RAPPORT

Toelichting omgevingsvergunningaanvraag beperkingengebiedactiviteit Noordzee - Aramis tunnel en ontvangstput

Klant: Aramis

Referentie: NL-ARM-040-ARM1-000002

Status: Definitief/02

Datum: 11 februari 2025

ARAMIS	**CCS-ARAMIS Project** **Environment Impact Assessment – Baseline report** Document No. NL-ARM-040-ARM1-000002 Document title Permit application Restricted area activity North Sea Revision Final 02

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1 Inleiding

1.1 Gegevens aanvrager

Gegevens aanvrager	
Statutaire naam aanvrager	N.V. Nederlandse Gasunie
Ligging project	Maasvlakte 1 en Territoriale zee
Handelsnaam	N.V. Nederlandse Gasunie
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1.2 Het Aramis initiatief

Integrale Aramis CCS-keten

Om de klimaatdoelstellingen te behalen, is er behoefte aan additionele CO_2-transportinfrastructuur, waarmee meerdere opslaglocaties op zee worden ontsloten voor opslag van CO2 van verschillende industriële emissiebronnen. Het Aramis initiatief speelt in op die behoefte door een nieuwe integrale en open CCS-keten mogelijk te maken. Het Aramis initiatief vormt een onderdeel van deze CCS-keten en bestaat uit de aanleg en exploitatie van een open CO_2-transportinfrastructuur. Samen met de afvanginfrastructuur en opslaginfrastructuur vormt dit de integrale CCS keten met onderstaande samenhangende onderdelen (zie Figuur 1-1 en 1-2).

CO_2-afvanginfrastructuur

1 CO_2-afvang bij industrie, en geschikt maken voor transport;

2 CO_2-transport naar het verzamelpunt op de Maasvlakte, middels de Porthos landleiding of per schip;

CO_2-transportinfrastructuur (Aramis initiatief) (zie Figuur 1-1)

3 CO_2-verzamelpunt op de Maasvlakte met een compressorstation en een terminal.

   a) Het compressorstation ontvangt gasvormig CO_2 dat via de Porthos landleiding wordt aangevoerd en brengt het op druk voor het transport per zeeleiding;

   b) De terminal ontvangt per schip aangevoerd vloeibaar CO_2. De terminallocatie bevat steigers, opslagtanks voor tijdelijke opslag van CO_2 en hogedrukpompen voor levering aan de zeeleiding.

   c) CO_2 uit het compressorstation en vanaf de terminal komen samen in de CO_2-zeeleiding;

   De CO_2 wordt in de zeeleiding vervoerd in de ‘dense phase’. De dense phase is een verdichtte toestand van een gas die eigenschappen van zowel vloeistof als gas vertoont. Onder dense phase wordt zowel ‘dense liquid’ (druk boven het kritische punt en temperatuur beneden het kritische punt) als ‘superkritische’ (zowel druk als temperatuur boven het kritische punt) condities verstaan.

4 CO_2-transport door de centrale CO_2-zeeleiding van naar het distributieplatform op de Noordzee. Er zijn tevens connectiepunten in de zeeleiding waarvandaan CO_2 aan andere platforms kan worden geleverd. De zeeleiding krijgt een lengte van ongeveer 230 km en start vanaf het mengpunt nabij het

compressorstation en loopt dan eerst over land, dan onder de zeewering en Maasgeul door en vervolgens over de zeebodem naar het distributieplatform (gelegen op ongeveer 80 km ten noordwesten van Den Helder).

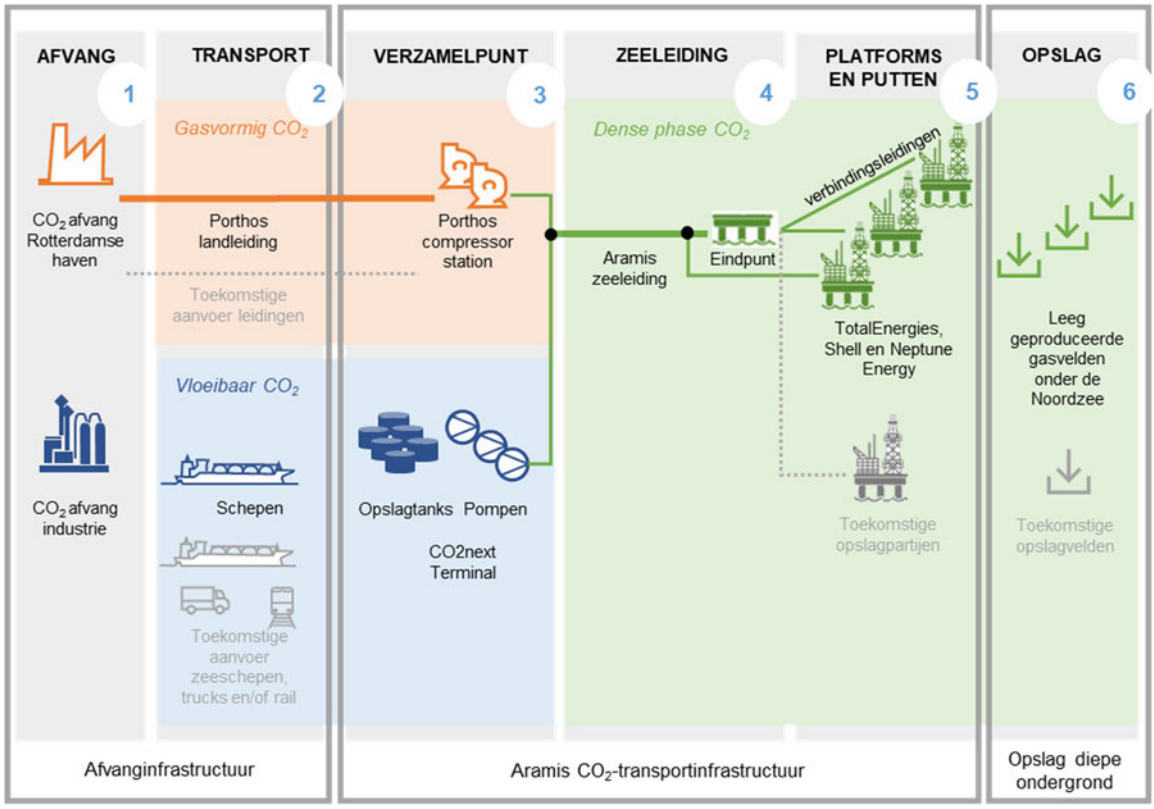
5 Het distributieplatform waarvandaan de CO_2 verder wordt gedistribueerd naar leeg geproduceerde gasvelden onder de Noordzee. Dit platform is uitgerust met een verdeelstation voor CO_2-toevoer naar de verschillende platforms. Het distributieplatform ligt nabij de zogenaamde K- en L-mijnbouwblokken waarin leeggeproduceerde gasvelden liggen waarin kan CO_2 worden opgeslagen. Vanaf het distributieplatform en vanaf connectiepunten op de zeeleiding kunnen opslagpartijen een verbindingsleiding leggen om hun platforms aan te sluiten. Via verbindingsleidingen komt de CO_2 vanaf het distributieplatform of vanaf de zeeleiding bij de injectieplatforms. Via putten op deze platforms wordt de CO_2 geïnjecteerd in leeggeproduceerde gasvelden in de diepe ondergrond van de Noordzee.

CO_2-opslag diepe ondergrond

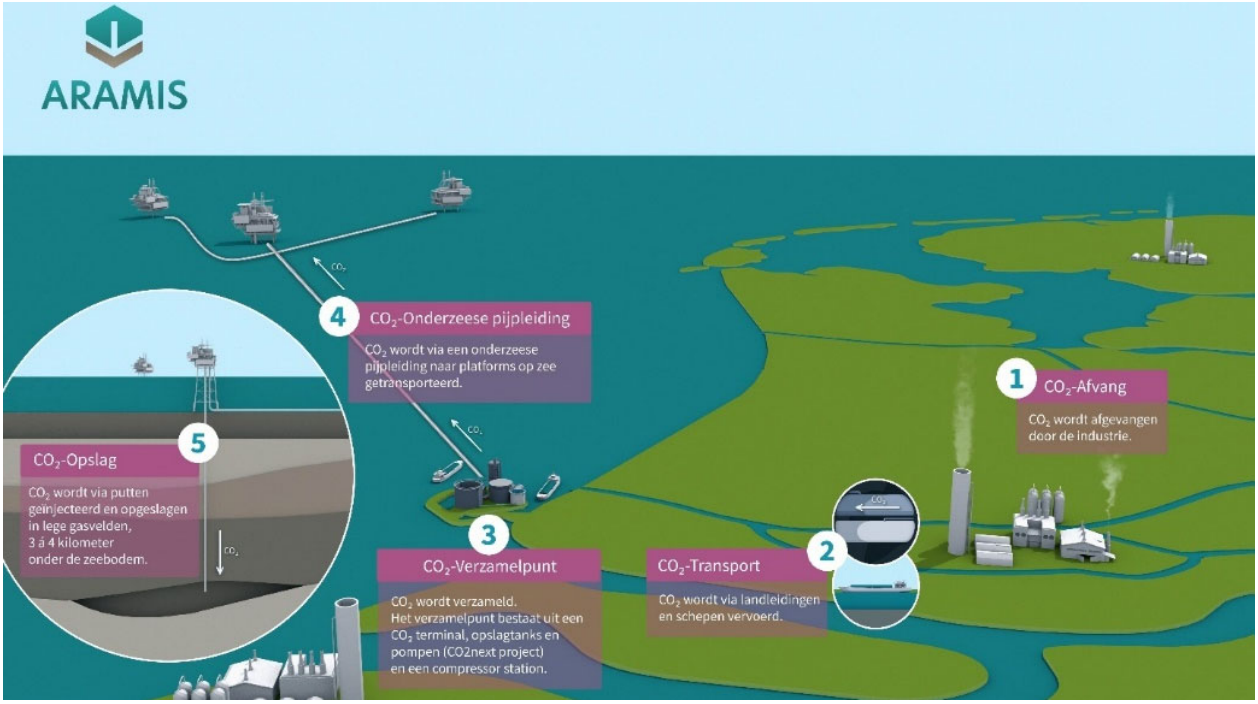
6 CO_2-opslag: permanente CO_2-opslag in de diepe ondergrond.

Het Aramis initiatief

Het Aramis initiatief heeft als doel het verzamelpunt (onderdeel 3), de zeeleiding (onderdeel 4) en de injectie (onderdeel 5) te realiseren. Hiervoor wordt door het Aramis-consortium (bestaande uit Shell, TotalEnergies, Gasunie en EBN) samengewerkt met CO2next (voor de terminal) en Porthos (voor het compressorstation). De opslag vindt plaats vanaf platforms van Shell, TotalEnergies en Eni (voorheen Neptune Energy). Voor het Aramis initiatief wordt een projectprocedure doorlopen en de procedures voor het projectbesluit en de benodigde vergunningen worden gecoördineerd door KGG. In dit kader is ook een MER opgesteld.



Figuur 1-1: Overzicht van de integrale CCS-keten met daarin de componenten die onderdeel zijn van de voorgenomen activiteit, namelijk: transport per schip, CO2next-terminal, uitbreiding compressorstation Porthos, zeeleiding met eindpunt en connectiepunten, aansluitleidingen en platforms. Daar waar in bovenstaande figuur Neptune Energy staat, moet Eni worden gelezen.



Figuur 1-2: Verbeelding van het Aramis CO_2-transport- en opslaginfrastructuur.

1.3 Scope aanvraag en voorgenomen activiteiten op hoofdlijnen

Deze aanvraag is opgesteld in het kader van Aramis project voor het aanbrengen van een microtunnel die start op land ten noorden van de Maasvlakteweg, en de ontvangstput op de zeebodem. Deze aanvraag ziet enkel op de besluitvorming over de omgevingsvergunning beperkingengebiedactiviteit Noordzee voor de onderdelen van het voorgenomen initiatief die binnen dit beperkingengebied vallen (zie figuur 1-3 en 1-4), namelijk de aanleg van de microtunnel onder de zeewering en het eindpunt van de microtunnel. De scope van de aanvraag is als volgt:

- het bouwen van een microtunnel in de bodem onder de zeewering en onder de Maasgeul;
- het lokaal ontgraven en weer aanvullen van een ontvangstput in de zeebodem ten bate van het terugwinnen van tunnelmaterieel en om de zeeleiding geleidelijk naar boven tot zeebodem niveau te begeleiden. De ontvangstput van de tunnelboormachine (TBM) is een tijdelijke open ontgraving met een vierkante vorm en flauwe taluds op de bodem van de Noordzee. De tunnel eindigt bij het eindpunt voor de tunnelboormachine. De scope van de aanvraag beperkt zich tot de activiteiten die in het beperkingengebied plaatsvinden.

Deze werkzaamheden vinden plaats in de Noordzee buiten de zone tussen de duinvoet en de laagwaterlijn.

Deze aanvraag ziet uitsluitend op de omgevingsvergunning voor de beperkingengebiedactiviteit Noordzee voor de aanleg van de microtunnel. Voor andere installaties, leidingen en milieubelastende activiteiten wordt apart toestemming gevraagd.
Geen onderdeel van deze aanvraag zijn de bouw van de startschacht en het aanbrengen van de on- en offshore CO_2-transportleiding. De startschacht ligt buiten het beperkingengebied.
Ook het aanbrengen van de CO_2-transportleiding (inclusief het deel in de tunnel binnen het beperkingengebied) is geen onderdeel van deze aanvraag aangezien deze activiteit valt onder artikel 94

en 95 van de Mijnbouwwet. Omwille van de specifieke zorgplicht uit artikel 7.6 van het Besluit activiteiten leefomgeving zijn deze activiteiten wel in deze aanvraag toegelicht, omdat deze activiteiten effecten kunnen hebben op de zeewering. Deze effecten zijn beschreven in paragraaf 6.2.

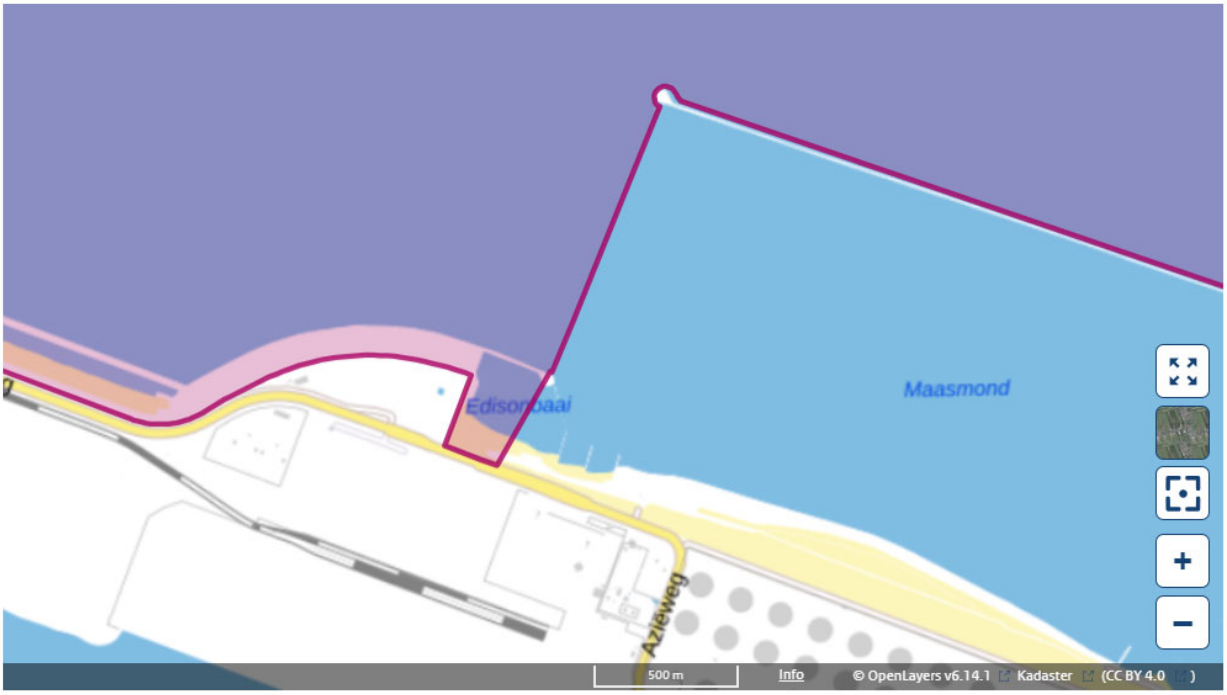
1.4 Leeswijzer

Hoofdstuk 1 geeft de scope van de aanvraag, het initiatief, het wettelijk kader, tijdvak, de locatie en de betrokken belanghebbenden weer. Hoofdstuk 2 gaat in op de details van de vergunningaanvraag, waaronder de het tracé, de ruimtelijke uitgangspunten, de wijze van aanleg en de voorgenomen activiteiten. Hoofdstuk 3 omvat de relevante effecten op het milieu en de omgeving in het kader van de vergunningaanvraag. Hoofdstuk 4 gaat in op de wijze van oprichting en constructie. Hoofdstuk 5 geeft een weergave van het onderhoudsplan. In hoofdstuk 6 worden de genomen veiligheidsmaatregelen en werkwijze bij eventuele calamiteiten toegelicht. Hoofdstuk 7 licht de uitbedrijfname toe.

1.5 Gebied van de aanvraag

De nautische gegarandeerde diepte van de Maasgeul is 23 meter ten opzichte van LAT (Lowest Astronomical Tide ofwel laagst mogelijke waterstand gebaseerd op de stand van zon en maan).
Het gebied is ingericht voor waterstaatkundige doeleinden. De op basis van het omgevingsplan voornaamste gebruiksfuncties van het gebied zijn zeewering en scheepvaart. Het gebied op land heeft op basis van het bestemmingsplan Maasvlakte 2 (onderdeel Omgevingsplan gemeente Rotterdam) ook de functie voor onder meer (spoor)wegen en paden, en kabels en (buis)leidingen).

In onderstaande figuren is met een rode lijn de begrenzing van het beperkingengebied Noordzee aangegeven. Deze aanvraag ziet slechts op de activiteiten die binnen dit gebied worden verricht.



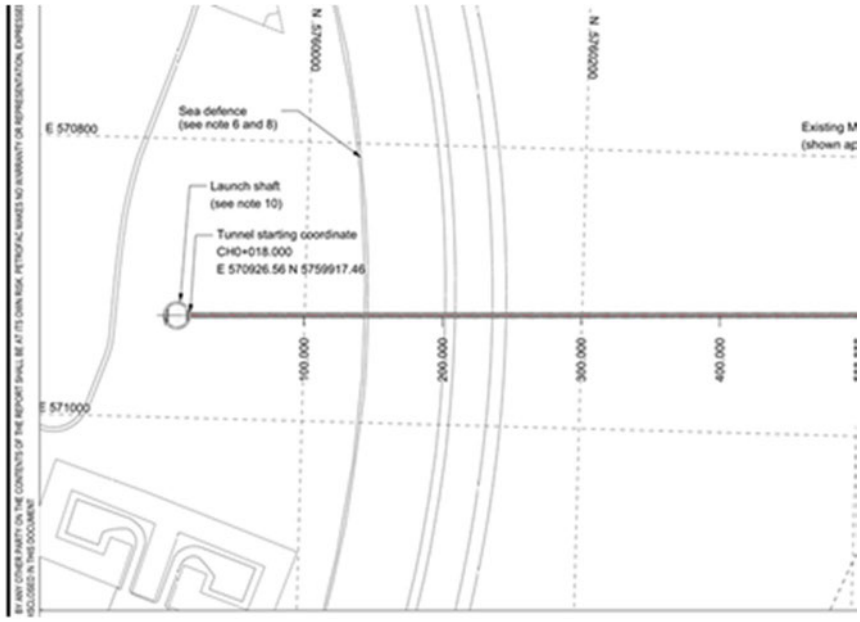
Figuur 1-3: Aanduiding beperkingengebied Noordzee (Bron: Omgevingsloket)



Figuur 1-4: Ligging schacht, tunnel en ontvangstput ten opzichte van beperkingengebied Noordzee en gemeentelijk ingedeeld gebied

Tabel 1-1: Coördinaten

Coördinatensysteem ETRS 1989 – UTM Zone 31 North	E	N
Startpunt tunnel (op land)	570926,56	5759917,46
Eindpunt (ontvangstput Noordzee)	570878,91	5761668,01



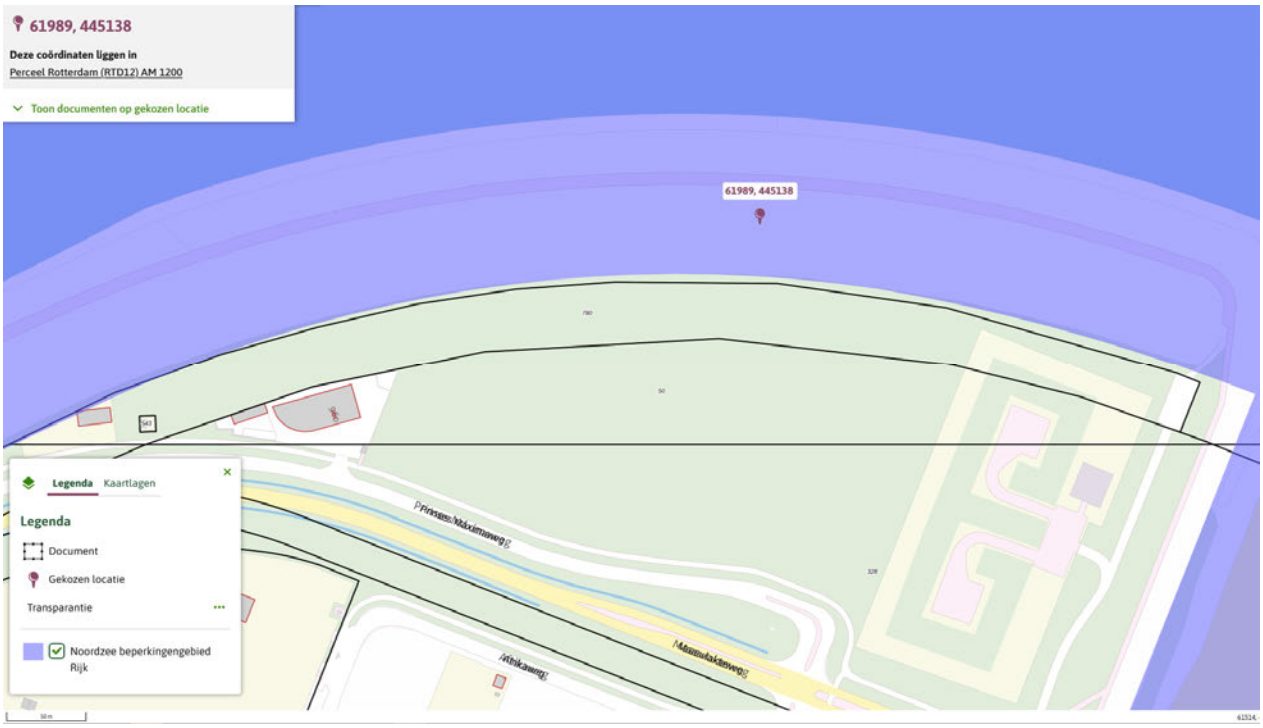
Figuur 1-5: Aanduiding coördinaten schacht (vanuit de FEED[1])

GEODETIC PARAMETERS					
EPSG CODE		25831	PROJECTION	NAME	UTM
DATUM	HORIZONTAL	ETRS 1989		ZONE	31 NORTH
	VERTICAL	LAT		CENTRAL MERIDIAN	3° EAST
SPHEROID	NAME	GRS 1980		LATITUDE OF ORIGIN	EQUATOR
	SEMI MAJOR AXIS	6 378 137.000		FALSE EASTING	500000m
	SEMI MINOR AXIS	6 356 752.314		FALSE NORTHING	0m
	INVERSE FLATTENING	298.257222101		SCALE FACTOR	0.9996

Figuur 1-6: Toelichting bij coördinaten

Kadastraal gezien zijn de volgende percelen betrokken bij de aanleg van de schacht en de tunnel: Kadastrale gemeente Rotterdam, (RTD12) sectie AM, nummers 328, 50, 780 en 1200. Deze laatste betreft hier hoofdzakelijk het beperkingengebied Noordzee, zie ook de volgende figuur.

[1] *FEED is Front-End Engineering Design*



Figuur 1-7: Betrokken kadastrale percelen met in perceel Rotterdam (RTD12) AM 1200 het beperkingengebied Noordzee

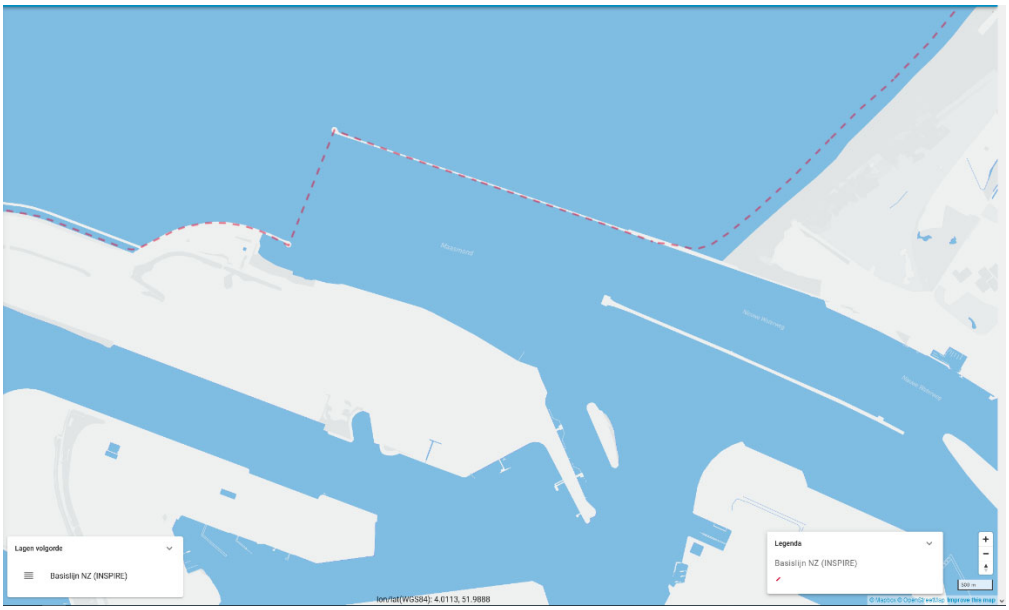
1.6 Scope van de aanvraag en wettelijk kader

Deze aanvraag betreft een beperkingengebiedactiviteit in of nabij een waterstaatswerk in beheer bij het Rijk als bedoeld in artikel 5.1, lid 2, sub f, onder 2 van de Omgevingswet (Ow) in het bijzonder voor activiteiten in de Noordzee buiten de zone tussen de duinvoet en de laagwaterlijn in de zin van artikel 7.1, lid 1, sub a/b van het Besluit activiteiten leefomgeving (Bal), voor het bouwen en in stand houden van bouwwerken en het plaatsen, in stand houden, veranderen of verwijderen van andere objecten ingevolge artikel 7.16, lid 1, sub a en c van het Bal juncto artikel 7.17, lid 1 sub d van het Bal, juncto artikel 7.20 van het Bal.

Het aanleggen van de microtunnel en de ontvangstput zijn niet opgenomen in bijlage V van het Omgevingsbesluit en dus niet mer-(beoordelings)plichtig.

Deze werkzaamheden vinden plaats in de Noordzee buiten de zone tussen de duinvoet en de laagwaterlijn conform 7.17 lid 1 sub d van het Bal. In onderstaande figuur is zichtbaar dat de laagwaterlijn gelijk loopt aan de ligging van de duinvoet. Dit brengt met zich mee dat een gebied binnen de duinvoet en de laagwaterlijn niet bestaat binnen het gebied waar de voorgenomen activiteiten plaatsvinden. Alle voorgenomen activiteiten voor het initiatief bevinden zich dan ook buiten deze zone.
De scope van deze vergunningaanvraag ziet op alle activiteiten die te maken hebben met:

- het bouwen of in stand houden van een bouwwerk, het aanleggen, plaatsen, in stand houden of veranderen van een werk dat geen bouwwerk is en het plaatsen of in stand houden van een ander object, anders dan bedoeld onder artikel 7.17 lid 1 sub a tot en met c van het Bal.



Figuur 1-8: Laagwaterlijn op de grens van de duinvoet/dijk/Noordzee.

Op grond van artikel 94 en 95 van het Mijnbouwbesluit is voor de aanleg van de CO_2-transportleiding reeds een vergunning aangevraagd. Deze transportleiding die in de microtunnel loopt, wordt niet beschouwd als vergunningplichtige beperkingengebiedactiviteit. In de eerste plaats niet omdat op grond van artikel 1.4 van de Omgevingswet de Omgevingswet niet van toepassing is op onderwerpen met betrekking tot de fysieke leefomgeving of onderdelen daarvan, die bij of krachtens een andere wet uitputtend zijn geregeld. Nu de vergunningplicht voor pijpleidingen al in artikel 94 respectievelijk 95 van het Mijnbouwbesluit wordt geregeld, ziet de reikwijdte van de Omgevingswet niet op deze pijpleiding. Dit wordt eveneens uitgesloten in artikel 7.16 lid 3 onder e van het Bal. De CO_2-transportleiding behoort daarom niet tot de scope van deze aanvraag voor een beperkingengebiedactiviteit. De scope van de aanvraag omvat alleen de ligging van de tunnel in de zeewering, de microtunnel en het eindpunt van de tunnel (ontvangstput) in het beperkingengebied Noordzee (zie figuur 1-4).

In de hoofdstukken 2 tot en met 7 van deze aanvraag wordt ingegaan op de in artikel 7.2 juncto 7.3 juncto 7.4 van de Omgevingsregeling algemene indieningsvereisten en de specifieke gegevens en bescheiden met betrekking tot de vergunningsaanvraag als bedoeld in de artikelen 7.159 juncto 7.177 van de Omgevingsregeling. In hoofdstuk 8 is een kruisverwijzing van de indieningsvereisten en de vindplaatsen in deze vergunningaanvraag opgenomen.

1.7 Planning/tijdvak van de aanvraag

De start van de werkzaamheden die vallen binnen het beperkingengebied Noordzee is afhankelijk van het tijdstip van in werking treden van de hiervoor benodigde (overige) vergunningen. Gepland is dat de voorbereidende werkzaamheden, zoals het bouwrijp maken van het terrein, het 2^{e} kwartaal van 2026 aanvangen. Hierna volgen respectievelijk de inrichting van het werkterrein in het 3^{e} kwartaal van 2026, de aanleg van de schacht omstreeks het 3^{e} kwartaal van 2026 en de start van de bouw van de microtunnel in het 1^{e} kwartaal 2027. De ingebruikname verwachten de Aramis initiatiefnemers in 2029.

1.8 Betrokken belanghebbenden

Participatie is een belangrijke pijler onder de Omgevingswet. Als iemand een vergunning aanvraagt, moet de aanvrager aangeven of burgers, bedrijven, maatschappelijke organisaties en bestuursorganen bij de plannen betrokken zijn geweest, hoe dat is gedaan en wat de uitkomst is.

In het kader van de projectprocedure voor Aramis is een participatieplan opgesteld waarin wordt aangegeven hoe aan de participatie vorm wordt geven. Dit houdt in dat naast de formele procedures de belanghebbenden ook rechtstreeks zijn/worden benaderd, naast de verplichte participatiemomenten. In bijlage 1, het Participatieplan, wordt toelichting gegeven over participatie binnen Aramis.

2 Activiteiten

2.1 Ruimtelijke uitgangspunten

Bij de ontwikkeling van de Aramis infrastructuur zijn al ruimtelijke keuzes gemaakt over de locatie van het verzamelpunt, de opslaglocaties en de tracés op de Noordzee. Deze zijn als uitgangspunten gebruikt bij de uitwerking van het technische ontwerp, het MER en de vergunningaanvragen, zijnde:

1 de ontwikkeling van een integraal en uitbreidbare infrastructuur;

2 verzamelpunt op de Maasvlakte;

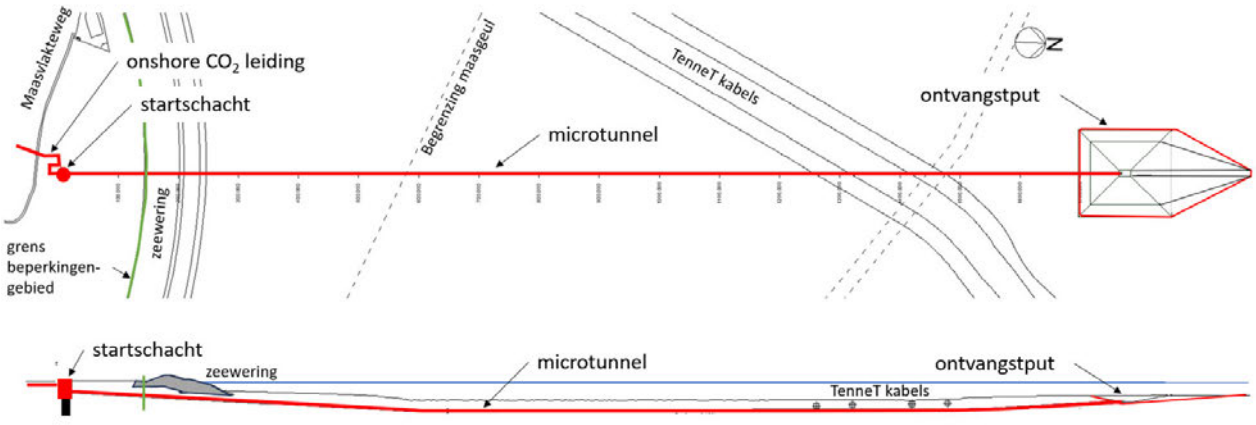
3 opslaglocaties;

4 tracé zeeleiding richting Noordelijke velden.

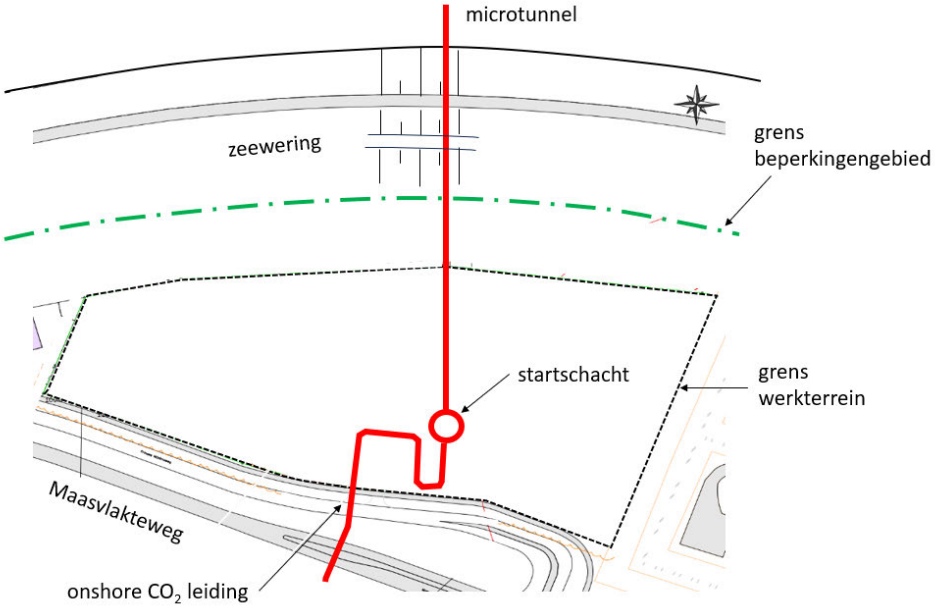
De benoemde ruimtelijke keuzes zijn opgenomen in het ontwerpprojectbesluit dat 13 september tot en met 24 oktober 2024 ter inzage heeft gelegen.

2.2 Beschrijving tracé microtunnel

De voorgenomen activiteiten in en nabij het beperkingengebied bestaat uit het aanbrengen van een microtunnel die start op land ten noorden van de Maasvlakteweg. De tunnel kruist de zeewering en de vaarroute (Maasgeul) en een kabelstrook van TenneT. Beginpunt van de microtunnel is de startschacht (buiten het beperkingengebied), eindpunt is de tijdelijke ontvangstput op de zeebodem. Van hieruit wordt de leiding op de zeebodem aangebracht. Zie Figuur 2-1 en Figuur 2-2, en tekening NL-ARM-040-ARU1-110323_01_IFD_20240919_01 (zie bijlage 2).



Figuur 2-1: Bovenaanzicht en langsdoorsnede van de voorgenomen activiteiten (figuur gebaseerd op NL-ARM-040-ARU1-110323_01_IFD_20240919_01, bijlage 2, en NL-ARM-040-PTC1-113855_01_IFD_20240820_01, bijlage 3).



Figuur 2-2: Bovenaanzicht voorgenomen activiteiten en begrenzing werkterrein ten noorden van de Maasvlakteweg (figuur gebaseerd op NL-ARM-040-PTC1-110261_03_IFD_20240816_01, bijlage 4).

2.3 Microtunnel

Om de CO_2-transportleiding onder de zeewering en de vaarroute (Maasgeul) mogelijk te maken is een tunnel voorzien. Het startpunt van de tunnel is de startschacht. Het eindpunt is de ontvangstput. De totale lengte van de tunnel bedraagt 1751 m. De tunnel wordt aangelegd in een rechte lijn (op het horizontale vlak) tussen startschacht en ontvangstput. Het alignement is aangegeven op tekening NL-ARM-040-ARU1-110323_01_IFD_20240919_01 (bijlage 2).

De tunnel start bij de startschacht op een diepte van ongeveer 19 m onder maaiveld (ongeveer 12,5 m onder NAP), en loopt onder een helling van 5% naar beneden. Met een startschacht op deze diepte wordt ervoor gezorgd dat de tunnel voldoende diep onder de zeewering ligt zodat de tunnel niet in aanraking komt met de onderkant van de zeewering. Ook komt de tunnel zo voldoende diep onder de Maasgeul te liggen. De vaarroute ligt op circa 600 m vanaf de startschacht. Hier ligt de tunnel nagenoeg horizontaal en ongeveer 16 m onder zeebodemniveau en 41 m onder NAP. Op circa 1300 m vanaf startpunt liggen TenneT kabels. De tunnel gaat hier op mimimaal 5 m onderdoor. Op ongeveer 1500 m vanaf startpunt buigt de tunnel omhoog naar het eindpunt op ongeveer 1750 m.

De tunnel heeft een inwendige diameter van 3 m en bestaat uit beton met een dikte van 250 mm en 300 mm (afhankelijk van het de gekozen bouwmethode, zie 4.2.2) en een groutschil aan de buitenzijde. Nadat de CO_2-transportleiding in de tunnel is aangebracht, wordt de microtunnel gevuld met grout[2]. Voor de methode van aanleg wordt verwezen naar paragraaf 4.2.

2.4 Ontvangstput

De ontvangstput van de tunnelboormachine (TBM) is een tijdelijke open ontgraving en flauwe taluds op de bodem van de Noordzee. De tunnel eindigt hier. De TBM wordt teruggewonnen en de CO_2-transportleiding zal vanaf hier op de bodem van de zee worden aangelegd. Vervolgens wordt de

[2] *Mogelijk dat in plaats van grout voor zand wordt gekozen.*

ontvangstput gedeeltelijk gevuld met zand en gedeeltelijk gevuld door natuurlijke verschijnselen zoals sedimenttransport van zand tot het niveau van de oorspronkelijke zeebodem.
Voorafgaand aan de start van de werkzaamheden levert Aramis een werkplan aan, waarin onder andere wordt beschreven hoe het overgangsgebied tussen het einde van de tunnel en daar waar de CO_2-transportleiding op de bodem van de zee ligt, wordt uitgevoerd.

Er wordt vergunning aangevraagd voor twee varianten van de ontvangstput: het Secondary Lining ontwerp, en het Monopile ontwerp. Deze varianten definiëren de wijze waarop de TBM vanuit de ontvangstput wordt teruggewonnen. Voor een verdere uitleg wordt verwezen naar paragraaf 4.3. Secondary Lining heeft de voorkeur. De diepte van de ontvangstput bedraagt dan 10,6 m, en afmetingen van de ontgravingsplot circa 150x300 m. Voor het Monopile ontwerp is de diepte 14,5 m, met een plot van ongeveer 200x400 m. De afmetingen van de ontvangstput zijn voor het voorkeursontwerp weergegeven op tekening NL-ARM-040-PTC1-113855_01_IFD_20240820_01 (bijlage 3). Omdat het niveau van de zeebodem in tijd varieert als gevolg van getijdewerking en sedimenttransport kan het uiteindelijke baggerprofiel afwijken van de geleverde tekeningen.

2.5 Mogelijke toekomstige activiteiten

Als mogelijk toekomstige ontwikkelingen ziet Aramis het toevoegen van een mantelbuis in de microtunnel, naast de CO_2-transportleiding.

3 Milieugevolgen Aramis infrastructuur

3.1 Huidige situatie

Landdeel
De Maasvlakte is een groot industriegebied dat is aangelegd in de Maasmonding. De vlakte ligt direct aan de Noordzee, ze maakt deel uit van de Rotterdamse haven en behoort tot de gemeente Rotterdam. Het projectgebied ligt deels op het terrein van Gate terminal en deels op het terrein van de MOT (Maasvlakte Olie Terminal). Op dit gedeelte van de Maasvlakte zijn geen kwetsbare gebieden aangewezen. Wel dient rekening gehouden te worden met trekvogels die op de Maasvlakte en de kust voor de Maasvlakte foerageren. Verder kunnen de beschermde soorten gladbiggenkruid en rugstreeppad hier voorkomen.

Op land volgt de CO_2-transportleiding de daar gelegen leidingstrook, waarin zich onder andere twee aardgasleidingen (Gasunie), twee waterleidingen (Evides) en een elektriciteitskabel (Stedin) bevinden en de toekomstige Porthosleiding gepland is. In westelijke richting loopt de leiding langs de kazerne van de Gezamenlijke Brandweer en het transformatorstation van TenneT. Langs het tracé staan enkele windturbines. Aan de westzijde worden nieuwe windturbines door Eneco op de zeewering geplaatst.

Rijkswaterstaat heeft aangegeven dat voor de waterkering langs de Maasvlakte dezelfde regels gelden als voor een primaire waterkering, alhoewel de Maasvlakte verhoogd gebied is dat niet beschermd hoeft te worden voor overstromingen ten gevolge van het getij en deze waterkering officieel geen onderdeel is van het primaire waterkering systeem in Nederland. Er bevinden zich wegen en een (haven-)spoorweg in het projectgebied, die tijdens de aanlegfase en gebruiksfase in gebruik moeten blijven.

Zeedeel
Het Natura-2000 gebied Voordelta grenst direct aan de Maasvlakte. Voor de gehele zeebodem geldt dat er mogelijk archeologische waarden aangetroffen kunnen worden. Het gebied wordt verder benut door de visserij en scheeps- en pleziervaart.

3.2 Milieueffecten leefomgeving, bodem, waterkwaliteit- en kwantiteit

Het landdeel bestaat uit het tracé vanaf het compressorstation naar kruising van de zeewering, de boring onder de zeewering en de Maasgeul. Er zijn neutrale, licht negatieve en negatieve effecten voorzien. De negatieve effecten van de microtunnel hebben betrekking op archeologische waarden en op omgevingsveiligheid in de gebruiksfase en gebiedsbescherming in de aanlegfase vanwege stikstofdepositie door baggerwerk.

Aanlegfase
Voor de kruising met de zeewering en Maasgeul vindt verstoring van de bodem plaats door vergraving bij de startschacht, vindt er afvoer van materiaal uit de boring plaats en verplaatsing van bagger voor aanleg van de sleuf van de leiding. Bij de microtunnel boring, wordt de baggerlaag op de waterbodem ongemoeid gelaten waardoor de hydrologische weerstand daar niet wijzigt en daarmee de hydrologische omstandigheden niet zullen veranderen. Er zijn luchtemissies, geluidhinder en onderwatergeluid als gevolg van de inzet van bouwmaterieel. Indirect heeft dit ook licht negatieve gevolgen op de natuur. Als gevolg van de werkzaamheden voor het aanleggen van de kruising van de leiding met de Maasgeul zijn nautische risico's en lichthinder te verwachten.

In de aanlegfase wordt geen oppervlaktewater onttrokken en vindt eveneens geen lozing plaats waardoor geen invloed op de waterkwantiteit wordt voorzien. Er wordt zoveel mogelijk voorkomen dat stoffen, die gebruikt worden tijdens de aanlegfase bij het bouwen van de tunnel en andere werkzaamheden benodigd

zijn, het water in de Noordzee aantasten. Daarmee wordt verwacht dat dit geen (negatieve) effecten voor de waterkwaliteit oplevert.

Ten aanzien van archeologie heeft de Rijksdienst voor het Cultureel Erfgoed het in opdracht van Aramis uitgevoerde archeologisch onderzoek goedgekeurd, zie ook bijlage 5.

Voor de mogelijke milieueffecten in de Noordzee kan op basis van de passende beoordeling zeegebieden als onderdeel van de omgevingsvergunningaanvraag Natura 2000-activiteit (ARM-PFE-B10-ENV-EIA-2004 van mei 2024) het volgende geconcludeerd worden:

- Wezenlijke effecten op platte oester, mossel en zandkokerworm door oppervlakteverlies kunnen worden uitgesloten;
- Wezenlijke effecten op zandkokerwormen door oppervlakteverlies kunnen niet worden uitgesloten;
- Wezenlijke effecten op relevante soorten door vertroebeling kunnen worden uitgesloten;
- Wezenlijke effecten op relevante soorten door verandering dynamiek kunnen worden uitgesloten;
- Significante effecten door onderwatergeluid op bruinvissen kunnen niet worden uitgesloten.

Mitigerende maatregelen
Ondanks het feit dat de zandkokerworm als soort niet wettelijk beschermd is, zal Aramis zijn best doen om de volgende mitigerende maatregelen uit te voeren om wezenlijke effecten te voorkomen:

- Bruinvissen komen in de Noordzee en specifiek in meerdere Natura 2000-gebieden voor. Hierbij geldt dat er in de Natura 2000-gebieden geen significante directe effecten optreden, maar indirecte effecten door onderwatergeluid op bruinvissen kunnen niet worden uitgesloten (indirecte effecten op de gehele Noordzee populatie). Daarom dienen de volgende maatregelen te worden genomen:
  - Er dient zoveel mogelijk gebruik gemaakt te worden van stille schepen om continu onderwatergeluid te minimaliseren.

3.3 Effecten voor de maatschappelijke functies van het waterstaatswerk

De maatschappelijke functie van het waterstaatswerk is om de krachten van het getij te weerstaan, zodat de Maasvlakte niet wordt beschadigd en afkalft. Het waterstaatswerk beschermt zo de havenactiviteiten tegen dagelijkse omstandigheden, maar ook tegen extreme stormen met een overschrijdingskans van 1/4000 per jaar (1/10.000 per jaar voor Maasvlakte 2). Deze functie wordt niet beïnvloed door de aanleg en de aanwezigheid van de startschacht en microtunnel.

4 Oprichtings- en constructieplan

4.1 Inleiding

In onderstaande paragrafen is het ontwerp en de uitvoeringsmethode van microtunnel en ontvangstput op hoofdlijnen toegelicht. Voor een meer gedetailleerde omschrijving wordt verwezen naar NL-ARM-040-ARU1-110306_02_IFD_2024-10-21_01 (zie bijlage 6).

4.2 Microtunnel

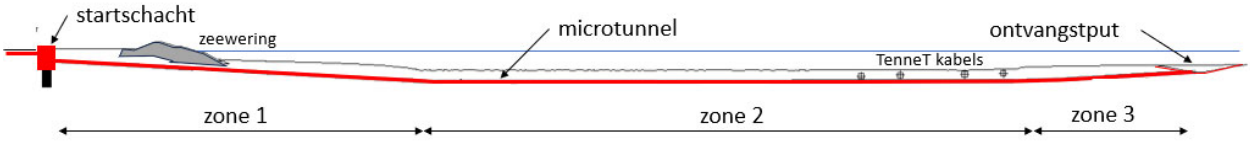
4.2.1 Alignement

Het horizontale alignement bevat geen horizontale bochten over de lengte van de tunnel. Zie tekening NL-ARM-040-ARU1-110323_01_IFD_20240919_01 (zie bijlage 2).

Het verticale alignement is onderverdeeld in drie zones:

- Zone 1: gradiënt in deze zone (-5%) is een functie van de operationele gradiënt van het transportsysteem in de tunnel tijdens de uitvoering (supply train locomotief) en de diepte van de schacht. De minimale dekking wordt bepaald door de zettingsanalyse onder de zeewering.
- Zone 2: gradiënt in deze zone (+0,1%) is een functie van het waterbeheer binnen de tunnel tijdens de bouw. Er is een laag punt in de tunnel tussen Zone 1 en Zone 2, waar een sump pomp mogelijk nodig is. De minimale dekking onder het scheevaartkanaal wordt bepaald door de frontstabiliteit van de TBM en de zettingsanalyse onder de TenneT-kabels.
- Zone 3: gradiënt in deze zone (+5%) is een functie van de operationele gradiënt van de locomotief. De minimale dekking wordt bepaald door de frontstabiliteit van de TBM bij de ontvangstput.

Zie ook Figuur 4-1 en Figuur 4-2. Het verticale alignement is gepresenteerd in tekening NL-ARM-040-ARU1-110323_01_IFD_20240919_01 (bijlage 2).



Figuur 4-1: Het verticale alignement van de tunnel in 3 zones

Asset	Criteria	Minimum cover above crown of tunnel
Sea defence	NEN 3651 Clause 9.6.2	3m
	Settlement Assessment	6.5m
	Recommendation	**Min. 6.5m**
Shipping channel	NEN 3651 Clause 8.1.6.2	4m
	Face stability Assessment	10 to 13m
	Recommendation	**Min. 13m**
TenneT cables	NEN 3651 Clause 9.6.2	3m
	Settlement Assessment	5m to the lowest TenneT cable **(Around 13 m to the seabed)**
	Recommendation	**Min. 5m to the lowest TenneT cable** **(Around 13 m to the seabed)**

Figuur 4-2: Criteria voor minimale dekking boven de kruin van de tunneldoorsnede

4.2.2 Uitvoeringsmethode

Voor de Aramis-tunnel wordt een hybride graafmethode voorgesteld. Voor de bouw van de tunnel in deze omstandigheden komen twee methoden in aanmerking: pipejacking en segmental tunnelen (zie het kader hieronder voor verdere uitleg van deze methoden). Bij beide methoden wordt gebruik gemaakt van een tunnelboormachine (TBM) die de grond met een graafwiel afschraapt en door de tunnel afvoert via de startschacht. Het boorfront wordt stabiel gehouden door daar een hoge water of bentonietdruk te handhaven die evenwicht maakt met de heersende gronddruk op tunneldiepte. Bij pipe jacking worden ronde buizen vanuit de startschacht door middel van hydraulische vijzels achter de TBM in de grond geduwd. Bij segmental tunneling wordt de tunnel opgebouwd uit betonnen segmenten die direct achter de TBM aan elkaar worden bevestigd en samen de tunnelbuis vormen. Het ontwerp van de tunnel is gebaseerd op drie 'omschakel' scenario's:

1. Hybride 'Vliegende start': een start van de tunnelbouw in pipejacking-modus en overschakelen naar segmental tunnelen na ongeveer 100-150 m.
2. Hybride oplossing voor maximale snelheid van de bouw met een geplande overschakeling van pipejacking naar segmental tunnelen tussen 1000 m en 1500 m, afhankelijk van de boven- en ondergrondse omstandigheden.
3. Overschakeling van pipejacking naar segmentele tunnelbouw op basis van de observatiemethode tijdens de bouw. Dit scenario biedt de EPC-contractor flexibiliteit om de locatie van de overschakeling te kiezen, rekening houdend met de werkelijke grondomstandigheden, de logistiek op de bouwplaats en potentiële risico's tijdens de bouw. In dit scenario is er ook de mogelijkheid om pipejacking toe te passen over de volledige lengte van de tunnel, met behoud van mogelijkheid om als noodmaatregel naar segmental tunnelen over te schakelen.

Kader uitleg pipe-jacking en segmental tunneling

Pipejacking
Bij de meeste microtunnelingoperaties wordt de tunnelbekleding in de vorm van buiselementen achter de machine geduwd. Dit pipe jacking (buisvijzel) proces wordt herhaald totdat de TBM het eindpunt bereikt. De wrijving van de grond aan de buitenzijde van de buizen neemt toe in verhouding tot de tunnelengte en diameter. Om deze te minimaliseren wordt geboord met oversnijding om speling te creëren tussen de verwijderde grond en de buitenrand van de tunnelbuis. De geboorde diameter is daarbij net iets groter dan de buitendiameter van de tunnelbuis. Een smeermiddel, vaak bentoniet suspensie, wordt in deze speling geïnjecteerd. De druk van het smeermiddel voorkomt dat de grond in elkaar valt en de speling zo tenietdoet.
Bij lange tunnels zijn er alsnog enorme krachten nodig om de machine en de tunnelbuizen vooruit te duwen. Het hydraulische jacking frame in de startschacht moet deze kracht leveren. De ingangsschacht moet sterk genoeg zijn om de krachten die het genereert te ondersteunen. Als de tunnellengte te groot wordt voor het jackingframe in de startschacht, kunnen ook kleinere vijzels (interjacks) tussen secties van de tunnelbekleding worden gestoken.
https://www.herrenknecht.com/en/products/productdetail/avn-machine/

Segmental tunneling
Bij segmental tunneling wordt de betonnen bekleding niet aan de achterzijde (in de startschacht) maar aan de voorzijde aangevuld. De consequentie hiervan is dat de bekleding in segmenten moet worden opgebouwd tot een ring. De segmenten worden via de startschacht middels een supply train door de tunnel naar de voorzijde getransporteerd. De TBM wordt afgevijzeld op de laatst aangebracht ring. Als de TBM voldoende grond heeft weggegraven worden de vijzels ingetrokken en kunnen er nieuwe segmenten worden geplaatst.
https://www.youtube.com/watch?v=2ERyP6JTPyo

4.2.3 Tunneldoorsnede

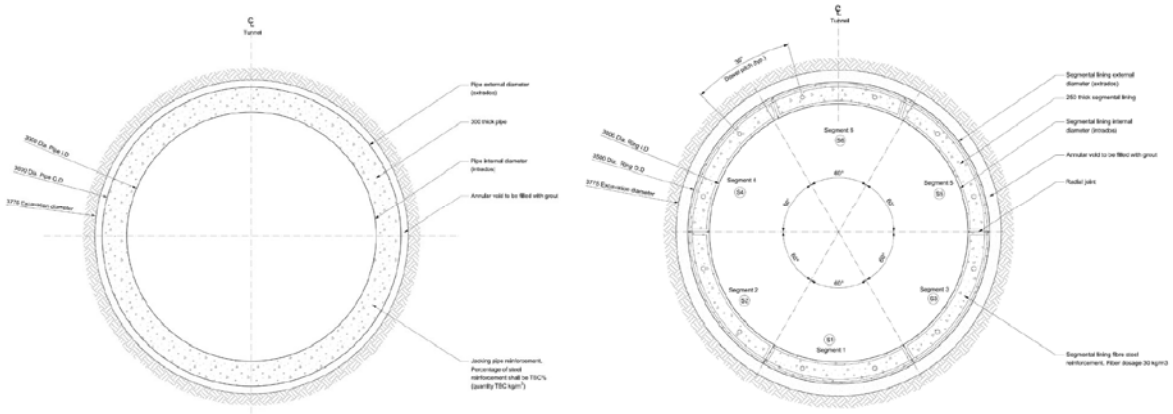
Er is in het FEED ontwerp aanbevolen een binnendiameter toe te passen van 3 m. Bij grotere diameters neemt de benodigde hoeveelheid af te voeren en aan te voeren grond substantieel toe. Daarnaast is er bij een standaardmaat van 3 m diameter een grotere kans op de beschikbaarheid van TBM's op de markt, zonder deze nieuw te hoeven bouwen. Het nieuw bouwen van een hybride TBM (voor pipejacking- en

segmentele bekledingsgraafmethoden) zou een geschatte doorlooptijd van ongeveer 12 maanden vergen, wat een potentiële beperking zou vormen voor het projectprogramma.

Er is een uitgebreide analyse uitgevoerd om te waarborgen dat alle noodzakelijke functies tijdens de uitvoering binnen het geplande tunnelvolume kunnen worden ondergebracht. Voor de CO_2-transportleiding is uitgegaan van een enkele circa 32 inch (812,8 mm) stalen buis (en bijbehorende ondersteuningen). De ruimte die hiervoor nodig is, is niet maatgevend voor de keuze van de inwendige tunneldiameter.

Op basis van de hybride graafmethode zoals beschreven in paragraaf 4.2.2, zijn er binnen het project twee typen tunnel-lining te onderscheiden: buizen (voor pipe-jacking), en segmenten (voor segmental tunneling), zie ook Figuur 4-3.

- Buizen. Dit betreft een voorgevormde betonnen buis met een binnendiameter van 3,0 m en een bekledingsdikte van 300 mm.
- Segmenten. Dit betreft voorgevormde betonnen segmenten met een binnendiameter van 3,0 m en een segmentdikte van 250 mm. Elke ring bestaat uit zes segmenten met een rechthoekige configuratie (6+0 opstelling).



Figuur 4-3: Doorsnede voor methode pipejacking (links) en methode Segmental tunneling (rechts) (uit NL-ARM-040-ARU1-110306_02_IFD_2024-10-21_01, bijlage 6).

4.2.4 Bouwterrein

De bouw van een microtunnel met de voorgenomen technieken is een complex proces. Alle werkstromen gaan via de startschacht. Het bouwterrein dat zich concentreert rondom de startschacht heeft onder meer de volgende functies:

- toegangswegen;
- gronddepots;
- opslagtanks;
- stroomvoorziening;
- scheidingsinstallatie voor afgegraven grond;
- opslag materialen zoals tunnelringen en lining;
- kranen;
- ketenpark.

Op tekening NL-ARM-040-PTC1-110261_03_IFD_20240816_01 (bijlage 4) is een inrichtingsvoorstel gegeven van het werkterrein. Alle functies liggen daarbij buiten het beperkingengebied.

4.3 Ontvangstput

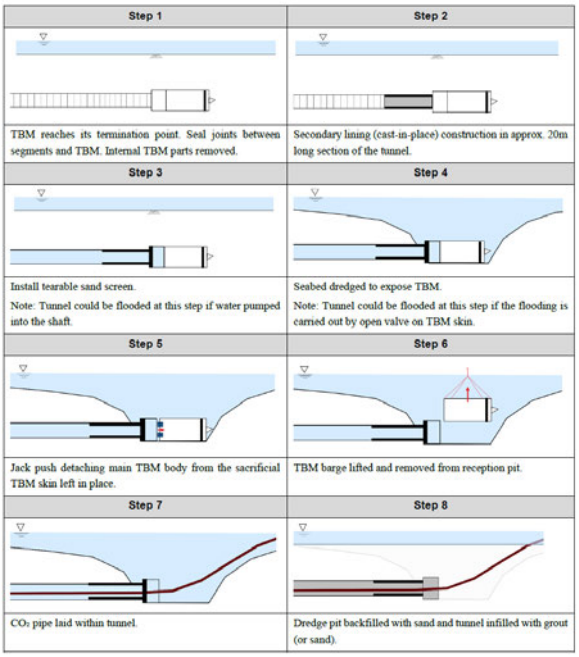
4.3.1 Algemeen

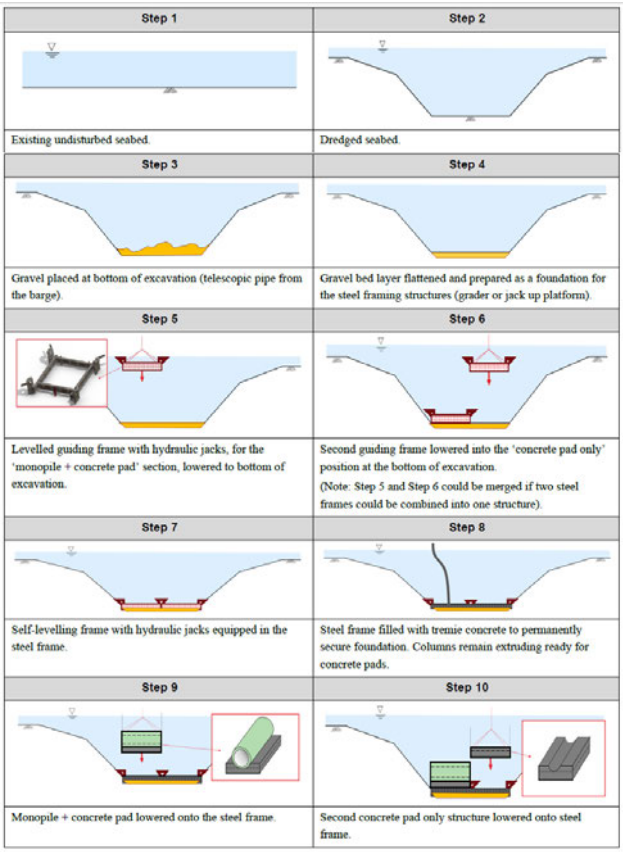
De ontvangstput van de tunnelboormachime (TBM) is een tijdelijke open ontgraving op de bodem van de Noordzee. De tunnel eindigt hier. De TBM wordt teruggewonnen en de CO_2-transportleiding zal vanaf hier op de bodem van de zee worden aangebracht. Vervolgens wordt de ontvangstput gevuld met zand tot het niveau van de oorspronkelijke zeebodem. De diepte van de ontvangstput wordt voornamelijk bepaald door de minimaal benodigde gronddekking boven de TBM voor een stabiel boorproces, en daarnaast de wijze waarop de TBM vanuit de ontvangstput wordt teruggewonnen. De afmetingen van de ontvangstput zijn weergegeven op tekening NL-ARM-040-PTC1-113855_01_IFD_20240820_01 (zie bijlage 3).

4.3.2 Terugwinnen tunnelboormachine

Voor het terugwinnen en boven water halen van de TBM zijn er twee hoofdoplossingen beschreven in het FEED ontwerprapport (NL-ARM-040-ARU1-110306_02_IFD_2024-10-21_01, bijlage 6) te weten; inwendig versterken van de tunnel (Secondary Lining optie) en uitwendig versterken van de tunnel (Monopile optie). De eerstgenoemde oplossing is de voorkeursoplossing. Beide opties omvatten maatregelen die ervoor zorgen dat de het einde van de tunnelbuis niet omhoogkomt (opdrijven), en constructief in staat is om belastingen op te nemen die optreden bij het proces van het terugwinnen van de TBM en het opengraven van de tunnelmond.

Bij de Secondary Lining optie wordt de tunnel hiertoe inwendig versterkt met een betonnen schil (dikte 225mm), over een lengte van circa 20m achter de TBM. Deze schil maakt de constructie sterker en zwaarder. Het proces is weergegeven in Figuur 4-4. Bij de Monopile optie wordt de noodzakelijke versterking van de tunnel uitwendig gerealiseerd door de tunnel te boren door een vooraf horizontaal afgezonken betonnen buis (Monopile). De uitwendige diameter van de Monopile bedraagt 4,5 m. Om de Monopile te verzwaren zodat hij niet tordeert bij het boren wordt deze bevestigd aan een circa 0,75 m dikke betonnen fundatieplaat aan de onderzijde. De Monopile constructie (buis en plaat) wordt afgezonken op een grondverbetering (gravel bed). De grondverbetering, de fundatieplaat en de dikte van de Monopile maken dat de ontgravingsdiepte van de ontvangstput circa 4m groter is dan bij de Secondary Lining optie, zie Figuur 4-5.





Figuur 4-4: (links) Uitvoeringsmethode Secondary Lining optie (uit NL-ARM-040-ARU1-110306_02_IFD_2024-10-21_01, bijlage 6).

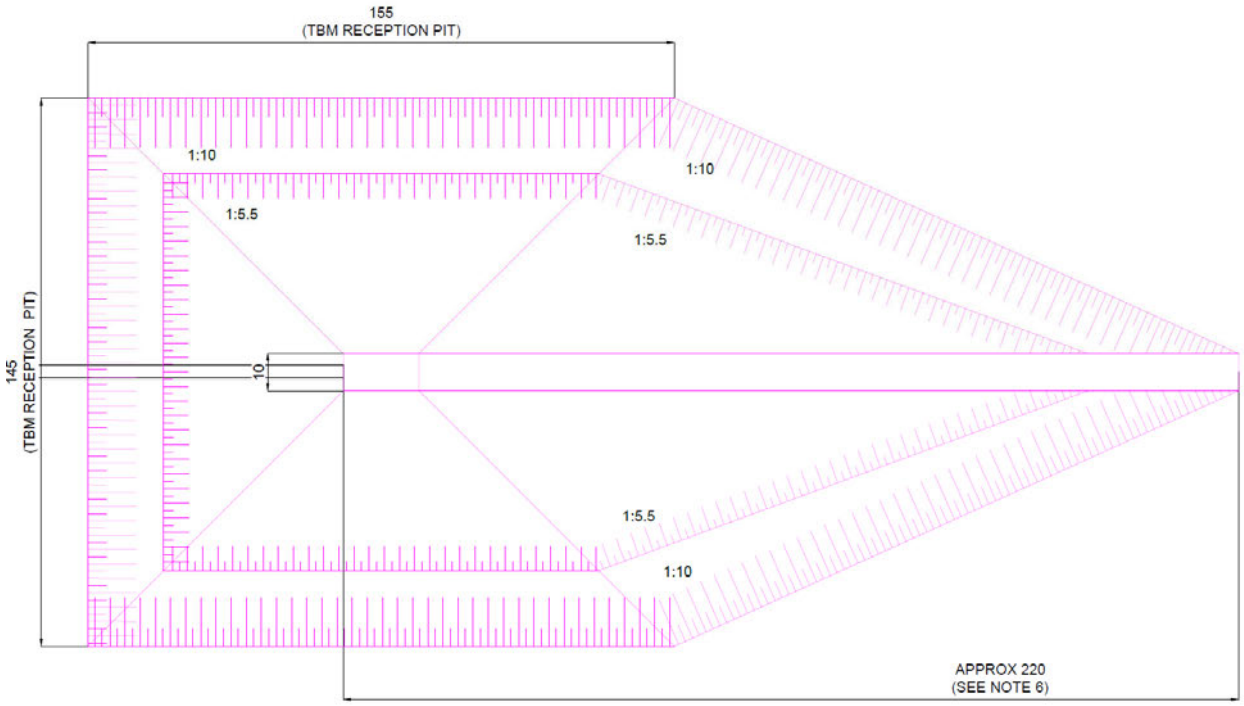
Figuur 4-5: (rechts) Uitvoeringsmethode Monopile optie (uit NL-ARM-040-ARU1-110306_02_IFD_2024-10-21_01, bijlage 6).

4.3.3 Afmetingen

Dwangpunten voor de geometrie van de tijdelijke open ontgraving zijn de volgende:

- het alignement van de tunnel met een helling van 5% en een minimale gronddekking van 7,55 m boven de tunnelmond om instabiliteit van het boorfront te voorkomen;
- voor de Secondary Lining optie een ontgravingsdiepte van ongeveer 10,6 m onder niveau zeebodem;
- voor de Monopile een ontgravingsdiepte van ongeveer 14,5 m onder niveau zeebodem;
- een horizontaal plateau in het diepste punt van de ontgraving van 10 x 20 m om de verschillende werkzaamheden (onder andere TBM terugwinnen, aanbrengen CO_2-transportleiding, en het vullen van de tunnel) uit te kunnen voeren;
- aan drie zijden een 1:5,5 onderwatertalud in de zandlagen en 1:10 onderwatertalud in de siltlaag onder de zeebodem. De vierde zijde heeft een 1:20 talud. Dit is de zijde waar de CO_2-transportleiding uit de tunnelmond onder helling naar de zeebodem loopt.
- Omdat het niveau van de zeebodem in tijd varieert als gevolg van getijdewerking en sedimenttransport, en ook de stratigrafie over de plot van de ontvangstput horizontaal varieert, kan het uiteindelijke baggerprofiel afwijken van de geleverde tekeningen in het FEED design.

In onderstaande figuur zijn de afmetingen van de Secondary Lining optie (voorkeur) aangegeven. Zie ook tekening NL-ARM-040-PTC1-113855_01_IFD_20240820_01 (bijlage 3).



Figuur 4-6: .Bovenaanzicht ontvangstput secondary lining optie. (NL-ARM-040-PTC1-113855_01_IFD_20240820_01, bijlage 3)

4.3.4 Blijvende objecten in ondergrond

Wat er na de operatie ter plaatse van de ontvangstput in de ondergrond achterblijft is afhankelijk van de gekozen optie voor het terugwinnen van de TBM. Bij de Secondary Lining optie zijn naast de tunnel(mond) met inwendige verdikking geen andere objecten die in de ontvangstput achterblijven. Bij de Monopile optie zijn dit:

- de Monopile om de tunnelmond, met daaronder de funderingsplaat;
- het gravelbed onder de funderingsplaat
- een frame met vijzels waarmee de monopile/funderingsplaat exact in positie kan worden gebracht.

5 Onderhoudsplan

Na ingebruikname worden er geen onderhoudswerkzaamheden uitgevoerd aan de tunnel en de startschacht. Dit is ook niet mogelijk omdat de tunnel wordt gevuld met grout (of zand) en de schacht met zand.

6 Veiligheids- en calamiteitenplan

6.1 Inleiding

De doelstelling van de Aramis initiatiefnemers is dat geen schade aan mensen, milieu of de maatschappij optreedt. Ook de nog te selecteren aannemers die de werkzaamheden voor de aanleg van de schacht en de tunnel zullen uitvoeren, worden hieraan gehouden.
In de ontwerpfase heeft Aramis op basis van veiligheidsstudies, zoals HAZID, HAZOP en HAZCON, potentiële risico's geïdentificeerd. Deze potentiële risico's vormen de basis voor het op te stellen veiligheidsplan (HSSE-plan).
De te selecteren aannemer moet een HSSE management system vaststellen om HSSE-plannen te kunnen ontwikkelen en implementeren (zowel in het ontwerp als de uitvoering van werkzaamheden) in lijn met de laatste industrie standaarden en 'best practices' en om zeker te stellen dat de risico's beheerst worden volgens ALARP (as low as reasonably practicable ofwel zo laag als redelijkerwijs mogelijk).
Daarnaast zal de tunnel aannemer ook een emergency repsonse plan (ERP) opstellen waarin maatregelen zijn uitgewerkt in het geval van een overschrijding van grenswaarden van zettingen van de zeewering.

In onderstaande paragrafen wordt ingegaan op de veiligheid van de zeewering en de aanwezige Tennet kabels en van de overige aspecten milieu, waterveiligheid, personeel, scheepvaart en visserij.

6.2 Zeeweringveiligheid

6.2.1 Algemeen

De vigerende projectplanning gaat ervanuit dat de tunnel wordt geboord in het gesloten seizoen waarin er geen werkzaamheden aan waterkeringen (of zeeweringen) mogen worden uitgevoerd. Bij de aanleg van de leidingkruising met de waterkering vinden er geen werkzaamheden plaats op of aan de zeewering, maar de werkzaamheden onder en in nabijheid van de zeewering hebben er indirect wel invloed op. Navolgend wordt aangetoond dat deze werkzaamheden geen invloed hebben op het functioneren van de kering, en dat risico's kunnen worden gemitigeerd. Hierbij wordt achtereenvolgens gekeken naar:

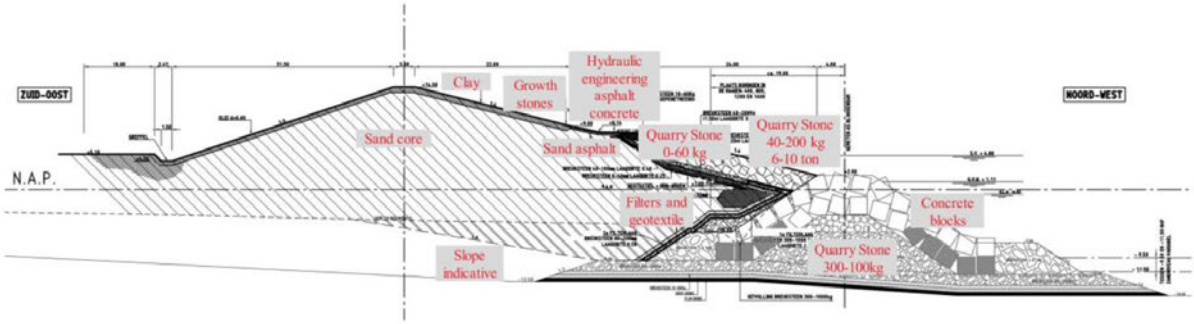
- de invloed van de bouw van de microtunnel;
- de invloed van de bouw van de schacht;
- de impact van het falen van de CO_2-transportleiding in de operationele fase.

De ontvangstput ligt zo ver van de waterkering dat de invloed ervan kan worden uitgesloten.

6.2.2 Beschrijving van de situatie

De voorgestelde tunnelbouw vindt direct onder de zeewering plaats, die zich uitstrekt van metrering 120 m tot 290 m langs de tunnel as.

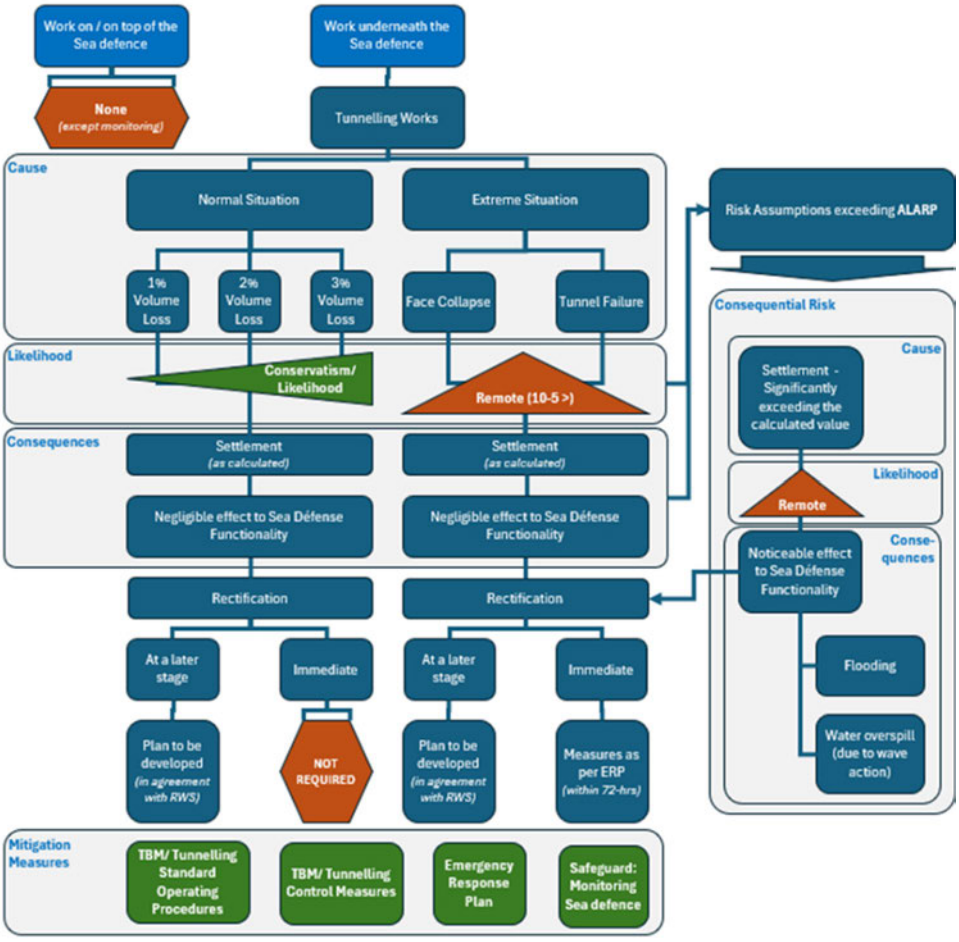
De zeewering is een zogenaamde aanleundijk, gebouwd tegen een bestaande blokkendam. De blokkendam vormt de teenconstructie van de huidige zeewering en is vanaf zeebodem (circa NAP-11,5m) opgebouwd uit verschillende graderingen breuksteen en afgedekt met grote betonblokken. Vanaf bovenzijde blokkendam (NAP+2 m) loopt het talud van de dijk onder 1:4 op tot een berm, en daarna tot de kruin op NAP +14,5 m. Het buitentalud is aan de onderzijde bekleed met breuksteen, op de berm met asfalt en aan de bovenzijde met klei. Het binnentalud is bekleed met klei. De kern van de zeewering is opgebouwd uit zand. Het maaiveld aan de binnenzijde ligt op NAP+5,1m.



Figuur 6-1: Opbouw van de zeewering ter plaatse van de kruising van de microtunnel (uit NL-ARM-040-ARU1-110381_02_IFD_2024-10-24_01, bijlage 7).

6.2.3 Impact microtunneling

Aramis heeft een risicoanalyse uitgevoerd om de potentiële risico's van de tunnelwerkzaamheden voor de zeewering te evalueren, zie bijlage 8. De risicoanalyse toont aan dat het risico acceptabel is, en dat de functionaliteit van de zeewering niet in gevaar komt, zelfs na de geringe waarschijnlijkheid van het ontwikkelen van een significante verzakking in de ondergrond. Het onderstaande stroomdiagram (Figuur 6-2) geeft een overzicht van de risico's voor de zeewering en de relatie tussen de directe en gevolgschade als gevolg van de tunnelwerkzaamheden.



Figuur 6-2: Stroomschema risico-analyse

Het lage risicoprofiel komt door de geringe waarschijnlijkheid van een extreem falen tijdens de tunnelwerkzaamheden en bijgevolg significante gevolgen van de tunnelwerkzaamheden op de zeewering, waarbij de frequentie van falen wordt geschat op minder dan 10^{-5}.

Voor de risico's die betrekking hebben op de operationele fase van de tunnel, wordt elke verzakking als gevolg van een tunnelstorting uitgesloten, aangezien de tunnel volledig met grout (of zand) zal worden opgevuld. Het risico op een explosie van de hoofdleiding in de microtunnel wordt geëlimineerd door de toepassing van de specificaties uit de NEN 3651. Het leidingtracé onder de kustverdediging een grotere wanddikte hebben (20% toename), en wordt hydrostatisch getest op een aanzienlijk hogere testdruk.

Naast de risicobeoordeling (bijlage 8) is een analyse uitgevoerd waarbij de zetting van de zeewering als gevolg van normale tunnelwerkzaamheden is voorspeld. Ook is daarbij gekeken naar een extreem geval van tunnelstorting hoewel een dergelijke faalscenario's als zeer onwaarschijnlijk wordt beschouwd, zie NL-ARM-040-ARU1-110381_02_IFD_2024-10-24_01 (bijlage 7). De voorspelde zettingen hebben impact op de hoogte van de kering, maar resulteren niet in een afname van de integriteit van de constructie als geheel. Eventuele schade is zeer lokaal en eenvoudig te herstellen.

Tabel 6-1: Prognose zettingen zeewering

	volumeverlies	Maximale zetting zeewering
Normale tunnelwerkzaamheden	1% volume verlies	19
	2% volume verlies	39
	3% volume verlies	58
Extreme gebeurtenis	Boorfront instabiliteit	245
	Bezwijken tunnel	360

Aramis zal de zettingen van de zeewering voor, tijdens en na de werkzaamheden monitoren. Het doel hiervan is om te identificeren, en ervoor te zorgen dat de zettingen binnen de verwachte waarden blijven, (1% volumeverlies). Als deze basiswaarde wordt overschreden zal dit de noodzakelijke maatregelen in gang zetten om de verzakkingswaarden binnen de basiswaarden te brengen door aanpassing van de procesparameters van het tunnelen.
Óngeacht de zetting stelt Aramis als onderdeel van het nog op te stellen werkplan een herstelplan op. In dit herstelplan worden de maatregelen die Aramis treft in kader van een extreme gebeurtenis (calamiteit) en de maatregelen die Aramis treft voor zettingen die bij de normale tunnelwerkzaamheden horen, opgenomen. Dit herstelplan stelt Aramis in nauw overleg met Rijkswaterstaat keringbeheer op.

6.2.4 Impact bouw startschacht

De startschacht ligt ongeveer 127 m buiten het beperkingengebied en is dus geen onderdeel van deze aanvraag. Vanuit zorgplicht is onderstaand aangetoond dat de bouw ervan geen invloed heeft op de zeewering.

De startschacht is een ronde bouwkuip met een diameter van ongeveer 19 m en een ontgravingsdiepte van ongeveer 23 m. De bouwkuipwanden zijn betonnen diepwanden. De waterdichte vloer van de bouwkuip bestaat uit beton. Trekankers zorgen dat de betonnen vloer niet door de grondwaterdruk omhoogkomt. De bouwfasering is weer gegeven in NL-ARM-040-ARU1-110332_01_IFD_20240904_01 (bijlage 9) en bestaat in hoofdlijnen uit:

1 het in een raster aanbrengen van verticale trekankers op de plot van de toekomstige schachtvloer;
2 het aanbrengen van een tijdelijk werkplatform op het maaiveld;

3 het installeren van 1,2 m dikke en 27 m lange diepwandpanelen vanaf het werkplatform. De diepwandpanelen vormen later samen de bouwkuipwanden van de ronde schacht;

4 het “in den natte” ontgraven van de grond tussen de diepwanden tot circa 23 m onder maaiveld;

5 het storten van de ongewapende onderwaterbetonvloer;

6 het leegpompen van de schacht;

7 storten van de constructieve betonvloer en het afzetblok voor de vijzels;

8 Na het aanbrengen van de leiding wordt de schacht gevuld met zand.

De werkzaamheden in stap 3 en 4 induceren grondvervormingen (zettingen) en hebben potentieel effect op de zeewering. Alle bovengenoemde werkzaamheden zijn trillingsvrij of trillingsarm.

Impact van de bouw van de startschacht op de zeewering zou kunnen optreden als gevolg van grondverplaatsingen (zetting) door het installeren van de diepwanden en door het ontgraven van de grond uit de schacht. Deze zetting is direct achter de schacht het grootst, en neemt met de afstand tot de schacht af. Op basis van een empirische analyse is geconcludeerd de zettingen al op 46 m achter de schacht verwaarloosbaar klein zijn. Zie rapport NL-ARM-040-ARU1-110381_02_IFD_2024-10-24_01 (bijlage 7).
Zetting kan bij bouwwerkzaamheden ook optreden door het verdichten van grondlagen als gevolg van trillingen. Omdat bij de bouw van de schacht met trillingsvrije uitvoeringstechnieken wordt gewerkt is dit hier niet aan de orde. De bouw van de startschacht heeft derhalve geen impact op de zeewering.

Tabel 6-2: Impact van de bouw van de startschacht op de zeewering

Locatie	Invloedszone bouw startschacht [m]	Afstand tot beperkingengebied [m]	Voldoet
1	46	127	Ja

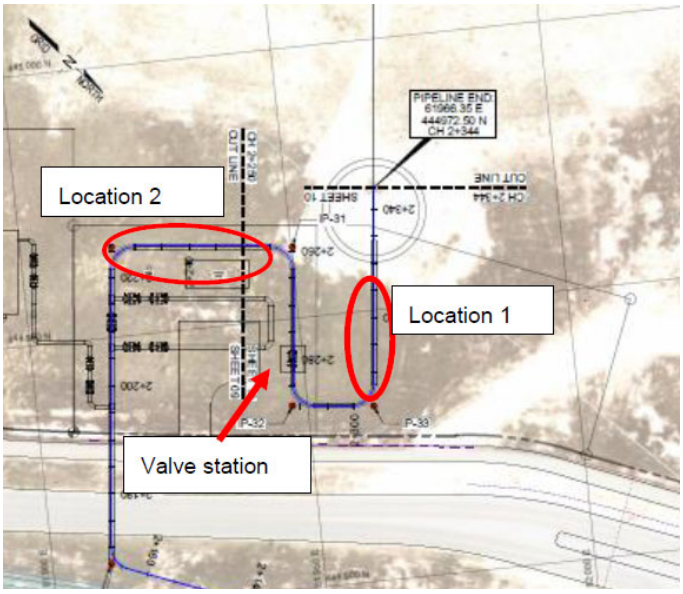
6.2.5 Impact van het falen van de onshore CO2-transportleiding

De onshore CO_2-transportleiding ten noorden van de Maasvlakteweg ligt buiten het beperkingengebied en is geen onderdeel van deze aanvraag. Vanuit zorgplicht is onderstaand aangetoond dat een eventuele calamiteit in de leiding geen invloed heeft op de zeewering.

De kleinste afstand van de onshore CO_2-transportleiding (het deel ten zuiden van de startschacht) tot het beperkingengebied bedraagt 142 m. Een analyse van veiligheidszones rondom de leiding op basis van NEN3651 is in deze situatie niet toereikend omdat NEN3651 (per uitgiftedatum van deze aanvraag) geen methoden beschrijft voor CO_2 in dichte fase. Dichte fase CO_2 is een sterk gecomprimeerd fluïdum dat eigenschappen vertoont van zowel een vloeistof als een gas. Vanwege de onzekerheden van de impact van een lek met dichte fase CO_2 is Deltares geconsulteerd voor de bepaling van de veiligheidszones. Zie rapport NL-ARM-040-PTC1-114043_02_IFD_20241014_02. Deltares neemt ook deel aan de actualisering van NEN 3651 om dichte fase CO_2 hierin op te nemen en beschikt over de state-of-the-art kennis op dit gebied. Nieuwe inzichten en formules die zullen worden bijgewerkt in NEN 3651 zijn geïmplementeerd in de genoemde rapportage.

De veiligheidszones zijn door Deltares voor 2 locaties beschouwd, zie Figuur 6-3: en Tabel 6-3. De afstand van de veiligheidszone is in de beschouwde maatgevende situaties kleiner dan de afstand tot het beperkingengebied van de zeewering. Hiermee wordt geconcludeerd dat het huidige FEED-ontwerp van

de schachtlocatie en de onshore pijpleiding voldoet ten aanzien de impact op de zeewering, en er geen aanvullende NEN 3651-vereisten van toepassing zijn op de onshore pijpleiding.



Figuur 6-3: Beschouwde locaties van leidingfalen (uit NL-ARM-040-PTC1-114043_02_IFD_20241014_02, bijlage 10).

Tabel 6-3: Impact van het falen van de onshore CO_2-transportleiding op de zeewering

Locatie	veiligheids zone (maatgevend) [m]	Afstand tot zeewering [m]	Voldoet
1	90	146	Ja
2	108	142	Ja

6.3 TenneT kabels

De beheerders van bestaande leidingen en kabels zijn of worden geïnformeerd en details met betrekking tot de ligging nabij bestaande kabels en leidingen (K&L) worden met hen afgestemd. Aan land wordt de Verordening Beheer Ondergrond Rotterdam gevolgd en wordt gewerkt met graafmeldingen (KLIC-meldingen). De eigenschappen, de aanleg, de ligging en het onderhoud van de pijpleiding voldoen aan de in de Wet informatie-uitwisseling bovengrondse en ondergrondse netten en netwerken (Wibon).

6.4 Overige aspecten

De mogelijke effecten van de overige aspecten milieu, waterveiligheid, personeel, scheepvaart en visserij zijn, net als de mogelijke effecten op de zeewering, in een risico-analyse beoordeeld. Voor zover relevant heeft Aramis de gevonden risico's opgenomen in het 'hazard register'. Aramis geeft de aannemers die de aanleg van de startschacht, tunnel en leiding zullen uitvoeren, opdracht om dit hazard register te implementeren in het HSSEplan.
Mitigerende maatregelen om potentiële negatieve effecten op gezondheid, veiligheid en milieu te voorkomen of te verminderen zijn vastgelegd in het FEED risk assessment report (ref NL-ARM-040-ARU1-110311) en het bijbehorende hazard register (kenmerk NL-ARM-040-ARU1-110377).

Uit het Hazard register is gebleken dat de te selecteren aannemer minimaal aan de volgende onderwerpen aandacht moet besteden om de risico's verder te reduceren van toelaatbaar naar acceptabel:

- Werkplaatsgevaren waaronder toegang en beveiliging, aanwezigheid van zware industriële installaties, graafwerkzaamheden, hijswerkzaamheden, werken op hoogte etc;
- De plaatselijke omstandigheden zoals het weer, verlichting, geluid, mogelijk verontreinigende stoffen;
- Blootstelling aan hoogspanning;
- Het werken in besloten ruimtes, waaronder het tunnelen, mogelijke overgang van de tunnelmethode, aanwezigheid van installaties en transportmiddelen in de tunnel;
- Het mogelijk falen van apparatuur zoals slurry leidingen, ventilatie, afsluitingen;
- Werken in ploegendienst;
- Mogelijk werken onder druk/werken met gecomprimeerde lucht;
- Werken in een mariene omgeving inclusief interactie met haven en scheepsvaartroute, risico van niet gesprongen explosieven;
- Duikwerkzaamheden;
- Interactie met het installeren van de CO_2-transportleiding.

7 Uitbedrijfname

Uitgangspunt is dat de microtunnel met daarin het leidingdeel blijft liggen. Momenteel is er geen techniek beschikbaar om specifiek het leidingdeel uit de tunnel te verwijderen.
Ook verwijdering van de microtunnel (inclusief stuk zeeleiding) is praktisch onuitvoerbaar. Er is geen techniek beschikbaar zonder grote maatschappelijke impact en risico's voor de waterveiligheid. Om de waterveiligheid enigszins te waarborgen zou mogelijkerwijs de aanleg van een tijdelijke tweede dijk achter de bestaande zeewering noodzakelijk zijn. De duur van deze verwijderingsoperatie wordt geschat op minimaal 2-jaar waarin de Maasgeul (gedeeltelijk) zou moeten worden afgesloten wat een ontwrichtend effect zou hebben op de Rotterdamse haven. Het laten liggen van de microtunnel met het leidingdeel is praktisch de beste uitvoerbare oplossing en heeft aanzienlijk minder milieueffecten dan verwijdering.
Omdat de tunnel initieel al voldoende diep is aangelegd, leidt deze ook na uit gebruik name niet tot hinder of gevaar.
Na de aanlegfase is de schacht gevuld met grond en ter plaatse van de schacht is de grond met 1,5m opgehoogd. Dit zal dan begroeid zijn met beplanting die al voorkomt in de directe omgeving. Deze situatie blijft ook zo na uitbedrijfname van de CO_2-transportleiding.

8 Kruisverwijzing

Tabel 8-1 bevat een kruisverwijzing tussen de indieningsvereisten conform paragraaf 7.2.1 en artikelen 7.177 en 7.159 van de Omgevingsregeling en de vindplaats(en) hiervan in deze aanvraag.
In de tabel is tevens vermeld welke informatie later komt en wanneer.

Tabel 8-1: Kruisverwijzing indieningsvereisten

Art Or	Indieningsvereisten	Vindplaats	Na te leveren gegevens
7.2	Deze afdeling is van toepassing op het verstrekken van gegevens en bescheiden bij een aanvraag om een omgevingsvergunning als bedoeld in artikel 5.1 van de wet en, voor zover het gaat om de artikelen 7.3 en 7.4, ook op het verstrekken van gegevens en bescheiden bij een aanvraag om een omgevingsvergunning als bedoeld in artikel 5.3 of 5.4 van de wet.	nvt	-
7.3	Bij de aanvraag worden de volgende gegevens en bescheiden verstrekt: **a. een beschrijving van de activiteit waarvoor de omgevingsvergunning wordt aangevraagd;** b. het telefoonnummer van de aanvrager; c. het adres, de kadastrale aanduiding of coördinaten van de locatie waarop de activiteit wordt verricht; d. een aanduiding van de begrenzing van de locatie waarop de activiteit wordt verricht; e. als de aanvraag wordt ingediend door een gemachtigde: naam, adres, telefoonnummer en woonplaats van de gemachtigde; f. als de aanvraag elektronisch wordt ingediend: het e-mailadres van de aanvrager of de gemachtigde; g. als wordt gevraagd een voorschrift aan de omgevingsvergunning te verbinden over regels als bedoeld in paragraaf 4.1.1 van de wet: een beschrijving van het onderwerp van dat voorschrift; en h. als wordt gevraagd om toestemming om een gelijkwaardige maatregel te treffen: gegevens waaruit blijkt dat met de gelijkwaardige maatregel ten minste hetzelfde resultaat wordt bereikt als met de voorgeschreven maatregel is beoogd.	hfdst 1	

Art Or	Indieningsvereisten	Vindplaats	Na te leveren gegevens
7.4	1. Bij de aanvraag wordt aangegeven of burgers, bedrijven, maatschappelijke organisaties en bestuursorganen bij de voorbereiding van de aanvraag zijn betrokken. 2. Als burgers, bedrijven, maatschappelijke organisaties en bestuursorganen bij de voorbereiding van de aanvraag zijn betrokken, verstrekt de aanvrager bij de aanvraag gegevens over hoe zij zijn betrokken en wat de resultaten daarvan zijn.	par 1.8	
	Specifieke vereisten		
7.177	(beperkingengebiedactiviteit: bouwwerken, werken en objecten in de Noordzee) Bij een aanvraag om een omgevingsvergunning voor het bouwen, in stand houden of slopen van bouwwerken, het aanleggen, plaatsen, in stand houden, veranderen of verwijderen van werken die geen bouwwerken zijn of het plaatsen, in stand houden of verwijderen van andere objecten in de Noordzee, bedoeld in artikel 7.17, eerste of tweede lid, van het Besluit activiteiten leefomgeving, worden de gegevens en bescheiden, bedoeld in artikel 7.159, verstrekt:	nvt	
	Als de aanvraag betrekking heeft op een locatie buiten het provinciaal en gemeentelijk ingedeelde gebied, wordt bij de aanvraag ook een rapport verstrekt waarin de archeologische waarde van de locatie in voldoende mate is vastgesteld	Bijlage 5	
7.159	(module: waterstaatswerk in beheer bij het Rijk)		
	Bij een aanvraag om een omgevingsvergunning voor een beperkingengebiedactiviteit met betrekking tot een waterstaatswerk in beheer bij het Rijk als bedoeld in hoofdstuk 6 van het Besluit activiteiten leefomgeving worden de volgende gegevens en bescheiden verstrekt:	nvt	
	een werkplan waarin wordt beschreven hoe de activiteit wordt verricht;	Hdst 2 en 4	Detail werkplan komt beschikbaar nadat aannemer is gecontracteerd
	een toelichtende tekening en de coördinaten van de activiteit met daarbij het ontwerp en de afmetingen van het werk of het tracé van de kabel of de leiding;	Bijlagen 2 t/m 4 en 6 t/m 10 en par 1.5	

Art Or	Indieningsvereisten	Vindplaats	Na te leveren gegevens
	de verwachte datum en het verwachte tijdstip van het begin van de activiteit en de verwachte duur ervan;	Par 1.7	
	een beschrijving van de gevolgen van de activiteit voor de waterkwaliteit, waterkwantiteit, waterveiligheid en maatschappelijke functies van het waterstaatswerk;	Hfst 3	
	contactgegevens van de partijen die direct bij het verrichten van de activiteit zijn betrokken; en	niet beschikbaar	contactgegevens komen beschikbaar nadat aannemer is gecontracteerd
	als een waterstaatswerk wordt gekruist door een boring: een boorplan met een beschrijving van de horizontaal gestuurde boring overeenkomstig Richtlijn Boortechnieken en open ontgraving voor kabels en leidingen, uitgegeven door Rijkswaterstaat	Nvt (geen horizontaal gestuurde boring)	

Bijlage

1. Participatieplan

Royal HaskoningDHV
Enhancing Society Together

PARTICIPATIEPLAN

ARAMIS-INITIATIEF

Fase voorkeursalternatief t/m planuitwerking

Herziene versie
Maart 2024
Documentnummer
NL-ARM-PFE-B10-ENV-GEA-0299

INHOUDSOPGAVE

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ALGEMEEN

Voor u ligt het geactualiseerde participatieplan van het Aramis-initiatief (hierna: Aramis). Het plan is opgesteld door Aramis in afstemming met CO_2next en het ministerie van Economische Zaken en Klimaat (EZK). In het participatieplan leest u hoe u en andere belanghebbenden worden geïnformeerd over en betrokken bij het Aramis-project.

Bij elke fase van het project actualiseren initiatiefnemers TotalEnergies, Shell, Energie Beheer Nederland (EBN) en Gasunie het participatieplan. Dat doen zij op basis van voortschrijdend inzicht, ontwikkelingen in het project, gesprekken met stakeholders, reacties op het participatieplan en een evaluatie van de voorgaande periode.

- De eerste versie van het participatieplan is samen met de kennisgeving *Voornemen en Voorstel Participatie voor het project Aramis* (kennisgeving van het V&P)[1] gepubliceerd in januari 2022.
- Naar aanleiding van gesprekken met stakeholders en reacties op de kennisgeving van het V&P is in juni 2022 een tweede versie van het plan gepubliceerd, gelijktijdig met de publicatie van de conceptversie van de Notitie Reikwijdte en Detailniveau (concept-NRD).
- In november 2022 werd de derde versie uitgebracht, die in het teken stond van de definitieve Notitie Reikwijdte en Detailniveau (NRD).
- De vierde versie van het participatieplan is ten tijde van de Integrale Effectenanalyse (IEA) opgesteld (onder behoud van het concept-milieueffectrapport (MER) fase 1), die de basis vormt voor de keuze van het voorkeursalternatief (VKA).
- Deze vijfde versie van het participatieplan omvat het tijdvak van begin 2024 tot de aanbesteding. In deze periode wordt het VKA nader uitgewerkt in een concreet ontwerp als onderdeel van de verschillende contracten die worden aanbesteed.

Het MER wordt medio 2024 samen met de ontwerpbesluiten ter inzage gelegd. Dan is er mogelijkheid tot reageren. In het vierde kwartaal van 2024 vindt de publicatie van de MER in de *Staatscourant* plaats. De definitieve besluiten op de vergunningaanvragen liggen dan ter inzage en daar kan eventueel bezwaar op worden ingediend.

In de voorliggende periode zal het VKA worden uitgewerkt, de zogeheten planuitwerkingsfase. Het ontwerp zal met stakeholders worden besproken. Indien de input van stakeholders leidt tot aanpassingen van het participatieplan, dan zal deze tussentijds worden geüpdatet.

LEESWIJZER

- Hoofdstuk 1 introduceert het Aramis-project en de rol van EZK in de te volgen procedure.
- Hoofdstuk 2 licht de doelen, uitgangspunten en het kader van het participatieplan toe.
- Hoofdstuk 3 beschrijft hoe de participatie aan het MER en de IEA tot en met de VKA er concreet uitziet.
- Hoofdstuk 4 geeft een overzicht van alle geplande participatiemomenten.

Voor aanvullende informatie ziet u een verwijzing naar websites en documenten.

[1] https://www.rvo.nl/sites/default/files/2021/12/Notitie-Voornemen-en-Voorstel-Participatie-CCS-Aramis.pdf

1. INLEIDING

1.1 OVER ARAMIS

Het klimaat verandert snel door de toename van CO_2- en andere broeikasgassen in de atmosfeer. In het Klimaatakkoord van Parijs zijn ambitieuze doelen vastgelegd om de CO_2-uitstoot te verlagen. Hierin is afgesproken de opwarming van de atmosfeer te beperken tot maximaal 2°C, maar bij voorkeur onder 1,5°C te houden. Het vormt een grote uitdaging om de uitstoot zodanig te verlagen dat de klimaatdoelstellingen voor 2050 worden behaald.

Verduurzaming van de industrie is een van de maatregelen om CO_2-uitstoot te verminderen. De komende decennia wordt het aandeel van fossiele brand- en grondstoffen in productieprocessen afgebouwd. Voor deze transitie is tijd nodig: het is niet mogelijk in één keer volledig fossielvrij te worden en alle industriële processen om te zetten naar groene waterstof en/of groene stroom.

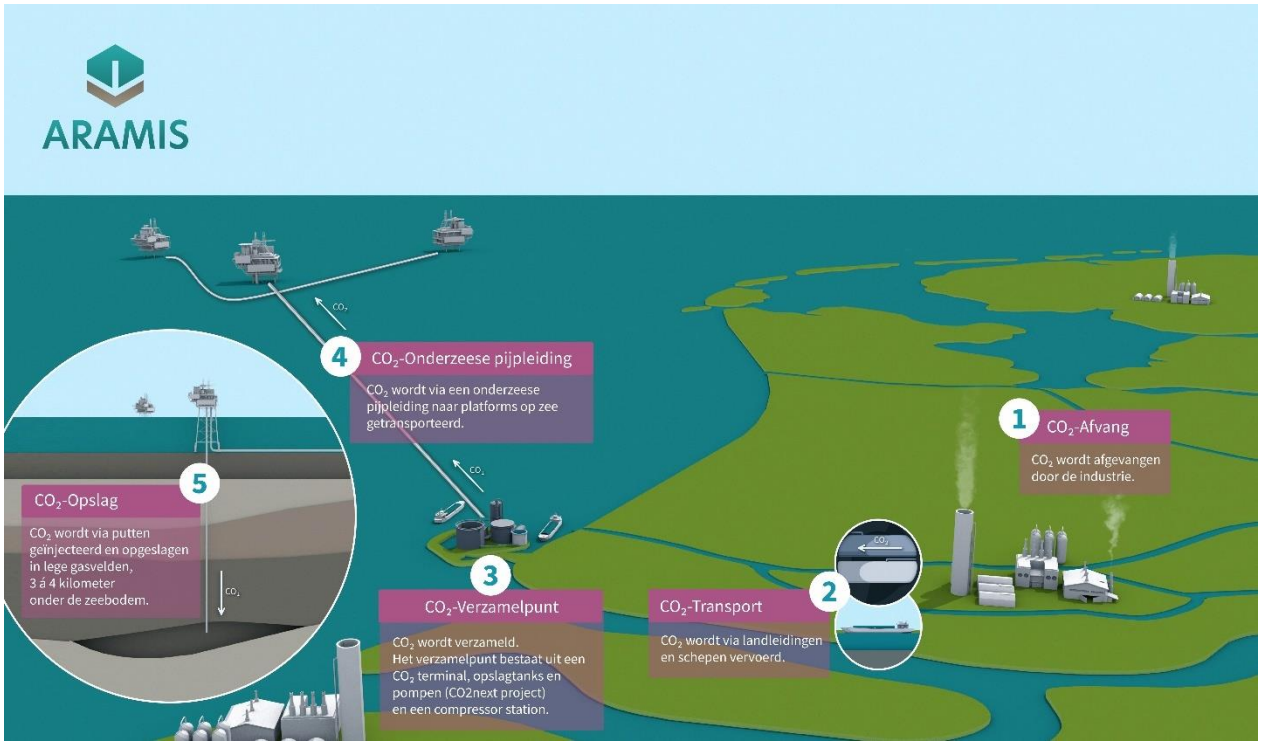
Totdat het gebruik van fossiele brandstoffen in industriële processen tot nul is gereduceerd, kan CO_2-uitstoot fors worden verminderd door afvang en ondergrondse opslag van vrijgekomen CO_2. Deze techniek wordt Carbon Capture and Storage (CCS) genoemd en vermindert de hoeveelheid broeikasgassen die in de atmosfeer terechtkomt.

Rapportages van het Intergovernmental Panel on Climate Change (IPCC) en het Internationale Energie Agentschap (IEA)[2] laten zien dat – zolang er onvoldoende alternatieven zijn – permanente CO_2-opslag noodzakelijk is voor moeilijk te verduurzamen industrie. In de Klimaatnota 2022 en de Klimaat- en Energieverkenning (KEV) 2023 staat aangegeven dat het grootste gedeelte van de industriële CO_2-reductie tot 2030 uit CCS zal komen. De overheid ziet het afvangen en opslaan van CO_2 als een belangrijke (overgangs)technologie en stimuleert daarom CO_2-opslag onder de Noordzee.

De opslag van de afgevangen CO_2 is voorzien in lege gasvelden diep onder de zeebodem. Om de bij de industrie afgevangen CO_2 naar deze opslaglocaties te brengen, wordt een nieuwe, open-access transportinfrastructuur ontwikkeld. ‘Open’ betekent dat andere partijen de mogelijkheid hebben om op de CCS-keten aan te sluiten, zowel aan de voorkant (de afvang) als aan de achterkant (de opslag).

Bij een open CO_2-transportinfrastructuur zijn veel verschillende partijen betrokken, elk met een eigen rol en elk met een eerder of later moment waarop zij aansluiten. Samen vormen deze partijen de integrale CCS-keten: van de afvang van CO_2 tot permanente opslag in lege gasvelden diep onder de Noordzee. De keten bestaat veelal uit zelfstandige onderdelen, die voor een goed functionerend geheel, nauw op elkaar moeten zijn afgestemd (zie afbeelding 1).

[2] IPPC rapportage 2022, Mitigation of Climate change



Afbeelding 1. Overzicht componenten van de CCS-keten, waar het Aramis-initiatief onderdeel van uitmaakt.

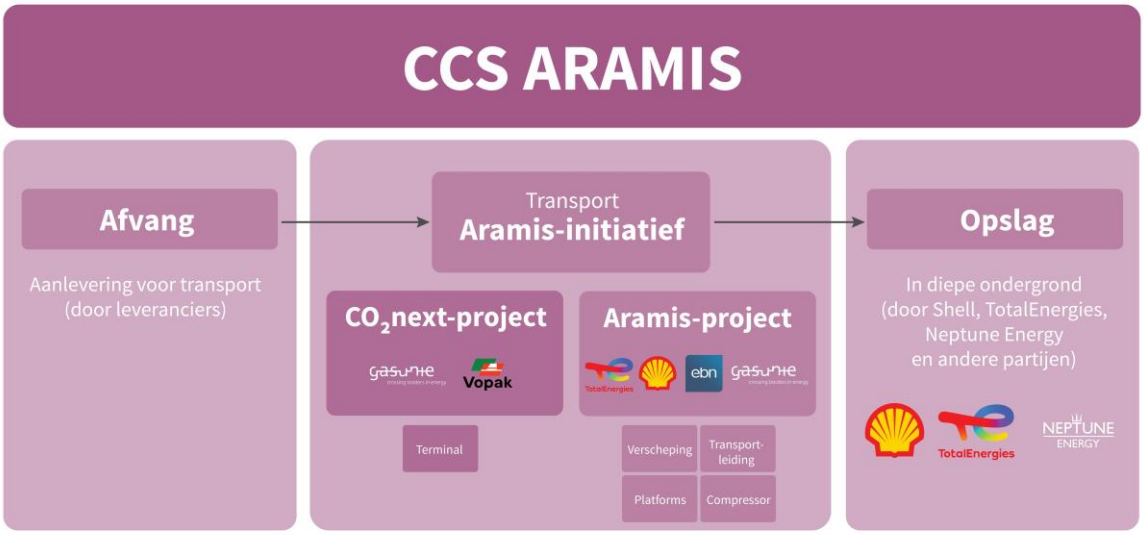
1. CO_2-afvang bij industrie en geschikt maken voor transport;
2. CO_2-transport naar de Maasvlakte via Porthos-landleiding, binnenvaart en zeevaart;
3. CO_2-verzamelpunt op de Maasvlakte met terminal en compressorlocatie. De terminal omvat steigers, tanks voor tijdelijke opslag van per schip aangevoerde CO_2, en hogedrukpompen voor levering aan de zeeleiding (CO_2next-project). De compressorlocatie ontvangt CO_2 via de landleiding en brengt dit op druk voor het transport per zeeleiding;
4. CO_2-transport door de centrale CO_2-zeeleiding naar platforms op de Noordzee;
5. Platform met leidingen vanaf de centrale CO_2-zeeleiding en met putten naar lege gasvelden diep onder de Noordzee.

Aramis heeft betrekking op het transport van CO_2 (onderdeel 2) naar het CO_2-verzamelpunt (onderdeel 3) en het transport via een zeeleiding naar de platforms op zee (onderdeel 4). In de CCS-keten van afvang, transport en opslag richt Aramis zich op het transportdeel: de CO_2-transportinfrastructuur. De CO_2-afvang (onderdeel 1) en de CO_2-opslag (onderdeel 5) vallen weliswaar buiten Aramis, maar vormen een samenhangend geheel met Aramis. Zodoende worden deze onderdelen in het verlengde van Aramis beschreven.

De transportinfrastructuur biedt andere partijen de mogelijkheid om op de CCS-keten aan te sluiten, zowel aan de voorkant (de afvang) als aan de achterkant (de opslag). Aramis voorziet daarmee in een cruciaal onderdeel van de CCS-keten. Het is niet mogelijk om op voorhand aan te geven welke partijen zich aansluiten en wanneer. Dat is inherent aan de aard van een open infrastructuur, die is gericht op toekomstige uitbreiding en aanpassing.

1.2 PROJECTORGANISATIE EN INITIATIEFNEMERS

Afbeelding 2 geeft weer hoe de verschillende onderdelen van Aramis zich verhouden tot elkaar en tot de Aramis-CCS-keten.



Afbeelding 2. Aramis binnen de Aramis-CCS-keten.

TotalEnergies, Shell, Energie Beheer Nederland (EBN) en Gasunie zijn de initiatiefnemers van de ontwikkeling van de Aramis- CO_2-transportinfrastructuur. Zij zijn zelf verantwoordelijk voor de compressie van CO_2 die afkomstig is van de landleiding, de centrale CO_2-zeeleiding en de platforms.

Door verschillende bedrijven zal CO_2 worden afgevangen. Vervolgens verzorgen verschillende leveranciers de aanlevering van CO_2 via leiding (gas) of schip (vloeibaar) naar het CO_2-verzamelpunt. Op het verzamelpunt worden de terminalfaciliteiten verzorgd door CO_2next. In CO_2next werken Gasunie en Koninklijke Vopak samen aan de bouw van een nieuwe CO_2-terminal op de Maasvlakte.

De aanleg van de centrale CO_2-zeeleiding is onderdeel van het Aramis-project, evenals de bouw van het compressorstation op het verzamelpunt. Voor het overige (steigers, tanks voor tijdelijke opslag van per schip aangevoerde CO_2, en hogedrukpompen voor levering aan de zeeleiding) valt het verzamelpunt onder CO_2next.

De opslagpartijen (onder meer Neptune Energy, Shell en TotalEnergies) zijn verantwoordelijk voor de opslag van CO_2, inclusief het transport vanaf hun platforms naar de ondergrondse reservoirs.

1.3 ROL VAN HET MINISTERIE EN KORTE TOELICHTING OP DE PROCEDURE

Het ministerie van Economische Zaken en Klimaat (EZK) en Aramis werken nauw samen aan dit project en hebben hierin elk een eigen taak en rol.

Rollen van EZK

Voordat Aramis en CO_2next kunnen worden gerealiseerd, is er een ruimtelijk besluit nodig en moeten de vereiste vergunningen zijn verleend. EZK coördineert de besluitvorming van energieprojecten met een nationaal belang. Dit heet nu nog de Rijkscoördinatieregeling (RCR). Onder de nieuwe Omgevingswet die op 1 januari 2024 ingaat heet dit projectprocedure. Aangezien de vergunningaanvragen na 1 januari 2024 worden ingediend, hebben we het hier verder over de projectprocedure.

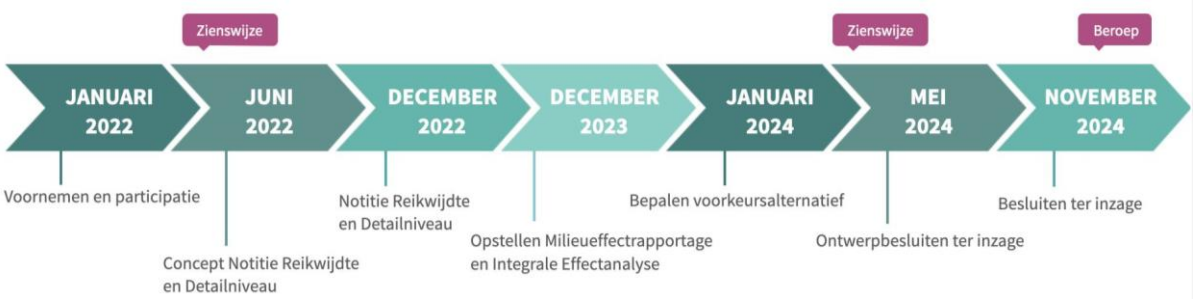
EZK coördineert de projectprocedure, waarbij de verschillende benodigde besluiten (vergunningen en eventueel ontheffingen) gelijktijdig worden genomen in afstemming met de overheden. Het gaat dan om zowel het ruimtelijk besluit als de uitvoeringsbesluiten. De coördinatie betekent ook dat alle stukken tegelijk ter inzage worden gelegd. Tegen de definitieve besluiten kan beroep worden aangetekend. Er is een beperkt aantal momenten waarin om een reactie wordt gevraagd, of men een zienswijze of beroep kan indienen.

Het ruimtelijk besluit wordt genomen door de minister voor Klimaat en Energie in overeenstemming met het ministerie van Binnenlandse Zaken en Koninkrijksrelaties. Het ruimtelijke besluit (in de nieuwe Omgevingswet: projectbesluit) wijzigt de huidige bestemmingen. Ook zijn er omgevingsvergunningen nodig, waaronder bouwvergunningen voor installaties op het verzamelpunt en voor de aanpassingen aan de platforms.

Andere vergunningen vallen onder de verantwoordelijkheid van andere bevoegde gezagen, bijvoorbeeld gemeente Rotterdam, Rijkswaterstaat en het ministerie van Landbouw, Natuur en Voedselkwaliteit (LNV). Vergunningen voor de afvang en opslag van CO_2 vallen buiten Aramis en worden aangevraagd door de opslagpartijen.

Nieuwe Omgevingswet en projectprocedure

Op 1 januari 2024 treedt de nieuwe Omgevingswet in werking. De formele besluiten voor Aramis worden niet voor deze datum genomen. Het ruimtelijk besluit van het Rijk heet onder de Omgevingswet niet meer rijksinpassingsplan (zoals in de Wet ruimtelijke ordening), maar projectbesluit. Aramis doorloopt de projectprocedure zoals weergegeven in afbeelding 3.



Afbeelding 3. Overzicht procedurestappen en tijdlijn.

Voornemen en voorstel participatie
Met de publicatie van de kennisgeving *Voornemen en Voorstel Participatie voor het project Aramis* (kennisgeving van het V&P) in de *Staatscourant* op 6 januari 2022 ging de projectprocedure officieel van start. EZK ontving zes reacties naar aanleiding van de kennisgeving. Op 19 en 24 januari 2022 heeft Aramis werksessies georganiseerd voor stakeholders van de Maasvlakte en de Noordzee. Bijlage 1 beschrijft de reacties en op welke manier die zijn gebruikt voor het actualiseren van dit participatieplan.

Concept-NRD
Bijlage 2 bevat het verslag van de stakeholdersessie op 21 juni 2022 waar de inhoud van de concept-NRD (Notitie Reikwijdte en Detailniveau) is besproken. In reactie op dit concept zijn acht zienswijzen ingediend. Op basis van deze zienswijzen is bekeken welke aanvullingen er nodig waren in de definitieve NRD. De definitieve NRD is in december 2022 vastgesteld.
Zowel de beantwoording van de vragen als de definitieve NRD is terug te vinden op de website van de RVO (https://www.rvo.nl/sites/default/files/2022-11/Vaststelling-NRD-en-Nota-van-Antwoord-concept-NRD-Aramis.pdf).

De inspraakprocedure heeft geresulteerd in twee aanpassingen aan de concept-NRD:

1. Als gevolg van de zienswijze van Neptune Energy worden de opslagfaciliteiten en bijbehorende infrastructuur van Neptune Energy als gelijkwaardig meegenomen in het MER, conform de opslagfaciliteiten voor TotalEnergies en Shell;
2. Het tracé van de zeeleiding is verder geoptimaliseerd, wat heeft geleid tot drie alternatieven en een variant, die alle in het MER worden getoetst.

IEA en MER
De volgende stap in het proces heeft plaats gevonden en behelste de voorbereidingen voor één integraal MER (fase 1 en fase 2 in één MER): een inventarisatie van de milieueffecten aan de hand van bureaustudies, onderzoeken en surveys. Op basis van de eerste resultaten van de milieuonderzoeken, evenals de aspecten kosten, omgeving, techniek en toekomstvastheid, stelt Aramis een Integrale Effectenanalyse (IEA) op. Deze analyse van de effecten van de verschillende routealternatieven en -varianten biedt tevens een uitgebreide analyse van zaken als de ruimtelijke inpassing. De resultaten van alle milieuonderzoeken zijn samengevoegd in het MER. Het MER onderbouwt zowel de vergunningaanvragen als het projectbesluit en wordt in 2024 bij de ontwerpbesluiten ter inzage gelegd.

In overeenstemming met de minister van Binnenlandse Zaken en Koninkrijksrelaties kiest de minister voor Klimaat en Energie op basis van de IEA het voorkeursalternatief (VKA). Over het VKA vindt afstemming plaats met andere overheden en belangenorganisaties. Het VKA wordt gepubliceerd op de website van de RVO: *Bureau Energieprojecten*. Het VKA vormt de grondslag voor het ruimtelijk besluit (projectbesluit) en de vergunningen. Naar verwachting worden in het derde kwartaal van 2024 alle besluiten in ontwerp ter inzage gelegd, waarop ieder die dat wenst een zienswijze kan indienen. De zienswijzen worden betrokken bij het opstellen van de definitieve besluiten, waartegen beroep openstaat.

2 DOELEN EN KADER VAN PARTICIPATIE

2.1 DOELEN VAN PARTICIPATIE

Participatie gaat in brede zin over het betrekken van belanghebbenden en belangstellenden bij een project (zie de uitleg van de participatieladder in paragraaf 2.3). Dit participatieplan loopt vooruit op de nieuwe Omgevingswet door naast de wettelijk geregelde inspraak op het projectbesluit (formele procedure) een bredere betrokkenheid te organiseren. Aramis betrekt ieder die dat graag wil bij het project en handelt daarmee nu al in de geest van de aankomende wet. Hiermee hebben wij de volgende doelen voor ogen:

1. We willen burgers, bedrijven en maatschappelijke organisaties op een passende wijze bereiken;
2. We willen hun vragen, kansen en zorgen kennen en begrijpen;
3. We willen bij de ontwikkeling van het project rekening houden met ieders belangen;
4. We willen heldere keuzes maken en daarbij duidelijk laten zien hoe we omgaan met belangen, aandachtspunten, kansen en zorgen vanuit de omgeving.

Bij het behalen van deze doelen zijn we altijd bereid tot een constructieve dialoog. Onze projectorganisatie gaat uiteraard zorgvuldig om met persoonsgegevens, conform de AVG.

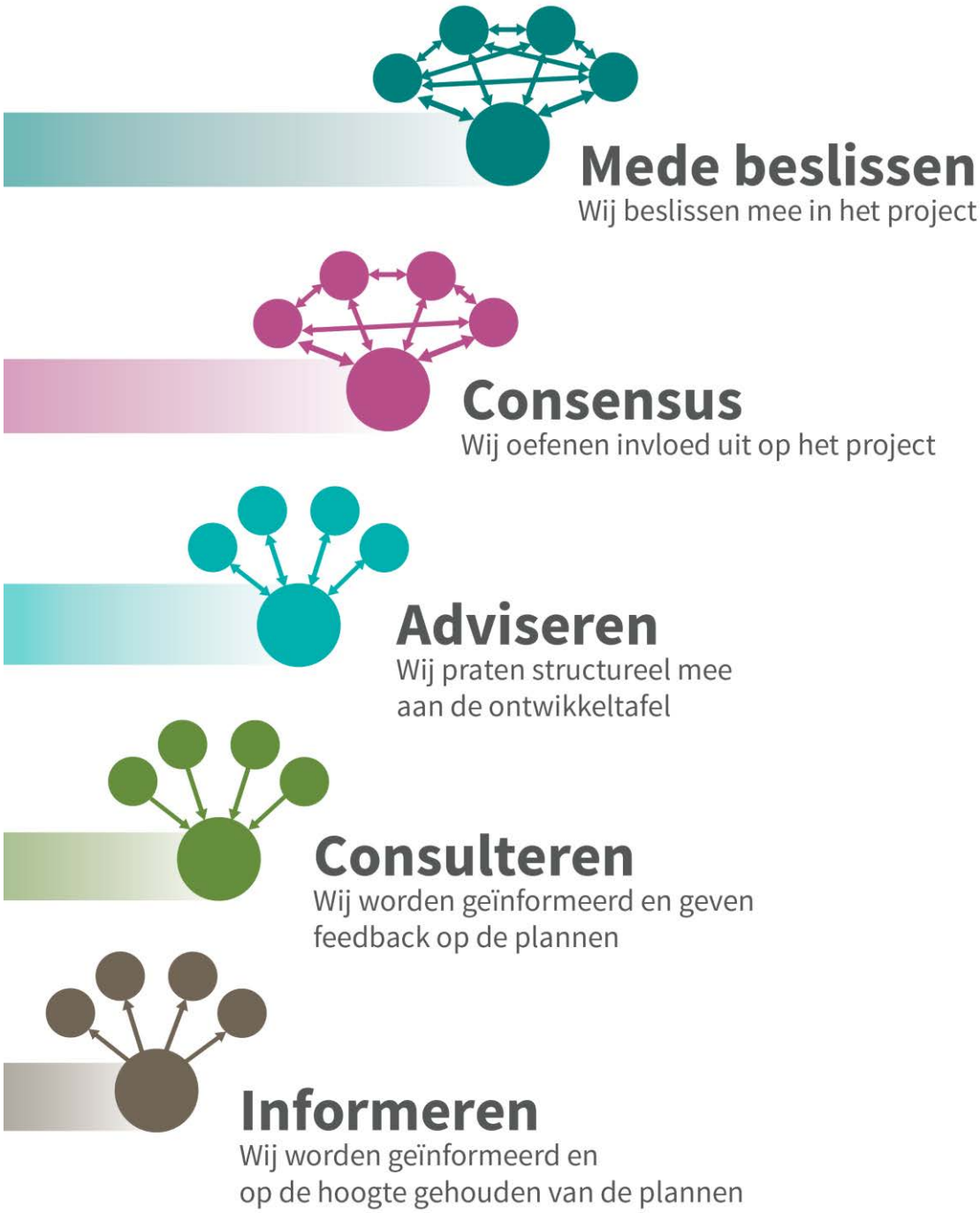
2.2 UITGANGSPUNTEN VAN PARTICIPATIE

We vinden het belangrijk dat participatie met betrekking tot Aramis begrijpelijk, betrouwbaar en toegankelijk is. Om te zorgen dat onze participatieaanpak zo goed mogelijk aansluit op de informatiebehoefte en wensen van belanghebbenden en belangstellenden, hanteren we de volgende uitgangspunten:

- We communiceren duidelijk, begrijpelijk en op maat;
- We bieden verschillende communicatiemiddelen aan, zodat iedereen de mogelijkheid heeft om onze informatie tot zich te nemen en indien gewenst met ons in dialoog te gaan;
- We communiceren tijdig en proactief;
- We kiezen voor een toegankelijke vorm die interactie en deelname aan inspraak stimuleert;
- We zijn goed bereikbaar en we reageren snel op vragen, klachten en verzoeken;
- We koppelen inhoud, toon en vorm aan elkaar, zodat we iedereen zo passend mogelijk bereiken.

2.3 KADER VAN PARTICIPATIE: HIER GAAT HET WEL/NIET OVER

Voor geslaagde participatie moet het duidelijk zijn waar belanghebbenden en belangstellenden wel en niet over kunnen meepraten en waar zij wel en geen invloed op hebben. De volgende drie vragen spelen hierbij een belangrijke rol: *waarom* we dit project willen doen, *waar* we dit project willen doen en *hoe*. Dit participatieplan maakt onderscheid tussen deze vragen en geeft per vraag de mate van participatie aan. Participatie kent namelijk verschillende gradaties, zoals hierna weergegeven in de participatieladder. Hoe hoger op de ladder, hoe meer invloed. Toch is ook op de onderste trede (informeren) sprake van participatie. Participatie is dus een heel breed concept.



Afbeelding 4. Participatieladder.

Waarom we dit willen doen?

De vraag waarom we Aramis willen uitvoeren is een vraag over nut en noodzaak van het initiatief. Aramis sluit aan op het regeringsbeleid, zoals geformuleerd in de brief van het kabinet aan de Tweede Kamer van 10 december 2021. In deze brief staat dat het afvangen, transporteren en opslaan van CO_2 een belangrijke (overgangs)technologie vormt voor de verduurzaming van Nederland en essentieel is om de CO_2-reductiedoelstelling voor 2030 te halen[3]. Ook in het Klimaatakkoord wordt verwezen naar CCS als een van de oplossingen om deze reductiedoelstelling te halen. Zie de Notitie Reikwijdte en Detailniveau (NRD) voor meer informatie over het Europese en Nederlandse klimaatbeleid en de rol van CCS hierin.

PARTICIPATIENIVEAU: INFORMEREN

Waar we dit willen doen?

De vraag waar we Aramis willen uitvoeren heeft betrekking op alternatieven en varianten van onder andere het tracé. De procedure voor de ruimtelijke inpassing, evenals de voorbereiding van het voorkeursalternatief, krijgt vorm in nauwe cocreatie met bevoegde instanties en betrokkenen bij andere activiteiten en ontwikkelingen in de buurt van Aramis. Met hen wordt ook gesproken over de gevolgen van de aanleg van onderdelen van het initiatief. Dit participatieplan beschrijft de verschillende manieren die belanghebbenden en belangstellenden hebben om hun suggesties kenbaar te maken. Ieder heeft de mogelijkheid om alternatieven aan te dragen, waarna deze worden afgewogen en mogelijk meegenomen. De uiteindelijke besluitvorming over het voorkeursalternatief is een taak van de ministers van EZK en BZK.

PARTICIPATIENIVEAUS: CONSULTEREN EN ADVISEREN

Hoe we dit willen doen?

De vraag hoe we Aramis willen uitvoeren is met name relevant in de dialoog met belanghebbenden en betrokkenen in de buurt van het project. Participatie draait hier om de gevolgen voor enerzijds de directe leef- en werkomgeving van mensen, en anderzijds de bedrijfsvoering van ondernemingen op de Maasvlakte en de Noordzee. Het gaat dus vooral om de impact van Aramis tijdens de uitvoering en ingebruikname. Naarmate het project zich verder ontwikkelt, concreter wordt en de uitvoering nadert, neemt de betrokkenheid van stakeholders in de directe omgeving toe. Gesprekken verplaatsen we dan naar lokaal niveau. Onderwerpen die hierbij aan bod komen zijn bijvoorbeeld de planning (start en duur) en uitvoering (tijdelijke overlast van bouwactiviteiten en veiligheid).

PARTICIPATIENIVEAU: CONSENSUS

[3] https://open.overheid.nl/repository/ronl-8fded76b-4d2c-4e79-817d-06bb14d9bb3a/1/pdf/kamerbrief-over-stand-van-zaken-ccs.pdf

3 PARTICIPATIEAANPAK

We betrekken graag personen en partijen bij Aramis wanneer het project hun belangen beïnvloedt, wanneer zij zich inhoudelijk betrokken voelen en/of wanneer zij belangrijk zijn voor de realisatie van Aramis. Hierbij onderscheiden we de volgende groepen:

- Burgers: mensen die dicht bij het project wonen of verblijven en om die reden vragen of zorgen hebben of anderzijds geïnteresseerd zijn. Wij denken dan vooral aan omwonenden;
- Bedrijven in de omgeving: bedrijven die dicht in de buurt van het project gevestigd zijn of daar werkzaamheden uitvoeren, zoals buurbedrijven op de Maasvlakte en op de Noordzee;
- Inhoudelijk betrokkenen: maatschappelijke organisaties en stakeholders die zich, los van de locatie, inhoudelijk betrokken voelen. Dit zijn bijvoorbeeld vertegenwoordigers van de scheepvaart, kustwacht, visserij, kabelexploitanten en operators van windparken. Wij denken verder aan ngo's die zich sterk maken voor natuur en milieu. Ook kennisinstellingen en organisaties die zich bezighouden met klimaat en CCS horen hierbij;
- Bestuursorganen: overheden op landelijk, provinciaal en lokaal niveau, zoals de provincie Zuid-Holland, gemeenten, Rijkswaterstaat (kruising zeewering, zandwinning, scheepvaart) en het waterschap Hollandse Delta. Ook semipublieke instellingen zoals ProRail, TenneT en Havenbedrijf Rotterdam zijn belangrijke stakeholders;
- Offshore storage-operators: operators van platforms op de Noordzee die in de toekomst wellicht toegang willen tot de CO_2-transportinfrastructuur van Aramis.

Deze personen en partijen hebben keuze uit individuele gesprekken en groepsbijeenkomsten, zowel online als live. De mate van participatie (informeren, consulteren, adviseren of verkrijgen van consensus) wordt vastgelegd en duidelijk gecommuniceerd. Zo willen wij een brede vertegenwoordiging van de samenleving bereiken en iedereen passend bedienen. Het is onze hoop dat deze werkwijze leidt tot meer betrokkenheid en meer waardering voor en acceptatie van Aramis.

We bieden de volgende informatiekanalen om geïnformeerd te blijven (informeren):

- Publicaties in de *Staatscourant* en huis-aan-huisbladen;
- Informatie op de websites van Aramis, CO_2next en Bureau Energieprojecten;
- (In)formele bijeenkomsten: (online) informatiebijeenkomst/seminar/kennissessie;
- Digitale nieuwsbrief;
- Persoonlijke of geclusterde gesprekken.

We bieden de volgende manieren om betrokken te blijven (consulteren of adviseren):

- (Online) informatiebijeenkomst;
- Bestuurlijke, regionale en landelijke overleggen;
- Persoonlijke of geclusterde gesprekken;
- Schriftelijke reactie op plannen.

Hieronder lichten we deze kanalen toe voor de periode vanaf het vaststellen van het milieueffectrapport (MER) en de Integrale Effectenanalyse (IEA) tot de publicatie van de ontwerpbesluiten.

3.1 MANIEREN OM GEÏNFORMEERD TE BLIJVEN (INFORMEREN)

In deze en de volgende paragraaf leest u hoe wij personen en partijen in de komende periode bij Aramis willen betrekken. Bijlage 3 beschrijft welke stappen in eerdere fases zijn genomen.

a. Publicaties Staatscourant en huis-aan-huisbladen
Formele stappen in de projectprocedure worden vooraf gepubliceerd in de *Staatscourant* en in huis-aan-huisbladen. Naar verwachting wordt in het derde kwartaal van 2024 de terinzagelegging van de ontwerpbeschikkingen in de *Staatscourant* gepubliceerd, waarop zienswijzen kunnen worden ingediend. Eind 2024/begin 2025 volgt naar verwachting de publicatie in de *Staatscourant* dat de definitieve besluiten op de vergunningaanvragen ter inzage liggen voor beroep.

b. Websites Aramis, CO_2next en Bureau Energieprojecten
Iedereen heeft toegang tot onze websites *www.aramis-ccs.com/nl* en *CO2next.nl*. Hier delen wij regelmatig updates en mijlpalen, waarbij we verwijzen naar de officiële documenten op de website van *Bureau Energieprojecten*. Het is voor iedereen mogelijk om een reactie achter te laten. De websites vermelden ook de e-mailadressen en telefoonnummers voor rechtstreeks contact. Wanneer het MER, de IEA en de (ontwerp)besluiten gereed zijn, worden die op de website van *Bureau Energieprojecten* gepubliceerd.

c. (In)formele bijeenkomsten: (online) informatiebijeenkomsten en symposia
In de komende periode worden de milieuonderzoeken uitgevoerd. Tijdens eerdere sessies hebben verschillende stakeholders aandachtspunten (eisen en wensen) aangedragen. Op basis van deze aandachtspunten bespreken we de tussentijdse resultaten van de milieuonderzoeken met de stakeholders. Zo kunnen we stakeholders met zorgen en vragen, bijvoorbeeld over geluid, Natura 2000-gebieden, veiligheid, gezondheid of de impact op de omgeving, specifiek en gedetailleerd informeren. Eventueel vindt er een informatiebijeenkomst of symposium plaats. Vooraf peilen we hiervoor de interesse en informatiebehoefte bij stakeholders. Bij voldoende interesse bepalen we een datum, die we tijdig aan de stakeholders kenbaar maken.

d. Digitale nieuwsbrief
Zo'n vier keer per jaar verschijnt een nieuwsbrief waarvoor iedereen zich via onze website kan aanmelden. De aankomende nieuwsbrieven staan gepland voor september en november. Deze planning staat niet vast en hangt onder andere af van de vraag of er voldoende nieuws is om te communiceren.

e. Persoonlijke of geclusterde gesprekken
De komende periode vinden zowel individuele als geclusterde gesprekken plaats met de diverse stakeholders. Deze gesprekken kunnen het gehele Aramis-initiatief tot onderwerp hebben, dus inclusief het onderdeel waarvoor CO_2next verantwoordelijk is. Maar het is ook mogelijk dat het gesprek zich beperkt tot uitsluitend het deel waarvoor Aramis of CO_2next verantwoordelijk is. Dit is afhankelijk van het onderwerp en de organisatie waarmee het gesprek plaatsvindt, bijvoorbeeld omliggende bedrijven, gemeenten, ngo's, Kamerleden enzovoort.

Tijdens deze gesprekken worden de eisen en wensen van de gesprekspartners zo concreet mogelijk gemaakt. Eisen en wensen die betrekking hebben op het tracé en de exacte ligging zijn in deze fase meegenomen, eisen en wensen die betrekking hebben op de uitvoering volgen in een later realisatiecontract.

De Integrale Effectenanalyse (IEA) brengt de effecten in kaart die de verschillende alternatieven hebben op milieu, kosten, omgeving, techniek en toekomstvastheid. Hier krijgen de opgehaalde eisen en wensen hun beslag. Belanghebbenden worden geïnformeerd over de uitkomsten van de IEA en geconsulteerd over de beoogde voorkeursalternatieven voor het Aramis initiatief.

3.2 MANIEREN OM BETROKKEN TE BLIJVEN (CONSULTEREN/ADVISEREN)

a. (Online) informatiebijeenkomst

Aramis organiseert een of meer kennissessies met als onderwerpen:

- wat zijn de uitkomsten uit de milieuonderzoeken;
- wat is daarvan de vertaling naar het ontwerp;
- wat zijn de consequentie voor de omgeving ten aanzien van de beoogde uitvoering?

b. Bestuurlijke, regionale en landelijke overleggen

Aramis en EZK vinden het belangrijk om direct betrokken overheden, adviesorganen en belangenorganisaties te betrekken bij de besluitvorming over het project. Voor zowel de ruimtelijke procedure als de uitvoeringsvergunningen vinden afstemmingsoverleggen plaats. Zo wordt in het Noordzeeoverleg met enige regelmaat een update gegeven van de onderzeese routealternatieven van Aramis en het overleg dat daarover heeft plaatsgevonden. Deze updates hebben tot doel de aanwezige organisaties mee te nemen in de totstandkoming van de IEA en het VKA, de basis voor het (ruimtelijk) projectbesluit. Ook wordenandere regionale overheden en belangenorganisaties geïnformeerd over het project.

c. Stakeholders

Aramis is in een eerder stadium geïntroduceerd bij onder meer programmamanagers, regioadviseurs, beleidsadviseurs en projectleiders van ministeries (EZK Wind-op-zee, Landbouw, Natuur en Voedselkwaliteit (LNV), Defensie, Binnenlandse Zaken en Koninkrijksrelaties (BZK), Infrastructuur en Waterstaat (IenW)), de Rijksdienst voor het Cultureel Erfgoed (RCE), lokale gemeenten (Rotterdam, Voorne aan Zee), de provincie (Zuid-Holland), Veiligheidsregio Rotterdam-Rijnmond (VRR), water(veiligheid)beheerders (waterschap Hollandse Delta, RWS Zee & Delta, Kustwacht), omgevingsdiensten (DCMR, ODH), wegbeheerder (RWS WNZ), railbeheerder (ProRail), belangengroepen (Deltalinqs, KVNR, Element NL, Nexstep, de Nederlandse Vissersbond, Nederlands Loodswezen, H-vision, NWEA, Verontruste Burgers van Voorne), ngo's (Bellona, Stichting de Noordzee, Natuur & Milieu, Greenpeace, Milieufederatie Zuid-Holland, Vogelbescherming, WNF), raakvlakprojecten (Porthos, Eneco), kabel- en pijpleidingeigenaren (TenneT, Stedin), offshore operators (o.a. Neptune Energy, Petrogas) en bedrijven op de Maasvlakte (Havenbedrijf Rotterdam, MOT, Euromax). Met deze stakeholders worden een-op-een- of clustergesprekken gevoerd.

d. Schriftelijke reactie op plannen

Iedereen krijgt in 2024 de mogelijkheid om schriftelijk een reactie te geven op het ontwerpbesluit en op het MER. De publicatie van het ontwerpbesluit staat gepland voor het derde kwartaal van 2024 en men heeft dan zes weken de tijd om te reageren. In het vierde kwartaal is de publicatie in de *Staatscourant*. De definitieve besluiten op de vergunningaanvragen liggen ter inzage, waarop bezwaar kan worden ingediend. Aramis brengt de stakeholders te zijner tijd op de hoogte van de publicatie, zodat zij in de gelegenheid zijn om tijdig een zienswijze op het ontwerpbesluit (inclusief het MER) in te dienen.

4. PARTICIPATIEKALENDER

4.1 PARTICIPATIEKALENDER

De onderstaande tabel geeft op hoofdlijnen de stappen van besluitvorming en participatie weer conform de projectprocedure (zie paragraaf 1.3 hierboven). In de tabel staat wanneer officiële documenten worden gepubliceerd en ter inzage worden gelegd, en wanneer ieder die dat wil kan meedenken, bijdragen en inspreken.

PROCESSTAP	WIJZE VAN PARTICIPATIE	STATUS
Voornemen en voorstel participatie (januari 2022)	**Informeren, consulteren en adviseren** EZK en Aramis hebben de brede omgeving van overheden, bevoegde instanties, inwoners, bedrijven en professionele stakeholders geïnformeerd over het projectvoornemen en de voorgestelde invulling van participatie. Iedereen kon een formele reactie geven met betrekking tot: 1. andere oplossingen voor de geschetste opgave, bijvoorbeeld andere manieren om CCS toe te passen (denk aan alternatieven en varianten); 2. andere voorstellen voor de wijze waarop derden worden betrokken. Alle verzamelde reacties zijn waar mogelijk verwerkt in de concept-NRD (Notitie Reikwijdte en Detailniveau). Participatie-instrumenten: • Publicatie in Staatscourant en huis-aan-huisbladen; • Openbare informatiebijeenkomst.	**Gereed**
Inventarisatie alternatieven en varianten en het beoordelingskader (januari-mei 2022)	**Consulteren en adviseren** EZK en Aramis hebben andere overheden, bevoegde instanties en belangenorganisaties geconsulteerd om op verschillende manieren mee te denken, informatie aan te leveren over tracé-alternatieven, en varianten en aandachtspunten aan te dragen voor de NRD en het MER. Participatie-instrumenten: • Geïntegreerde interactieve werksessies; • Een-op-een- of clustergesprekken; • Nieuwsbrief Aramis.	**Gereed**
Concept Notitie Reikwijdte en Detailniveau (concept-NRD) (juni 2022)	**Informeren, consulteren en adviseren** Iedereen kon een formele zienswijze indienen over de vragen: • of de participatie beter kan; • of er iets ontbreekt bij de onderzoeken; • of de juiste onderdelen worden onderzocht; • of er andere tracé-alternatieven en/of -varianten onderzocht moeten worden. Waar relevant zijn deze meegenomen in de definitieve NRD. Participatie-instrumenten: • Publicatie in Staatscourant en huis-aan-huisbladen; • Publicatie op www.rvo.nl/onderwerpen/bureau-energieprojecten;	**Gereed**

	• Websites Aramis en CO_2next; • Raadpleging Commissie MER; • Een-op-een- of clustergesprekken; • Formele en informele informatiebijeenkomst op 21 juni 2022; • Nieuwsbrief Aramis.	
Vaststellen definitieve NRD (december 2022)	**Informeren** EZK en Aramis hebben de brede omgeving geïnformeerd over de definitief vastgestelde NRD. Participatie-instrumenten: • Publicatie op www.rvo.nl/onderwerpen/bureau-energieprojecten; • Websites Aramis en CO_2next; • Nieuwsbrief Aramis.	**Gereed**
Integrale Effectenanalyse (IEA) (december 2023)	**Informeren, consulteren en adviseren** EZK en Aramis consulteren de brede omgeving over de afwegingen van de IEA op basis van de aspecten milieu, kosten, omgeving, techniek en toekomstvastheid. Participatie-instrumenten onder andere: • Publicatie op www.rvo.nl/onderwerpen/bureau-energieprojecten; • Websites Aramis en CO_2next; • Overleggen (door EZK); • Een-op-een- of clustergesprekken; • Nieuwsbrief Aramis.	**Gereed**
Keuze voorkeursalternatief (VKA) (februari 2024)	**Informeren, consulteren en adviseren** EZK en Aramis raadplegen decentrale overheden en andere departementen over het VKA. De minister van EZK bepaalt op basis van dit advies het voorkeursalternatief. Participatie-instrumenten onder andere: • Een-op-een- of clustergesprekken met belanghebbenden; • Overleggen (door EZK); • Websites Aramis en CO_2next; • Nieuwsbrief Aramis.	**Gereed**

Milieueffectrapport (MER) als onderdeel van de vergunningaanvragen (medio 2024)	**Informeren, consulteren en adviseren** EZK en Aramis consulteren de brede omgeving over het MER. Reageren op het MER is mogelijk bij de terinzagelegging van de ontwerpbesluiten (zie de stap Publicatie ontwerp-projectbesluit en ontwerp-vergunningen hieronder). Participatie-instrumenten onder andere: • Resultaten van het MER zullen aan het eind worden gedeeld; • Een-op-een- of clustergesprekken met belanghebbenden; • Openbare informatiebijeenkomsten; • Websites Aramis en CO_2next; • Nieuwsbrief Aramis.	*Gepland*
Publicatie ontwerp-projectbesluit en ontwerp-vergunningen (eind 2024)	**Informeren en horen** De bevoegde instanties stellen op basis van de aanvragen van Aramis het ontwerp-projectbesluit en de ontwerp-vergunningen op. EZK publiceert het ontwerp-projectbesluit en de ontwerp-vergunningen, inclusief het MER. Iedereen die dat wil kan een formele zienswijze indienen. De commissie van de m.e.r. geeft een advies over het MER. Participatie-instrumenten onder andere: • Publicatie in Staatscourant en huis-aan-huisbladen; • Publicatie op www.rvo.nl/onderwerpen/bureau-energieprojecten; • Openbare informatiebijeenkomst(en); • Een-op-een- of clustergesprekken met belanghebbenden; • Websites Aramis en CO_2next; • Nieuwsbrief Aramis.	*Gepland*
Publicatie definitief projectbesluit en definitieve vergunningen (eind 2024/begin 2025)	**Informeren en beroep** EZK publiceert het definitief projectbesluit en de definitieve vergunningen. Iedereen kan reageren op het projectbesluit en de vergunningen door hiertegen beroep in te stellen. Participatie-instrumenten onder andere: • Publicatie in Staatscourant en huis-aan-huisbladen; • Publicatie op www.rvo.nl/onderwerpen/bureau-energieprojecten; • Hoger beroep; • Websites Aramis en CO_2next; • Nieuwsbrief Aramis.	*Gepland*
Onherroepelijk projectbesluit en vergunningen (zonder beroep)	Uitspraak Raad van State na behandeling van mogelijke beroepen.	n.t.b.

4.2 WE HOREN GRAAG UW REACTIE OP DIT PARTICIPATIEPLAN

Zoals in paragraaf 1.1 aangeven, actualiseren we het participatieplan minstens eenmaal per projectfase. Het volgende participatieplan verschijnt naar verwachting in het najaar van 2024, voorafgaand aan de publicatie van het projectbesluit.

Heeft u vragen of suggesties voor verbetering van dit plan? Wij horen graag van u!
U kunt uw reactie per e-mail sturen naar: info@aramis-ccs.com.

BIJLAGES

BIJLAGE 1 SAMENVATTING INBRENG STAKEHOLDERS

Het doel van de stakeholderparticipatie is het ophalen van informatie, gebiedskennis, aandachtspunten, ideeën en kansen uit de omgeving. Zo hebben er sinds zomer 2021 kennismakingsgesprekken met stakeholders, één-op-één overleggen en persoonlijk contact met verschillende belanghebbenden plaatsgevonden. Van 7 januari tot 17 februari 2022 heeft de notitie 'Voornemen en Voorstel Participatie' ter inzage gelegen. In die periode was het mogelijk om te reageren door een schriftelijke reactie te geven op deze notitie. Er zijn zes reacties binnengekomen bij EZK. Er is formeel een antwoord gegeven op deze reacties via de nota van antwoord die is opgesteld door EZK in afstemming met het Aramis initiatief. Deze nota van antwoord is tegelijkertijd met de concept NRD en dit Participatieplan gepubliceerd.

Daarnaast werden er op 19 en 24 januari 2022 werksessies met verschillende stakeholders op respectievelijk 'land' en op 'zee' georganiseerd en heeft het ministerie van Economische Zaken en Klimaat op 26 januari 2022 een informatieavond gehouden. Een aantal aanwezigen bij de informatieavond heeft aangegeven de Aramis nieuwsbrief te willen ontvangen: zij hebben inmiddels de eerste Aramis nieuwsbrief ontvangen en worden op de hoogte gehouden door volgende nieuwsbrieven. Tijdens de verschillende gesprekken en werksessies zijn de plannen toegelicht en is er veel gebiedskennis verzameld. In het onderstaande wordt een samenvatting van aandachtspunten gegeven die door stakeholders zijn benoemd. Hierbij is onderscheid gemaakt tussen het onderdeel 'aanlanding en landdeel' (A) en het onderdeel 'zeedeel' (B). Daarnaast volgt een lijst van geraadpleegde stakeholders per onderdeel.

1 Samenvatting aandachtspunten Maasvlakte – aanlanding en landdeel

Omgevingsveiligheid, geluid & stikstof depositie
Veel partijen stellen vragen over omgevingsveiligheid, geluid en stikstofdepositie door de aanleg en aanwezigheid van de terminal en het compressor station, pompen en andere installaties. Ook over het 'entry' punt van de micro-tunnel (één van de twee voorlopige aanlandingslocaties op de Maasvlakte) stellen partijen vragen met het oog op het risico op calamiteiten, aangezien de 'vuurwerk ompak' locatie op de Prinses Maximaweg zich nabij bevindt. Verder wordt voor de stikstofdepositie in relatie tot scheepvaartbewegingen (ten behoeve van de vloeibare intake van CO_2) aandacht gevraagd.

Overslag CO_2 na aanlanding per schip
De terminalfaciliteiten, bestaande uit de overslag van CO_2 van schepen, tijdelijke opslag en verpomping van vloeibaar CO_2 naar de zeeleiding worden door CO_2next uitgevoerd.

Aanlanding vanuit zee op Maasvlakte
Voor de aanlanding van de pijpleiding vanuit zee naar de Maasvlakte zijn twee opties in beeld. Ten eerste via een Horizontale boring (HDD) onder de harde zeewering of ten tweede via een micro-tunnel die op diepte ligt onder de Maasgeul. De stakeholders vragen aandacht voor het feit dat beide aanlegmethodes ook op het land van de Maasvlakte permanente ruimte en werkterreinen behoeven. Hiervoor is tijdige afstemming met meerdere stakeholders, onder meer Port of Rotterdam van belang.

De suggestie wordt gedaan om een overleg te hebben met de stakeholders die gebiedskennis hebben over de aanlanding middels een HDD op de Maasvlakte. De beschikbare ruimte is beperkt gezien de ligging van TenneT kabels (Net op zee HKZ), de voorziene ligging van de Porthos CO_2 leiding, de aanwezige leidingenstrook op de Maasvlakte en het voorziene windpark van Eneco op de zeewering.

Een van de opties, een microtunnel, zou mogelijkheden en kansen kunnen bieden voor medegebruik zoals het 'Net op zee' van TenneT voor nog toekomstige windparken.
Ongeacht de aanlandingsopties wordt aandacht gevraagd voor dat de scheepvaart in de Maasgeul geen hinder mag ondervinden.

Andere functies en industrie op de Maasvlakte
In veel gesprekken komt naar voren dat de industrie volcontinu in bedrijf is. De dagelijkse werkzaamheden moeten 24/7 door kunnen gaan tijdens de aanlegfase van de projecten. Ook dient de toegang van hulpdiensten te allen tijde zijn gegarandeerd. Eveneens dient de bereikbaarheid van de kazerne van de Gezamenlijke Brandweer aan de Prinses Maximaweg 24/7 gegarandeerd te blijven.
De leiding komt deels binnen en buiten de leidingenstrook te liggen. Dit vergt afstemming met zowel Port of Rotterdam als het Leidingenbureau van gemeente Rotterdam. De krappe ligging in de leidingenstrook en de drukte in de ondergrond zijn aandachtspunten.

Autoriteiten en andere stakeholders – aanlanding en landdeel
Autoriteiten: Het Ministerie van EZK, DCMR, ProRail regio Randstad-Zuid, Gemeente Rotterdam (RO, leidingenbureau Rotterdam), Veiligheidsregio Rotterdam-Rijnmond, Rijkswaterstaat (WNZ, Zee & Delta), Omgevingsdienst Haaglanden, Provincie Zuid-Holland
Ngo's: Vereniging Natuurmonumenten Zuid Holland, Natuur- en Milieufederatie Zuid-Holland
Kabel en pijplijn eigenaren: TenneT
Industrie & Business & andere projecten Maasvlakte: Deltalinqs, Havenbedrijf Rotterdam, Divisie Havenmeester van het Havenbedrijf Rotterdam, Eneco, Euromax, Gate terminal, Porthos , MOT, ProRail, ECT Rotterdam
Scheepvaart: het Nederlands Loodswezen
Overige: Gezamenlijke brandweer Prinses Maxima kazerne

1 Samenvatting aandachtspunten - zeedeel

Zeeleiding op of in de zeebodem
Partijen hebben vragen over de installatie van de zeeleiding op of in de zeebodem. Dit heeft te maken met verschillende belangen van verschillende stakeholders. Zo dient de leiding overvisbaar te zijn en moet scheepvaartveiligheid gegarandeerd zijn in geval van (nood)ankeren boven de leiding. Daarnaast zijn er vragen over de gevolgen van meerdere leidingen en kabels die gekruist worden in de aanlooproute voor de scheepvaart; ontstaan er dan niet lokale verondiepingen op de zeebodem als gevolg van de kruisingsconstructies op de zeebodem? Nautische partijen vragen verder om het beperken van hinder voor de scheepvaart door het vermijden van ankergebieden en het zoveel mogelijk haaks kruisen van hoofdvaarroutes en geulen. Daarnaast wordt er aandacht gevraagd voor het mogelijke effect van CO_2 lekkage op het milieu. Ook is er sprake van de aanwezigheid van mogelijke obstakels op de zeebodem (zoals wrakken en mogelijk WO II resten).

Andere functies op de Noordzee
Partijen geven aan dat er nieuwe windparken op zee worden gepland. Dit heeft mogelijk ook gevolg voor een militair oefengebied op zee dat verplaatst moeten worden. Partijen vragen of er bij de tracering van de leiding rekening wordt gehouden met deze ontwikkelingen. Dit betekent ook nieuwe hoogspanningskabels van het net op zee, waarin in de tracering rekening gehouden moet worden (t.a.v. minimumafstanden en kruisingen).
Andere olie- en gasoperators hebben interesse getoond voor het eveneens aansluiten op de centrale leiding, zodat ook van hun opslagmogelijkheden gebruik gemaakt kan worden. Voor deze groep van stakeholders is op 9 maart 2022 een aparte bijeenkomst georganiseerd.
Partijen vragen aandacht voor andere gebruiksfuncties op de drukke Noordzee; zoals zandwinning. Deze gebieden dienen zo veel mogelijk vermeden te worden.
Met de stakeholders zijn twee tracé opties (Opties A en B) in het noordelijke deel op zee besproken. Alleen vanuit de toekomstige windpark belangen is er een voorkeur uitgesproken voor route-optie A omdat deze route-optie minder impact heeft op het toekomstige windenergiegebied. Overige partijen hebben geen onderscheidende aandachtspunten per tracé optie aangegeven.

Natuurversterkende maatregelen en andere kansen
In de contacten met partijen werden ook kansen benoemd voor de Noordzee; zoals het natuur-inclusief aanleggen van de benodigde infrastructuur op de zeebodem en een eventuele koppeling met andere CCS projecten.

Autoriteiten en andere stakeholders - zeedeel
Autoriteiten: Ministerie van EZK, Rijkswaterstaat (Zee & Delta), Ministerie van LNV, Ministerie van Defensie/ Dienst der Hydrografie, Ministerie van I en W
Ngo's: Vereniging Natuurmonumenten Zuid Holland, Natuur- en Milieufederatie Zuid-Holland, Stichting de Noordzee, Natuur & Milieu
Kabel en pijplijn eigenaren: TenneT, Stedin
Industrie & Business: Divisie Havenmeester van het Havenbedrijf Rotterdam
Scheepvaart: het Nederlands Loodswezen, Scheepvaart Adviesgroep Noordzee, KVNR
Visserij: Nederlandse Vissersbond, Voormalig VisNED
Olie en gas: Element NL
Zandwinning: LaMER
Overig: Kustwacht

Terugkoppeling werksessies
In de terugkoppeling naar deze stakeholders hebben we initieel een korte reactie gegeven op alle aandachtspunten. Hierin is aangeven dat we contact opnemen om een afspraak te maken en in individuele gesprekken hun aandachtspunten verder willen bespreken. Het Aramis initiatief heeft na de werksessie contact gehad met het Havenbedrijf Rotterdam, Euromax, Deltalinqs (bij de Klimaattafel) en DCMR. Op 7 april 2022 is er ook een gezamenlijk gesprek geweest met de gemeente Rotterdam, EZK, Gate terminal, MOT, Aramis en CO_2next over de aanpak voor het wijzigen van het huidige bestemmingsplan van Gate terminal en MOT en de rol van de bevoegde gezagen. Er is een vervolgoverleg ingepland om helderheid te verschaffen aan de te volgen procedure. Alle reacties zijn als input meegewogen voor de concept NRD en het technisch ontwerp waar we momenteel mee bezig zijn.

BIJLAGE 2 VERSLAG STAKEHOLDERSESSIE 21 JUNI 2022

Onderwerp	Stakeholderbijeenkomst Aramis en CO_2next
Project	Aramis
Datum bijeenkomst	21 juni 2022
Plaats	Hoek van Holland
Bijlage(n)	Presentatie Aramis
Aanwezig	Ministerie van EZK, EZK Wind-op-zee, TenneT, RWS, Koninklijke Vereniging van Nederlandse Reders, Kustwacht, Neptune, Carbon Collectors, Noordgastransport, Porthos, AECOM, Buis Consultancy, TNO, Port of Rotterdam (nautisch beheer), Omgevingsdienst Haaglanden, DCMR, Provincie Zuid- Holland, RWS (WNZ), LNV, Veiligheidsregio Rotterdam -Rijnmond.

Verslag stakeholderbijeenkomst

Algemeen

Op 21 juni jl. heeft een stakeholderbijeenkomst plaatsgevonden. Het doel van de bijeenkomst was het ophalen van informatie, gebiedskennis, aandachtspunten voor het MER ideeën, zorgen, wensen en kansen uit de omgeving. Onderstaand het verslag van de bijeenkomst.

Plenaire opening

Er wordt gestart met een toelichting op de concept Notitie Reikwijdte en Detailniveau en de stand van zaken van Aramis. Er wordt aangegeven wat de planning is en op welke momenten er nog ruimte is voor participatie.

Thematafels

Na het plenaire gedeelde wordt er uiteen gegaan in drie thematafels: de Maasvlakte, de Aflanding en de Noordzee.

Samenvatting aandachtspunten Maasvlakte

Aan deze tafel gingen vragen onder meer over:

- technisch gerelateerde zaken zoals de aanleg van pijpleidingen: land-trace's en de constante flow van de CO_2 in relatie tot een flexibel aanbod van de CO_2
- de schepen: emissieloos bouwen, stikstofdepositie en duur van het bouwen, soort schepen, capaciteit steigers, en aanbod walstroom
- het bevoegd gezag voor het deel van de aanlanding en de Maasvlakte (in dit geval gecoördineerd door EZK).
- de situatie met betrekking tot het compressorstation en de relatie tussen Aramis, Porthos en CO_2next.
- de scope tussen Aramis emitters en andere emitters, als ook over de capaciteit en prioritering voor de opslagvelden en voldoende beschikbaarheid van schepen voor de aan- en afvoer van vloeibare CO_2.
- punten in relatie tot de veiligheid, zoals het meenemen van de windturbines in de risicoanalyse, de gevolgen voor Hoek van Holland, aanvaringsrisico's, tankrisico's, de ligging van de brandweer kazerne bij een verkeerde wind.

Samenvatting aandachtspunten Aflanding
Aan deze tafel is onder andere gevraagd naar de technische uitdaging in dit project, en de beschikbare ruimte in relatie tot de beoogde Porthos leiding. Verder hebben TenneT en Porthos vooral hun ervaringen gedeeld, opgedaan bij eerdere aanleg van leidingen in het gebied, respectievelijk bij de voorbereiding daarop. Zo is uitdrukkelijk meegegeven aandacht te hebben in het vervolgtraject voor aanwezige niet gesprongen explosieven, archeologische waarden, bodemgesteldheid, stabiliteit van de zeewering, en beschermde soorten. Dit zowel uit technisch oogpunt als voor wat betreft de benodigde vergunningen en toestemmingen en de tijd die daarmee gemoeid is. Aangeboden wordt waar mogelijk gegevens van bijvoorbeeld boringen te delen, zonder daarbij de eigen verantwoordelijkheid van Aramis uit het oog te verliezen. Vanuit Nautisch Beheer van Port of Rotterdam wordt aandacht gevraagd voor het veilig en ongestoord doorgang vinden van de scheepvaart en de eisen die daaraan worden gesteld. In dat kader is als aandachtspunt meegegeven dat het Port of Rotterdam niet altijd duidelijk is op welke wijze de verschillende initiatiefnemers in de Maasmond met elkaar samenwerken.

Samenvatting aandachtspunten Noordzee
Aan deze tafel werd de ligging van de leiding toegelicht aan de hand van een tracétekening. Daarna is er de mogelijkheid gegeven aan de aanwezigen om te reageren op deze tekening.

Veel van de ingebrachte punten waren suggesties ter verbetering van de ligging van de leiding en het kaartmateriaal.

- EZK Wind-op-zee merkt op dat de zoekgebieden voor Hollandse Kust Zuidwest en Noordwest vervallen. Deze moeten nog van de tracétekening worden afgehaald.
- De Kustwacht geeft aan dat in de bepaling van de tracékeuze aandacht moet zijn voor multifunctioneel ruimtegebruik, bijvoorbeeld gaswindgebieden en bijbehorende aanvliegroutes en defensie oefengebied.
- De Kustwacht geeft als suggestie dat bestaande pijpleidingen gevolgd kunnen worden om een corridor te creëren.
- Neptune Energy geeft aan dat de Riser Tower of site tap op ‘gelijke’ afstand van hun velden moet liggen als van de velden van TotalEnergies en Shell.
- De Kustwacht geeft aan dat de leiding overvisbaar moet zijn, geen ankerplekken mag kruisen en zoveel mogelijk parallel moet liggen aan de vaarroutes.
- EZK Wind-op-zee ziet graag dat de leiding wordt gelegd buiten de (beoogde) windgebieden.

Daarnaast worden er verschillende punten ingebracht ter verbetering van de c-NRD en om mee te nemen in het MER:

- EZK Wind-op-zee vindt dat de ruimtelijke keuzes voor de ligging van het tracé nog beter omschreven mogen worden in de c-NRD.
- Neptune Energy voegt daaraan toe dat ze graag nog beter de mogelijkheden voor toekomstige aan- en aftakkingen op de leiding omschreven zien.
- De Kustwacht geeft aan dat er in het MER onderzocht moet worden wat het effect van lekkage is.

KNVR geeft tot slot de tip om MARIN te benaderen voor meer informatie over hun onderzoek naar de mogelijkheden om windmolens te beschermen tegen op drift geraakte schepen, omdat de uitkomsten hiervan ook nuttig voor Aramis kunnen zijn.

De middag is afgerond met een plenaire terugkoppeling, waarbij de gevoerde gesprekken per thematafel zijn samengevat, en is benadrukt dat op meerdere momenten in het vervolg van het proces participatie mogelijk is. Aramis zal de opgehaalde informatie verwerken in het MER en zal het gesprek van de thematafels voort zetten met de verschillende stakeholders.

BIJLAGE 3 AFGERONDE ACTIES VAN PARTICIPATIE (UIT H3)

MANIEREN OM GEÏNFORMEERD TE BLIJVEN (INFORMEREN)

a. Publicaties Staatscourant en huis-aan-huis bladen
Op 9 juni 2022 is in de Staatscourant (en in diezelfde week ook in huis-aan-huis bladen) gepubliceerd dat de concept NRD en dit participatieplan ter inzage lagen voor reacties. Op 2 december 2022 is in de Staatscourant gepubliceerd dat de definitieve NRD is vastgesteld.

b. Websites projecten Aramis, CO_2next en Bureau Energieprojecten
Op 10 juni 2022 is de concept NRD gepubliceerd op de website van *Bureau Energieprojecten*. Hierop kon iedereen de concept NRD en het geactualiseerde participatieplan inzien. Iedereen had de mogelijkheid tot het indienen van een zienswijze. Er zijn acht zienswijze ingediend die formeel zijn beantwoord. Op 2 december 2022 is de definitieve NRD inclusief de nota van antwoord gepubliceerd op de website van *Bureau Energieprojecten*.

c. (In)formele bijeenkomsten: Informatiebijeenkomst, symposium en kennissessies
Op 21 juni 2022 hebben EZK en het Aramis initiatief een formele informatiebijeenkomst gehouden, ten tijde van de terinzagelegging van de concept NRD. We hebben de concept NRD toegelicht, welke alternatieven en varianten we in het MER gaan onderzoeken, hoe we dat gaan doen en in welk detailniveau. Tijdens deze bijeenkomst waren projectleden van het Aramis initiatief aanwezig om vragen over het project en de concept NRD te beantwoorden. Medewerkers van EZK waren ook aanwezig om vragen over de procedure te beantwoorden.

Naast de formele bijeenkomst heeft Aramis een informele bijeenkomst georganiseerd voor alle (zakelijke) stakeholders. Doel was om de deelnemers van deze bijeenkomst te informeren over de status van het project aan de hand van de concept NRD en om alle vragen die er leven te beantwoorden. Met deze bijeenkomst heeft het Aramis initiatief ook voldaan aan de verplichting van een openbare raadpleging die volgt uit de PCI-status (Project of Common Interest).

d. Digitale nieuwsbrief
We hebben eind april 2022 de eerste nieuwsbrief en in juli 2022 de tweede nieuwsbrief uitgebracht. De eerste twee nieuwsbrieven waren in het Nederlands. De derde nieuwsbrief (in het Engels) is in november 2022 verspreid en de vierde in april 2023. De laatste twee neiuwsbrieven zijn in juli en december 2023 verstuurd. Alle nieuwsbrieven zijn toegankelijk via de Aramis website.

e. Persoonlijk of geclusterde gesprekken
Afgelopen periode zijn individuele en ook geclusterde gesprekken met de diverse stakeholders gevoerd. Uitkomsten daarvan zijn en worden verwerkt in Dialog (= klanteisen managementsysteem).

MANIEREN OM BETROKKEN TE BLIJVEN (INFORMEREN/CONSULTEREN/ADVISEREN)

a. Informatiebijeenkomst
Tijdens de informatiebijeenkomst op 21 juni 2022 konden de aanwezigen op een laagdrempelige manier in gesprek gaan met projectmedewerkers van het Aramis initiatief en het ministerie van EZK. Ook was het voor de aanwezigen mogelijk tijdens deze bijeenkomst een mondelinge reactie (zienswijze) in te dienen. Uiteindelijk zijn er acht schriftelijke reacties ingediend op de concept NRD.

b. Bestuurlijke en landelijke overleggen
Het Aramis initiatief en het ministerie van EZK vinden het belangrijk om gemeenten, provincie en andere bestuursorganen actief te betrekken bij de besluitvorming over het project.

Het Aramis initiatief en het ministerie van EZK betrekken bestuurlijke partners van de gemeenten, de provincie Zuid-Holland en andere departementen met betrekking tot de Noordzee actief bij het besluitvormingsproces van het projectbesluit. Bestuurders van deze partners worden bij elke formele zienswijze periode op de hoogte gehouden van de voortgang in een op te richten Bestuurlijk Overleg (BO), geïnitieerd door EZK.

Op 15 november 2022 heeft het eerste coördinatieoverleg vergunningen plaatsgevonden. Dit is een tweemaandelijks overleg met alle bevoegde gezagen in het kader de vergunningen onder de Rijkscoördinatieregeling (RCR).

c. Persoonlijke of geclusterde gesprekken
Wij hebben het project al eerder geïntroduceerd o.a. aan programma-managers, regioadviseurs, beleidsadviseurs en projectleiders van ministeries (EZK Wind, Landbouw, Natuur en Voedselkwaliteit (LNV), Rijksdienst voor het Cultureel Erfgoed (RCE) , Defensie, Binnenlandse Zaken en Koninkrijksrelaties (BZK), Infrastructuur en Waterstaat (IenW)), lokale gemeenten (Rotterdam, Brielle, Westvoorne), Provincie (Zuid-Holland), VRR, water(veiligheid)beheerders (Waterschap Hollandse Delta, RWS Zee & Delta, Kustwacht), omgevingsdiensten (DCMR, ODH), wegbeheerder (RWS WNZ), railbeheerder (ProRail), belangengroepen (Deltalinqs, KVNR, Element NL, Bellona, Nexstep, de Nederlandse Vissersbond, Stichting de Noordzee, Nederlands Loodswezen, H-vision, NWEA, Verontruste Burgers van Voorne), ngo's (Natuur & Milieu, Greenpeace, Milieufederatie Zuid-Holland), raakvlakprojecten (Porthos, Eneco), kabel- en pijpleiding eigenaren (TenneT, Stedin), offshore operators (o.a. Neptune Energy , Petrogas) en bedrijven op de Maasvlakte (Havenbedrijf Rotterdam, MOT, Euromax). Dit ambtelijke en persoonlijke contact zetten wij voort in deze komende fase.

Hieronder staat een overzicht met welke belanghebbenden en over welke onderwerpen wij spreken.

- Havenbedrijf Rotterdam: de aanlanding, uitwerking verschillende tracés en locatie alternatieven en varianten in het havengebied;
- Provincie Zuid-Holland: de ruimtelijke kwaliteit (o.a. openheid en natuur) van het gebied in relatie tot het tracé en locatiealternatieven en -varianten, vergunningen;
- RWS Zee & Delta en Kustwacht: nautische veiligheid, het kruisen van scheepvaartroutes, de tracering en locatie alternatieven en varianten, vergunningen op zee;
- RWS WNZ: uitwerking van tracé- en locatiealternatieven en varianten bij kruising van waterkeringen, hoofdwatergangen, aandachtspunten van diverse uitvoeringsmethodes en vergunningen;
- Waterschap Hollandse Delta, DCMR en ODH: benodigde water vergunningen, vergunningen in het kader van de wet algemene bepalingen omgevingsrecht en natuurvergunningen en ontheffingen;
- Gemeente Rotterdam: voor de benodigde vergunningenoverzicht en rol van bevoegde gezag en invloed op CCS op de energietransitie;
- TenneT, Stedin: raakvlakken projecten en invloeden van tracé- en locatiekeuzes, met name bij de kruising van de waterkering (TenneT) en energievoorziening en beschikbare ruimte in de Leidingenstrook (Stedin);

- Eneco: raakvlakken en veiligheidsrisico's van windmolens op de Maasvlakte;
- MOT, ECT Rotterdam, Euromax: impact op 24/h bedrijfsvoering en overlast (geluid, trillingen);
- Ministeries: raakvlakken (toekomstige) windparken op de Noordzee zoals Lagelander, impact op het milieu en visserij, raakvlakken (toekomstige) zandwinningsgebieden, gebieden van hoge cultuur-historische waarde en vergunningen;
- Wij informeren de bij ons bekende maatschappelijke organisaties (Milieufederatie Zuid-Holland, Natuur & Milieu, Greenpeace, Milieudefensie en Stichting de Noordzee) rechtstreeks over het project en de procedures. In de studies die we uitvoeren voor de vergunningen en het milieueffectrapport (MER) besteden we nadrukkelijk aandacht aan milieu, natuur en andere belangrijke maatschappelijke waarden. Daarnaast onderzoeken we met Stichting de Noordzee, Natuur & Milieu, het Koninklijk Nederlands Instituut voor Onderzoek der Zee, de Wageningen University & Research en het Nederlandse Organisatie voor Wetenschappelijk Onderzoek of we het project Aramis natuurversterkend kunnen aanleggen;
- Porthos: afstemming omgevingsmanagement en aansluiting op Porthos;
- Commissie MER: afstemming en advies voor concept NRD en MER;
- ProRail: impact op kruising van en werken nabij het spoor (veiligheid en bedrijfsvoering);
- Veiligheidsregio's: veiligheidsrisico's in het havengebied en de nabije omgeving (toegangswegen);
- Het Aramis initiatief is meermalen aangeschoven bij het Noordzeeoverleg (NZO). De NZO-leden zijn: de ministeries (Infrastructuur en Waterstaat, Economische Zaken en Klimaat, en Landbouw, Natuur en Voedselkwaliteit), Energiesector (Nederlandse Wind Energie Associatie, TenneT, Element NL, Energie Beheer Nederland), Zeevaartsector (Branche Organisatie Zeehavens, Koninklijke Vereniging Nederlandse Rederijen, Havenmeesters), natuur en milieuorganisaties (WNF Nederland, Greenpeace (geen permanent lid), Stichting De Noordzee, Vogelbescherming Nederland, Natuur & Milieu) en Voedsel&Visserij (NetVisWerk en Producentenorganisaties Urk & Delta Zuid). Het project Aramis informeert regelmatig over de stand van zaken tijdens dit NZO-overleg. Aanwezigen van dit overleg wordt gevraagd om input te leveren vanuit hun organisatie, bijv. over scheepvaartbelemmering op zee of kruising Maasgeul, gevoelige infrastructuur op de zeebodem, raakvlak (toekomstige) windmolenparken, impact op natuur, onderwater geluid, etc.);
- NEa (Nederlandse Emissieautoriteit): onafhankelijke autoriteit voor toezicht op de uitstoot van broeikasgassen;
- Er is een gezamenlijke bijeenkomst geweest waarin het project Aramis gepresenteerd werd aan alle operators en waar operators kenbaar konden maken of men wilde aansluiten, en zo ja, wanneer. Met operators met concrete belangstelling en betrokkenheid zijn er individuele overleggen gevoerd;
- Eind 2021 is door *CO_2next* een Open Season proces gestart. Het primaire doel van het Open Season was het verkrijgen van een beter inzicht in het marktpotentieel. Dit is mede van belang voor de vergunningaanvraag waarin de eindsituatie dient te worden omschreven. Bovendien is waardevolle informatie verzameld voor het verdere engineering proces zodat al vroegtijdig kan worden nagedacht over bijvoorbeeld tie-in point en over-dimensionering. Een secundair doel van het Open Season proces was om te voldoen aan de criteria voor Open Access en Non-discriminatory Access. Hierdoor wordt

gerechtvaardigd dat er een of enkele launching customers zijn.
In een intensieve samenwerking en onder speciale voorwaarden kan met deze launching customers de keten worden opgezet. In een volgende fase zouden andere partijen dan onder de dan geldende voorwaarden kunnen aansluiten.

d. Schriftelijke reactie op de plannen geven
Iedereen heeft in 2022 de mogelijkheid gehad een schriftelijke reactie te geven op de concept NRD (een zienswijze indienen). Er zijn acht zienswijzen ingediend. Al deze zienswijzen zijn gebundeld (zienswijzebundel) en in de nota van antwoord is een toelichting gegeven of en hoe deze zijn meegenomen bij het opstellen van de definitieve NRD of in het verdere proces.

Het Aramis initiatief heeft advies aan de commissie MER op de concept NRD gevraagd. Dit advies is op de site van de commissie op 18 augustus 2022 gepubliceerd. Het ministerie van EZK heeft op basis van de ingekomen zienswijzen en het advies van de commissie MER de definitieve NRD vastgesteld en gepubliceerd op 2 december 2022.

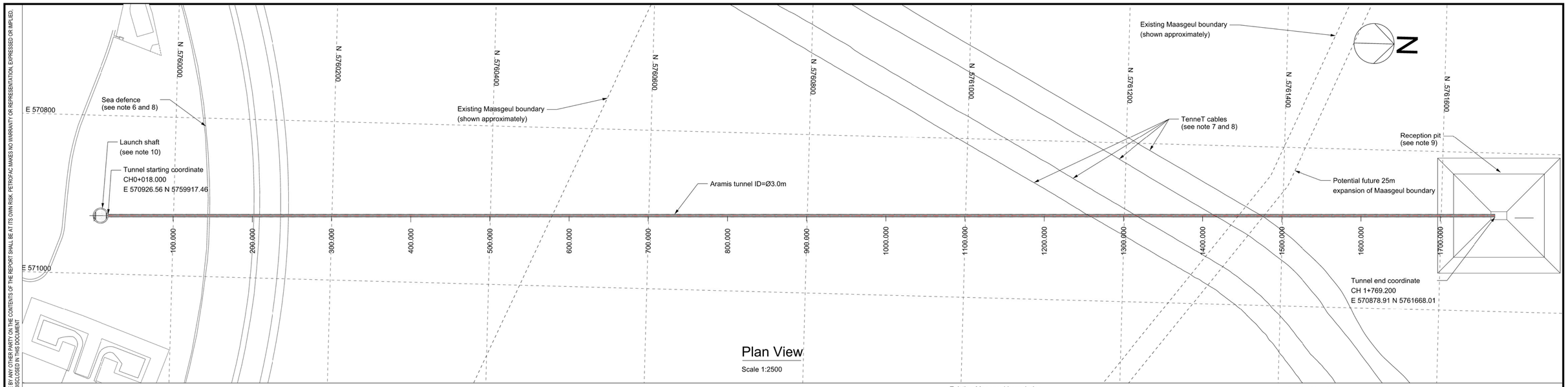
Bijlage

2. Tunnel plan view

NL-ARN-040-ARU1-110323_01_IFD_20240919_01

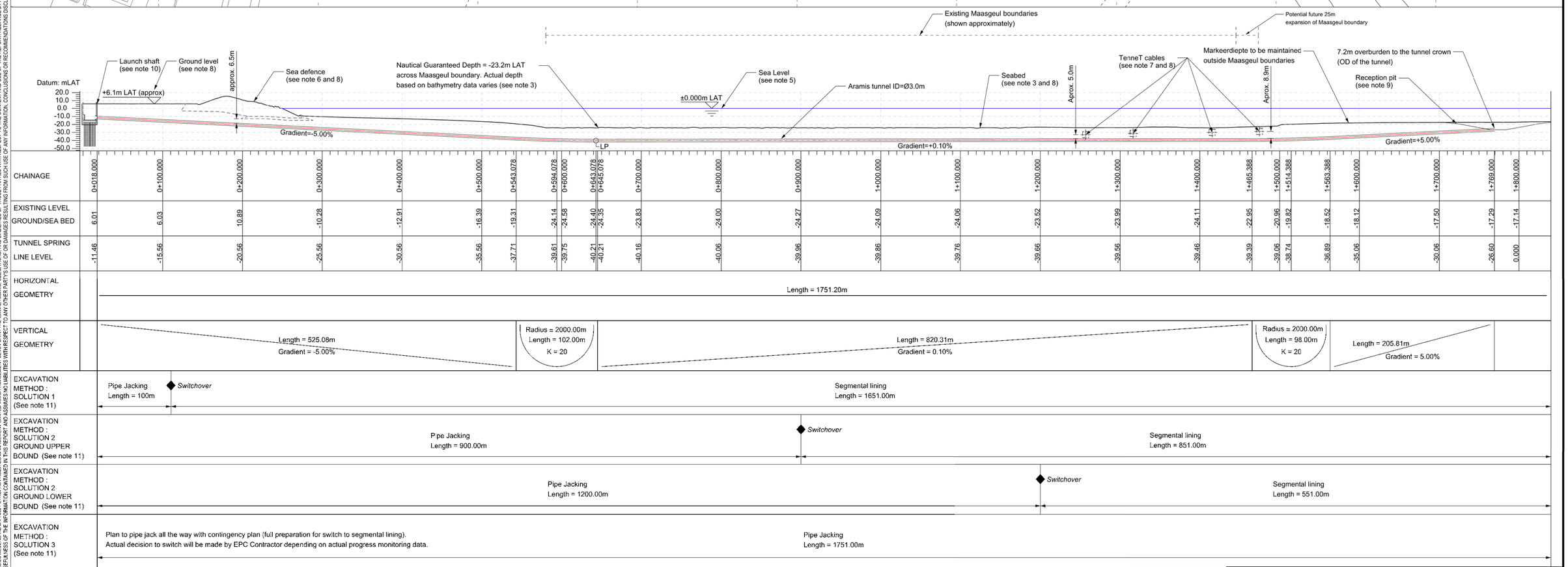
Royal HaskoningDHV

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Plan View

Scale 1:2500



Longitudinal Profile View

Scale 1:2500

Scale Bar 1:2500 @ A1
0m 50m 100m 150m 200m 250m

NOTES CONTINUED

10. LAUNCH SHAFT IS SHOWN INDICATIVELY. SEE DRAWING NUMBERS NL-ARM-040-ARU1-110331 TO NL-ARM-040-ARU1-110338.
11. THE ESTIMATED PIPE JACKING EXCAVATION LENGTHS FOR SOLUTION 2 VARY BASED ON TWO GROUND MODEL CONDITIONS: THE UPPER BOUND AND THE LOWER BOUND. THESE LENGTHS CONSIDER A MAXIMUM ANGULAR JOINT DEFLECTION OF 0.25 DEGREES. FOR MORE DETAILS REFER TO THE FEED REPORT NL-ARM-040-ARU1-110306. THE PRESENTED SWITCHOVER LOCATION(S) SHALL BE FURTHER EVALUATED BY EPC CONTRACTOR DURING DETAILED DESIGN. CURRENT BASE CASE ASSUMPTION IS TO ADOPT SOLUTION 3.

NOTES

1. REFER TO GENERAL TUNNEL NOTES, SEE DRAWING NUMBER NL-ARM-040-ARU1-110322.
2. ALL DIMENSIONS ARE IN METERS (m) UNLESS NOTED OTHERWISE.
3. BATHYMETRY DATA SOURCE: WIP MODIFIED - SUBSAMPLED DATA. BATHY FILES DATED 24/01/2024 FROM THE CLIENT.
4. COORDINATE SYSTEM UTM Z31N ETRS89 AND LEVELS IN LOWEST ASTRONOMICAL TIDE (LATm)
5. SEA WATER LEVEL VARIES AND IS SHOWN INDICATIVELY.
6. SEA DEFENCE GEOMETRY AND LOCATION EXTRACTED FROM CROSS SECTION AA FROM PDF FILE GZ079_419 DATED 24/01/2024 FROM THE CLIENT.
7. TENNET CABLES LOCATIONS ARE BASED ON DATA SOURCE: TENNET CABLE AS BUILT SHAPE FILES DATED 24/01/2024 FROM THE CLIENT.
8. GROUND SURFACE, SEABED AND EXISTING STRUCTURES ARE SHOWN INDICATIVELY AND MUST BE CONFIRMED IN THE DETAILED DESIGN.
9. RECEPTION PIT IS SHOWN INDICATIVELY. SEE DRAWING NUMBERS NL-ARM-040-ARU1-110329 AND NL-ARM-040-ARU1-110330.

LEGEND

- TUNNEL CENTRE LINE
- TUNNEL LINING
- SEA LEVEL
- K CURVE COEFFICIENT
- TENNET CABLES
- LP CURVE LOWER POINT
- Switchover

Rev.	Date
01	19/09/2024
00	21/06/2024

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Tunnel FEED - Tunnel
Plan view and longitudinal profile
hybrid excavation solution

Syst / S-Syst : N/A	Discipline : CIV				
NL-ARM-040-ARU1-110323	Rev:	01	Scale:	1:2500	
NL-ARM-040-ARU1-110323	Format:	A1	Folio:	1 of 1	

Bijlage

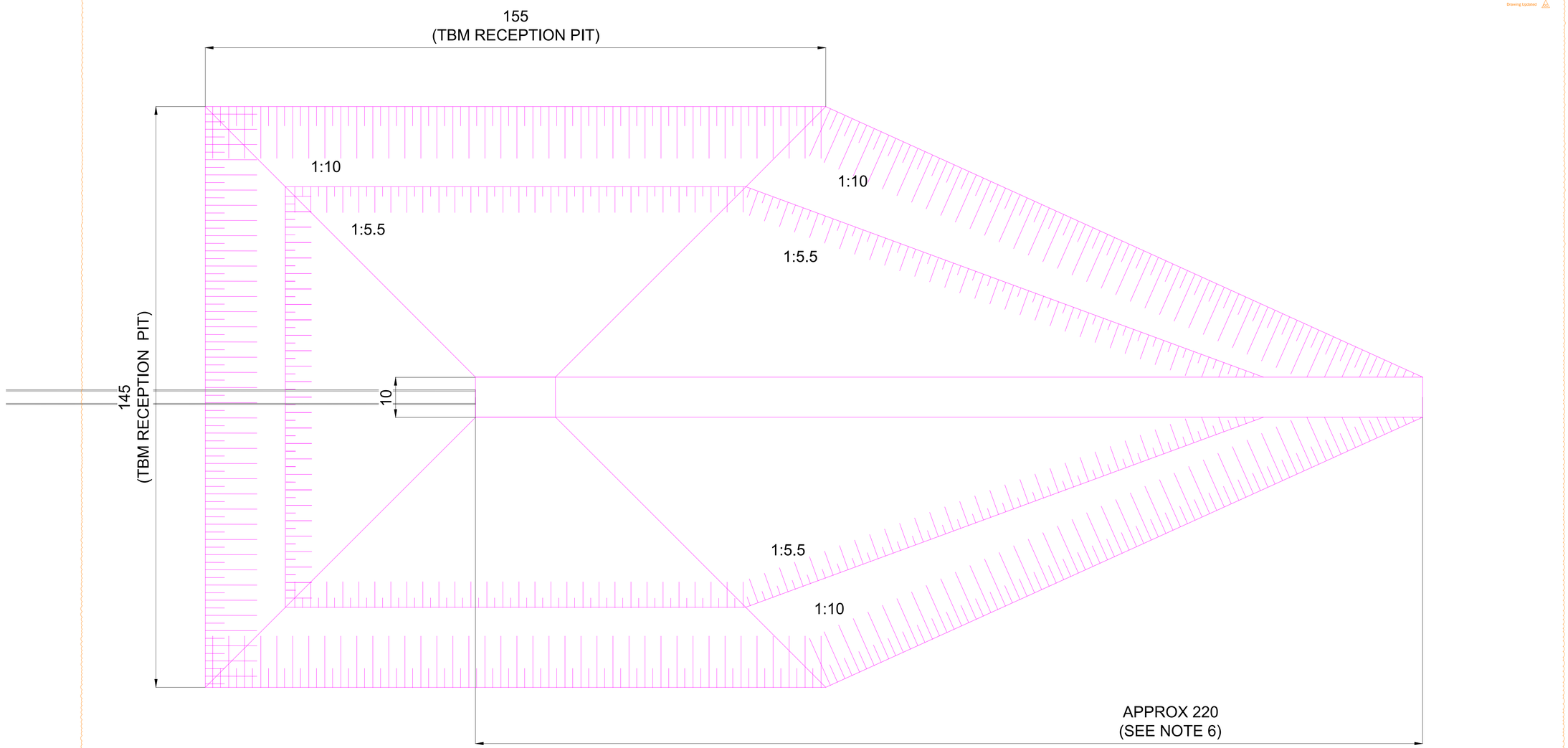
3. Offshore tunnel exit

NL-ARM-040-PTC1-113855_01_IFD_20240820_01

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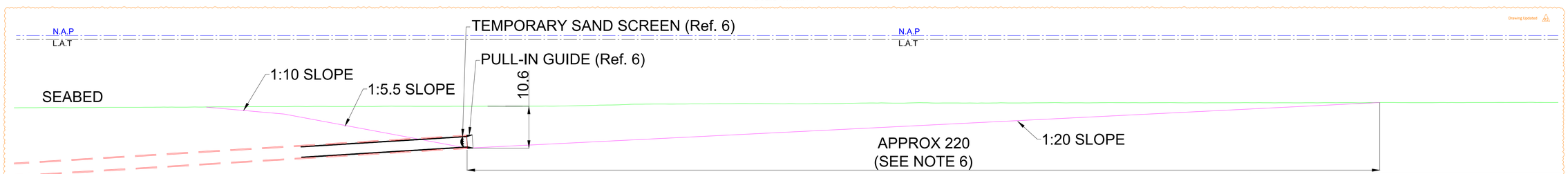
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Drawing Updated



Plan View

Scale 1:550



Side View

Scale 1:550

Notes Updated

NOTES

1. All dimensions are in meters, unless noted otherwise.
2. LAT is 1.03 m below NAP.
3. TBM retrieval reception pit is presented in Ref. 1.
4. Pipeline beyond the pull in trench shall be post trenched, Ref. 4.
5. Borehole data is available in Ref. 5.
6. Seabed levels vary across the reception pit footprint, therefore geometry shown is schematic and approximate.
7. Reference is made to Ref. 2 with regards to sedimentation assessment.
8. The drawing is developed on a base case scenario, where TBM reception pit and Pipeline pull in transition zone are dredged simultaneously.

Legend Updated

LEGEND

- NAP
- LAT
- SEABED LEVEL
- TUNNEL WALLS PLAN/PROFILE
- TBM RECEPTION PIT

Reference Updated

REFERENCE DOCUMENTS

NO	DOCUMENT NUMBER	TITLE
1	NL-ARM-040-ARU1-110329	Tunnel FEED - Tunnel - Reception pit - General Arrangement
2	NL-ARM-040-ARU1-110306	Tunnel FEED Design Report
3	NL-ARM-040-ARU1-110323	Tunnel FEED - Tunnel - Plan view and longitudinal profile
4	NL-ARM-050-PTC1-113905	Alignment sheet - Offshore Pipeline KP0 to KP5
5	NL-ARM-100-ARM1-100013	Aramis Landfall and Nearshore Pipeline Route Measured and Derived Geotechnical Parameters and Final Results
6	NL-ARM-040-PTC1-114088	Offshore Pipeline pull-in Operation Drawings - Temporary Sand Screen at Receiver Pit - General Arrangement

Rev.	Date	Status	Revision Description	Iss.	Check.	Appr.
01	20/08/24		Issued for Design			
00	12/06/24		Issued for Review			
A	21/05/24		Issued for Inter. Discipl. Check			

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Offshore Tunnel Exit Drawings

Reception Pit for Pull-in Operations

Doc Type : DRW	Syst / S-Syst : N/A	Discipline : PLR		Electronic Filename	NL-ARM-040-PTC1-113855.dwg	
Company Doc. N°:	NL-ARM-040-PTC1-113855	Rev:	01	Scale:	1:550	
Contractor Doc. N°:		Format:	A1	Folio:	1 of 1	

Bijlage

4. Shark fin construction site layout

NL-ARM-040-PTC1-110261_03_IFD_20240816_01

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P:\Q2023-107 Petrofac - CCS trunkline FEED\5. Deliverables\01 OI Deliverables\05 WP2\5.1 Shark Fin Construction Site Layout\02 Native files\Rev03\0. IndexSheet.dwg

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REFERENCES

DOCUMENT TITLE	DOCUMENT NUMBER

REVISIONS

Rev.	Date	Status	Revision Description	Iss.	Check.	Appr.
03	16/08/2024	[redacted]	Issued For Design	[redacted]	[redacted]	[redacted]
02	23/07/2024	[redacted]	Issued For Design	[redacted]	[redacted]	[redacted]
01	31/05/2024	[redacted]	Issued for Client Review	[redacted]	[redacted]	[redacted]
00	07/03/2024	[redacted]	Issued for Client Review	[redacted]	[redacted]	[redacted]
A	22/02/2024	[redacted]	Issued for Inter. Discipl. Check	[redacted]	[redacted]	[redacted]

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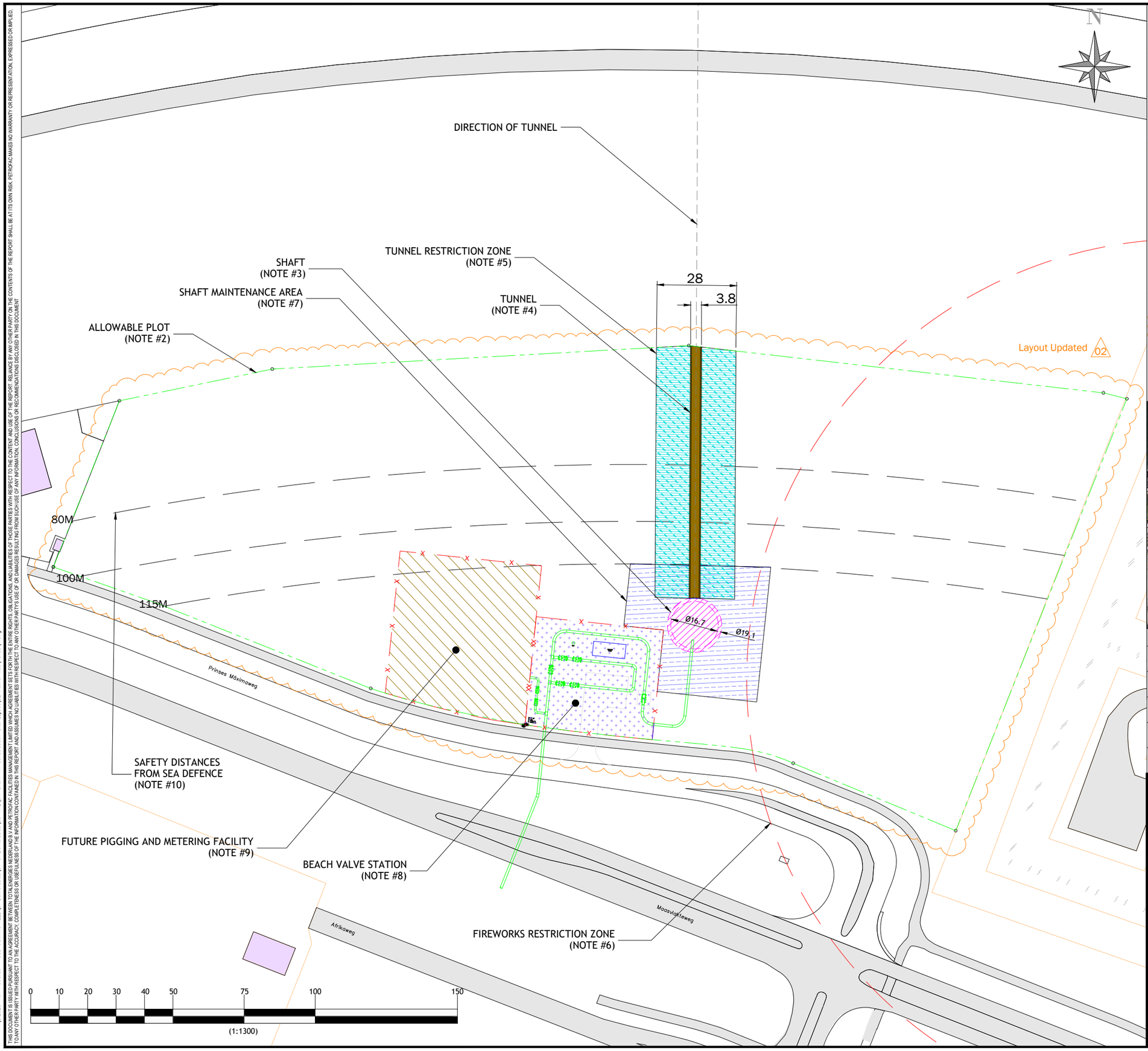
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SHARK FIN CONSTRUCTION SITE LAYOUT
DRAWING INDEX

Doc Type : LAY	Syst / S-Syst : N/A	Discipline : CIV	Electronic Filename: NL-ARM-040-PTC1-110261.dwg
Company Doc. N°:	NL-ARM-040-PTC1-110261	Rev: 03	Scale:
Contractor Doc. N°:		Format: A3	Folio: 0 of 6

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P:\Q02023-107 Petrofac - CCS trunkline FEED\5. Deliverables\01 01 Deliverables\05 WP2\5.1 Shark Fin Construction Site Layout\02 Native files\Rev03\1.dwg



NOTES:

Notes Updated 02

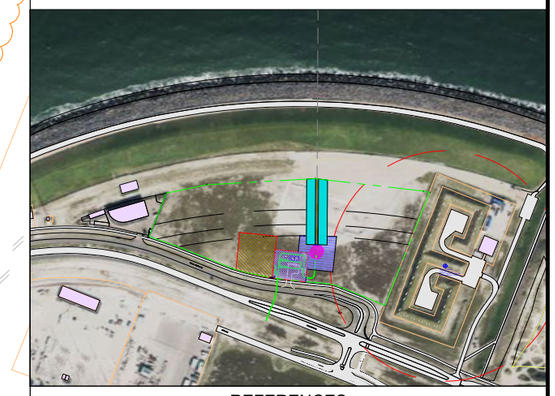
1. All dimensions are in meters, unless noted otherwise. Coordinate system: Onshore - CRS Amersfoort / RD New (EPSG:28992).
2. Available site boundaries as defined in NL-ARM-040-PTC1-TQ0003_Rev2.
3. Shaft OD = 19.1, ID = 16.7m. Ref. #4
4. Tunnel OD = 3.8m, ID = 3.0m. Ref. #4
5. Tunnel restriction zone (area for limited future excavation works) is defined by 14m from the tunnel centerline in each side. Ref. #2
6. Fireworks restriction zone NL-ARM-ARM1_ONPLN-CR_PORITS-000003. Within this designated area, no permanent buildings & temporary offices will be permitted.
7. Shaft maintenance area, extends circa 15m around the shaft. Ref. #2
8. Beach Valve facility circa 1740m². Ref. #3
9. Future pigging and metering facility circa 2695m². Ref. #3
10. Safety Distances from Sea Defence Toe. Ref. #1

LEGEND:

Legend Updated 02

- Available plot
- Tunnel
- Tunnel restriction zone
- Fireworks restriction zone
- Shaft
- Shaft maitenance area
- Future pigging and metering facility
- Beach valve station

KEYPLAN:



REFERENCES

NO.	DOCUMENT TITLE	DOCUMENT NUMBER
1	SAFETY ZONE CALCULATION SEA DEFENCE	NL-ARM-040-PTC1-114043
2	LAYOUT ARRANGEMENT JUSTIFICATION NOTE	NL-ARM-040-PTC1-110262
3	BEACH VALVE STATION PLOT PLAN	NL-ARM-040-PTC1-115028
4	TUNNEL FEED – PRELIMINARY DESIGN REPORT	NL-ARM-040-ARU1-110302

REVISIONS

Rev.	Date	Status	Revision Description	Iss.	Check.	Appr.
02	23/07/2024		Issued For Design			
01	31/05/2024		Issued for Client Review			
00	07/03/2024		Issued for Client Review			
A	22/02/2024		Issued for Inter. Discipl. Check			

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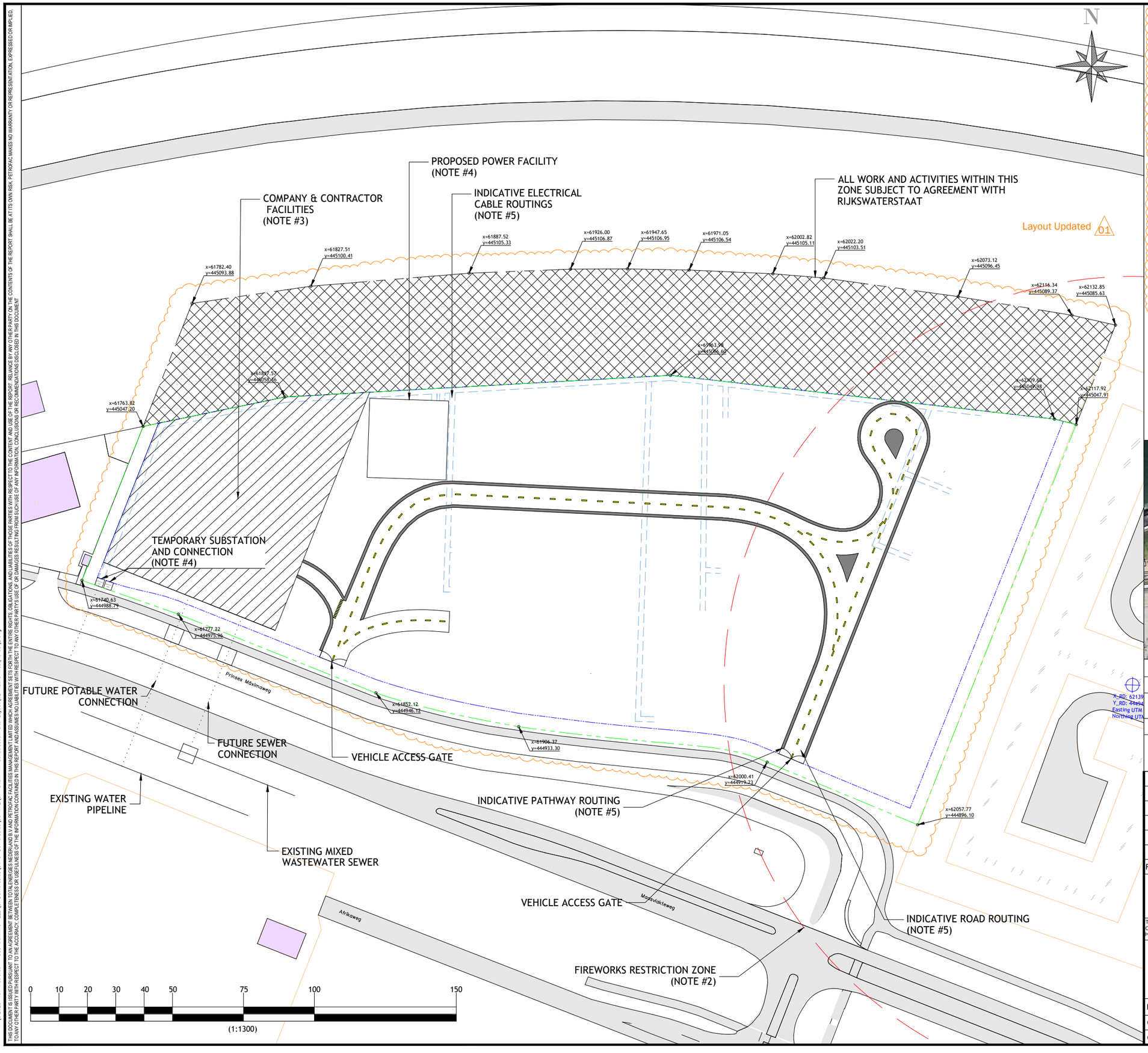
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SHARK FIN CONSTRUCTION SITE LAYOUT
FINAL LAYOUT AFTER CONSTRUCTION AND INITIAL OPERATION

Doc Type : LAY	Syst / S-Syst : N/A	Discipline : CIV	Electronic Filename NL-ARM-040-PTC1-110261.dwg		
Company Doc. N°:	NL-ARM-040-PTC1-110261	Rev:	02	Scale:	1:1300
Contractor Doc. N°:		Format:	A3	Folio:	1 of 6

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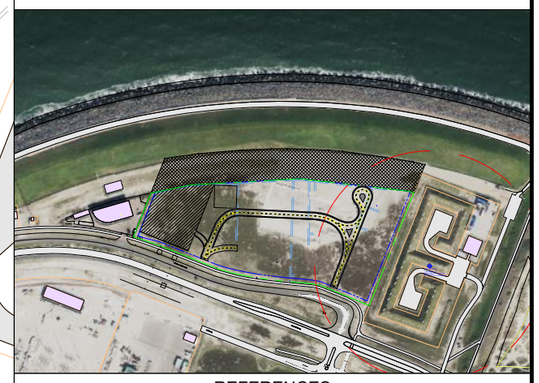
NOTES: Notes Updated 01

1. All dimensions are in meters, unless noted otherwise. Coordinate system: Onshore - CRS Amersfoort / RD New (EPSG:28992).
2. Fireworks restriction zone NL-ARM-ARM1_ONPLN-CR_PORITS-000003. Within this designated area, no permanent buildings & temporary offices will be permitted.
4. Maximum extent of land anticipated for Company and Contractor Facilities, including a Temporary Control Room as described in Ref.#3.
4. Power facility & substations are shown indicatively. Substations are located outside the construction fencing.
5. Electrical cable, road and pathway routings are shown indicatively.
6. Construction equipment & facilities are shown indicatively.
7. Infrastructure interface points are shown indicatively. Ref.#2

LEGEND: Legend Updated 01

- Allowable plot
- Construction fence
- Fireworks restriction zone
- Shaft area
- Indicative electrical cable routing
- Indicative pathway routing
- Company & contractor facilities
- Zone subject to agreement with Rijkswaterstaat

KEYPLAN:



REFERENCES

NO.	DOCUMENT TITLE	DOCUMENT NUMBER
1	LAYOUT ARRANGEMENT JUSTIFICATION NOTE	NL-ARM-040-PTC1-110262
2	SITE PREPARATION LAYOUT	NL-ARM-030-PTC1-110251
3	OFFICES & FACILITIES – FUNCTIONAL DESCRIPTION AND ARCHITECTURAL SPECIFICATIONS	NL-ARM-020-PTC1-110264

REVISIONS

Rev.	Date	Status	Revision Description	Iss.	Check.	Appr.
01	23/07/2024		Issued For Design			
00	31/05/2024		Issued for Client Review			

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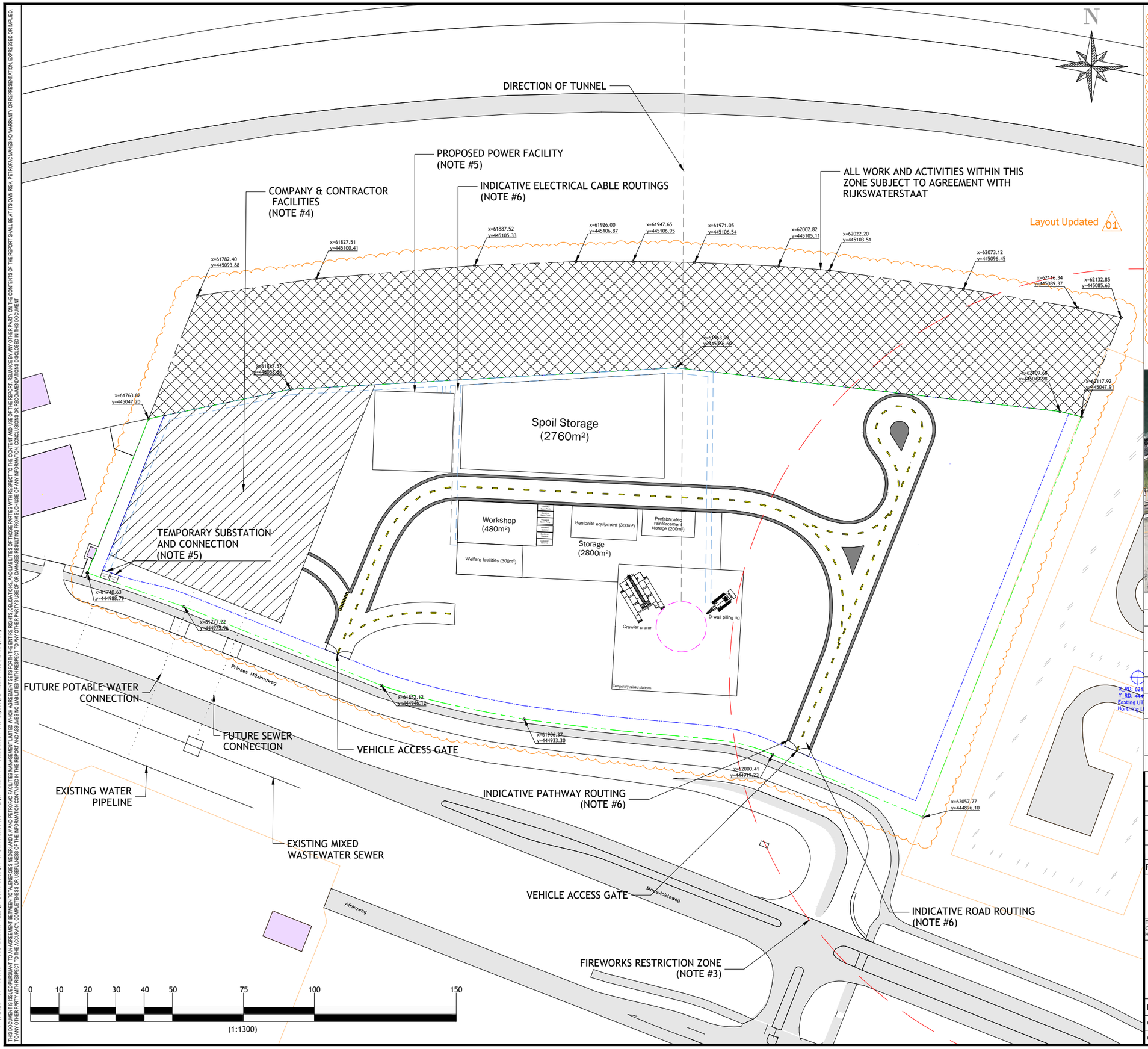
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SHARK FIN CONSTRUCTION SITE LAYOUT
SITE PREPARATION LAYOUT

Doc Type : LAY	Syst / S-Syst : N/A	Discipline : CIV	Electronic Filename NL-ARM-040-PTC1-110261.dwg		
Company Doc. N°:	NL-ARM-040-PTC1-110261	Rev:	01	Scale:	1:1400
Contractor Doc. N°:		Format:	A3	Folio:	2 of 6

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NOTES:

Notes Updated 01

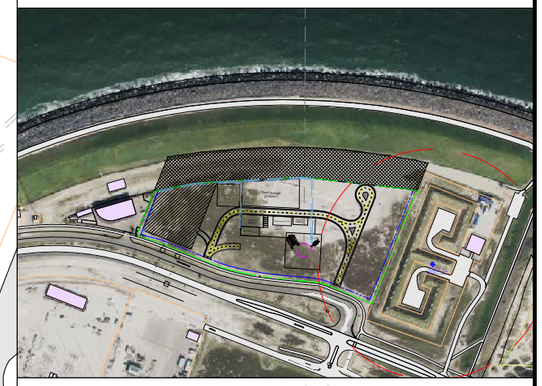
1. All dimensions are in meters, unless noted otherwise. Coordinate system: Onshore - CRS Amersfoort / RD New (EPSG:28992).
2. Shaft OD = 19.1, ID = 16.7m Ref.#2
3. Fire restriction zone NL-ARM-ARM1_ONPLN-CR_PORITS-000003. Within this designated area, no permanent buildings & temporary offices will be permitted.
4. Maximum extent of land anticipated for Company and Contractor Facilities, including a Temporary Control Room as described in Ref.#5
5. Power facility & substations are shown indicatively. Substations are located outside the construction fencing.
6. Electrical cable, road and pathway routings are shown indicatively.
7. Indicative shaft construction layout subject to Tunnel FEED. Ref.#4
8. Construction equipment & facilities are shown indicatively.
9. Infrastructure interface points are shown indicatively. Ref.#6

LEGEND:

Legend Updated 01

- Allowable plot
- Construction fence
- Fireworks restriction zone
- Shaft
- Indicative electrical cable routing
- Indicative pathway routing
- Company & contractor facilities
- Zone subject to agreement with Rijkswaterstaat

KEYPLAN:



REFERENCES

NO.	DOCUMENT TITLE	DOCUMENT NUMBER
1	LAYOUT ARRANGEMENT JUSTIFICATION NOTE	NL-ARM-040-PTC1-110262
2	TUNNEL FEED – PRELIMINARY DESIGN REPORT	NL-ARM-040-ARU1-110302
3	TUNNEL FEED - SITE LAYOUT PLAN	NL-ARM-040-ARU1-110339
4	PRELIMINARY CONSTRUCTION EXECUTION PLAN	NL-ARM-040-ARU1-110315
5	OFFICES & FACILITIES – FUNCTIONAL DESCRIPTION AND ARCHITECTURAL SPECIFICATIONS	NL-ARM-020-PTC1-110264
6	SITE PREPARATION LAYOUT	NL-ARM-030-PTC1-110251

REVISIONS

Rev.	Date	Status	Revision Description	Iss.	Check.	Appr.
01	23/07/2024		Issued For Design			
00	31/05/2024		Issued for Client Review			

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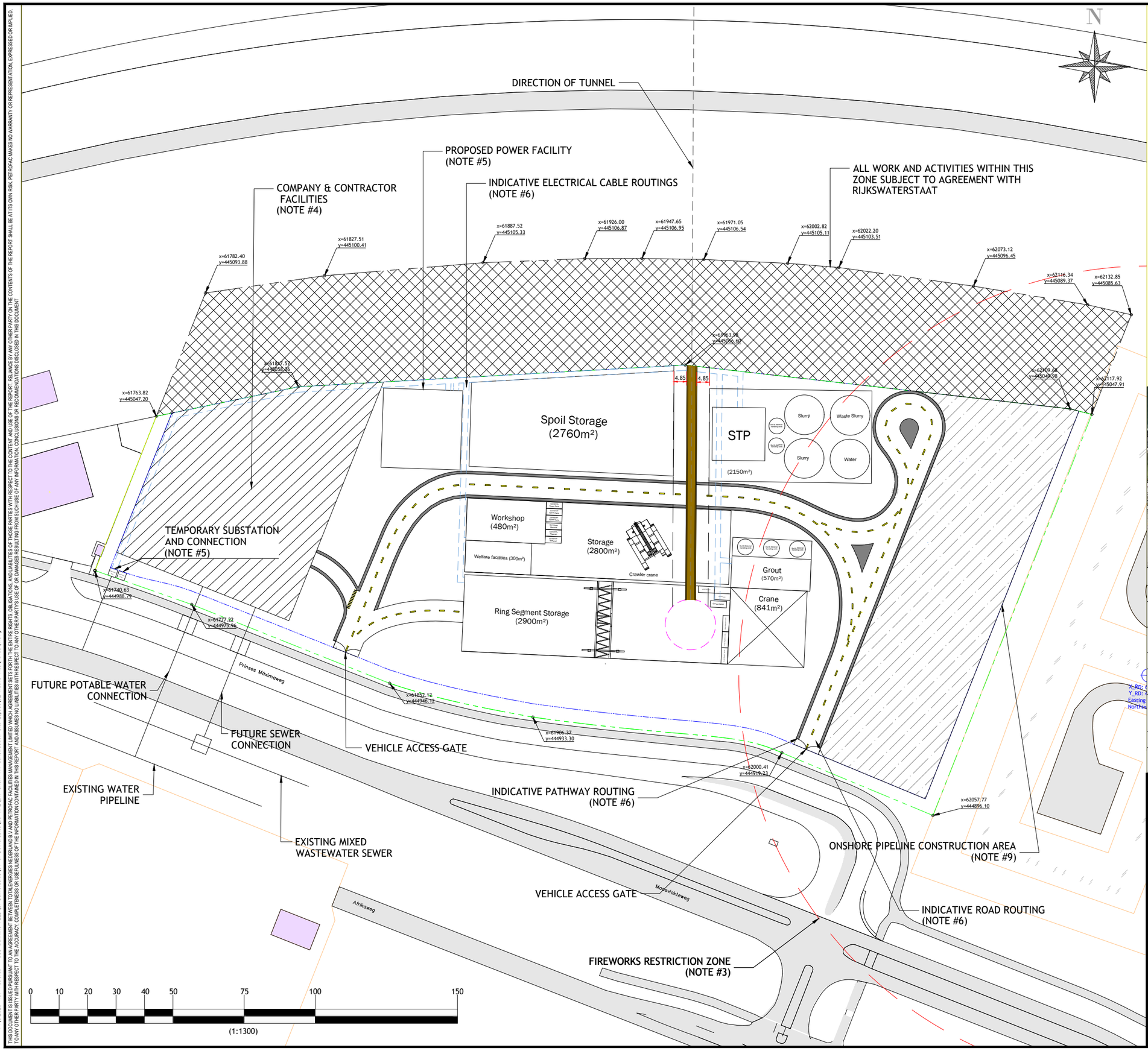
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SHARK FIN CONSTRUCTION SITE LAYOUT
SHAFT CONSTRUCTION LAYOUT

Doc Type : LAY	Syst / S-Syst : N/A	Discipline : CIV	Electronic Filename NL-ARM-040-PTC1-110261.dwg		
Company Doc. N°:	NL-ARM-040-PTC1-110261	Rev:	01	Scale:	1:1400
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NOTES:

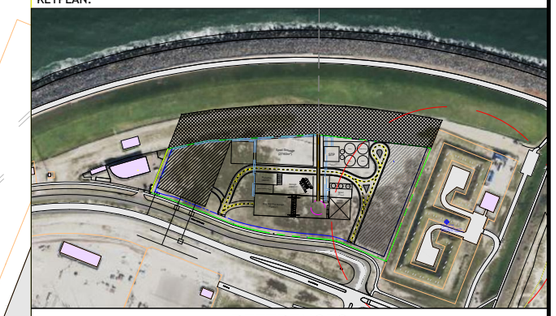
1. All dimensions are in meters, unless noted otherwise. Coordinate system: Onshore - CRS Amersfoort / RD New (EPSG:28992).
2. Shaft OD = 19.1, ID = 16.7m & Tunnel OD = 3.8m, ID = 3.0m Ref.#2.
3. Fire restriction zone NL-ARM-ARM1_ONPLN-CR_PORITS-000003. Within this designated area, no permanent buildings & temporary offices will be permitted.
4. Maximum extent of land anticipated for Company and Contractor Facilities, including a Temporary Control Room as described in Ref.#5.
5. Power facility & substations are shown indicatively. Substations are located outside the construction fencing.
6. Electrical cable, road and pathway routings are shown indicatively.
7. Indicative tunnel construction layout subject to Tunnel FEED. Ref.#4
8. Construction equipment & facilities are shown indicatively.
9. The designated area for the onshore pipeline construction of 6742m2 may be available for a limited period only as this area is currently planned to be cleared and made available for pre-commissioning activities when the offshore pipeline pull in has been completed. Should the Contractor responsible for the onshore pipeline construction require space on the shark fin for a longer period, the Contractor responsible for the overall site layout and operations shall optimize the site layout in collaboration with relevant Contractors to accommodate this request.
10. Infrastructure interface points are shown indicatively. Ref.#6

Note Updated 02

LEGEND:

- Allowable plot
- Construction fence
- Tunnel
- Fireworks restriction zone
- Shaft
- Indicative electrical cable routing
- Indicative pathway routing
- Onshore pipeline construction area
- Company & contractor facilities
- Zone subject to agreement with Rijkswaterstaat

KEYPLAN:



REFERENCES

NO.	DOCUMENT TITLE	DOCUMENT NUMBER
1	LAYOUT ARRANGEMENT JUSTIFICATION NOTE	NL-ARM-040-PTC1-110262
2	TUNNEL FEED – PRELIMINARY DESIGN REPORT	NL-ARM-040-ARU1-110302
3	TUNNEL FEED - SITE LAYOUT PLAN	NL-ARM-040-ARU1-110339
4	PRELIMINARY CONSTRUCTION EXECUTION PLAN	NL-ARM-040-ARU1-110315
5	OFFICES & FACILITIES – FUNCTIONAL DESCRIPTION AND ARCHITECTURAL SPECIFICATIONS	NL-ARM-020-PTC1-110264
6	SITE PREPARATION LAYOUT	NL-ARM-030-PTC1-110251
7	OPTIONAL ONSHORE PIPELINE CONSTRUCTION YARD	NL-ARM-030-PTC1-114065

REVISIONS

Rev.	Date	Status	Revision Description	Iss.	Check.	Appr.
02	16/08/2024		Issued For Design			
01	23/07/2024		Issued For Design			
00	31/05/2024		Issued for Client Review			

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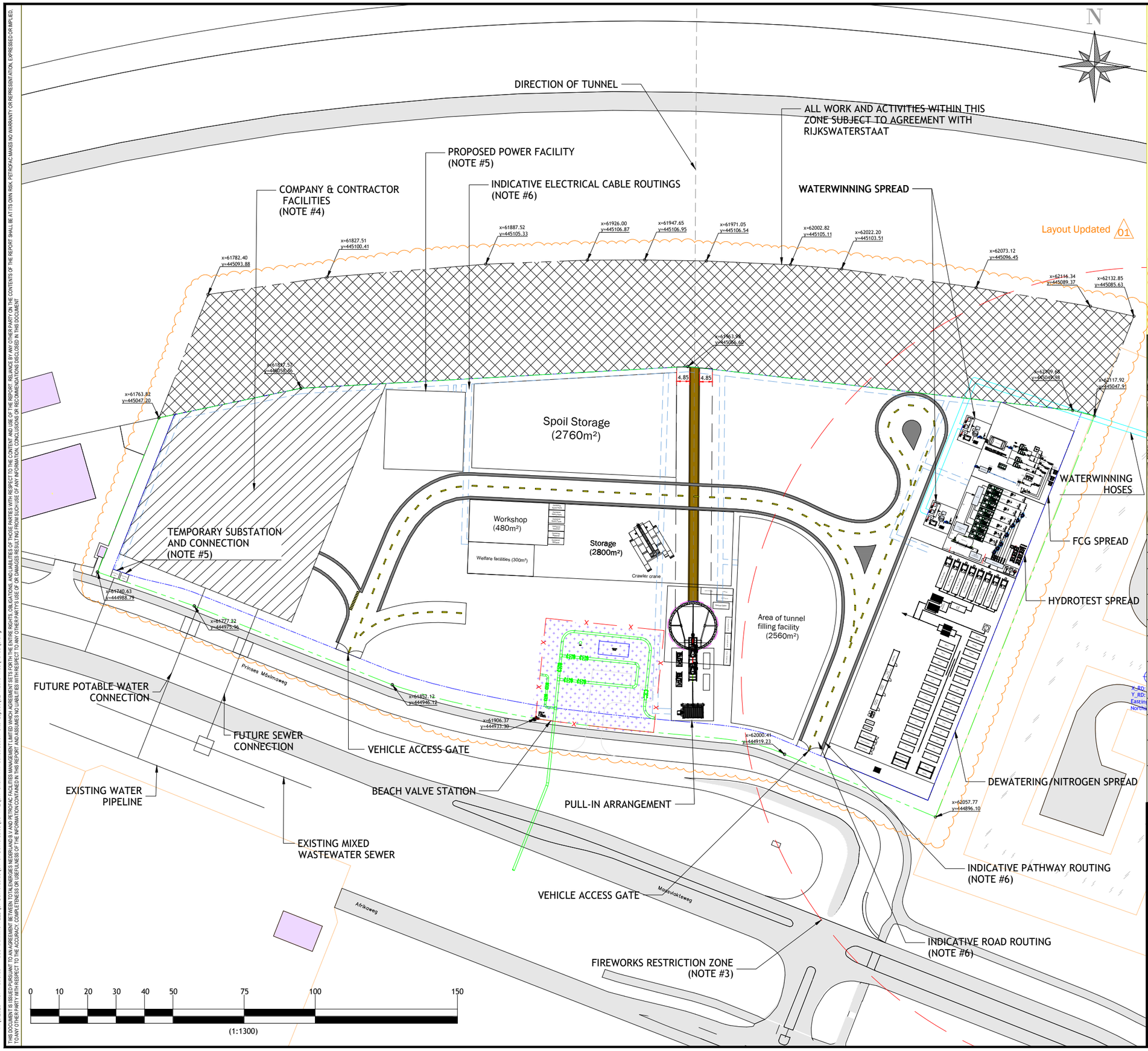
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SHARK FIN CONSTRUCTION SITE LAYOUT
TUNNEL & ONSHORE CONSTRUCTION LAYOUT

Doc Type : LAY	Syst / S-Syst : N/A	Discipline : CIV	Electronic Filename NL-ARM-040-PTC1-110261.dwg		
Company Doc. N°:	NL-ARM-040-PTC1-110261	Rev:	02	Scale:	1:1400
Contractor Doc. N°:		Format:	A3	Folio:	4 of 6

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NOTES:

Notes Updated 01

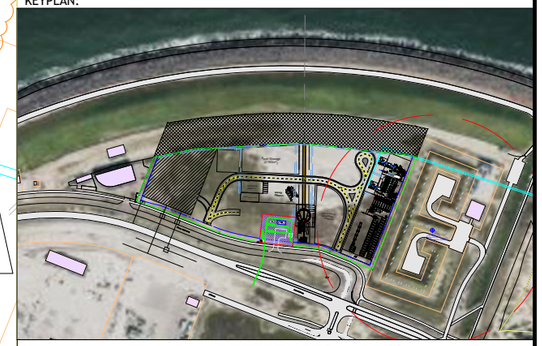
1. All dimensions are in meters, unless noted otherwise. Coordinate system: Onshore - CRS Amersfoort / RD New (EPSG:28992).
2. Shaft OD = 19.1, ID = 16.7m & Tunnel OD = 3.8m, ID = 3.0m Ref.#2
3. Fire restriction zone NL-ARM-ARM1_ONPLN-CR_PORITS-000003. Within this designated area, no permanent buildings & temporary offices will be permitted.
4. Maximum extent of land anticipated for Company and Contractor Facilities, including a Temporary Control Room as described in Ref.#5
5. Power facility & substations are shown indicatively. Substations are located outside the construction fencing.
6. Electrical cable, road and pathway routings are shown indicatively.
7. Construction equipment & facilities are shown indicatively.
8. Infrastructure interface points are shown indicatively. Ref.#6.

LEGEND:

Legend Updated 01

- Allowable plot
- Construction fence
- Tunnel
- Fireworks restriction zone
- Shaft
- Indicative electrical cable routing
- Indicative pathway routing
- Beach valve station
- Company & contractor facilities
- Zone subject to agreement with Rijkswaterstaat
- Waterwinning hoses

KEYPLAN:



REFERENCES

NO.	DOCUMENT TITLE	DOCUMENT NUMBER
1	LAYOUT ARRANGEMENT JUSTIFICATION NOTE	NL-ARM-040-PTC1-110262
2	TUNNEL FEED – PRELIMINARY DESIGN REPORT	NL-ARM-040-ARU1-110302
3	TUNNEL FEED - SITE LAYOUT PLAN	NL-ARM-040-ARU1-110339
4	PRELIMINARY CONSTRUCTION EXECUTION PLAN	NL-ARM-040-ARU1-110315
5	OFFICES & FACILITIES – FUNCTIONAL DESCRIPTION AND ARCHITECTURAL SPECIFICATIONS	NL-ARM-020-PTC1-110264
6	SITE PREPARATION LAYOUT	NL-ARM-030-PTC1-110251
7	OPTIONAL ONSHORE PIPELINE CONSTRUCTION YARD	NL-ARM-030-PTC1-114065
8	CONSTRUCTION CONCEPT SOLUTION TECHNICAL NOTE	NL-ARM-030-PTC1-114070
9	PRE-COMMISSIONING AND COMMISSIONING PHILOSOPHY ONSHORE PIPELINE	NL-ARM-040-PTC1-110501

REVISIONS

Rev.	Date	Status	Revision Description	Iss.	Check.	Appr.
01	23/07/2024		Issued For Design			
00	31/05/2024		Issued for Client Review			

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SHARK FIN CONSTRUCTION SITE LAYOUT

TUNNEL END ACTIVITIES, BEACH VALVE, LANDFALL PULL-IN, TUNNEL FILL, PRE-COMMISSIONING

Doc Type : LAY	Syst / S-Syst : N/A	Discipline : CIV	Electronic Filename NL-ARM-040-PTC1-110261.dwg		
Company Doc. N°:	NL-ARM-040-PTC1-110261	Rev:	01	Scale:	1:1400
Contractor Doc. N°:		Format:	A3	Folio:	5 of 6

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DIRECTION OF TUNNEL

ALL WORK AND ACTIVITIES WITHIN THIS ZONE SUBJECT TO AGREEMENT WITH RIJKSWATERSTAAT

PROPOSED POWER FACILITY (NOTE #5)

INDICATIVE ELECTRICAL CABLE ROUTINGS (NOTE #6)

COMPANY & CONTRACTOR FACILITIES (NOTE #4)

WATERWINNING SPREAD (NOTE #10)

Layout Updated 01

x=61782.40 y=445093.88
x=61827.51 y=445100.41
x=61887.52 y=445105.33
x=61926.00 y=445106.87
x=61947.65 y=445106.95
x=61971.05 y=445106.54
x=62002.82 y=445105.11
x=62022.20 y=445103.51
x=62073.12 y=445096.45
x=62116.34 y=445089.37
x=62132.85 y=445085.63
x=61963.98 y=445056.60
x=61897.57 y=446058.46
x=61763.82 y=445047.20
x=62109.08 y=445049.98
x=62117.92 y=445047.91

TEMPORARY SUBSTATION AND CONNECTION (NOTE #5)

WATERWINNING HOSES

FCG SPREAD (NOTE #7)

HYDROTEST SPREAD (NOTE #8)

x=61240.63 y=444988.79
x=61277.22 y=444975.96
x=61852.12 y=444946.12
x=61906.37 y=444933.30
x=62000.41 y=444919.23
x=62057.77 y=444896.10

Prinses Máximoweg

FUTURE POTABLE WATER CONNECTION

FUTURE SEWER CONNECTION

VEHICLE ACCESS GATE

PIG LAUNCHER SPREAD (NOTE #12)

BEACH VALVE STATION (NOTE #11)

DEWATERING / NITROGEN SPREAD (NOTE #9)

EXISTING WATER PIPELINE

EXISTING MIXED WASTEWATER SEWER

INDICATIVE PATHWAY ROUTING (NOTE #6)

VEHICLE ACCESS GATE

Mosselvlaketeweg

Afrikaweg

INDICATIVE ROAD ROUTING (NOTE #6)

FIREWORKS RESTRICTION ZONE (NOTE #3)

0 10 20 30 40 50 75 100 150

(1:1300)

N

NOTES:

Notes Updated 01

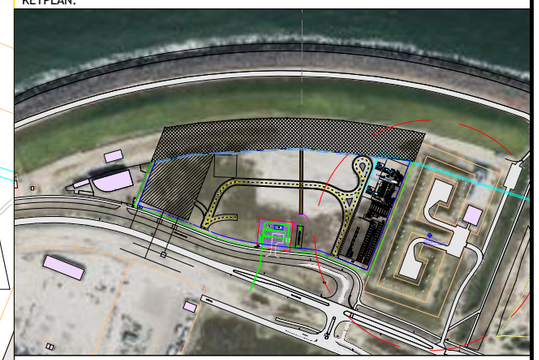
1. All dimensions are in meters, unless noted otherwise. Coordinate system: Onshore - CRS Amersfoort / RD New (EPSG:28992).
2. Shaft OD = 19.1, ID = 16.7m & Tunnel OD = 3.8m, ID = 3.0m Ref.#2.
3. Fire restriction zone NL-ARM-ARM1_ONPLN-CR_PORITS-000003. Within this designated area, no permanent buildings & temporary offices will be permitted.
4. Maximum extent of land anticipated for Company and Contractor Facilities, including a Temporary Control Room as described in Ref.#5.
5. Power facility & substations are shown indicatively. Substations are located outside the construction fencing.
6. Electrical cable, road and pathway routings are shown indicatively.
7. FCG spread area occupation is 30x30m. Ref.#8.
8. Hydrotest spread area occupation is 30x30m. Ref.#8.
9. Dewatering/Drying spread area occupation is 42.5x90m. Ref.#8.
10. Waterwinning spread area occupation is 10x10m. Ref.#8.
11. Beach Valve facility circa 1740m². Ref.#7
12. Pig launcher spread area occupation is 8x30m. Ref.#8.
13. Construction equipment & facilities are shown indicatively.
14. Infrastructure interface points are shown indicatively, Ref.#6.

LEGEND:

Legend Updated 01

- Allowable plot
- Construction fence
- Fireworks restriction zone
- Indicative electrical cable routing
- Company & contractor facilities
- Zone subject to agreement with Rijkswaterstaat
- Tunnel
- Shaft
- Indicative pathway routing
- Beach valve station
- Waterwinning hoses

KEYPLAN:



REFERENCES

NO.	DOCUMENT TITLE	DOCUMENT NUMBER
1	LAYOUT ARRANGEMENT JUSTIFICATION NOTE	NL-ARM-040-PTC1-110262
2	TUNNEL FEED – PRELIMINARY DESIGN REPORT	NL-ARM-040-ARU1-110302
3	TUNNEL FEED - SITE LAYOUT PLAN	NL-ARM-040-ARU1-110339
4	PRELIMINARY CONSTRUCTION EXECUTION PLAN	NL-ARM-040-ARU1-110315
5	OFFICES & FACILITIES – FUNCTIONAL DESCRIPTION AND ARCHITECTURAL SPECIFICATIONS	NL-ARM-020-PTC1-110264
6	SITE PREPARATION LAYOUT	NL-ARM-030-PTC1-110251
7	BEACH VALVE STATION PLOT PLAN	NL-ARM-040-PTC1-115028
8	PRE-COMMISSIONING AND COMMISSIONING PHILOSOPHY EPCC OFFSHORE PIPELINE, ILT'S, RISER AND PIG RECEIVER DHUB	NL-ARM-040-PTC1-100504

REVISIONS

Rev.	Date	Status	Revision Description	Iss.	Check.	Appr.
01	23/07/2024		Issued For Design			
00	31/05/2024		Issued for Client Review			

ARAMIS

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SHARK FIN CONSTRUCTION SITE LAYOUT
REMAINING PRE-COMMISSIONING LAYOUT

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Bijlage

5. Toetsing RCE

Royal HaskoningDHV

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Onze referentie

Aantal Bijlagen
-

Datum 21 augustus 2023
Betreft 22A030-01 Geofysisch conceptrapport Aramis

Geachte mevrouw [redacted]

Hierbij deel ik mede dat de Rijksdienst voor het Cultureel Erfgoed (RCE) het conceptrapport Aramis geofysisch van d.d. 03-08-2023 met referentienummer 22A030-01 goedkeurt.

Dit advies is verzorgd door:

- [redacted]

Mochten er naar aanleiding van dit advies nog vragen zijn, neem dan gerust contact op via de contactgegevens aan de rechterzijde.

Met vriendelijke groet,

[redacted]
Rijksdienst voor het Cultureel Erfgoed

Bijlage

6. Design Report

NL-ARM-040-ARU1-110306_02_IFD_20241021_01

Royal HaskoningDHV

Tunnel FEED – Design Report				**Document Reference** NL-ARM-040-ARU1-110306
				Revision: 02 / **Step**: [redacted]
				Date: 21/10/2024
Document Type: REP	**Family Equip:** n/a	**Discipline:** CIV	**Class:** 1	**Page** 1 of 154

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Rev	Step	Date	Revision memo	Issued by	Checked by	Approved by
00	[redacted]	04/07/2024	Issued for Review	Various	[redacted]	[redacted]
01	[redacted]	27/09/2024	Issued for Approval	Various	[redacted]	[redacted]
02	[redacted]	21/10/2024	Issued for Design	Various	[redacted]	[redacted]

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ABBREVIATIONS

Abbreviation	Description
BOD	Basis of Design
CHI	Cutterhead Interventions
GBR	Geotechnical Baseline Report
GDR	Geotechnical Design Report
GIR	Ground Investigation Report
FEED	Front End Engineering Design
HI	Hyperbaric Interventions
ID	Internal Diameter
LB	Lower Bound
LM	Lower Mean
OD	Outer Diameter
PCTL	Precast Concrete Tunnel Lining
PDR	Preliminary Design Report
RWS	Rijkswaterstaat (Ministerie van Infrastuctuur en Waterstaat)
SFRC	Steel Fibre Reinforced Concrete
TBM	Tunnel Boring Machine
TSHD	Trailing Suction Hopper Dredger
UM	Upper Mean
UB	Upper Bound

DEFINITIONS

Terminology	Definition
Company	Aramis, a joint development by TotalEnergies, Shell Netherlands, Energie Beheer Nederland (EBN) and NV Nederlandse Gasunie.
Aramis	Refers to assets owned and operated by Aramis
Project	Aramis Project consisting large-scale infrastructure to transport CO2 captured by other industrial enterprises and to deliver this offshore to depleted gas fields in which the CO2 will be stored permanently.
FEED Contractor	Petrofac Facilities Management Ltd.
EPC Contractor	Party or organisation selected by the Company to execute the detailed engineering, procurement (supply) and construction (and commissioning) phase of the project.
Supplier	Party or organisation that manufactures, supplies equipment packages and/or services to perform duties specified by the Company, FEED Contractor or EPC Contractor.
Shall	Is used to indicate a mandatory requirement.
Should	Is used to indicate that the provision is preferred.
May	Is used to indicate alternatives, which are equally acceptable, at Contractor discretion.
Must	Is used to indicate compulsory, legally binding requirement.

HOLDS

No.	Section	Hold
-	-	No holds identified

1 INTRODUCTION

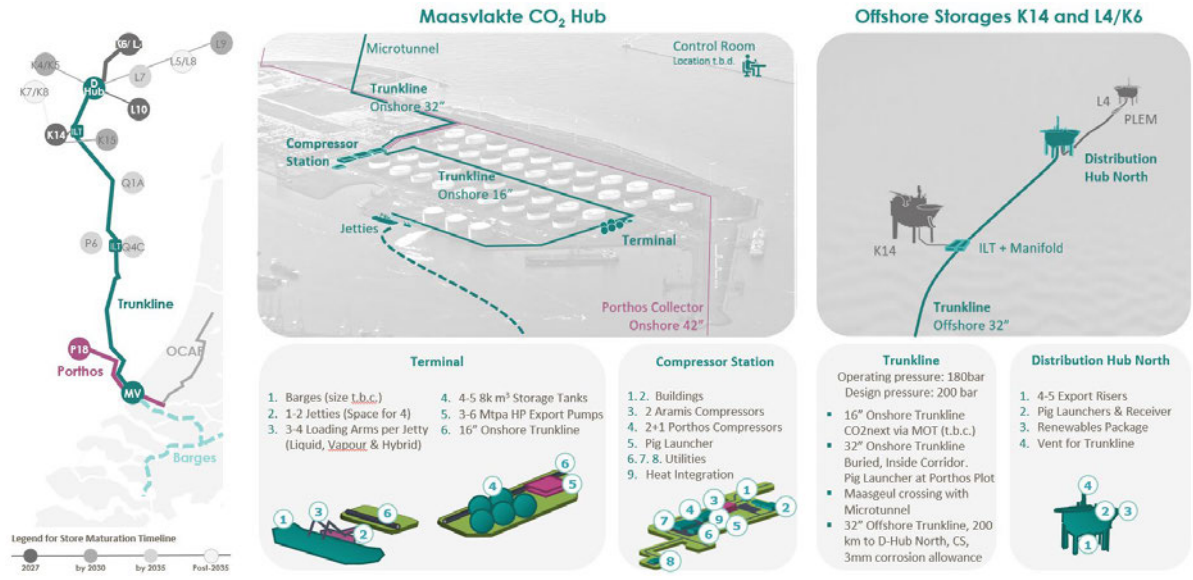
1.1 Background

The Aramis project is a cooperation between Shell, TotalEnergies, Gasunie and EBN to utilise depleted gas fields and potentially aquifers in the Dutch North Sea to establish an open-access CCS infrastructure for transport and storage, targeting hard-to abate industrial emissions in the Netherlands and Europe.

This part of the project provides the transport of CO_2 from the onshore terminal to an offshore platform, which is used as distribution hub (D-Hub). Other parts of the project are provided by CO2next and Porthos, while the actual storage in the depleted offshore gas fields under the North Sea is performed by Shell and TotalEnergies as part of their offshore operations.

In Figure 1-1 below an overview of the envisaged CCS infrastructure is given. The left panel illustrates the trunkline that transports the CO_2 from the Maasvlakte to the offshore storage. The middle panels illustrate the Maasvlakte Hub with the Porthos compressor station and the CO2next Terminal. The right panel shows the platforms of TTE and Shell.

Figure 1-1 - Overview of CCS Infrastructure



1.2 Purpose of Document

This report summarises the design verifications undertaken for the Aramis Tunnel FEED design comprising shaft, tunnel, reception pit and TBM retrieval method. A high-level summary of the design is provided in Section 2, which also directs the reader to other deliverables (including the basis of design and associated drawings). In case of any inconsistency between documents, the more stringent requirement takes precedence and should be considered during EPC stage.

2 SUMMARY OF DESIGN

2.1 Basis of design

FEED design has been undertaken in accordance with the Tunnel FEED Basis of Design NL-ARM-040-ARU1-110301 (Rev 02 Issued for Design being current at time of issue) [1]. The design has been developed based on the preliminary design documented in the Tunnel FEED Preliminary Design Report NL-ARM-040-ARU1-110302 [2] and the preferred alignment shown in the preliminary design tunnel alignment NL-ARM-040-ARU1-110321.General description of FEED design

The design described in the report comprises the following key components:

- 16.7m internal diameter circular diaphragm wall launch shaft with concrete base slab, capping beam, and tension piles.
- 3.0m internal diameter tunnel, constructed partly using concrete pipes (with pipe-jack technique), with a switch to segmental lining technique prior to arriving at the reception pit.
- Reception pit constructed as an open underwater dredged excavation to extract the Tunnel Boring Machine (TBM) prior to pipeline installation. With modifications to the completed tunnel lining required prior to dredging in order to provide additional tunnel strengthening and stability.

It is anticipated that during EPC tender (or following EPC award) that the EPC Contractor will develop the detailed design in consideration of their own preferences and assessment of risks, for example as may be influenced by their own existing in-house capabilities, supply chain, preferred working practices, etc. For EPC detailed design, it is recommended that consideration is given to local CUR guidelines as relevant to the design, for example CUR-231 (Handbook for diaphragm walls), CUR-077 (Rules for unreinforced underwater concrete), CUR-236 (Micropiles).

2.2 Drawings

This design is presented in drawings (Table 2-1), which should be read alongside this report.

Table 2-1 – FEED drawings

Drawing number	Drawing title
NL-ARM-040-ARU1-110323	Tunnel FEED - Tunnel - Plan view and longitudinal profile - hybrid excavation
NL-ARM-040-ARU1-110324	Tunnel FEED - Tunnel - Typical cross section geometry
NL-ARM-040-ARU1-110325	Tunnel FEED - Tunnel - Jacking pipe
NL-ARM-040-ARU1-110326	Tunnel FEED - Tunnel - Segmental lining - ring arrangement
NL-ARM-040-ARU1-110327	Tunnel FEED - Tunnel - Segmental lining - segments details
NL-ARM-040-ARU1-110328	Tunnel FEED - Tunnel - Construction Sequence (method switchover)
NL-ARM-040-ARU1-110329	Tunnel FEED - Tunnel - Reception pit - General Arrangement
NL-ARM-040-ARU1-110330	Tunnel FEED - Tunnel - Reception pit - Construction Sequence (TBM retrieval)
NL-ARM-040-ARU1-110332	Tunnel FEED - Shaft - Construction sequence

Drawing number	Drawing title
NL-ARM-040-ARU1-110333	Tunnel FEED - Shaft - Section A -A (vertical section through the tunnel)
NL-ARM-040-ARU1-110334	Tunnel FEED - Shaft - Section B -B (vertical section perpendicular to A-A)
NL-ARM-040-ARU1-110335	Tunnel FEED - Shaft - Plan at Ground Level
NL-ARM-040-ARU1-110336	Tunnel FEED - Shaft - Plan Section at -1.000m LAT
NL-ARM-040-ARU1-110337	Tunnel FEED - Shaft - Plan Section at Tunnel Opening
NL-ARM-040-ARU1-110338	Tunnel FEED - Shaft - Plan at Base Slab including Pile Layout

2.3 Other design documents

Further to the drawings listed in section 2.2, this report should be read in consideration of the other Tunnel FEED deliverables listed in Table 2-2.

Table 2-2 – Other FEED deliverables

Drawing number	Drawing title
NL-ARM-040-ARU1-110301	Tunnel FEED - Basis of Design Report
NL-ARM-040-ARU1-110302	Tunnel FEED - Preliminary Design Report
NL-ARM-040-ARU1-110303	Tunnel FEED - Ground Investigation Report
NL-ARM-040-ARU1-110305	Tunnel FEED - Geotechnical Design Report
NL-ARM-040-ARU1-110307	Tunnel FEED - Geotechnical Baseline Report
NL-ARM-040-ARU1-110308	Tunnel FEED - Geological Long Section
NL-ARM-040-ARU1-110309	Tunnel FEED - Phase 1 Contaminated Land Risk Assessment
NL-ARM-040-ARU1-110311	Tunnel FEED - Risk Assessment - Environmental, Health and Safety
NL-ARM-040-ARU1-110312	Tunnel FEED - Risk Assessment - Schedule and Cost
NL-ARM-040-ARU1-110315	Tunnel FEED - Preliminary Construction Execution Plan
NL-ARM-040-ARU1-110316	Tunnel FEED - Tunnelling and Shaft Sinking Specification for EPC
NL-ARM-040-ARU1-110317	Tunnel FEED - TBM Specification
NL-ARM-040-ARU1-110318	Tunnel FEED - Preliminary Design - Alignment Plan and Profile
NL-ARM-040-ARU1-110319	Tunnel FEED - Preliminary Design - Shaft spaceproofing
NL-ARM-040-ARU1-110320	Tunnel FEED - Preliminary Design - Tunnel spaceproofing
NL-ARM-040-ARU1-110321	Tunnel FEED - Preliminary Design - Alignment Plan and Profile (Extension)
NL-ARM-040-ARU1-110381	Tunnel FEED - Settlement Impact Analysis and Monitoring Plan - Sea Defence
NL-ARM-040-ARU1-110382	Tunnel FEED - Settlement Impact Analysis and Monitoring Plan - TenneT Cables

3 GEOTECHNICAL AND GEOENVIRONMENTAL

Geotechnical information is presented in three standalone reports. To avoid duplication of information sources, readers are referred to the following reports, with a description of the contents of each report included in the following sections 3.1 to 3.3.

- Ground Investigation Report [GIR] (NL-ARM-040-ARU1-110303).
- Geotechnical Design Report [GDR] (NL-ARM-040-ARU1-110305).
- Geotechnical Baseline Report [GBR] (NL-ARM-040-ARU1-110307).

In addition to the above a geological long section presenting the anticipated ground conditions has been prepared for the proposed shaft, tunnel and reception pit alignment.

- Plan and Geological Profile (NL-ARM-040-ARU1-110308)

Geoenvironmental (Contaminated Land Risk Assessment) information is presented in a separate standalone report.

- Contaminated Land Risk Assessment (NL-ARM-040-ARU1-110309)

3.1 Ground Investigation Report (GIR)

The GIR summarises geotechnical and geological information provided for FEED design and provides characteristic geotechnical parameters for use in design. Parameters are limited to fundamental geotechnical properties of the ground strata, and do not include specific design parameters which are included in the GDR.

To understand the sensitivity of designs to inherent soil variation, four characteristic soil profiles have been developed:

- LB (Lower Bound): An estimate of the weakest ground properties that could apply over a localised zone of approximately 3 m (i.e. a tunnel diameter).
- LM (Lower Mean): An estimate of the mean which incorporates a margin of uncertainty towards lower-strength soils, i.e. a weaker estimate of mean soil properties for the mass of ground as a whole.
- UM (Upper Mean): An estimate of the mean which incorporates a margin of uncertainty towards higher-strength soils, i.e. a stronger estimate of mean soil properties for the mass of ground as a whole.
- UB (Upper Bound): An estimate of the strongest ground properties that could apply over a localised zone of about 3 m (i.e. a tunnel diameter).

The selection of characteristic ground profiles and parameters for design and the rationale for the section will be detailed in the Geotechnical Design Report for each design situation (i.e. shaft design, tunnel design and recovery pit design); in some cases, stronger soils may be more onerous than weaker soils.

Note that ground investigation data used to produce the GIR was scoped, specified (including Laboratory testing requirements) and supervised by others.

3.2 Geotechnical Design Report (GDR)

The purpose of this document is to record the design methodologies and design assumptions for the FEED design as part of the Aramis onshore and nearshore works at the Maasvlakte hub. Available relevant data, geotechnical and geochemical parameters and material properties are summarised therein.

Geotechnical parameters, assumptions, and calculations to support FEED design of the following assets are covered within this report:

- Launch shaft: 16.7m ID, 1.2m thick cast in-situ concrete D-wall panels,
- Pipe jacked and segmental mTBM tunnels: 3m ID with precast concrete tunnel rings.
- Reception pit: underwater dredged slopes into native material for recovery of mTBM and connection to trunkline.

Reference in the GDR is made to the following documents:

- GIR: NL-ARM-040-ARU1-110303
- Tunnel FEED Basis of Design: NL-ARM-040-ARU1-110301

The design life of the infrastructure is taken as 30 years as per the Tunnel FEED Basis of Design report. Where an increased design life of 100 years is expected to significantly influence design, e.g. on issues of durability, soil behaviour, return periods, comment has been provided on the impact.

3.3 Geotechnical Baseline Report (GBR)

This purpose of this Geotechnical Baseline Report (GBR) is:

- To describe the site geology and sub-surface conditions along the project proposed alignment,
- To assist prospective contract tenderers in their evaluation of the sub-surface conditions for construction of the project,
- To assist Company in assessing tendered submissions,
- To establish sub-surface baseline conditions to be used as the only basis for evaluating changed sub-surface conditions.

3.4 Phase 1 Contaminated Land Risk Assessment

This document is a review of historic data from previous ground investigations, geological literature, and other publicly available data. The aim is to provide a review of the geoenvironmental conditions present within the soil and groundwater and provide an initial assessment of risks from the ground to proposed design and construction of the onshore and nearshore infrastructure.

The scope of the preliminary contamination risk assessment comprises:

- A review of desk-based information collated for the site relating to historical and current uses,

- Outline the local geology, hydrogeology and hydrology and provide an appraisal of the environmental setting and sensitivity,
- Consideration of relevant information (e.g. ground investigation) and details of the proposed works,
- Development of a conceptual site model and risk assessment, and provide an assessment of the potential for ground contamination and the nature of any potential contamination sources; and
- Recommendations, including the requirement for further assessment and mitigation.

4 MATERIALS

This section summarises the material properties used for the design of the shaft and tunnel structures. Properties have been established following the Tunnel FEED Basis of Design [1].

4.1 Material Properties

4.1.1 Concrete – General

The concrete mix design shall comply with the materials and durability requirements of the relevant codes and project standards (such as Dutch Eurocodes 2: NEN EN-1992, and NEN-EN 206 Standards). The cast-in-place concrete and PCTL watertightness, exposure, and design chemical classes comply with the materials and durability requirements described in section 11.

4.1.2 Shaft – Concrete

Concrete properties for the shaft are listed in Table 4-1.

Table 4-1 – Concrete properties for shaft

Property	Symbol	Unit	Minimum Value
Strength Grade	-	-	C40/50
Unit weight	γ	kg/m^3	2400
Characteristic compressive strength, cylinder	f_{ck}	MPa	40
Characteristic tensile strength	$f_{ctk0.05}$	MPa	2.46
Mean secant elastic modulus (short term)	E_{cm}	MPa	35220
Mean secant design elastic modulus (long term)	$E_{cd,ef}$	MPa	13341
Poisson's ratio	n	-	0.2
Concrete material factor	γ_c	-	1.5

4.1.3 Tunnel – Concrete and Fibre Reinforcement

The minimum performance requirements in terms of strength of concrete for the segmental lining are presented in Table 4-2. In the segmental lining, steel fibres are added to the concrete to improve crack resistance, resistance to bursting effects, enhanced load-bearing capacity and reduction of conventional reinforcement. The fibres shall also meet the minimum requirements defined in NEN EN 14889-1, Group 1 Table ZA.2 – system of attestation of conformity. Only steel fibres controlled and certified in accordance with "System 1" and bearing the corresponding EC certificate of conformity shall be used.

The minimum performance requirements in terms of strength of concrete for the precast pipe-jack lining sections are presented in Table 4-3. Contentional reinforcement (no steel fibres) is proposed for use in the precast concrete pipejack section.

Table 4-2 – Concrete and fibre properties for tunnel segments

Property	Symbol	Unit	Minimum Value
Strength Grade	-	-	C50/60
Unit weight	γ	kg/m^3	2400
Characteristic compressive strength, cylinder	f_{ck}	MPa	50
Characteristic tensile strength	$f_{ctk0.05}$	MPa	2.85
Mean secant elastic modulus (short term)	E_{cm}	MPa	37277
Mean secant design elastic modulus (long term)	$E_{cd,ef}$	MPa	14120
Poisson's ratio	n	-	0.2
Concrete material factor	γ_c	-	1.5
Performance Specification of SFRC			
Characteristic flexural strength LOP	f_{Lk}	MPa	5.5
Characteristic residual flexural strength at $CMOD_1$	$f_{R1,k}$	MPa	4.0
Characteristic residual flexural strength at $CMOD_3$	$f_{R3,k}$	MPa	3.5
Dramix or similar steel fibre type	-	-	3D 80/60 BG 4D 65/60 BG 5D 65/60 BG
Dosage of steel fibres	-	kg/m^3	30-40

Table 4-3 - Concrete properties for jacking pipe sections

Property	Symbol	Unit	Minimum Value
Strength Grade	-	-	C50/60
Unit weight	γ	kg/m^3	2400
Characteristic compressive strength, cylinder	f_{ck}	MPa	50
Characteristic tensile strength	$f_{ctk0.05}$	MPa	2.85
Mean secant elastic modulus (short term)	E_{cm}	MPa	37277
Mean secant design elastic modulus (long term)	$E_{cd,ef}$	MPa	14120
Poisson's ratio	n	-	0.2
Concrete material factor	γ_c	-	1.5

4.1.4 Steel reinforcement

Material properties for steel reinforcement (for all reinforced concrete parts of the shaft and tunnel) are listed in Table 4-4. For the segmental lining reinforcement is required to resist bursting stresses. For the precast pipejack lining reinforcement will be placed throughout, to meet minimum reinforcement requirements of concrete sections as stated in EN 1992-1-1.

Table 4-4 – Steel reinforcement properties

Property	Symbol	Unit	Value
Characteristic yield strength of reinforcement – B500	f_{yk}	MPa	500
Reinforcing steel material factor	γ_s	-	1.15

5 TUNNEL DESIGN

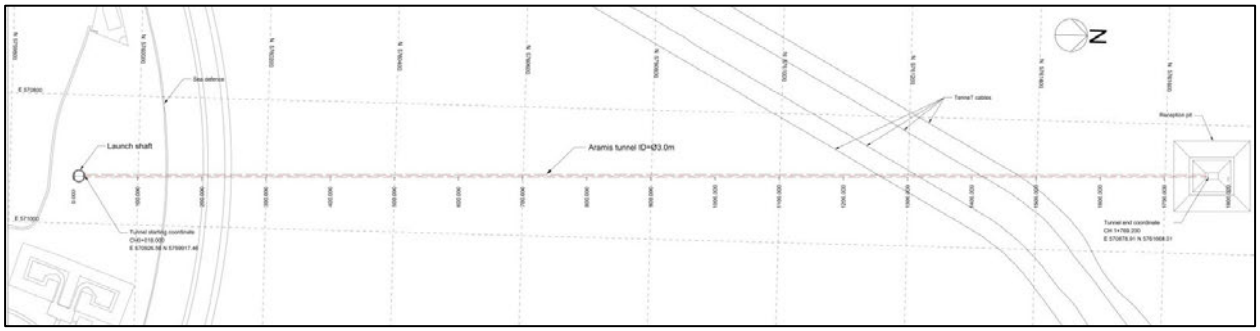
5.1 Description of the tunnel

5.1.1 Tunnel alignment

5.1.1.1 Horizontal Alignment

The horizontal alignment of the CO2 pipeline, as presented in the Figure 5-1, has been pre-defined by Contractor as described in the Tunnel FEED Basis of Design [1]. The alignment does not include any horizontal curves along the extent of the tunnel length. The horizontal alignment is presented in drawing number NL-ARM-040-ARU1-110323.

Figure 5-1 - Aramis Horizontal alignment



5.1.1.2 Vertical Alignment

The vertical alignment has been developed considering tunnel gradient, overburden, and vertical clearances. These considerations are based on project-specific constraints, design calculations, and construction requirements previously outlined in the Tunnel FEED Preliminary Design Report [2]. The tunnel gradient is determined by factors such as construction machinery requirements, locomotive transportation systems, and health and safety guidelines.

The general alignment parameters for the tunnel are summarised in Table 5-1.

Table 5-1 - General alignment information

Parameter	Unit	Value
Total tunnel length	m	1751
Vertical radius (equivalent parabola with K=20)	m	2000
Vertical minimum slope	%	-5.0
Vertical maximum slope	%	+5.0
Minimum overburden to the tunnel crown	m	13
Maximum overburden to the tunnel crown	m	35

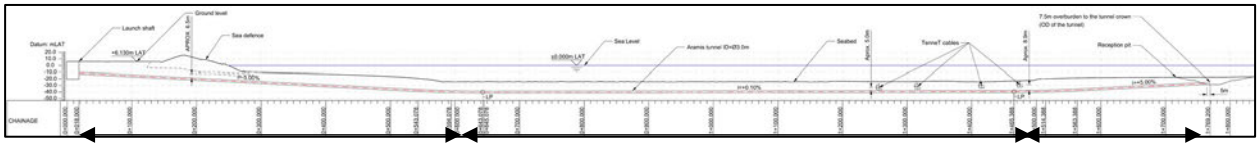
The vertical alignment is grouped into three zones (see Figure 5-2). Note that gradients are expressed as positive or negative when looking in direction from shore towards reception pit:

- Zone 1: The vertical gradient in this zone (-5%), is a function of the operational gradient of the locomotive and the depth of shaft. The minimum cover is governed by the settlement analysis under the sea defence.
- Zone 2: The vertical gradient in this zone (+0.1%) is a function of the water management inside the tunnel during the construction. There is a low point in the tunnel between Zone 1 and Zone 2, where a sump pump could be required. The minimum cover is governed by the face stability under the shipping channel and settlement analysis under the TenneT cables.
- Zone 3: The vertical gradient in this zone (+5%) is a function of the operational gradient of the locomotive. The minimum cover is governed by face stability analysis at the reception pit.

The vertical alignment corresponding to a 5% maximum gradient at the beginning and the end of the tunnel has been presented in NL-ARM-040-ARU1-110323.

In the Preliminary Design Report [2] and Tunnel Alignment technical note (NL-ARM-040-ARU1-110372), alignments with gradients including 3%, 4% and 6% were also analysed before recommending 5% gradient based on the consideration of the shaft depth, overburden, length of tunnel and market engagement. In order to optimise the shaft depth (to circa 20m from the ground surface to the top of the base slab) and the depth of reception pit, while minimising the length of the tunnel, the 5% gradient was recommended at the beginning and the end of the tunnel with 13m minimum ground cover under the shipping channel. For the minimum cover at the end of the tunnel at the reception pit location, refer to section 8.

Figure 5-2 - Tunnel vertical alignment gradient zones



Zone 1 **Zone 2** **Zone 3**

For more information regarding the horizontal and vertical alignment for Aramis tunnel see the drawing number NL-ARM-040-ARU1-110323.

5.1.2 Excavation methods

In accordance with the Tunnel FEED scope and the Tunnel FEED Basis of Design [1], a TBM driven hybrid solution consisting of a partial pipe jacked and partially segmentally lined tunnel with a slurry shield TBM machine has been considered.

For the hybrid solution different variants have been studied:

1) Hybrid 'Flying start': a start-up of tunnelling in pipe jack mode and switch to segmental tunnelling after approx. 100-150m. This option combines the advantage of pipe jacking (fast start up and a smaller shaft) and segmental tunnelling (assumed higher drive speed and a low risk of getting stuck).
2) Hybrid solution for maximized speed of construction with a planned switch from pipe jacking to segmental tunnelling in the vicinity of 1.000m - 1.500m drive length: This option results in a small shaft and maximum drive speed assuming the friction losses remain low in the anticipated ground conditions and the need for intermediate jacking is very limited.
3) Switch from pipe jacking to segmental tunnelling based on actual production speed and actual friction forces, without prior information on the exact location: This option aims to maximise the drive speed by switching only when necessary, depending on actual production speed and actual friction forces.

As an additional option, the ability to pipe jack the full length of tunnel, while retaining full planning for switchover as a contingency measure, has been considered. Similar to option 3 above, this would be dependent on on-site verification of actual tunnelling forces and progress etc. Further details are provided regarding the construction sequence in section 8.6.

5.1.3 Geometry and spaceproofing

The FEED design considers an internal diameter of 3.0m for the tunnel, based on the spaceproofing exercise undertaken during preliminary design, summarised as follows.

5.1.3.1 Spaceproofing During Tunnel Construction

The minimum spaceproofing requirements required for the tunnel during construction are summarised in the Preliminary Design Report [2] and represented on the tunnel spaceproofing drawing folio (NL-ARM-040-ARU1-110320). The assessment considers services and equipment within the TBM including walkways, refuges chambers dimensions, TBM compressed air working chamber, locomotive, ventilation duct, cables, etc. The spaceproofing requirements have been established using existing standards, guidance documents and contractor experience.

5.1.3.2 Spaceproofing During Tunnel Operation

For the spaceproofing assessment during the tunnel operation, a single 32 inch (812.8 mm) steel pipe (and associated supports) has been considered. The annulus between the steel pipeline and the concrete tunnel intrados will be backfilled after installation. The current design caters for a single pipeline only.

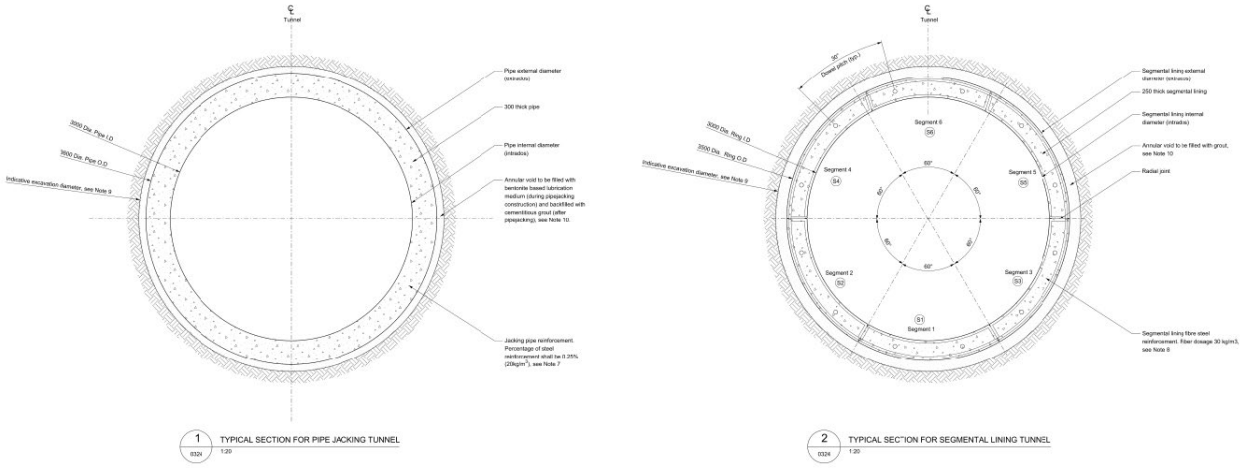
5.1.4 Lining Type

Based on the hybrid excavation method described in Section 5.1.2, two lining types have been defined along the tunnel alignment: jacking pipes and segmental lining (see also Figure 5-3):

- Jacking pipe - Pre-cast concrete pipe with an internal diameter of 3.0m and lining thickness of 300mm. The geometry and material properties has been described in the drawing numbers NL-ARM-040-ARU1-110324 and 110325.

- Segment - Pre-cast concrete segments with an internal diameter of 3.0m and segment thickness of 250mm. Each ring is formed of six segments with a rectangular configuration (6+0 arrangement). The geometry and material properties has been described in the drawing numbers NL-ARM-040-ARU1-110324, 110326 and 110327.

Figure 5-3 - Tunnel FEED – Tunnel - Typical Cross Section Geometry (from NL-ARM-040-ARU1-110324)



5.2 Loading

According with the Tunnel FEED Basis of Design [1], the tunnel lining is designed to consider the relevant permanent, variable, transient and accidental loads.

5.2.1 Load conditions

5.2.1.1 Construction Loads

- Annular grouting load

Primary and secondary grouting loads have been considered for the lining design.

Primary grouting, typically injected before the ground load is fully transferred to the segment lining/jacking pipes through the TBM tail skin. Annular grouting load has been considered as a hydrostatically distributed load around the lining. According to the BTS Specification for Tunnelling 4th edition [3] and the PAS 8810:2016 [4], the grout pressure assumed for the design is 1bar above the hydrostatic pressure.

During the pipejacking operations, lubrication slurry shall be injected to the overcut gap, and this pressure will be maintained during the pipejacking work. Upon completion of the pipejacking this lubricant shall be replaced with a cementitious grout by injecting it in the gap from the same ports that were used for lubrication injection with a slightly higher pressure. Primary grout loading has been considered as part of the structural design; lubrication loading is considered similar to grout loading, therefore, a separate load for lubrication has not been assessed.

Secondary grouting is carried out for a specific ring or segment only when primary grouting proves insufficient. To simulate this effect, the loading involves applying a localised triangular pressure of 1.0 bar above the hydrostatic head to the segments where the secondary grouting pressure is exerted. This 1.0bar pressure is applied to 1/10 of the lining perimeter at various locations within the tunnel as per recommendations by the ITA Working Group 2 guidelines [5] .

- Jacking loads

The jacking loads have been derived considering the face pressure to advance the shield, self-weight of the jacking pipes, friction around the jacking pipes due to ground closure or unstable soils, and misalignment or time delays during jacking. These loads could induce transverse splitting stresses beneath the shoe and the circumferential flexural stress between the shoes along the jacking pipes joints.

- TBM rams

TBM ram loads have been assumed with the estimated face pressure along the alignment, and it is defined as a maximum force of 511kN per shoe. Re-assessment of the lining should be undertaken once the TBM has been selected.

5.2.1.2 Permanent loads

- Self-Weight

The self-weight of the lining is considered as a dead load. The self-weight of the precast concrete tunnel lining (jacking pipes or segments) has a unit weight of the concrete of 24 kN/m³ (see material properties presented in Table 4-2)

- Ground Loads

The ground loads are defined as the self-weight of the ground above the tunnel. Unit weight of the ground has been estimated based on the geotechnical parameters defined for the Project stated in Section 3.13.

Ground loading scenarios have been included in the analysis includes full overburden and soil arching where appropriate. Soil arching assessment has been performed to estimate the ground load for the tunnel design and is presented in section 5.3.2.

- External water

The groundwater table and sea water level has been defined in accordance with the Basis of Design and the geotechnical parameters defined for the Project stated in Section 3.13.

For tunnel design, groundwater pressure is defined by the maximum (+4.0mLAT) and minimum (0.0mLAT). Sea water level is defined by the maximum (+2.6m) and minimum (0.0m) (see table 4-6 Basis of Design [1]).

- Internal hydraulic pressure

Load acting on the lining due to the internal water pressure has been considered as the tunnel needs to be flooded during the pipeline installation. Groundwater pressure is modelled as hydrostatic pressure equivalent to the external water pressure described above.

- Tunnel infill during operation

As part of the tunnel operation, it was assumed that a single pipeline with a nominal outside diameter of 32 inches would be installed inside the tunnel. According with the Landfall Construction Concept Selection Technical Note, Section 3.4, NL-ARM-040-PTC1-114070, the tunnel will be fully grouted after the pipeline installation. A unit weight of 11 kN/m³ is assumed for infill material.

5.2.1.3 Accidental Loads

- Sunken ship

For FEED design, an accidental load associated with a sunken ship scenario has been considered in accordance with the ROK-0715 [6]. It is assumed that a sunken ship load of 150 kN/m^2 acts on the tunnel lining (corresponding to a seagoing vessel, including dynamic effect, as per ROK).

- Anchor loads

Dragging anchor: Along the majority of the tunnel length (including underneath the shipping lane) a minimum 13m overburden is provided above the tunnel crown (minimum 7.2m at reception pit location), which exceeds the minimum requirements for protection of a pipeline from dragging anchors per NEN 3651 [7]. Thus, dragging anchor scenario is assumed not critical. This should be verified at EPC detailed design.

Dropped anchor: Based on the overburden depth, a simplified dropped anchor scenario has been considered in FEED; for completeness, more detailed verifications should be undertaken during EPC detailed design in accordance with ROK guidelines which consider actual shipping data relevant to the location.

For FEED, an assessment for a dropped anchor load and its impact on the tunnel lining has been undertaken using a maximum 500 tonne point load distributed through the soil overburden to calculate an equivalent uniformly distributed load on the tunnel.

- Extreme water load

In consideration of natural phenomena and changing environmental conditions such as tidal and surge levels, the 200 years return period has been used to inform the extreme high and low sea levels estimated at 4.87mLAT and -1.42mLAT, respectively. For further information, refer to Basis of Design Report - NL-ARM-040-ARU1-110301 [4].

Temporary loads, prior to the lining installation, shall be considered by the EPC contractor in the detailed design These loads concern the temporary forces on the jacking pipes or segments and are those that they experience prior to installation within the tunnel (demoulding, lifting, handling, transportation and stacking of the structural elements).

5.2.2 Load factors and Application

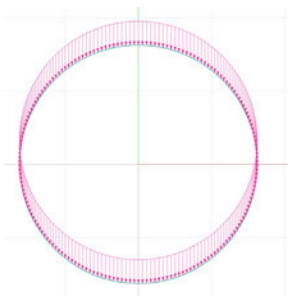
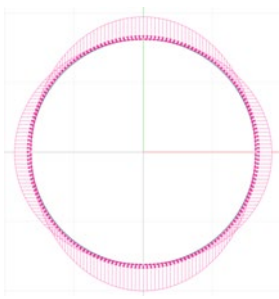
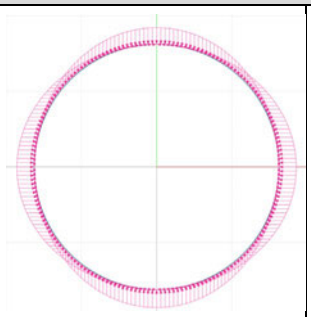
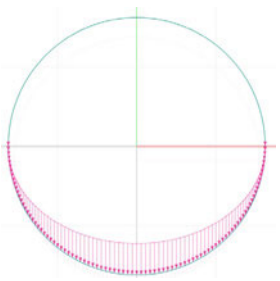
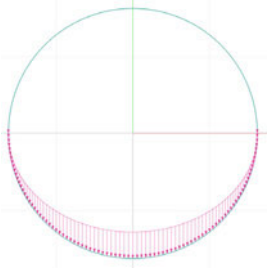
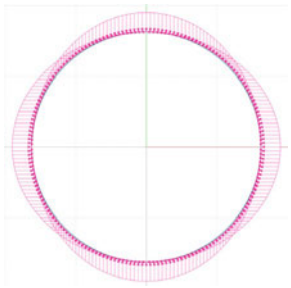
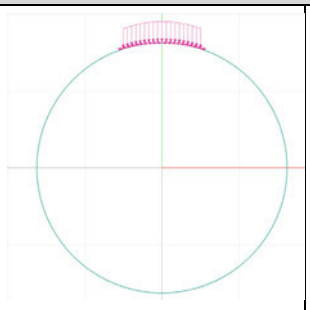
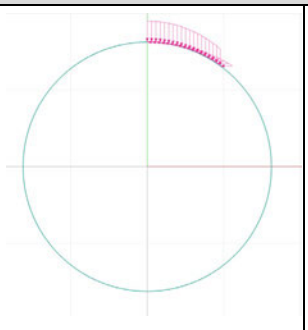
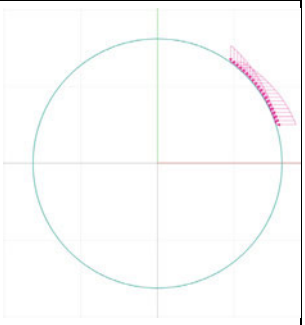
Load factor for Ultimate Limit State (ULS), Service Limit State (SLS) and Uplift (UPS) are assigned according to Eurocode 2 (Dutch National Annex) NEN-EN-1997-1+C1 [8] and NEN-EN-1990+A1:2006+A1:2006/C2:2019 [9] [10]. Table 5-2 Presents the load factor considered for the tunnel lining design, and Table 5-3 shows the loads diagrammatically for the readers convenience.

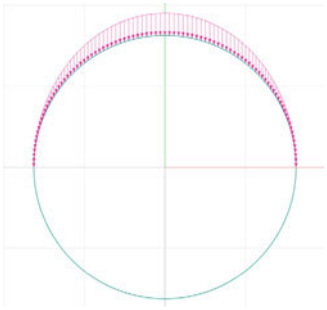
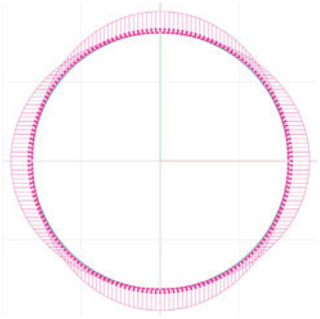
Note that the load factors correspond to Consequence Class 2 – for commentary on impact of potentially adopting a different consequence class, refer to Section 11.7.

Table 5-2 – Loads and load factors

Description	ULS	SLS	UPS
Permanent loads			
Self-Weight (L1)	1.35	1.00	0.90
Ground load (UB and LB) (L2)	1.35	1.00	0.90
External water load (high and low) (L3)	1.20	1.00	1.10
Internal water load (1/2 Full and Full) (L4)	1.20	1.00	-
Internal grout fill load (L5)	1.35	1.00	-
Construction Loads			
Primary annular grouting load (L6)	1.50	1.00	
Secondary annular grouting load (L7)	1.50	1.00	-
TBM rams or Jacking load	1.50	1.00	-
Accidental loads			
Ship impact (L8)	1.00	N/A	-
200 years return period external water load (extreme high and low) (L9)	1.00	N/A	-

Table 5-3 – Diagram of load cases

Permanent Load Cases			
			
Self-Weight (L1)	Ground load (L2)	External water load (L3)	Internal water load (L4)
			
Internal grout fill load (L5)			
Construction Load Cases			
			
Primary annular grouting load (L6)	Secondary annular grouting load (L7) (example of locations shown - localised grout loads applied entire way round lining)		

Accidental Load Cases	
	
Ship impact (L8)	200 years return period external water load (extreme high and low) (L9)

5.2.3 Load combinations

Load combinations are developed separately for ULS and SLS design situations. Each combination of actions should include a leading variable action or an accidental action. The SLS combinations used in the design are identified in Eurocode 2 (Dutch National Annex) NEN-EN-1997-1+C1 [8] and NEN-EN-1990+A1:2006+A1:2006/C2:2019 [9]. The list of loads and combination factors are given in Table 5-4.

Table 5-4 – Loads and load combinations lining design

Load combinations	ULS	SLS
Construction		
LC1: Self-Weight + External water load + Primary annular grouting load	1.35L1+1.2L3+1.5L6	1.0L1+1.0L3+1.0L6
LC2: Self-Weight + Ground load + External water load + Secondary annular grouting load	1.35L1+1.35L2+1.2L3+1.5L7	1.0L1+1.0L2+1.0L3+1.0L7
LC4: Self-Weight + Ground load + External water load	1.35L1+1.35L2+1.2L3	1.0L1+1.0L2+1.0L3
LC3: Self-Weight + Ground load + External water load + Internal water load	1.35L1+1.35L2+1.2L3+1.2L4	1.0L1+1.0L2+1.0L3+1.0L4
Permanent		
LC5: Self-Weight + Ground load + External water load + Internal grout fill load	1.35L1+1.35L2+1.2L3+1.35L5	1.0L1+1.0L2+1.0L3+1.0L5

Load combinations	ULS	SLS
Accidental		
Sunken ship load		
LC6: Self-Weight + Ground load + External water load + Ship impact load	1.0L1+1.0L2+1.0L3+1.0L8	N/A
LC7: Self-Weight + Ground load + External water load + Internal grout fill load + Ship impact load	1.0L1+1.0L2+1.0L3+1.0L5+1.0L8	N/A
200yr Return Period G.W.P – Extreme water load		
LC8: Self-Weight + Ground load + External water load + 200yr Return external water	1.0L1+1.0L2+1.0L3+1.0L9	N/A
LC9: Self-Weight + Ground load + External water load + Internal grout fill load + 200yr return period external water	1.0L1+1.0L2+1.0L3+1.0L5+1.0L9	N/A
Notes:		
1. Where water pressure is indicated, there is both a high and low water level cases 2. Where ground load is indicated, there is both a high and Low ground load case (UB and LB soil properties) 3. Where secondary grout load is defined, there are 10 different load location cases analysed. 4. For internal water pressure two cases exist (1/2 full section, full section). 5. For 200yr return period external water pressure there is both a high and low max/min. case.		

5.3 Design calculations

5.3.1 Design sections

Different design scenarios have been defined along the tunnel alignment to determine the critical sections for the assessment of pipe jacking and segmental lining thickness. Table 5-5 and Figure 5-4, present the design sections considered in the tunnel design.

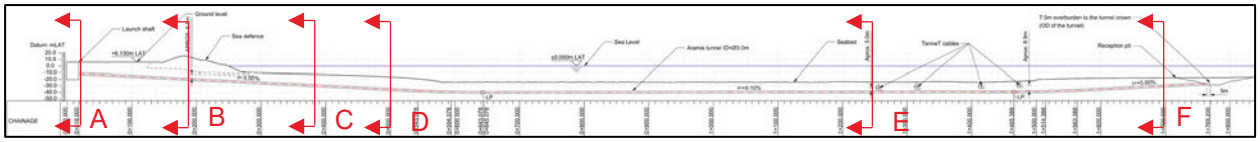
The following factors have been considered for determining critical sections for design:

- Change in geology;
- Change in overburden (shallow and deepest section);
- Change in groundwater level;

Table 5-5 - Tunnel design sections

Design Section	Chainage	Ground surface/seabed level [mLAT]	Groundwater/sea level [mLAT]	Tunnel Axis level [mLAT]	Overburden to the tunnel crown [m]
~~A~~	+18.00	6.1	4.0 /0.0	-11.5	15.7
B	+178.00	15.5	4.0 /0.0	-19.5	33.1
C	+288.00	-10.0	2.6 /0.0	-25.0	13.1
D	+500.00	-16.3	2.6 /0.0	-35.5	17.3
E	1+257.00	-24.0	2.6 /0.0	-39.6	13.7
F	1+700.00	-17.5	2.6 /0.0	-30.0	10.6

Figure 5-4 – Tunnel design sections

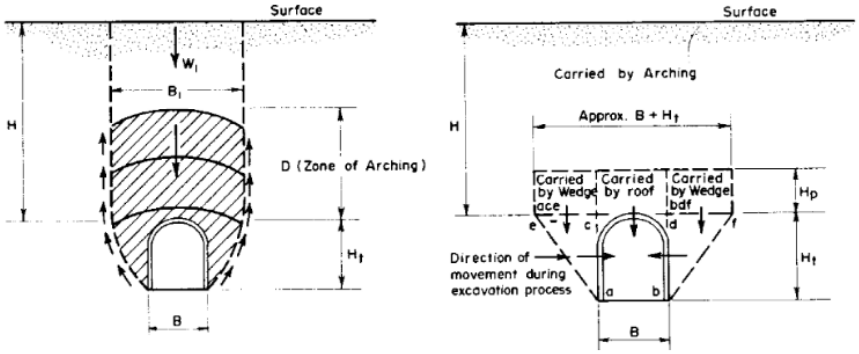


5.3.2 Ground load estimation

The full overburden and arching condition have been checked to estimate the ground load for the tunnel lining design. Terzaghi's method (Figure 5-5) was adopted to estimate the ground loading on the tunnel considering the ground arching effect; note this diagram shows the theory in a generic manner and the theory has been applied to the circular cross section.

These calculated ground arching loads for the lower bound (LB) and upper bound (UB) ground models are summarised in Table 5-6 and Table 5-7 respectively. These load models are a simplified version of the ground model presented in the GDR, which are considered appropriate for ground load estimation calculation.

Figure 5-5 - Terzaghi's method for the calculation of load acting on tunnel linings



$$q = \frac{\gamma a_1 - c\left(1 - e^{-\frac{K_0 \tan\Phi}{a_1} H}\right)}{K_0 \tan\Phi}$$

$$a_1 = \frac{B}{2} + h \tan\left(45^0 - \frac{\Phi}{2}\right)$$

Note: q = vertical uniform load acting on the supporting structure, γ = surrounding soil unit weight, h_s = depth of the tunnel crown, B_1 = span of the arch, α_1 = half-span of the arch, B = span of the tunnel, ϕ = real internal friction angle of the soil mass, c = cohesion, and K_0 = lateral pressure coefficient of the soil. This diagram shows a generic illustration for soil arching theory, for the Aramis tunnel it is confirmed that saturated unit weights are considered for the calculations.

Table 5-6 – Ground arching loads – Lower bound ground model

Lower bound ground model								
Section	h_s [m]	$h_{w,SL}$ [m]	$y_{soil\ (submerged\ unit\ weight)}$ [kN/m³]	c' [kPa]	Φ [°]	k_0 [-]	σ'_v [kPa]	σ'_h [kPa]
B	33.5	19.8	7.7	0	30	0.52	70.6	36.7
E	14.1	39.9	8.2	0	30	0.52	60.0	31.2

Table 5-7 – Ground arching loads – Upper bound ground model

Upper bound ground model								
Section	h_s [m]	$h_{w,SL}$ [m]	$y_{soil\ (submerged\ unit\ weight)}$ [kN/m³]	c' [kPa]	Φ [°]	k_0 [-]	σ'_v [kPa]	σ'_h [kPa]
B	33.5	19.8	10.2	0	42	0.65	45.3	29.5
E	14.1	39.9	12.2	0	43	0.65	50.0	32.5

Note: hs = depth of the tunnel crown, $h_{w,SL}$ = height of water/seawater to the springline, y_{soil} = soil unit weight, ϕ = internal friction angle of soil mass, c' = cohesion, and k_0 = lateral pressure coefficient of the soil, σ'_v = arched effective vertical stress, σ'_h = arched effective horizontal stress]

The calculation of ground loads using Terzaghi arching theorem aligns with the proposed method within PJA guidance [11] for the determination of ground loading in cohesionless soils. Pressure due to ground water itself is considered additionally.

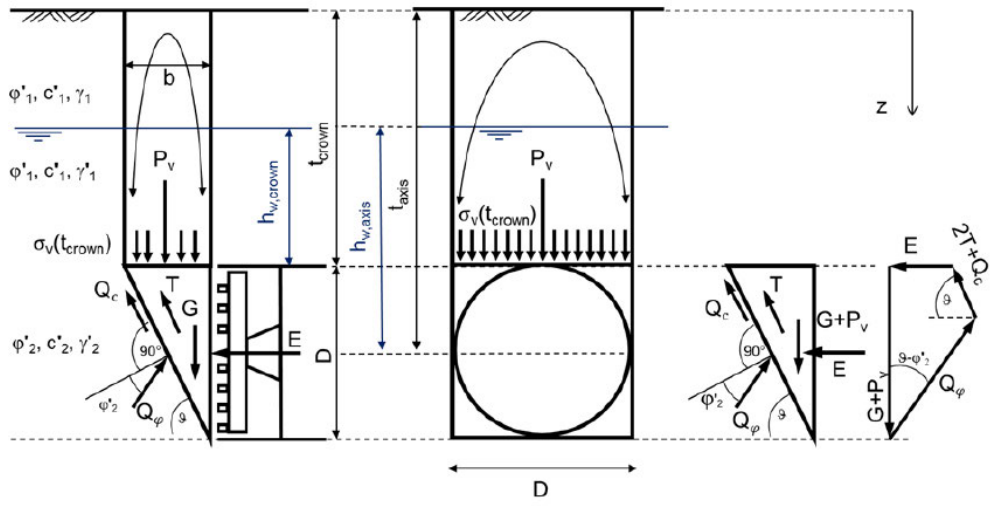
5.3.3 Face pressure assessment

The face stability of the tunnel is of considerable importance in pipe jacking or segmental lining design. Ground movements above the tunnel may cause damage to surface structures or services. Ground conditions must be carefully assessed to anticipate possible face instabilities particularly in soft ground, which is the current ground condition for this project.

The evaluation of the theoretical face pressure range is based on DAUB document: Recommendations for face support pressure calculations for shield tunnelling in soft ground, October 2016 [12].

The limit equilibrium method, Jancsecz & Steiner (1994) , suggested in the DAUB 2016 [12] was used to determine the required support pressure due to the acting earth pressure or water (see Figure 5-6). The effective (drained) shear parameters of a soil are assumed in this case. In addition, the face stability analysis method will also include the Draw down.

Figure 5-6 - Forces acting on the wedge

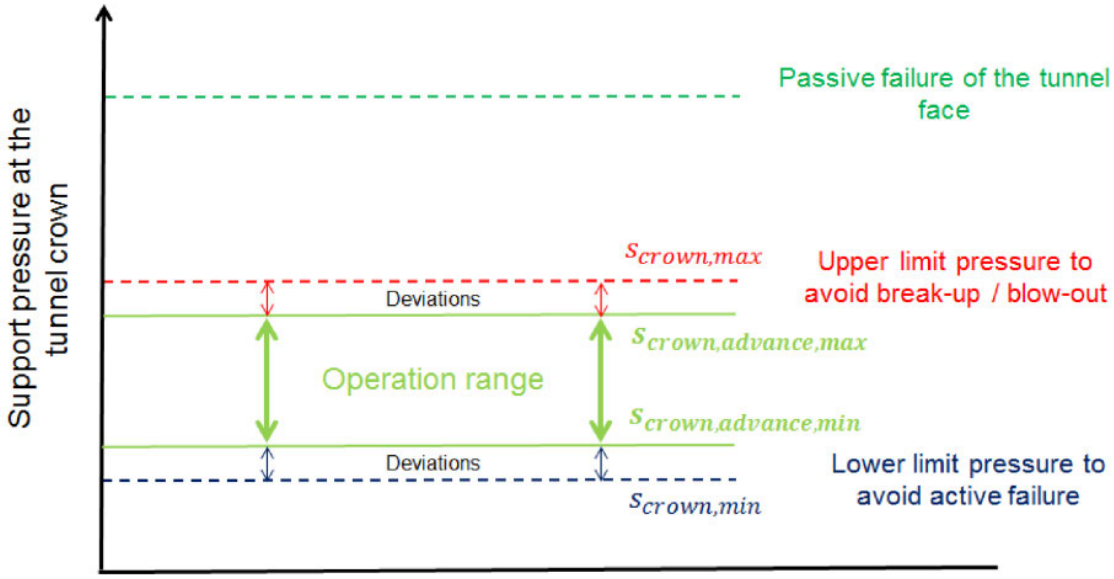


Note: E_{re} = support force due to the earth pressure, G = own weight of wedge, Pv = vertical load from the soil prism, T = Shear force on the vertical slip surface, ϑ = Sliding angle, φ' = Friction angle of soil, c = cohesion of the soil, D = shield diameter and Q = shear force on inclined surface, γ = unit weight of soil.

Upper and lower operational limits were evaluated to define the operational range of the support pressure (see Figure 5-7). The lower support pressure limit must ensure a minimal support force to prevent ground collapse. The upper support pressure limit must ensure a limiting pressure to avoid a break-up of the overburden or blow-out of the support medium. The safety factors for the earth pressure and water pressure considered are 1.5 and 1.05 respectively.

The support pressure deviations considered in the analysis is +/-10 kPa for the slurry shield (including compressed air support).

Figure 5-7 - Allowable operational pressures at the tunnel crown of a shield machine



We have considered two scenarios: lower and upper bound ground parameters based on the ground interpretation presented in Section 3. Minimum and Maximum of the groundwater level (see Section 5.2.1.2) have been considered to analyse the possible effects of the water pressure variations on the face pressure.

Table 5-8 and Table 5-9 present the results of the face pressure assessment at the design sections for lower (LB) and upper bound (UB) model respectively. In summary, the water pressure required for the Aramis tunnel have been estimated in a range approx. between 1.0 bar (minimum) and 5.9 bar (maximum) to maintain the face pressure stability and to avoid a break-up blow-out of the support medium.

Based on the sections analysed, Figure 5-8 and Figure 5-9 presents the approx. variation of the face pressure along the tunnel alignment for the lower and upper ground models, for the maximum water table expected along the tunnel. For more details, see calculations in Appendix A.1.

Results show that the face operation range needs to be controlled accurately during the construction process, especially at the beginning of the construction, the sea defence location and at the Tennet cables. In addition, Section 8 presents the calculation of the face pressure at the end of the tunnel at the reception pit location.

Table 5-8 – Results face pressure at tunnel crown – Lower bound ground model

Critical Sections	Chainage	Water Level	Ground Surface to Crown	Minimum Support Pressure at Tunnel Crown for Regular Advance, scr,adv,min	Maximum Support Pressure at Tunnel Crown for Regular Advance, scr,adv,max	Operation Range
		mLAT	m	bar	bar	bar
Maximum Groundwater						
A	+18.00	4.0	15.7	1.6	2.4	0.8
B	+178.00	4.0	33.1	2.4	5.5	3.1
C	+288.00	4.0	13.1	2.9	3.2	0.2
D	+500.00	4.0	17.3	4.1	4.5	0.4
E	1+257.00	4.0	13.7	4.5	4.7	0.2
F	1+700.00	4.0	10.6	3.5	3.6	0.1
Minimum Groundwater						
A	+18.00	0.0	15.7	1.2	2.4	1.2
B	+178.00	0.0	33.1	2.0	5.5	3.5
C	+288.00	0.0	13.1	2.5	2.8	0.3
D	+500.00	0.0	17.3	3.6	4.1	0.5
E	1+257.00	0.0	13.7	4.1	4.4	0.3
F	1+700.00	0.0	10.6	3.1	3.3	0.2

Figure 5-8 – Face pressure at tunnel crown – Lower bound ground model and maximum water level

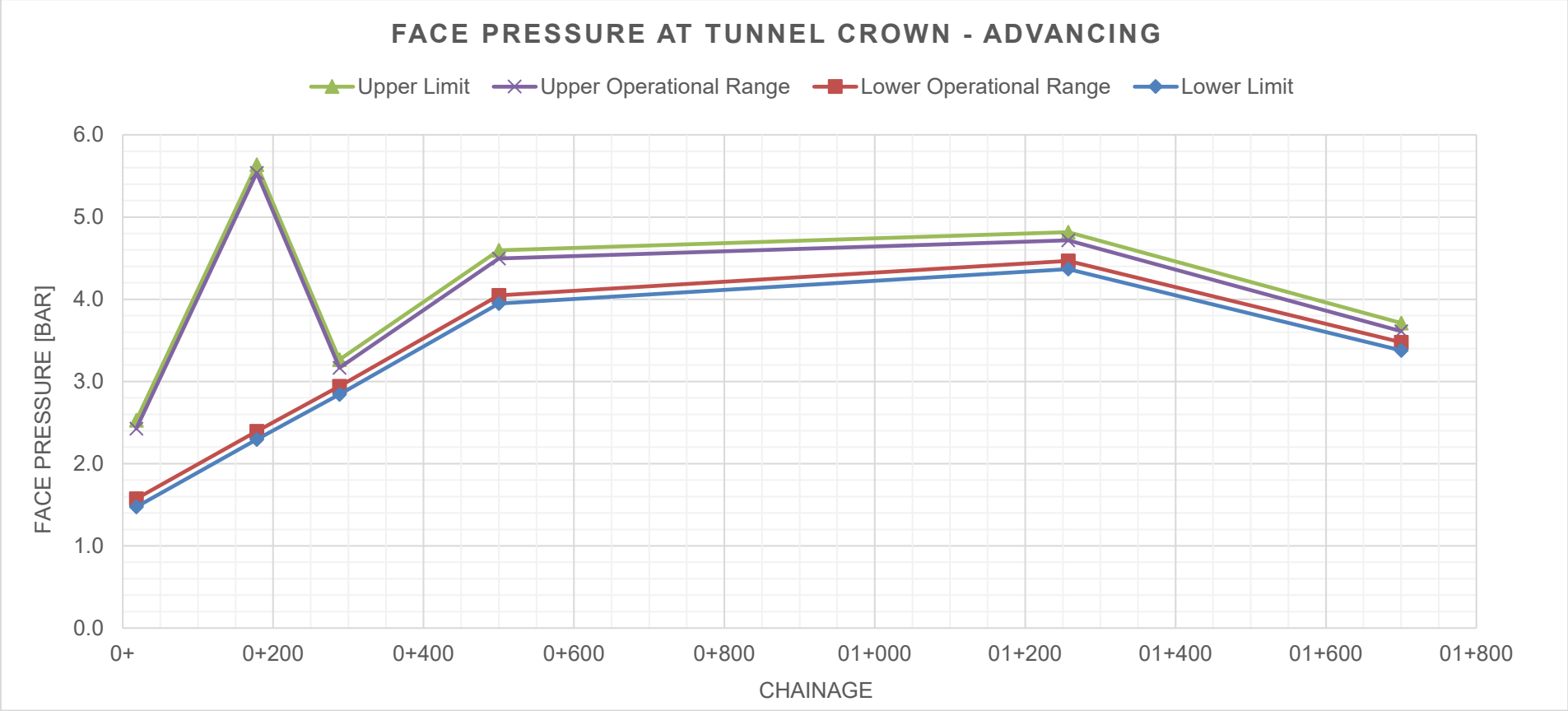
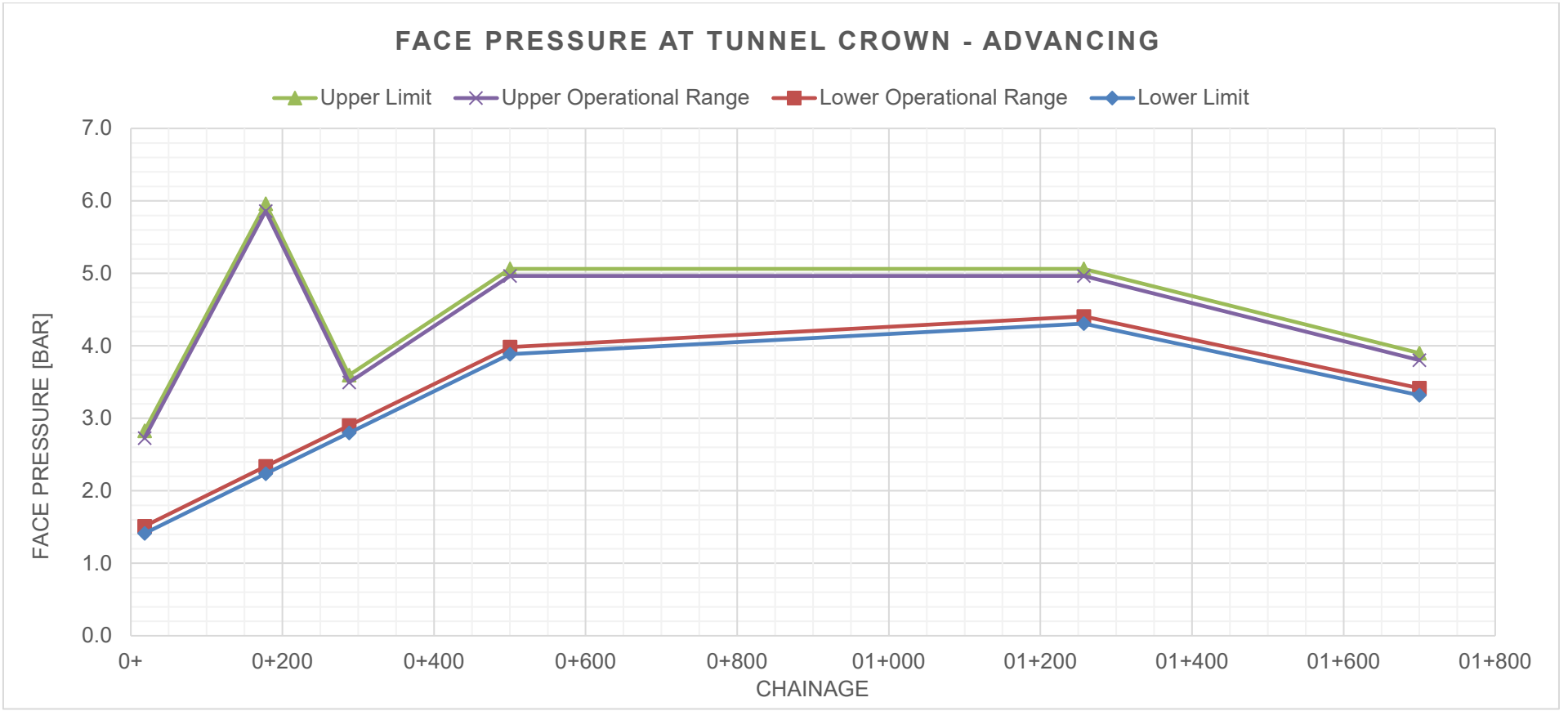


Table 5-9 – Results face pressure at tunnel crown – Upper bound ground model

Critical Sections	Chainage	Water Level	Ground Surface to Crown	Minimum Support Pressure at Tunnel Crown for Regular Advance, scr,adv,min	Maximum Support Pressure at Tunnel Crown for Regular Advance, scr,adv,max	Operation Range
		mLAT	m	bar	bar	bar
Maximum Groundwater						
A	+18.00	4.0	15.7	1.5	2.7	1.2
B	+178.00	4.0	33.1	2.3	5.9	3.5
C	+288.00	4.0	13.1	2.9	3.5	0.6
D	+500.00	4.0	17.3	4.0	5.0	1.0
E	1+257.00	4.0	13.7	4.4	5.0	0.6
F	1+700.00	4.0	10.6	3.4	3.8	0.4
Minimum Groundwater						
A	+18.00	0.0	15.7	1.1	2.7	1.6
B	+178.00	0.0	33.1	1.9	5.9	3.9
C	+288.00	0.0	13.1	2.5	3.1	0.7
D	+500.00	0.0	17.3	3.6	4.6	1.0
E	1+257.00	0.0	13.7	3.9	4.6	0.7
F	1+700.00	0.0	10.6	3.0	3.5	0.5

Figure 5-9 – Face pressure at tunnel crown – Upper bound ground model and maximum water level



5.3.4 Gasket and sealing system definition

5.3.4.1 Segmental Lining Gasket

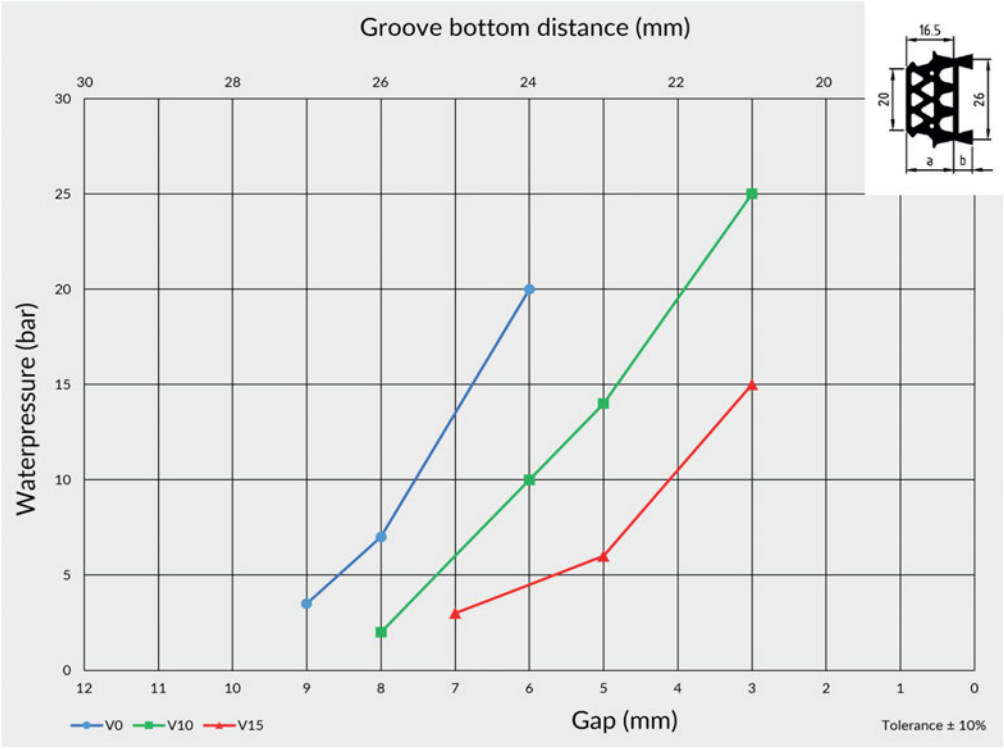
An elastomeric (EPDM) gasket between the segment joints has been defined to resist the design water pressure for the segmental lining design working life.

For the segmental lining, it's assumed for design an EPDM anchored gasket M80296 Type "RIO LOMAS", (by Sealable or similar). Properties for gaskets can be found in Table 5-10 and Figure 5-10. For the segmental lining gaskets shall comply with the manufacturing and tolerances requirements of the BTS Specification for Tunnelling [3].

Table 5-10 - EPDM Gasket M80296 Type "Rio Lomas", technical data

Characteristic	Value
Cross section (see Figure 5-10)	a=245,3 mm2 b=72,6 mm 2
Groove width at bottom	26 mm
Groove depth[1)]	9 mm
Groove angle	11,5°
Groove dimension	258,3 mm2
At full compression the groove is used with	95 %

Figure 5-10- EPDM Gasket M80296 Type "Rio Lomas" gap pressure curve



The design shall cater for the relaxation of the gasket material so that the gasket can resist twice the maximum anticipated working or design pressure in the long term with maximum construction tolerance.

The maximum water pressure expected at the tunnel lining its approximately 4.4bar. The gasket recommended has been defined to withstand 200% of the working water pressure (4.4barx200%=8.8bar) in the lining at the maximum specified gasket differential gap of 5mm and bearing offset of 10mm.

5.3.4.2 Pipe Jacking Sealing System

The design of the sealing system for jacking pipes is dependent on the manufacturer chosen for pipe fabrication/casting. For the sealing system for the jacking pipes, sealing gaskets available on the market are able to achieve the necessary 4.4bar sealing pressure required with a 0.5° joint angular deflection. Gasket design shall be further developed at detailed design stage, ensuring that the manufacturer adheres to BS EN 1916:2002 and BS EN-681-1.

For reinforced concrete pipes, designed for the combined effects of external and internal loads, the minimum requirement for factory test pressures is 120% of the working pressure [13]. For the case of the Aramis tunnel project this would equate to approximately 5.3bar testing pressure.

5.3.5 Pipejacking lining design

5.3.5.1 Thrust forces assessment

To develop provisional pipejacking lengths, the PJA (Pipe Jacking Association) ‘Guide to best practice for the installation of pipe jacks and microtunnels’ (1995) [14] [5], has been used as reference. Pipejack capacities have been calculated using PJA guidance and benchmarked against pipe manufacturers capacity data. In accordance with the basis of design [1], in addition to EN 1992, the jacking pipes have been assessed in consideration of the following standards which should be considered in full during EPC detailed design:

- BS 59111:2021, Concrete pipes and ancillary concrete, Part 1: Unreinforced and reinforced concrete pipes (including jacking pipes) and fittings with flexible joints (complementary to BS EN 1916:2002)
- DWA-A 161E Static Calculation of Jacking pipes.
- ASCE/CI 36 Standard Design and Construction Guidelines for Microtunnelling.
- Guide to Best Practice for the Installation of Pipe Jacks and Microtunnels (PJA)

The key assumptions for the pipejacking calculation are as follows. Note that these are assumptions used for confirming feasibility of pipe-jacking during FEED stage, along with the associated pipe jack design developed at FEED stage only. EPC Contractor should make and validate their own assumptions based on their own detailed design development.

- Length of TBM: 18m
- Length of pipe sections: 3.0m

- Angular deflection: 0.25°, 0.375°, 0.50°
- TBM stoppage factor: 24hrs (1.50)*
- Misalignment factor: 20% (1.20)*
- Partial factor (ULS condition): 2.0
- Lubrication friction: 0.5kN/m^2
- Joint packing material: Assumed 20mm plywood, E_p=70Mpa – In practice, the modulus of elasticity of joint packing material, Ep, after multiple loadings, should be obtained from tests [15]. The EPC contractor shall ensure that any packer/pressure transfer ring meets requirements outlined in DWA-A 161E.

**Values within brackets ( ) are factors applied to the value of friction calculated on the lubricated pipe sections for the corresponding situation.*

Table 5-11 provides the section capacity in tonnes utilised in this assessment for a 3m long pipe section.

Table 5-11 - Table of section capacities

Concrete Grade	Angular Deflection			Units
	0.25° (Very Good Workmanship)	0.375° (Good Workmanship)	0.5° (Standard Workmanship)	
C50/60	3208	2400	1984	tonnes

Both upper and lower bound ground conditions were assessed along the alignment (from section A to Section F), along with the impact of workmanship.

Table 5-12 presents the maximum and minimum face pressures experienced across the alignment, due to ground and water conditions, along with the sum of the friction on the unlubricated sections of pipe for both upper and lower bound cases.

Table 5-12 - Table of face pressures and friction

Condition	Lower Bound	Upper Bound
Face Pressure (Max./Min.)	4416kN / 1527kN	6132kN / 2857kN
TBM + 2 Pipes Friction + Face Pressure (Max./Min.)	11562kN / 7370kN	15662kN / 8875kN

Figure 5-11 and Table 5-13 below provide a summary of the estimated tunnelling lengths, completed for feasibility purposes only at FEED stage, which may be achieved using various permutations of upper and lower bound ground conditions, workmanship levels (angular deflection), and different numbers of interjacks. These lengths should not be taken as a specification and are provided as indicative values to inform FEED stage only. For all cases the worst credible scenario has been assessed assuming TBM stoppage (24hrs) + 20% misalignment. It is noted that the quantity of interjacks is based on theoretical calculation, and EPC contractor may consider the need to install additional intejacks to allow some level of redundancy, reduce the risks associated with interjacking over long lengths, and to introduce additional interjacks to suit their construction programme if necessary.

It is expected that actual progress during pipejacking will inform the potential need to switch to segmental lining mode. It should be noted that depending on construction progress, the optimum switchover location may lie within the 900 - 1300m range based on the theoretical number of interjacks required and increased stoppage time associated with higher numbers of interjacks. In addition, it may be preferable to switchover in advance of tunnelling under the first TenneT cable i.e. at approximately 1200m along the tunnel.

Figure 5-11 - Schematic of jacking lengths from shaft

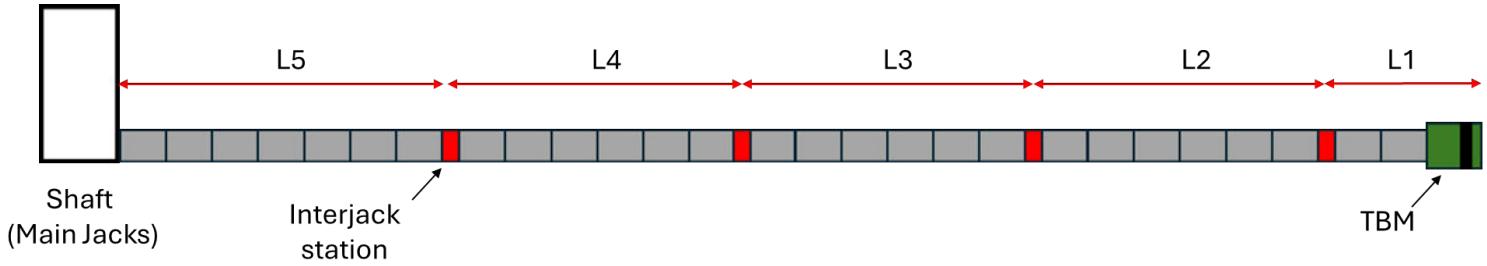


Table 5-13 - Estimated Jacking lengths

Ground model parameters	Angular Deflection	L1	L2	L3	L4	L5	L6	L7	L8	L9	No. of Interjack Stations to tunnel 1751m
LB	0.25	450	425	425	400	-	-	-	-	-	3
UB	0.25	325	300	300	300	300	225	-	-	-	5
LB	0.375	350	325	300	315	300	150	-	-	-	5
UB	0.375	250	225	225	225	225	217	217	167	-	7
LB	0.5	275	275	275	250	275	250	150	-	-	6
UB	0.5	200	200	133	233	183	183	183	183	183	8

5.3.5.2 Bedded Beam Spring Model

Oasys GSA software has been used to generate the bedded beam spring model (non-linear analysis), and to estimate the internal forces and the thickness of the lining. Within the model, radial springs have been used to simulate the ground reaction of the soil around the tunnel lining, and all the load conditions defined for the tunnel lining design have been applied in the model, according with the load combinations described in Section 0.

Bedded Beam Spring models consist of three key elements:

- **Tunnel Lining:** Made up of a series of beam elements, capable of carrying tension and compression. Each beam element is connected at a series of evenly spaced nodal points around the excavation perimeter. The geometry and material properties has been defined as defined in Sections 4 and 5.1.3.
- **Radial Springs:** The soil or rock medium is represented by radial springs; these elements define the properties of the ground lining reaction and are modelled as compression only elements.

  Radial and tangential springs simulate the elastic interaction between the lining and the ground. The following empirical relationship, derived from standard ground stiffness tests, has been considered for calculating the spring stiffness (K, subgrade reaction coefficient of the ground):

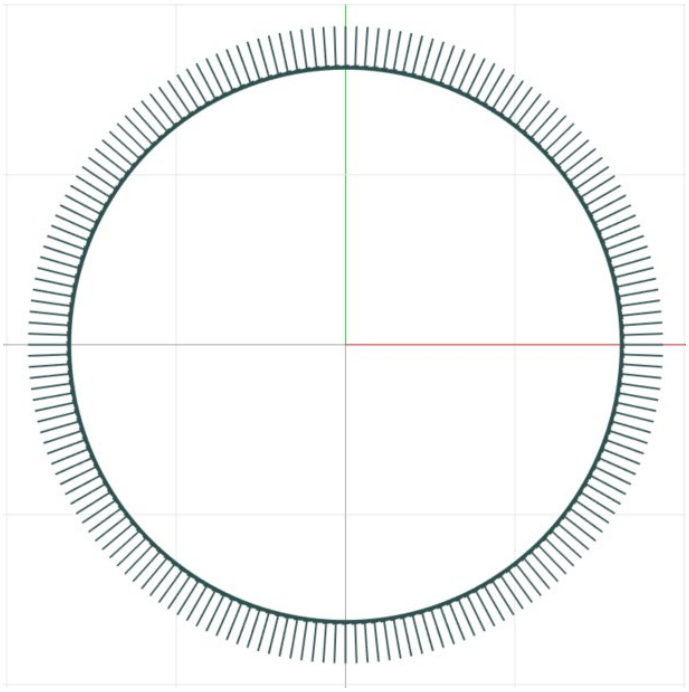
  From Duddeck & Erdmann (1985) [16], $K = 0.6\ Eg\ (1 - \nu) / r\ [(1+\nu)(1-2\nu)]$,

  with Eg, elastic modulus of the ground, ν, Poisson's Ratio & r, centroidal excavation radius.

  To calculate radial spring stiffness, the spring stiffness / subgrade reaction is simply applied over the loaded area for each spring $Kr = K\ w\ s$ (where $[s = \pi D / n]$; n = number of radial springs around tunnel circumference, w = width of tunnel lining).
- **Tangential Springs:** Represent the frictional capacity of the ground against shear and rotation of the tunnel.

  To determine the tangential stiffness in soft ground, estimated as 20% of the radial spring stiffness, the methodology given in Dixon (1971) [17] has been used.

Figure 5-12 – Bedded Beam Spring Model



The following tables in the section present the estimated radial spring values and ground model conditions according with the ground properties described in the Section 3. In addition, for the load conditions analysed, the maximum/minimum moments, axial forces and shear forces determined from the non-linear static analysis.

Ground loads have been analysed considering Terzaghi arching condition and the lining thickness, bursting reinforcement and shear reinforcement has been determined based on the arching condition. Sensitivity analysis has been performed to investigate the lining capacity with full overburden condition, the flexural capacity of a 250mm thick segmental lining and a 300mm thick pipe jacking lining is considered adequate under full overburden condition, however local bursting and shear reinforcement will need to be increased where full overburden is experienced.

Section B, lower bound ground model

Table 5-14 summarises the radial spring properties used to represent the lower bound properties of the ground at section B and Table 5-15 summarises the soil arching results for the section B lower bound ground model.

Table 5-14 – Radial spring values section B, lower bound

Springs location	Material description	E_g, [MPa]	n [-]	K [kN/m^3]	Radial Stiffness model (kN/m)
Crown	Clay	18	0.35	10505	605
Sides	Silty Sand	33	0.25	14400	829
Invert	Peat	1.5	0.45	2069	119

Table 5-15 – Pipejacking Lining – Lower Bound – GSA Analysis Results Summary

		Terzaghi Soil Arching
Bending Moment (kNm)	**Min.**	-50
	Max.	65
Axial (kN)	**Min.**	-725
	Max.	-298
Shear (kN)	**Min.**	-91
	Max.	82

Section B, Upper bound ground model

Table 5-16 summarises the radial spring properties used to represent the lower bound properties of the ground at section B and Table 5-17 summarise the results for section B upper bound ground model for the pipejack lining.

Table 5-16 - Radial spring values section B, Upper bound

Springs location	Material description	E_g, [MPa]	n [-]	K [kN/m^3]	Radial Stiffness model (kN/m)
Crown	Sand	53	0.2	21414	1233
Sides	Sand	80	0.2	32323	1862
Invert	Sand	75	0.2	30303	1745

Table 5-17 – Pipejacking Lining - Upper Bound – GSA Analysis Results Summary

		Terzaghi Soil Arching
Bending Moment (kNm)	**Min.**	-39
	Max.	51
Axial (kN)	**Min.**	-725
	Max.	-334
Shear (kN)	**Min.**	-77
	Max.	74

Figure 5-13 to Figure 5-14 illustrate the maximum internal forces considering Terzaghi arching conditions for the section B lower bound ground model (maximum absolute forces in the lining).

Figure 5-13 – Section B, lower bound Pipejack Lining Results – Terzaghi Soil Arching – Maximum Bending Moment

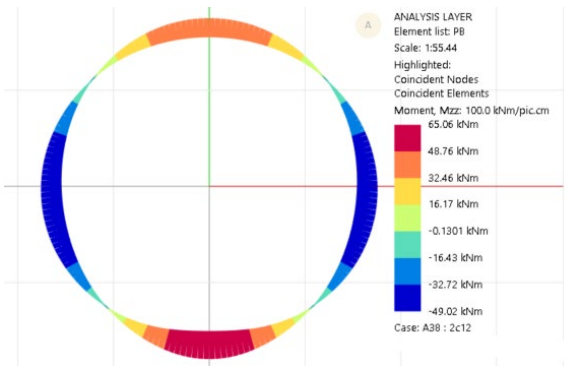
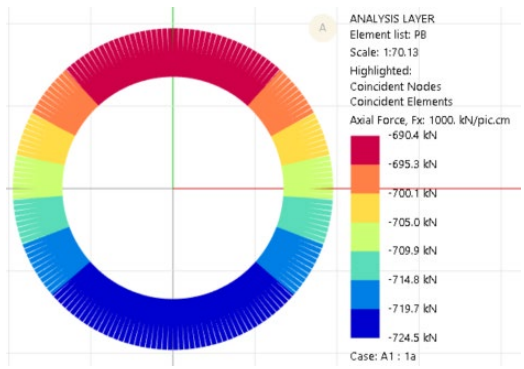


Figure 5-14 – Section B, lower bound Pipejack Lining Results – Terzaghi Soil Arching – Maximum Axial Force (negative - compression)



Section E, Lower bound ground model

Table 5-18 summarises the radial spring properties used to represent the lower bound properties of the ground at section E. The associated soil arching results for the section E lower bound ground model are summarised in Table 5-19.

Table 5-18 – Radial spring values section E, lower bound

Springs location	Material description	E_g, [MPa]	n [-]	K [kN/m^3]	Radial Stiffness model (kN/m)
Crown	Sand	77	0.2	29333	1664
Sides	Silty Sand	76	0.25	31269	1774
Invert	Sand	81	0.2	30857	1750

Table 5-19 Pipejacking Lining - Section E - Lower Bound – GSA Analysis Results Summary

		Terzaghi Soil Arching
Bending Moment (kNm)	**Min.**	46
	Max.	61
Axial (kN)	**Min.**	-1096
	Max.	-31
Shear (kN)	**Min.**	-95
	Max.	88

Section E, Upper bound ground model

Table 5-20 summarises the radial spring properties used to represent the upper bound properties of the ground at section E. The associated soil arching results for the section E upper bound ground model are shown in Table 5-21.

Table 5-20 – Radial spring values section E, Upper bound

Springs location	Material description	E_g, [MPa]	n [-]	K [kN/m^3]	Radial Stiffness model (kN/m)
Crown	Sand	101	0.2	38476	2182
Sides	Sand	107	0.2	40762	2312
Invert	Sand	101	0.2	38476	2182

Table 5-21 - Pipejacking Lining - Section E - Upper Bound – GSA Analysis Results Summary

		Terzaghi Soil Arching
Bending Moment (kNm)	**Min.**	-45
	Max.	62
Axial (kN)	**Min.**	-1096
	Max.	-33
Shear (kN)	**Min.**	-94
	Max.	87

Figure 5-15 and Figure 5-16 illustrate the maximum internal forces considering Terzaghi arching conditions for the section B lower bound ground model (maximum absolute forces in the lining).

Figure 5-15 - Section E, lower bound Pipejack Lining Results – Terzaghi Soil Arching – Maximum Bending Moment

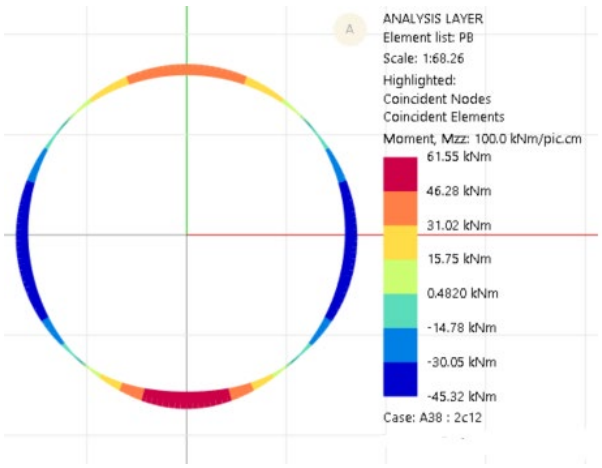
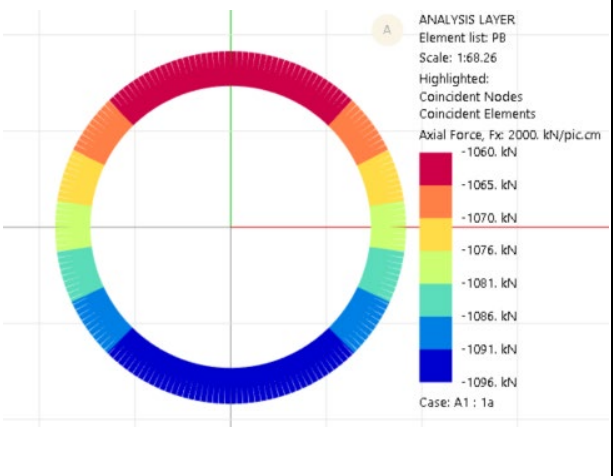


Figure 5-16 - Section E, lower bound Pipejack Lining Results – Terzaghi Soil Arching – Maximum Axial Force (negative - compression)



5.3.5.3 Concrete design ULS and SLS checks

The pipejack lining will be subject to combinations of axial thrust and bending moment, and shear force resulting from the different load combinations. The design checks have been performed on selected sections throughout the alignment, specifically in critical locations which may govern the design: Section B and section E.

- **Ultimate Limit State: *PCTL flexural design***

The flexural capacity at ultimate limit state of the lining has been calculated in accordance with NEN EN 1992, Eurocode 2 – Design of concrete structures, CEN. The imposed axial forces and bending moments from the design load cases have been checked against the lining capacity curves.

Capacity curves for critical sections and load cases for the pipejack lining sections are presented below (Figure 5-17 to Figure 5-20). A complete set of all analyses results are presented in Appendix A.2. Please note that the blue capacity curve is that of a plain 290mm thick concrete section (300mm – 10mm durability allowance presented in Section 11.3), and orange curve is that with minimum reinforcement requirements – 0.25% in line with BS EN 1916:2002.

Figure 5-17 - Section B, lower bound Pipejack Lining Results – Terzaghi Soil Arching – Construction Load Cases – Flexural Checks

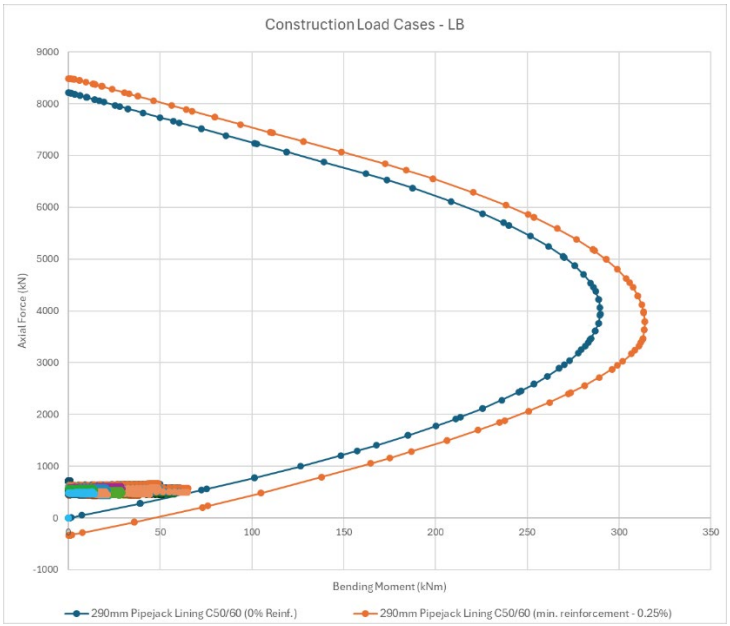


Figure 5-18 - Section B, lower bound Pipejack Lining Results – Terzaghi Soil Arching – Permanent Load Cases – Flexural Checks

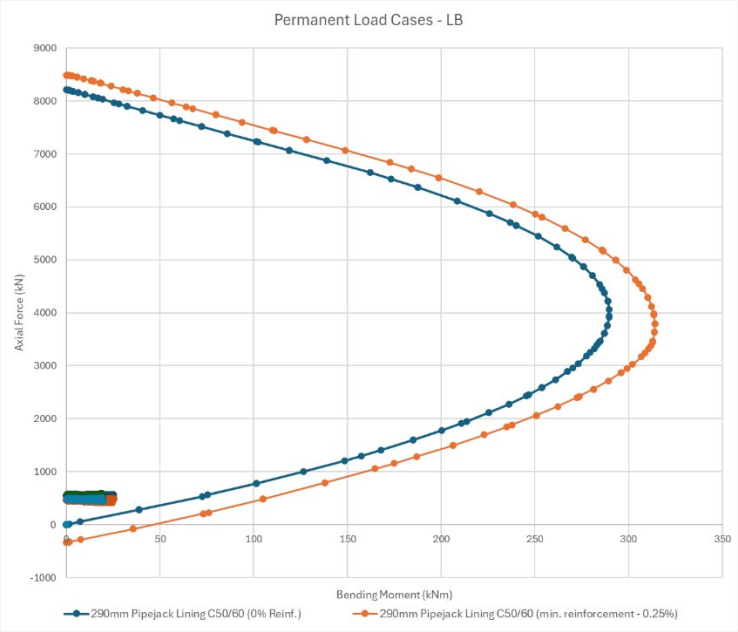


Figure 5-19 - Section E, lower bound Pipejack Lining Results – Terzaghi Soil Arching – Construction Load Cases – Flexural Checks

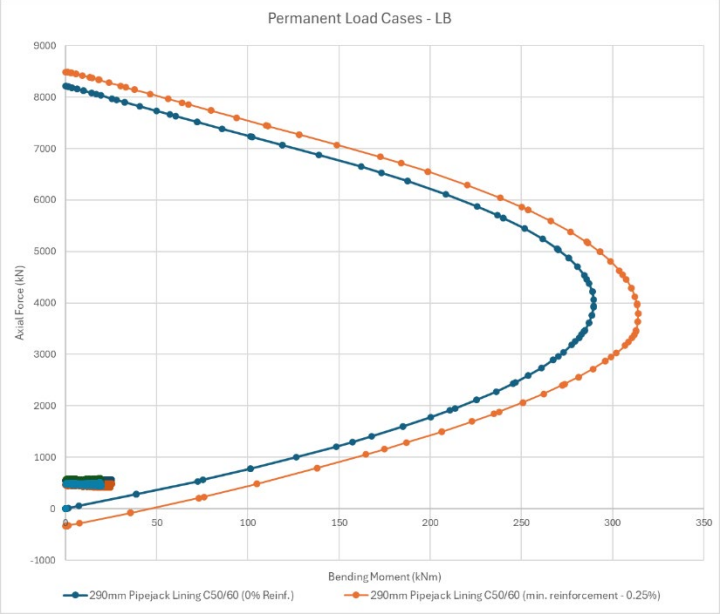
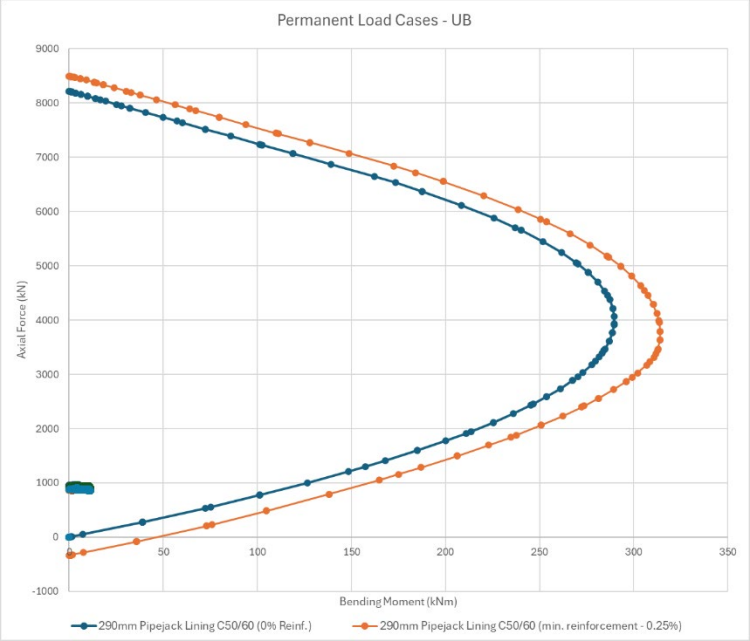


Figure 5-20 - Section E, lower bound Pipejack Lining Results – Terzaghi Soil Arching – Permanent Load Cases – Flexural Checks



It is evident from the plots above and those presented in Appendix A.2, that a 300mm thick segmental lining is sufficient to meet the axial forces/bending moments developed by the applied loads. It is evident from the plots above and those presented in Appendix A.2A.2, that a 300mm thick segmental lining is sufficient to meet the axial forces/bending moments developed by the applied loads.

- **Ultimate Limit State: *PCTL shear design***

The highest Q (Shear) with respect to N (axial thrust) have been defined to verify the shear capacity of the concrete. The shear capacity at ultimate limit state of the lining has been calculated in accordance with NEN EN 1992, Eurocode 2 – Design of concrete structures, CEN.

Shear reinforcement requirements within the tunnel lining are calculated considering the concrete as unreinforced (VRd,c). Based on results indicated in Table 5-22 which summarises the shear force for section E and section B, no shear reinforcement is required for these critical design sections, for details refer to Appendix A.3.

Table 5-22 - Summary of pipe lining shear design

Analysed Section	Axial Force [kN/m]	Shear Force [kN/m]	Shear Capacity [kN/m]	Utilisation Factor
Section B (LB)	619	91	239	0.38
Section B (UB)	625	77	239	0.32

Section E (LB)	995	95	286	0.33
Section E (UB)	996	94	286	0.33

- **Service Limit State: Crack width**

The maximum crack width has been calculated as 0.1mm considering the minimum reinforcement ratio (0.25%) provided, which is within the limit of 0.2mm for durability class XS2.

5.3.6 Segmental lining design

5.3.6.1 Bedded Beam Spring Model

As presented in the previous section, Oasys GSA software has been used for the segmental lining analysis. Where a lining is composed of segments, the stiffness at the joints may be appreciably less than elsewhere. The lining stiffness will be reduced based on the number of segments (n) greater than 4, the effective second moment of area (I_{ring}) will be used for lining design, as shown in Figure 5-21.

Figure 5-21 - Reduction in I for Joint Flexibility

Full second moment of area, I_{full}	$I_{full} = b.d^3 / 12$
Effective second moment of area, I_{ring}	$I_{ring} = I_{joint} + I_{full} \times (4 / \text{no. of joints})^2$ with $I_{ring} < I_{full}$

Section B, lower bound ground model

The same critical ground sections have been assessed in the design of the segmental lining as the pipejack lining. As such, the radial spring properties used to represent the lower bound properties of the ground at section B for the segmental lining are the same as those in Table 5-14. Table 5-23 summarises the soil arching results for the section B lower bound ground model.

Table 5-23 - Segmental Lining - Lower Bound – GSA Analysis Results Summary

		Terzaghi Soil Arching
Bending Moment (kNm)	**Min.**	-46
	Max.	61
Axial (kN)	**Min.**	-714
	Max.	6
Shear (kN)	**Min.**	-89
	Max.	79

Section B, Upper bound ground model

The same critical ground sections have been assessed in the design of the segmental lining as the pipejack lining. As such, the radial spring properties used to represent the upper bound properties of the ground at section B for the segmental lining are the same as those in Table 5-16.

Table 5-24 summarises the soil arching results for the section B upper bound ground model.

Table 5-24 - Segmental Lining - Upper Bound – GSA Analysis Results Summary

		Terzaghi Soil Arching
Bending Moment (kNm)	**Min.**	-33
	Max.	47
Axial (kN)	**Min.**	-716
	Max.	-5
Shear (kN)	**Min.**	-75
	Max.	70

Figure 5-22 and Figure 5-23 illustrate the maximum internal forces considering Terzaghi arching conditions for the section B lower bound ground model (maximum absolute forces in the lining).

Figure 5-22 - Section B, lower bound Segmental Lining Results – Terzaghi Soil Arching – Maximum Bending Moment

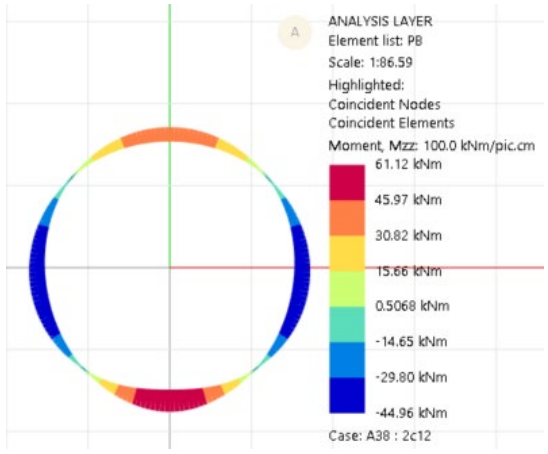
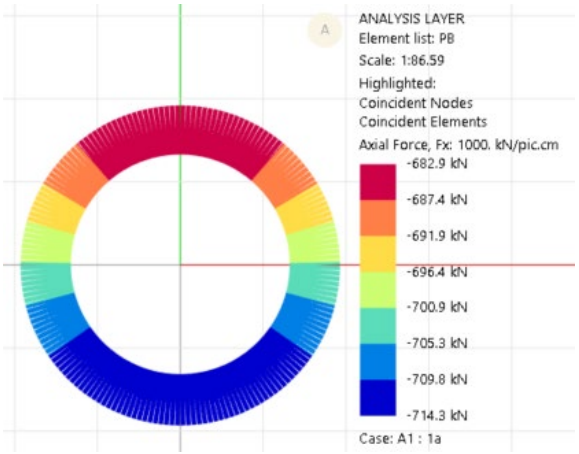


Figure 5-23 - Section B, lower bound Segmental Lining Results – Terzaghi Soil Arching – Maximum Axial Force (negative - compression)



Section E, lower bound ground model

The same critical ground sections have been assessed in the design of the segmental lining as the pipejack lining. As such, the radial spring properties used to represent the lower bound properties of the ground at section E for the segmental lining are the same as those in Table 5-18.

Table 5-25 summarises the soil arching results for the section E lower bound ground model.

Table 5-25 – Segmental Lining - Lower Bound – GSA Analysis Results Summary

		Terzaghi Soil Arching
Bending Moment (kNm)	**Min.**	-40
	Max.	55
Axial (kN)	**Min.**	-1080
	Max.	-35
Shear (kN)	**Min.**	-90
	Max.	82

Section E, Upper bound ground model

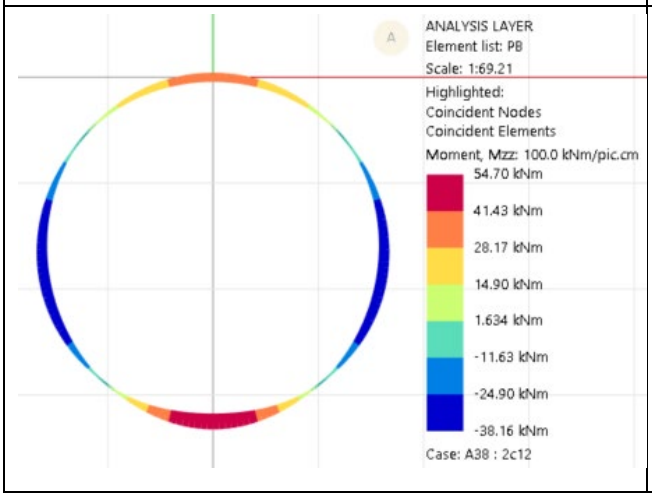
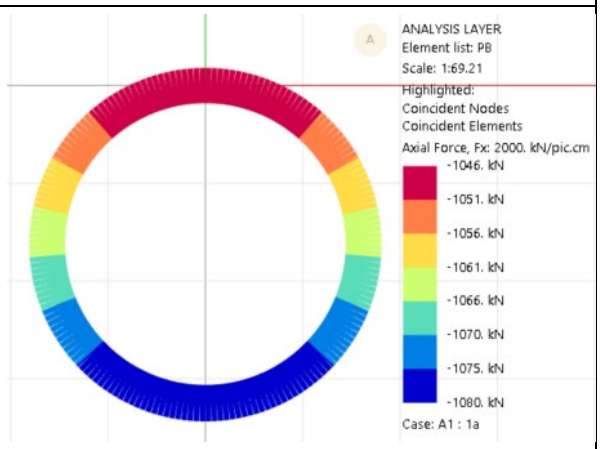
The same critical ground sections have been assessed in the design of the segmental lining as the pipejack lining. As such, the radial spring properties used to represent the lower bound properties of the ground at section B for the segmental lining are the same as those in Table 5-20.

Table -5-26 summarises the soil arching results for the section E upper bound ground model.

Table -5-26 – Segmental Lining - Upper Bound – GSA Analysis Results Summary

		Terzaghi Soil Arching
Bending Moment (kNm)	**Min.**	-40
	Max.	55
Axial (kN)	**Min.**	-1080
	Max.	-37
Shear (kN)	**Min.**	-90
	Max.	81

Figure 5-24 and Figure 5-25 illustrate the maximum internal forces considering Terzaghi arching conditions for the section B lower bound ground model (maximum absolute forces in the lining).

Figure 5-24 - Section E, lower bound Segmental Lining Results – Terzaghi Soil Arching – Maximum Bending Moment	Figure 5-25 - Section E lower bound Segmental Lining Results – Terzaghi Soil Arching – Maximum Axial Force (negative - compression)
	

5.3.6.2 Concrete design ULS and SLS checks

The checks in this section concern the forces that are mainly acting on the lining.

The tunnel lining will be subject to combinations of axial thrust and bending moment, axial thrust and shear force resulting from the different load combinations of ground, internal water pressure and hydrostatic loadings.

The concrete design checks have been performed on selected sections throughout the alignment, specifically in critical locations which may govern the design: Section B and section E.

- **Ultimate Limit State: *PCTL flexural design***

The worst combinations of M (bending moment) and N (axial thrust) are plotted on M-N capacity envelopes to define the overall structural capacity.

The flexural capacity at ultimate limit state of the lining has been calculated in accordance with NEN EN 1992, Eurocode 2 – Design of concrete structures, CEN. The imposed axial forces and bending moments from the design load cases have been checked against the lining section capacity curves.

Capacity curves for critical sections and load cases for the segmental lining sections are presented below (Figure 5-26 onwards). Additional results are presented in Appendix A.3. Please note that the blue capacity curve is that of a plain 240mm thick concrete section (250mm – 10mm durability allowance presented in Section 11.3), and orange curve is that with 30kg/m^3 steel fibre dosage (TR63).

Figure 5-26 - Section B, lower bound Segmental Lining Results – Terzaghi Soil Arching – Construction Load Cases – Flexural Checks

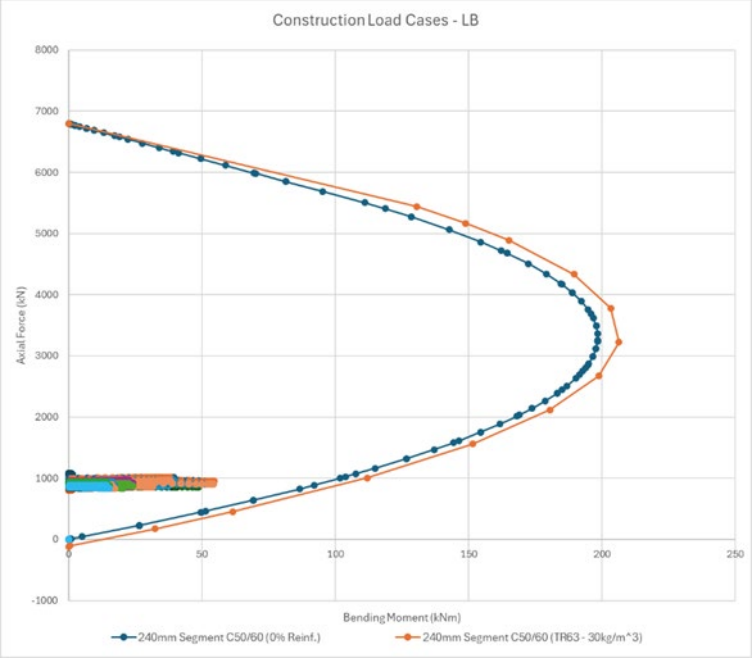


Figure 5-27 - Section B, lower bound Segmental Lining Results – Terzaghi Soil Arching – Permanent Load Cases – Flexural Checks

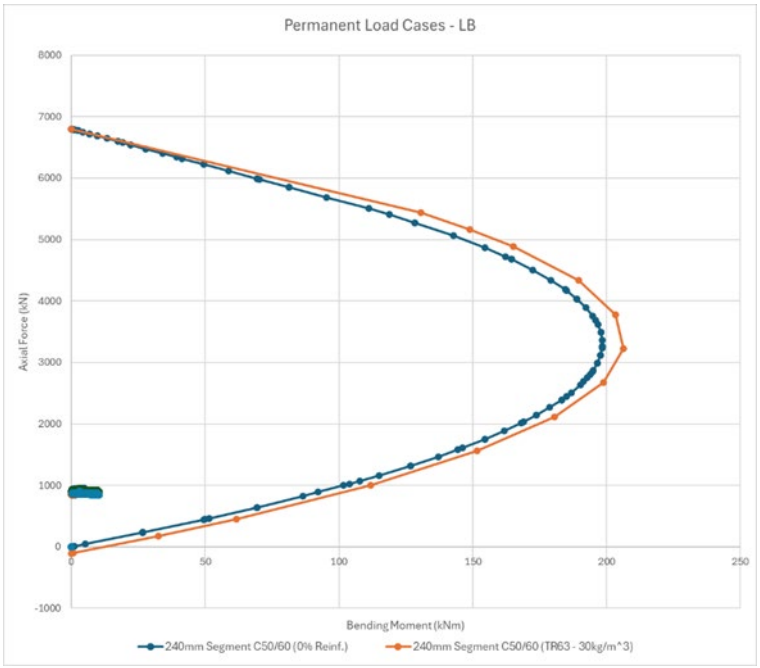


Figure 5-28 - Section E, lower bound Segmental Lining Results – Terzaghi Soil Arching – Construction Load Cases – Flexural Checks

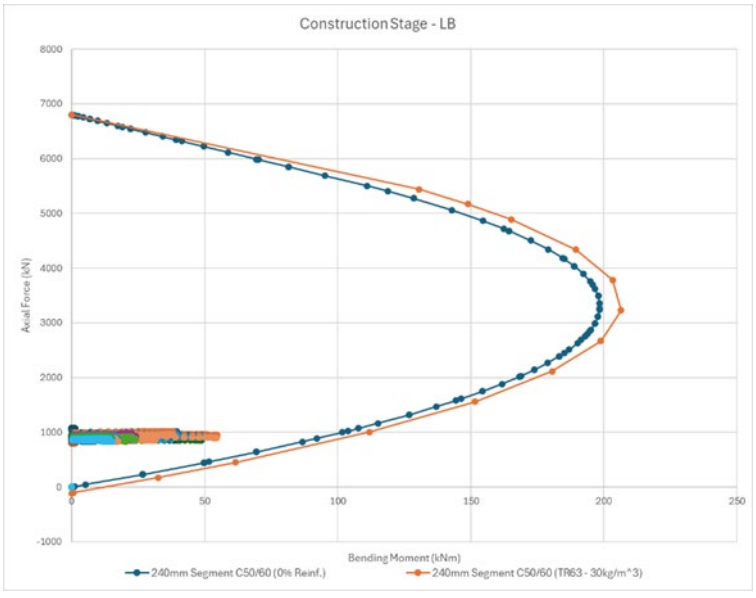
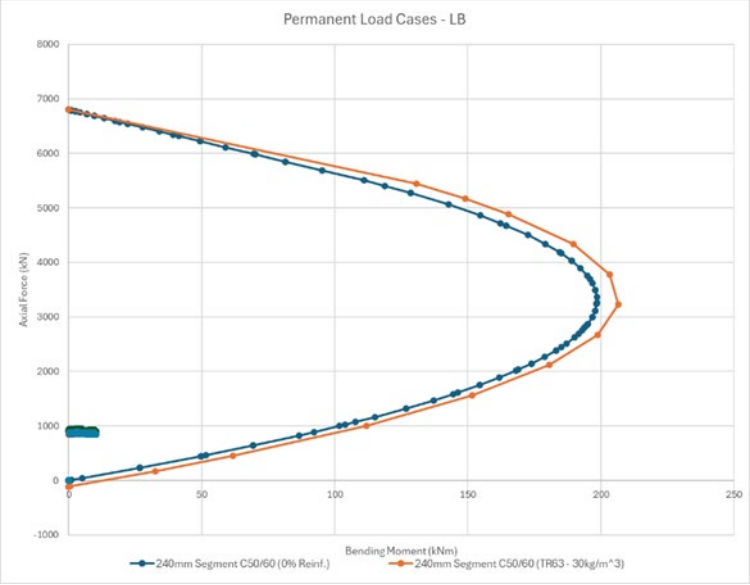


Figure 5-5-29 - Section E, lower bound Segmental Lining Results – Terzaghi Soil Arching – Permanent Load Cases – Flexural Checks



It is evident from the plots above and those presented in Appendix A.2, that a 250mm thick segmental lining is sufficient to meet the forces/bending moments developed by the applied loads.

- **Ultimate Limit State: *PCTL shear design***

The worst combinations of Q (Shear) and N (axial thrust) have been defined to verify the shear capacity of the concrete. The shear capacity at ultimate limit state of the lining has been calculated in accordance with NEN EN 1992, Eurocode 2 – Design of concrete structures, CEN.

Shear reinforcement requirements within the tunnel lining are calculated considering the concrete as unreinforced (VRd,c). Based on the shear internal forces, shear reinforcement is not required.

Table 5-27 summarises the shear design for the segmental lining. The table displays both section E and section B. Results are presented in Appendix A.3.

Table 5-27 Summary of segmental lining shear design

Analysed Section	Axial Force [kN/m]	Shear Force [kN/m]	Shear Capacity [kN/m]	Utilisation Factor
Section B (LB)	614	89	214	0.42
Section B (UB)	596	70	212	0.33
Section E (LB)	986	90	258	0.35

Section E (UB)	989	88	259	0.34

- **Service Limit State: Crack width**

The minimum requirements indicated at the NEN 1992, Eurocode 2 – Design of concrete structures, CEN, and lining design guidelines, regarding the lining crack widths and deformation tolerance will be followed during the lining capacity checks. The maximum crack width observed is 0.1mm, which falls within the limit of 0.2mm crack width durability limit of XS2.

5.3.6.3 Joint design

- Circumferential Joints

During construction, the TBM will advance by pushing the jacking pipe or segments with rams (hydraulic jacks).

High compressive stresses can be expected to develop under the jacking pad which result in the formation of tensile stresses within the lining.

The tensile stresses have been designed to be resisted in full by the steel reinforced element for the maximum and operational TBM thrust taking into account the tolerances, gap and eccentricity and also the force exerted by the compressed gasket.

The assessment has been carried out to estimate the tensile forces beneath the jacking pads by using the commonly accepted elastic approach by Leonhardt to calculate the tensile distribution within the segment for a given thrust. A force of 511kN per pad has been estimated using face pressures, and an approximate pad/shoe area of 0.128m^2.

Table 5-28 - Bursting Reinforcement Requirements - Ram Load

Segmental Lining - Bursting Reinforcement Requirements - Ram Load		
Model	**Area [mm^2]**	**Bars**
Ram Load	157	2-B10

- Radial Joints (segmental lining)

When the radial joints are subject to hoop load, similar bursting stresses as in the circumferential joints due to TBM ram loads are formed. In a similar fashion, the induced bursting stress shall either be resisted in full by the tensile strength of the concrete or be carried in full by the conventional reinforcement (see Table 5-29).

The stepping tolerance of 5mm and/or the rotation between the two segments are considered in the assessment. The preliminary elastic joint design has been carried out based on the following methodology:

- Establish joint contact stress and distribution incorporating effects of the ring ovalisation of 1%;

- Estimate maximum joint bursting stress and resulting tensile force using Leonhardt closed-form solution;
- Derive design bursting strength by applying appropriate material factor to the characteristic concrete tensile splitting strength;
- Ensure ultimate joint bursting is less than design bursting strength;
- Verification of bearing stresses in accordance with NEN EN 1992, Eurocode 2 – Design of concrete structures, CEN.

Table 5-29 - Bursting Reinforcement Requirements - Radial Joint

Bursting Reinforcement Requirements – Radial Joint			
Model	**Area [mm²]**	**Bars**	
Terzaghi Soil Arching Section B	235	3-B10	Intrados + Extrados
Terzaghi Soil Arching Section E	314	4-B10	Intrados + Extrados

5.3.7 Tunnel flotation

Uplift calculations were carried out in accordance with NEN 1997-1 to check that the design value of the destabilising vertical actions ($V_{dst;d}$) is less than or equal to the sum of the stabilising permanent vertical actions ($G_{stb;d}$) and any additional resistance to uplift (R_d).
Partial factors were adopted based uplift limit state verifications (UPL) as stated in NEN 1997-1. Stabilising forces are multiplied by a factor of 0.9 and destabilising forces are multiplied by a factor of 1.1.
Figure 5-30 provides a simplified illustration of the forces present within the flotation check. Figure 5-31 presents the N-M results of the flotation check for LB ground conditions for section B and E under Terzaghi Arching soil loading conditions.

Figure 5-30 – Forces acting on tunnel.

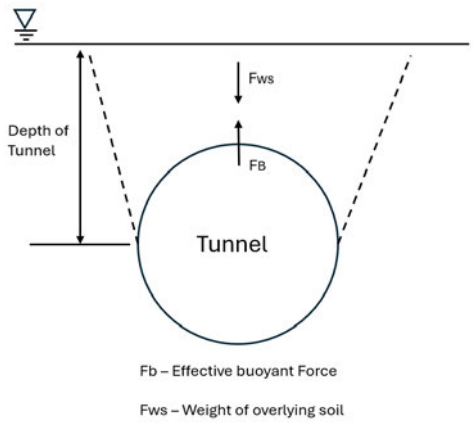
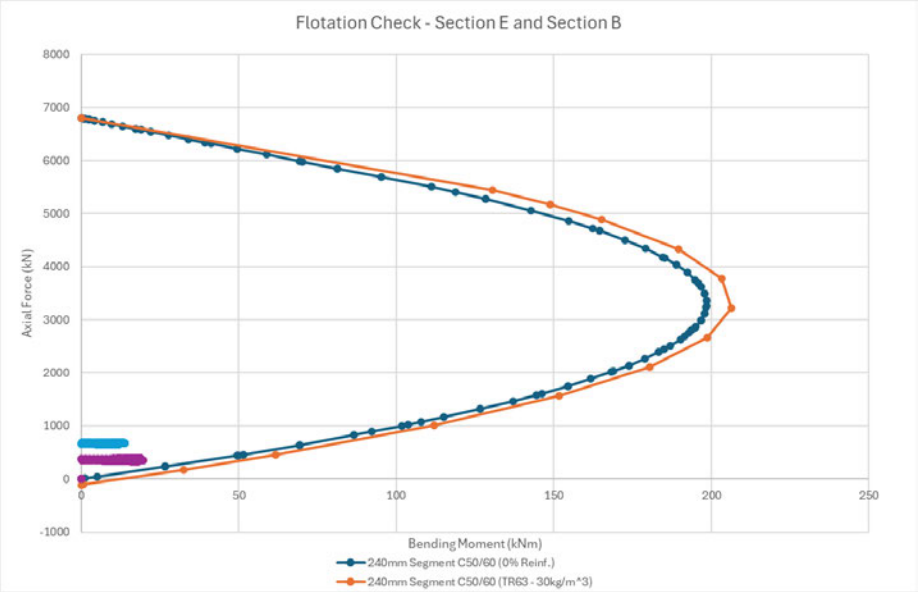


Figure 5-31 – N-M results – Segmental Lining flotation check



For information, to resist the buoyant forces and avoid uplift during the construction case (before tunnel is backfilled), approximately 6.5m of overburden above the tunnel is required to achieve a factor of safety of 1. Please refer to section 8.1.3 for checks on flotation during the retrieval process.

6 SHAFT DESIGN

6.1 Description of the shaft

6.1.1 Structural form

The shaft structure will be located in the sharkfin site, with an approx. clearance of 20m between the site boundary and the shaft wall (with shaft / tunnel interface at point KP0.018). The location of the shaft has been fixed as described in the 'Shaft Location Selection Technical Note' (Doc. Ref.: NL-ARM-040-PTC1-110265_01) issued by Contractor. The shaft is constructed within the Made Ground (dredged material from the reclaim of Maasvlacte 2) and Naaldwijk Formation, which are primarily sandy, high permeability soils with low-strength strata/pockets of clays and peat, as described in the GIR (Doc. Ref.: NL-ARM-040-ARU1-110303) and in the GDR (Doc. Ref.: NL-ARM-040-ARU1-110305).

The shaft will serve the dual purpose of:

- Launching the pipe jacking and TBM for the construction of the tunnel;
- Facilitating the installation of the pipe (installation of the winching system in the shaft and receiving the pipeline from the offshore reception pit through the tunnel).

From the vertical shaft the tunnel will exit at a gradient of -5% to pass under the sea defence embankment.

During preliminary design, a comparison exercise was carried out to define the most appropriate construction methodology for the shaft. The construction cost and programme, health, safety and environmental requirements, availability of plant and watertightness of the shaft wall were the main criteria assessed. A pre-cast segmental shaft and a diaphragm wall shaft were concluded to be the most appropriate options for the launch shaft. Based on previous experience in the Netherlands and the possible limited availability of contractors who have experience in building segmental shafts, the diaphragm wall structure has been progressed for the purpose of FEED.

6.1.2 Geometry and spaceproofing

The proposed shaft structure will have a circular shape and an internal diameter of 16.7m. This diameter:

- was confirmed to be adequate for the operations within the shaft (construction of the tunnel and installation of the pipeline);
- will allow for an off-the-shelf segment selection in case a segmental shaft is the preferred option by the Client and selected Contractor in the future.

For the purpose of FEED, the shaft structure for the Diaphragm Wall option comprises:

- A 1.2m thick diaphragm wall (which is assumed to be constructed using three-bite panels in accordance with Dutch construction practice), which will extend 1m above the existing ground level and with a total depth of 27.6m. Depending on the D-Wall panel size, chosen by the EPC contractor to suit their equipment, the resulting shaft may have a more polygonal appearance than the idealised circular concept;
- A 3m thick structural slab, consisting of an initial unreinforced layer 2m thick and a second fully reinforced layer 1m thick;
- 91No. 150mm diameter tension piles, anchored into the 1st unreinforced layer of the structural base slab. The tension piles will be designed to resist the buoyancy forces at the bottom of the shaft structure and will be reinforced with Dywidag or Macalloy reinforcing systems;
- A capping beam 1.4m wide and 1m high. The capping beam is raised 1m above the temporary piling platform to act as a protection against falls during the construction phase;
- A thrust wall with an average thickness of 2m;
- A tunnel eye with an average thickness of 1m.

Shaft spaceproofing has been assessed as described in the Preliminary Design Report [2] to accommodate the tunnel construction and the winching operation during the installation of the pipeline. The internal diameter of 16.7m is deemed to be adequate for the tunnel construction equipment for both construction options of pipe-jacking and TBM methods, as shown on spaceproofing drawing NL-ARM-040-ARU1-110319.

The pipeline design consultant, Offshore Independents, have also confirmed that the shaft spaceproofing is adequate for the proposed winching equipment.

6.2 Loading

6.2.1 Construction actions

6.2.1.1 Surcharge

The shaft construction will require plant to operate at the surface (excavators, gantry crane etc.), storage of reinforcing cages, tunnel segments, excavated spoil etc. A surface surcharge of 20kPa has been considered uniformly on the surface around the shaft to represent random plant movements. Note that an 1m thick piling platform is also proposed, to accommodate a stable support for the d-wall rigs etc. and to mitigate the hazard of a ground surface with poor bearing capacity.

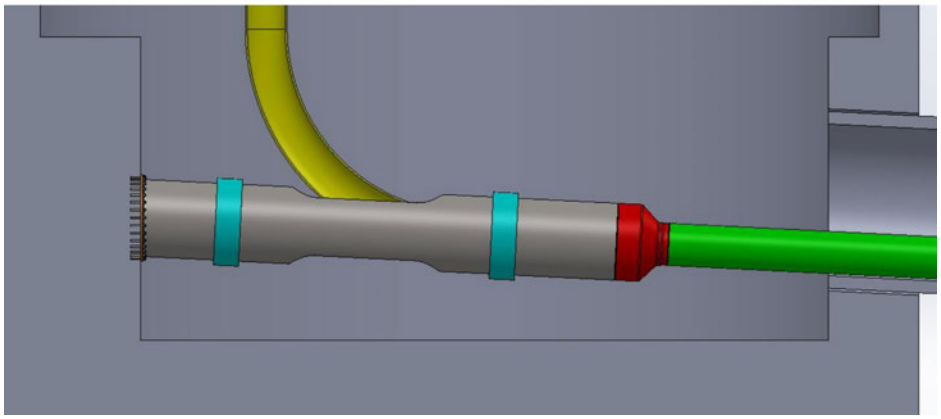
6.2.1.2 Tunnel Jacking Thrust Action

A maximum design jacking force of 3208 tonnes has been considered as outlined in Table 5-11. This has been modelled as a pressure on the thrust wall, over an area 5x5m. In reality, there will be several jacks with spreader plates, however these have not been designed in detail at this stage.

A friction factor of 0.5 has been used for the tunnel jacking seal to determine the drag force of the jacking force on the tunnel eye, taken from an example product data sheet shown in Appendix B below. This results in a drag force of 1604 tonnes applied around the tunnel eye.

The 'Landfall Construction Concept Selection Technical Note' (Doc. Ref.: NL-ARM-040-PTC1-114070) issued by Offshore Independents on 7th June 2024 describes the proposed riser configuration and the anchor flange support structure, as shown below in Figure 6-1. As per the aforementioned technical note, the anchor system, including the thrust wall, should have a nominal compressional capacity of 1830t, which is considerably lower than the characteristic jacking force of 3208t used in the design. The anchor system will also be required to have a tensile capacity of 380t, and therefore the bolted connection to the thrust wall will need to be designed for this load. Please note that the bolted connection will be designed at the detailed design stage, and the EPC contractor shall consider the impact of cyclic/fatigue loading resulting from repeated expansion and contraction of the pipeline during EPC detailed design.

Figure 6-1 - Anchor flange support structure in the shaft (Ref.: Figure 2-9, NL-ARM-040-PTC1-114070)



6.2.1.3 Pipe Pulling Winch Action

The winching loads on the shaft wall have been applied in accordance with the 'Offshore Pipeline Pull-in Operation – Sheave Support Structure Design Report' (Doc. Ref.: NL-ARM-PTC1-113860 Revision 00).

6.2.1.4 Crane Loads

A mobile crane for short time-frame construction activities, such as lifting in the D-wall panel reinforcing cages and lifting the TBM into the shaft, with an approximate total pressure of 48tonnes/patch applied at four locations to account for four patches. Note that this is a conservative assumption for the design. Different mobile cranes with lower weights could be used (calculation assumes 96t crane). A gantry crane for more frequent construction operations, to lift the tunnel precast segments into the shaft for the tunnel construction. A total load of 10tonnes/patch, conservatively assuming one patch for the most onerous scenario of a concentrated load. The lateral stresses on the shaft, generated by the crane operations, have been calculated based on the assumption that Ko conditions apply. Given the fact that the shaft is predominantly located in sandy ground, Ko=0.52, rather than K=1.0 for undrained clays, has been used for the design. If extensive clays are found on site at detailed design stage, this assumption and subsequently the design will need to be revisited.

6.2.2 Self-weight, ground and water actions

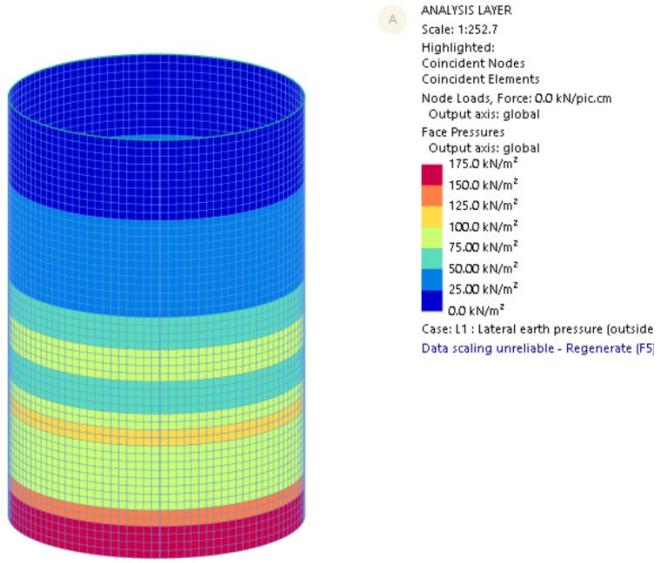
6.2.2.1 Self-weight

Self-weight of structural elements is considered as the product of the volume and the density of the structural component.

6.2.2.2 Ground Pressure

For the preliminary load assessment due to the ground pressures acting on the launch shaft lining, the close-form solution of Duddeck and Erdmann (1985) [16] was used. At this FEED stage, the lateral ground pressures and the corresponding ground spring stiffness are applied to the shaft numerical model. See Figure 6-2 below.

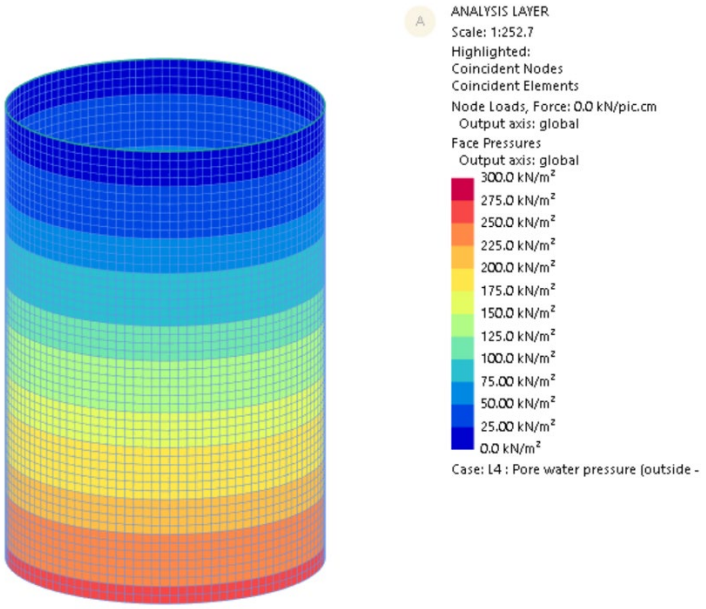
Figure 6-2 – Ground pressures applied to the shaft model



6.2.2.3 Groundwater Pressure

Groundwater pressure on the launch shaft and base slab is calculated as a hydrostatic pressure load defined by the design maximum ground water level with an increased unit weight of 10.25kN/m3 to account for the presence of sea water. See Figure 6-3 below.

Figure 6-3 – Groundwater pressures applied to the shaft model



6.2.2.4 Material factors

The design has generally considered a limit state approach in accordance with Netherlands Eurocodes, using the partial factor method. The Ultimate Limit State (ULS), Uplift (UPL) and Serviceability Limit State (SLS) of the structure have been checked.

The design value of a material or product shall be taken as:

$$X_d = X_k/\gamma_M$$

Where:

X_k = Characteristic value of the material

γ_M = Partial factor of the material

The partial factors for the materials to be used are presented in Table 6-1.

Table 6-1 - Partial factors for materials

Material	ULS	Reference
Concrete	1.5	NEN EN 1992-1-1 Table 2.1N
Steel	1.15	NEN EN 1992-1-1 Table 2.1N
Ground (geotechnical material)	1.0[1]	NEN EN 1997

Notes:

1. NEN *EN 1997 requires a partial factor to the effective ground strength of 1.25, however a conservative factor may be used when deriving the stiffness values from CPTs and a large sensitivity range is calculated in the design process. This is significantly higher than the material partial factor proposed in* NEN *EN 1997. The resulting ground load is multiplied by a partial load factor and is applied to conservative closed form solutions to calculate structural stresses.*

6.2.3 Load combinations

Load and combination factors are assigned according to the respective limit state in accordance with Netherland Eurocodes. Load factors for actions are summarised in Table 6-2 and Table 6-3, assuming a structural consequence class of 2 (for further details on the consequence class, please see section 11).

Table 6-2 - Load factors for actions

Load	ULS (STR)	UPL	Reference
Permanent loads (γ_G) *Including ground and horizontal groundwater loads*	1.35	0.9	NEN EN 1990 Table A2.4(B)
Variable loads (γ_Q)	1.5	0.0	NEN EN 1990 Table A2.4(B)
Groundwater loads for buoyancy	-	1.1	NEN EN 1990 Table A2.4(B)

Table 6-3 - Summary of loads

Description	ULS	UPL
Self-weight	1.35	0.9
Ground pressure	1.35	-
Groundwater pressure	1.35	1.1
Surcharge	1.50	-
Tunnel jacking thrust action	1.50	-
Pipe pulling winch action	1.50	-

6.3 Design calculations

6.3.1 Analysis model

6.3.1.1 Methodology

Oasys General Structural Analysis (GSA) version 10.1 has been used to create a 3D structural model of the launch shaft, as shown in Figure 6-4. The shaft geometry was input as summarised in Section 6.1.2.

Figure 6-4 - Launch shaft GSA model



The diaphragm wall, unreinforced structural slab, reinforced structural slab, tunnel eye and thrust wall have been modelled as 2D shell elements. These consist of 2D plate elements, predominantly ‘quad 4’ type (4-noded quadrilateral elements). The capping beam has been modelled as a beam element (2-noded line beam).

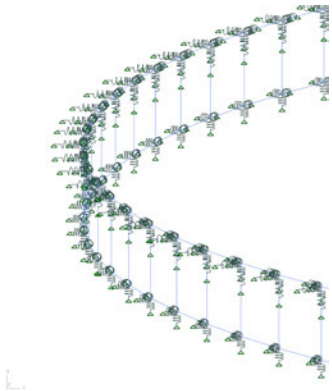
The shaft was modelled in several consecutive stages, to align with the construction staging, as shown on NL-ARM-040-ARU1-110332.

A non-linear static solver was used to analyse the models targeting a maximum out-of-balance force and moment value of 0.1%.

6.3.1.2 Boundary conditions

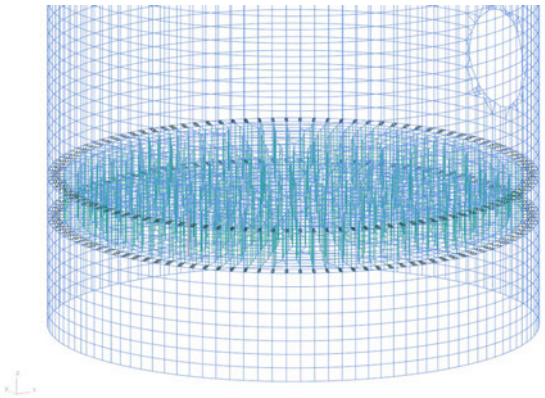
A series of radial compression only springs were applied to the diaphragm wall to represent the soil confinement and representative stiffness. Tangential springs of negligible stiffness are also provided to stabilise the model against global rotation by representing the frictional resistance of the soil to small “out-of-balance” forces against the walls of the shaft. Each spring is restrained at one end with pinned supports, as shown in Figure 6-5.

Figure 6-5 - Shaft lining boundary conditions



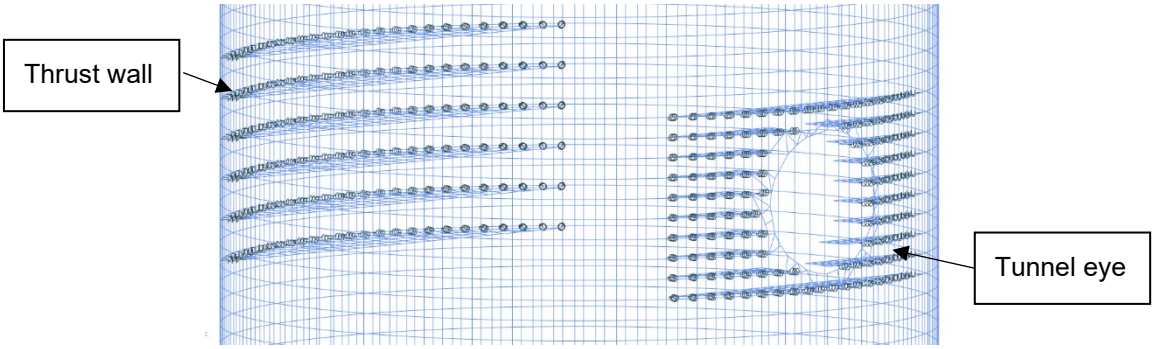
Similarly, compression only springs were modelled to represent a connection between the two layers of the structural slab and diaphragm wall elements, with spring stiffness set to represent the concrete material. Bar elements (transmitting forces but not moments) were used to connect the lower unreinforced structural slab and upper reinforced structural slab. See Figure 6-6.

Figure 6-6 - Structural slab connections



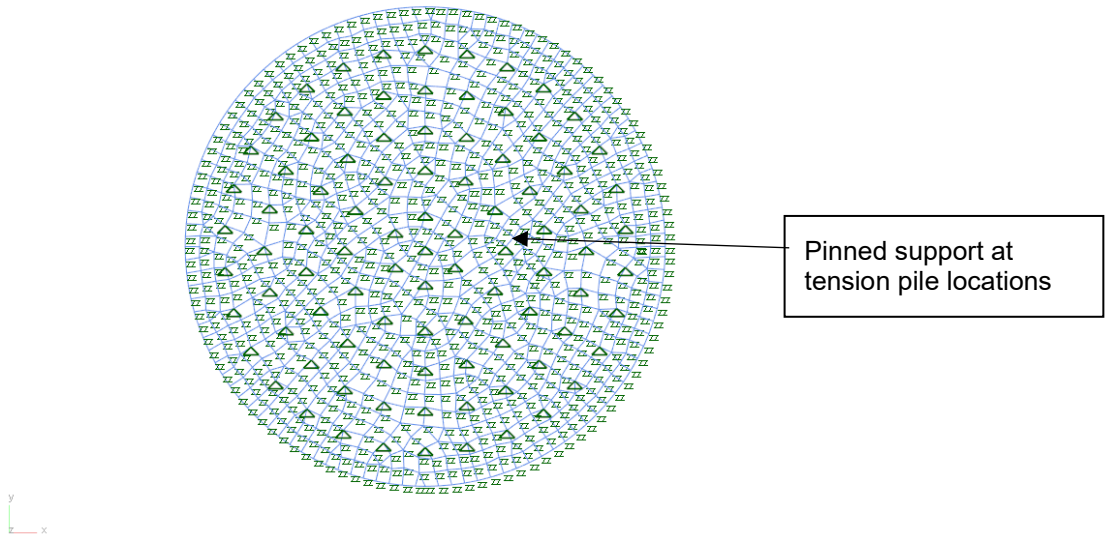
General springs were used to simulate the connection between the tunnel eye and the shaft lining, and also the connection between the thrust wall and the shaft lining. See Figure 6-7.

Figure 6-7 - Tunnel eye & thrust wall connections



Pinned restraints were modelled to represent the connection between the mass concrete plug and the tension piles beneath the plug. Figure 6-8.

Figure 6-8 - RC base slab boundary conditions



6.3.2 Shaft flotation

As described in the Preliminary Design Report [2], the shaft had initially been assessed against buoyancy based on the weight of the structural shaft elements (structural base slab, d-wall, capping beam) and the corresponding uplift force at the soffit of the base slab and diaphragm wall toe level. This assessment, however, relied on a structural connection between the d-wall panels and the structural base slab, that would require complex diving activities during construction. Moreover, the assessment concluded that additional measures were required for the shaft to withstand the uplift forces.

Following discussions with Company in the HAZID workshop and based on previous experience in the Netherlands, an alternative method using vertical tension piles constructed from the surface prior to the excavation and tied into the shaft structural base slab, was chosen as the preferred measure to prevent the shaft from floating. This approach has the further safety benefit of considerably reducing diving activities.

The number and length of the piles is based on the assumption that only the weight of the structural base slab acts as a stabilising force, alongside the friction mobilised by the piles' tensile capacity. A structural connection between the d-wall panels and the base slab is not required for this option, eliminating the associated Health & Safety risks. However, the steel anchorage plates at the top of the tension piles will still need to be installed by divers.

Therefore, 91 no. 150mm diameter tension piles are to be constructed in the wet from the ground level, with 50mm diameter Dywidag or Macalloy reinforcement installed from the first, unreinforced base slab pour to the toe of the piles, as shown on drawings NL-ARM-040-ARU1-110332, 110333 and 110334.

6.3.3 Diaphragm wall design

6.3.3.1 Flexural

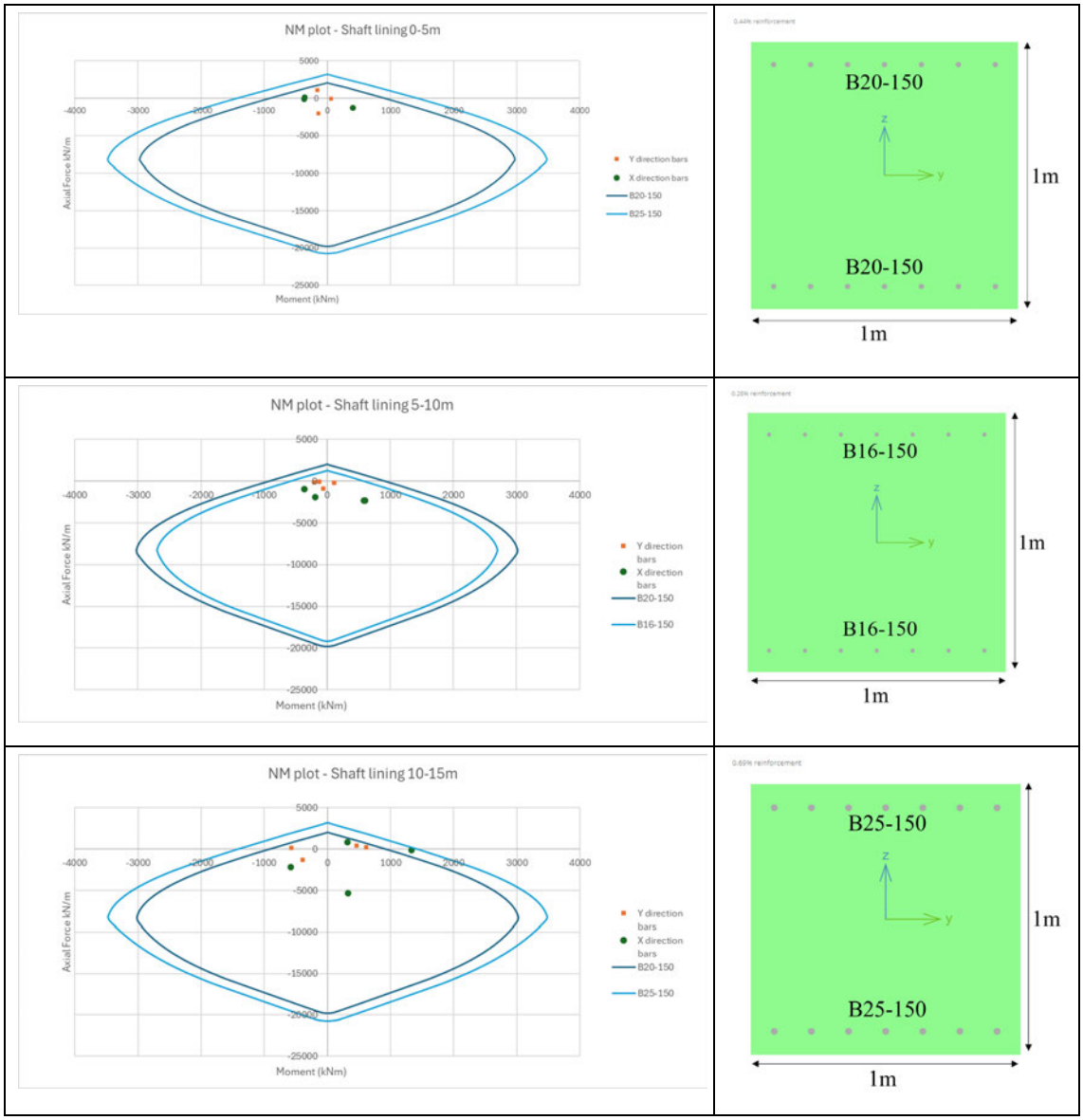
The flexural design for the diaphragm walls has assumed a reduced wall thickness of 1m to account for construction tolerances.

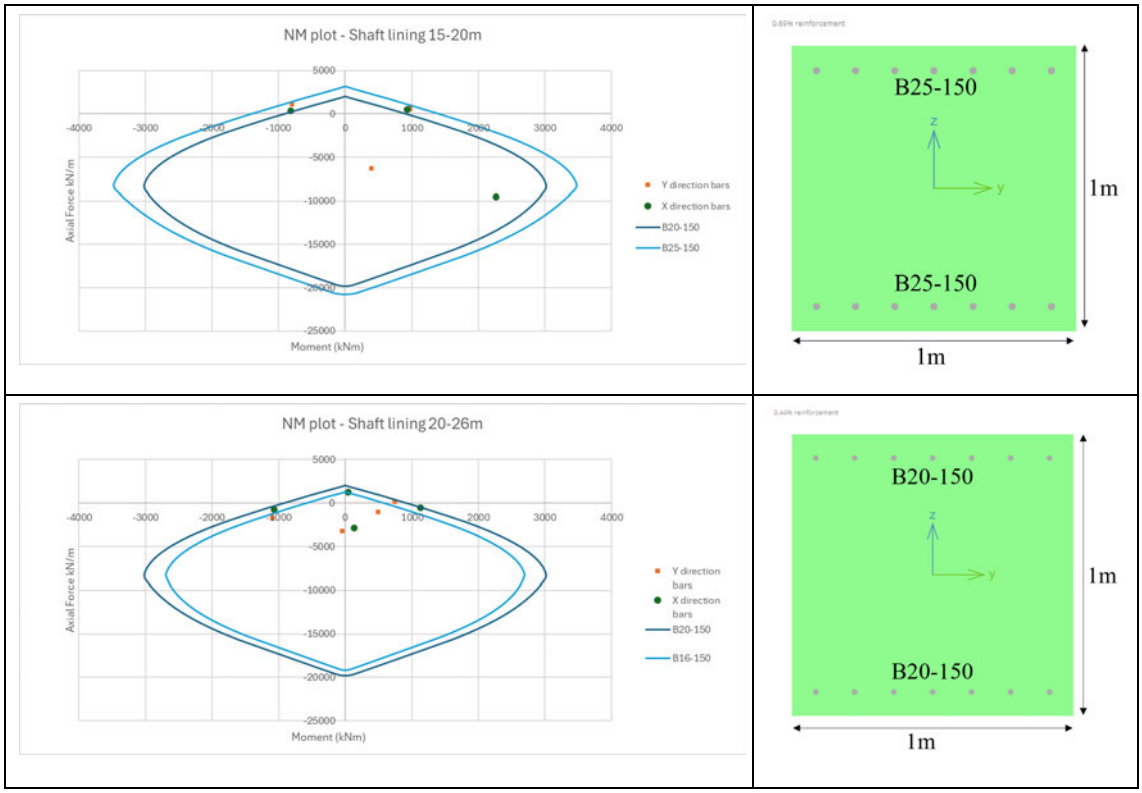
The results of the non-linear static analysis summarised in Section 6.3.1.1 have been processed using Oasys Adsec version 10.0. For the governing construction stages, the maximum/minimum moments and forces with coexisting forces and moments were determined at 5m intervals.

The coexisting loads were used to determine the required reinforcement arrangement within each 5m interval, using Oasys Adsec version 10.0. The forces and bending moments have been plotted on an interaction diagram against the section design capacity curves. The resulting N-M interaction plots for each 5m height are shown in Table 6-4. In summary, at the location of maximum forces, B25 at 150mm spacing is required in both faces. See calculations in Appendix B.

The diaphragm wall in the tunnel eye area is likely to be glass fibre reinforced polymer (GFRP) to aid breakout.

Table 6-4 - NM plot for reinforced diaphragm wall

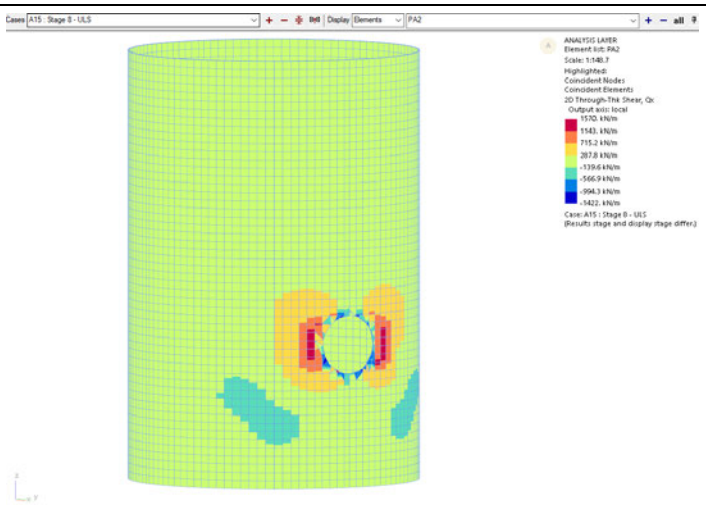
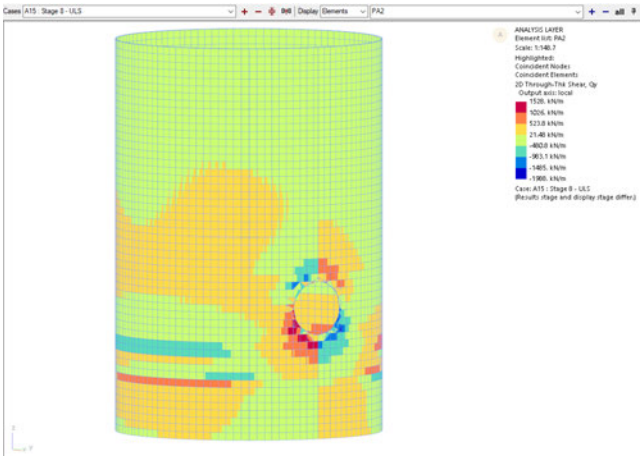




6.3.3.2 Shear

Shear reinforcement requirements within the diaphragm wall are calculated according to NEN EN 1992-1-1, considering the concrete as unreinforced ($V_{Rd,c}$). For the majority of the diaphragm wall, shear reinforcement is not required. For the governing construction stage, there are some localised shear spikes requiring localised shear reinforcement, as shown in Table 6-5. 2 legs of B16 links at 150mm spacing are required in these regions. See calculations in Appendix B.

Table 6-5 – Diaphragm wall 2D shear diagrams

x direction (Circumferential or Hoop Plane)	
y-direction (Vertical plane)	

6.3.3.3 Early/late thermal cracking

The RC diaphragm wall in-situ concrete element is designed to Ciria C766 to identify the cracking potential and the minimum reinforcement required to prevent uncontrolled cracking. The concrete is designed according to the following mix parameters:

- Limestone aggregate (thermal conductivity of 1.8 W/m°C, as per Ciria 766 Table A);
- 0% of CEM replacement;
- A placing temperature of 15°C;
- A specific heat of 1 kJ/kg°C (in the absence of concrete trials);
- No effective wind speed (conservative assumption which results in a higher temperature drop);

- Ambient humidity of 50%;
- Coefficient of thermal expansion of concrete of 9.5με/°C;
- Early age creep factor of 0.65;
- A maximum restraint of 0.7 (Edge restraint);

Ciria C766 acknowledges that the restraint factor is a significant contributing factor for the calculation of the likelihood for cracking and the resulting crack widths. An estimation has been made based on calculated restraint as recommended in Ciria C766 and the resulting restraint factor from calculation is 0.7.

The resulting early age/long term crack width is less than the 0.2mm limit according to NEN EN 1992-1-1. See calculations in Appendix B.

Considering potential for water leakage arising from potential through-width cracking in the D-walls during construction, the short term crack width has been checked against NEN EN 1992-3 Water Tightness Class 1 criteria and the associated maximum crack width limit of approximately 0.15mm is satisfied using 40mm bars at 150mm spacing. (Note that the maximum crack width requirement is a function of water depth and wall thickness and this should be further verified during EPC detailed design.)

6.3.3.4 SLS Crack width

A SLS crack width check to NEN EN 1992-1-1, using a limit of 0.2mm for flexural cracking, has been completed using SLS bending moments. The maximum flexural crack width with B40 reinforcement at 150mm spacing is 0.17mm i.e. acceptable. See calculations in Appendix B.

6.3.4 Base slab design

6.3.4.1 Flexural

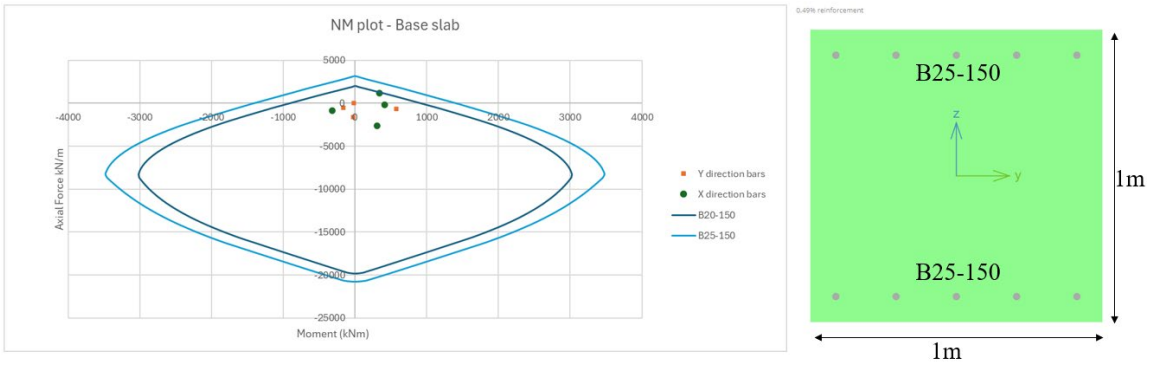
The base slab is made up of an unreinforced (no-steel bar) layer 2m thick and a fully reinforced (steel bar) layer 1m thick above.

The design of the lower 2m thick slab is to be refined at the detailed design stage - fibre reinforcement may be used if required.

The fully reinforced 1m structural slab above is conservatively designed as a simply supported beam in accordance with STR limit state design.

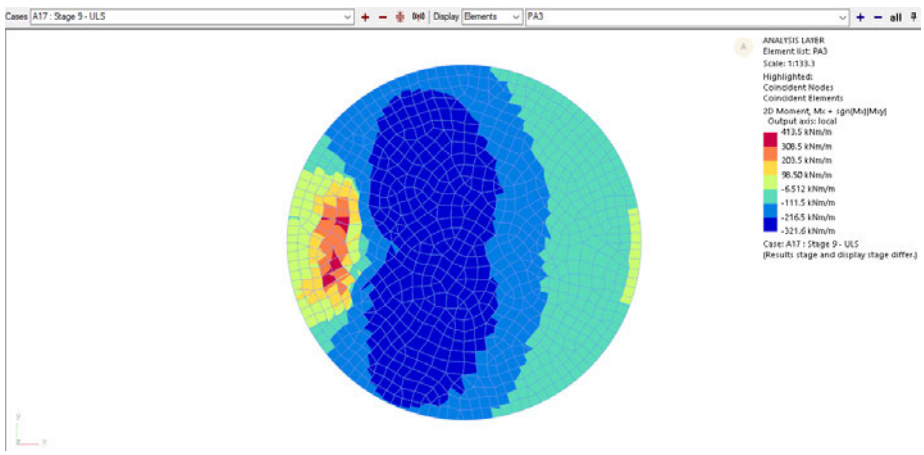
For the governing construction stage, the maximum/minimum moments and forces with coexisting forces and moments were determined from the non-linear static analysis summarised in Section 6.3.1. They were used to determine the reinforcement arrangement for the 1m thick base slab section, using Oasys Adsec version 10.0. The forces and bending moments have been plotted on an interaction diagram against the section design capacity curves. The resulting NM plot is shown in Figure 6-9, and concludes B25 at 150mm spacing is required in both directions.

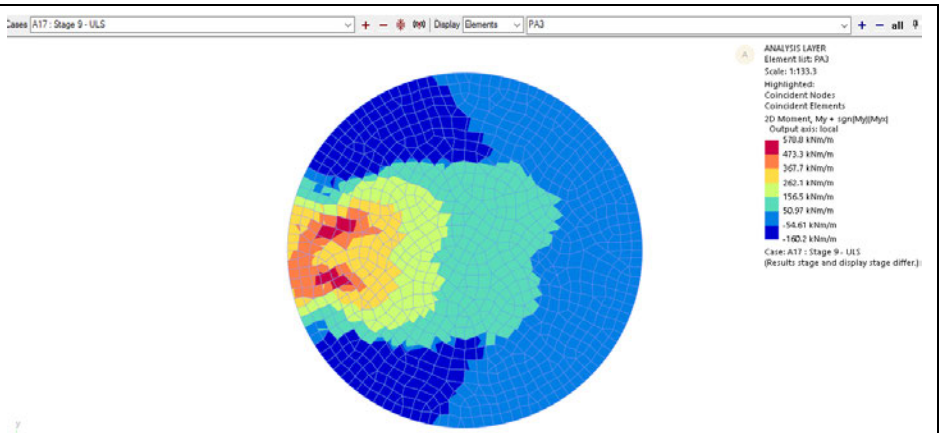
Figure 6-9 - NM plot for reinforced base slab



At locations with positive moments in Table 6-6, the base slab is in hogging since the winch attachment is pulling the base slab upwards. Elsewhere the slab is in sagging. For conservatism, B25 at 150mm spacing is proposed in both top and bottom layers.

Table 6-6 - Base slab 2D moment diagrams

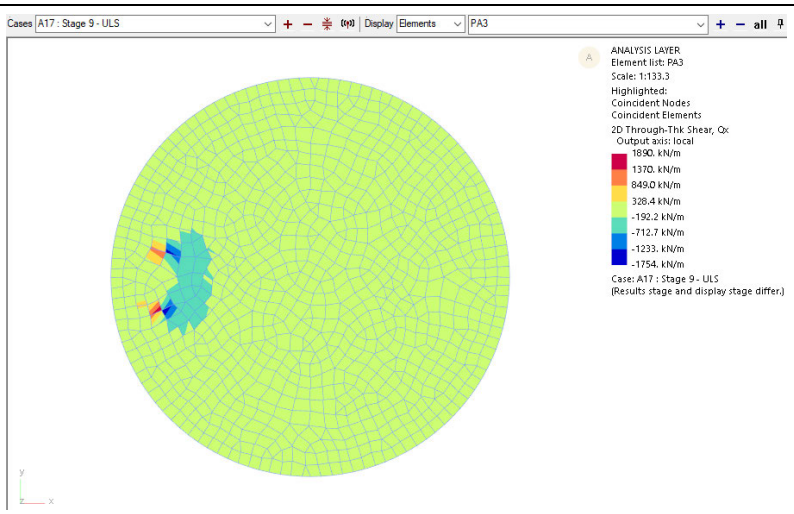
x direction	

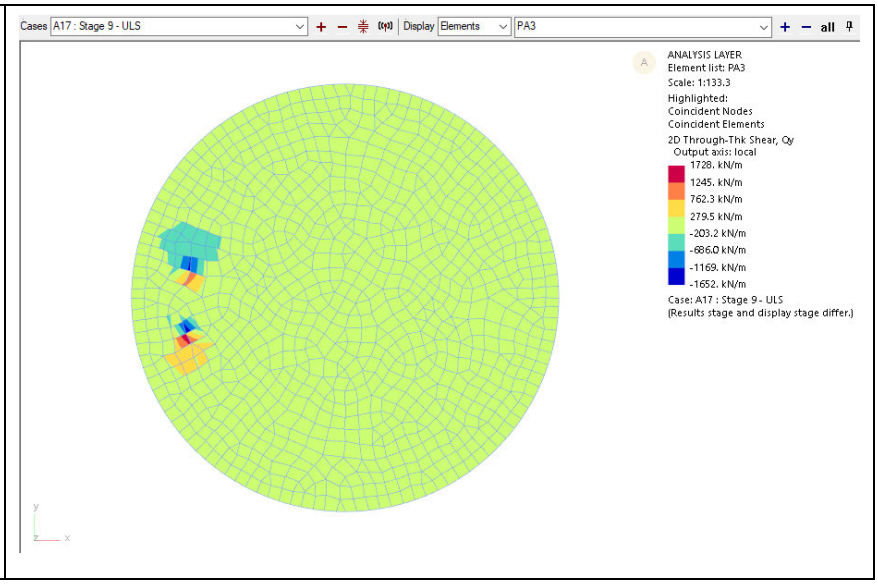
y-direction	

6.3.4.2 Shear

Shear reinforcement requirements within the base slab are calculated according to NEN EN 1992-1-1, considering the concrete as unreinforced ($V_{Rd,c}$). For the majority of the slab, shear reinforcement is not required. For the governing construction stage, there are some localised shear spikes at the winching locations, as shown in Table 6-7, requiring localised shear reinforcement. 2 legs of B16 links at 150mm spacing are required in these regions. See calculations in Appendix B.

Table 6-7 - Base slab 2D shear diagrams

x direction	

y-direction	

6.3.4.3 Early/late thermal cracking

The RC base slab in-situ concrete element is designed to Ciria C766 to identify the cracking potential and the minimum reinforcement required to prevent uncontrolled cracking. The concrete is designed according to the following mix parameters:

- Limestone aggregate (thermal conductivity of 1.8 W/m°C, as per Ciria 766 Table A);
- 0% of CEM replacement;
- A placing temperature of 15°C;
- A specific heat of 1 kJ/kg°C (in the absence of concrete trials);
- No effective wind speed (conservative assumption which results in a higher temperature drop);
- Ambient humidity of 50%;
- Coefficient of thermal expansion of concrete of 9.5με/°C;
- Early age creep factor of 0.65;
- A maximum restraint of 0.14 (Edge restraint);

Ciria C766 acknowledges that the restraint factor is a significant contributing factor for the calculation of the likelihood for cracking and the resulting crack widths. An estimation has been made based on calculated restraint as recommended in Ciria C766 and the resulting restraint factor from calculation is 0.14.

The resulting early age/long term crack width is less than the 0.2mm limit according to NEN EN 1992-1-1. See calculations in Appendix B.

Considering potential for water leakage arising from potential through-width cracking in the base slab during construction, the short term crack width has been checked against NEN EN 1992-3 Water Tightness Class 1 criteria; the slab is deemed uncracked in the short term due to early thermal cracking effects.

6.3.4.4 SLS crack width

A SLS crack width check to NEN EN 1992-1-1, using a limit of 0.2mm for flexural cracking, has been completed using SLS bending moments. The maximum crack width with B25 reinforcement at 150mm spacing is 0.13mm which is within the limit. See calculations in Appendix B.

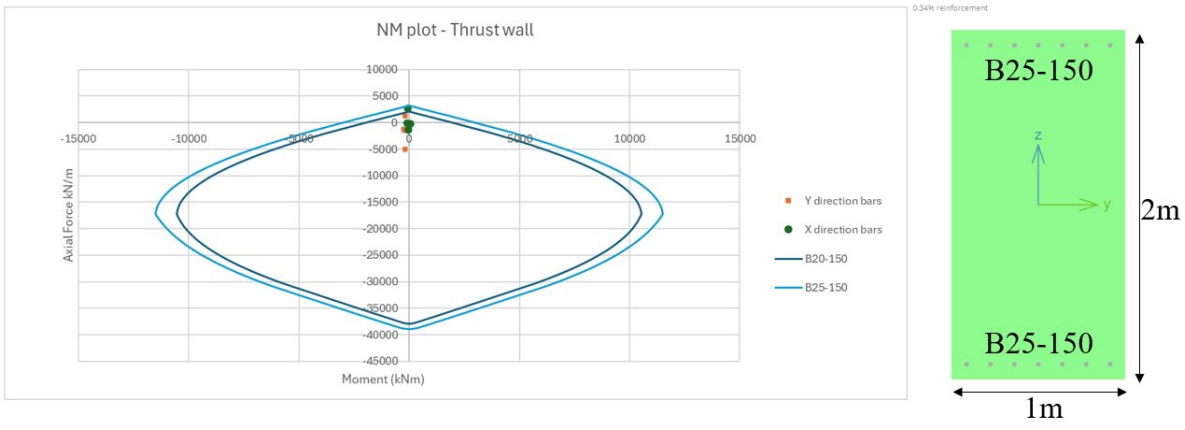
6.3.5 Thrust wall design

6.3.5.1 Flexural

The fully reinforced thrust wall is conservatively designed as a simply supported beam in accordance with STR limit state design.

For the governing construction stage, the maximum/minimum moments and forces with coexisting forces and moments were determined from the non-linear static analysis summarised in Section 6.3.1. They were used to determine the reinforcement arrangement for the thrust wall section, using Oasys Adsec version 10.0. The forces and bending moments have been plotted on an interaction diagram against the section design capacity curves. The resulting NM plot is shown in Figure 6-10, and concludes B25 at 150mm spacing is required in both directions.

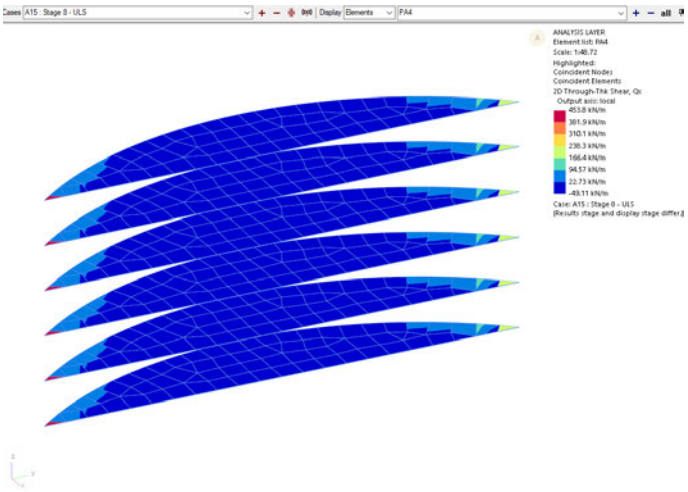
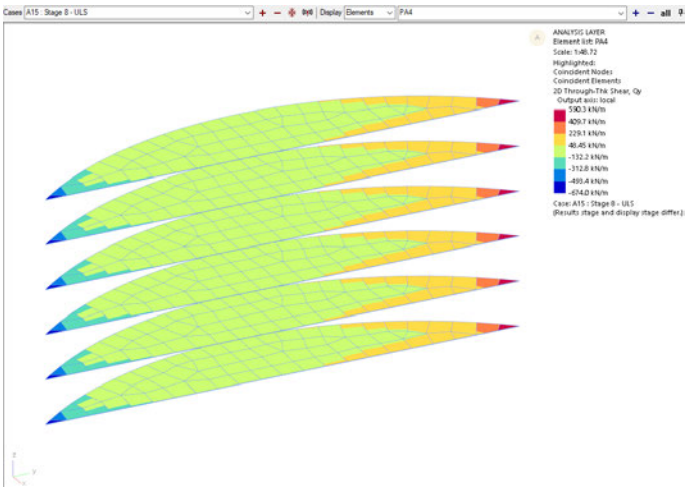
Figure 6-10 - NM plot for reinforced thrust wall



6.3.5.2 Shear

Shear reinforcement requirements within the thrust wall are calculated according to NEN EN 1992-1-1, considering the concrete as unreinforced ($V_{Rd,c}$). For the majority of the wall, shear reinforcement is not required. For the governing construction stage, there are some localised shear spikes at the corners, as shown in Table 6-8, requiring localised shear reinforcement. 2 legs of B16 links at 150mm spacing are required in these regions. See calculations in Appendix B.

Table 6-8 – Thrust wall 2D shear diagrams

x direction	
y-direction	

6.3.5.3 Early/late thermal cracking

The RC thrust wall in-situ concrete element is designed to Ciria C766 to identify the cracking potential and the minimum reinforcement required to prevent uncontrolled cracking. The concrete is designed according to the following mix parameters:

- Limestone aggregate (thermal conductivity of 1.8 W/m°C, as per Ciria 766 Table A);
- 0% of CEM replacement;
- A placing temperature of 15°C;
- A specific heat of 1 kJ/kg°C (in the absence of concrete trials);
- No effective wind speed (conservative assumption which results in a higher temperature drop);
- Ambient humidity of 50%;
- Coefficient of thermal expansion of concrete of 9.5με/°C;
- Early age creep factor of 0.65;
- A maximum restraint of 0.42 (Internal restraint);

Ciria C766 acknowledges that the restraint factor is a significant contributing factor for the calculation of the likelihood for cracking and the resulting crack widths. As recommended in Ciria C766, the restraint factor of 0.42 has been used given internal restraint.

The resulting early age/long term crack width is less than the 0.2mm limit according to NEN EN 1992-1-1. See calculations in Appendix B.

6.3.5.4 SLS crack width

A SLS crack width check to NEN EN 1992-1-1, using a limit of 0.2mm for flexural cracking, has been completed using SLS bending moments. The maximum crack width with B32 reinforcement at 150mm spacing is 0.15mm which is within the limit. See calculations in Appendix B.

6.3.6 Tunnel eye design

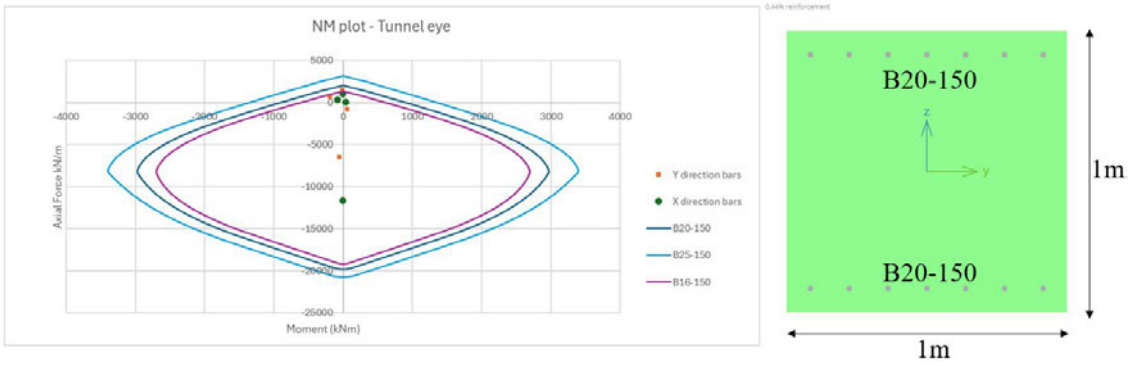
6.3.6.1 Flexural

The fully reinforced tunnel eye is conservatively designed as a simply supported beam in accordance with STR limit state design. The tunnel eye will have a weak plug that will be excavates as the first tunnel ring is pushed through.

For the governing construction stage, the maximum/minimum moments and forces with coexisting forces and moments were determined from the non-linear static analysis summarised in Section 6.3.1. They were used to determine the reinforcement arrangement for the tunnel eye section, using Oasys Adsec version 10.0. The forces and bending moments have been plotted on an interaction diagram against the section design capacity curves. The resulting NM plot is shown in Figure 6-11, and concludes B20 at 150mm spacing is sufficient in both directions.

Note, the analysis model is to be refined in the tunnel eye area at the next detailed design stage.

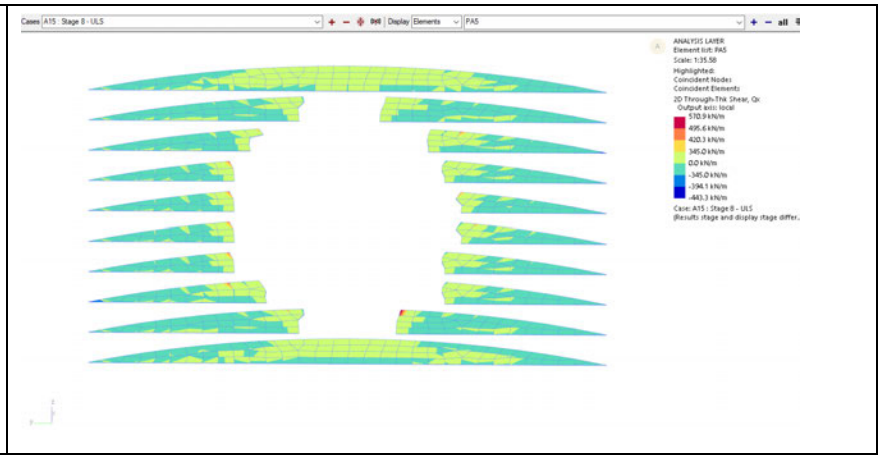
Figure 6-11 - NM plot for reinforced tunnel eye

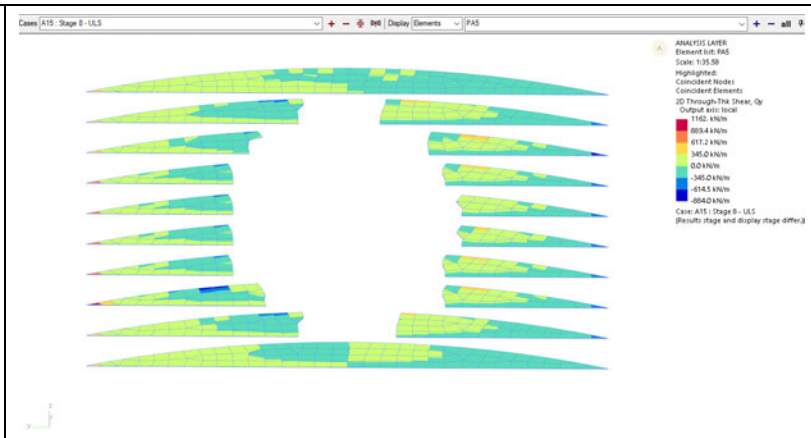


6.3.6.2 Shear

Shear reinforcement requirements within the tunnel eye are calculated according to NEN EN 1992-1-1, considering the concrete as unreinforced ($V_{Rd,c}$). For the governing construction stage, there are some localised shear spikes around the tunnel adit as shown in Table 6-9 which are unrealistic (considered to be “Hot Spots” caused by accumulation of numerical rounding errors or ill-conditioned element geometry) and so have been ignored. Shear reinforcement is not required within the tunnel eye, see calculations in Appendix B.

Table 6-9 – Tunnel eye 2D shear diagrams

x direction	

y-direction	

6.3.6.3 Early/late thermal cracking

The RC tunnel eye in-situ concrete element is designed to Ciria C766 to identify the cracking potential and the minimum reinforcement required to prevent uncontrolled cracking. The concrete is designed according to the following mix parameters:

- Limestone aggregate (thermal conductivity of 1.8 W/m°C, as per Ciria 766 Table A);
- 0% of CEM replacement;
- A placing temperature of 15°C;
- A specific heat of 1 kJ/kg°C (in the absence of concrete trials);
- No effective wind speed (conservative assumption which results in a higher temperature drop);
- Ambient humidity of 50%;
- Coefficient of thermal expansion of concrete of 9.5με/°C;
- Early age creep factor of 0.65;
- A maximum restraint of 0.42 (Internal restraint);

Ciria C766 acknowledges that the restraint factor is a significant contributing factor for the calculation of the likelihood for cracking and the resulting crack widths. As recommended in Ciria C766, the restraint factor of 0.42 has been used given internal restraint.

Since the tunnel eye is temporary, only the early thermal age crack width has been checked. The resulting early age crack width is less than the 0.2mm limit according to NEN EN 1992-1-1. See calculations in Appendix B.

6.3.6.4 SLS crack width

A SLS crack width check to NEN EN 1992-1-1, using a limit of 0.2mm for flexural cracking, has been completed using SLS bending moments and the section remains uncracked. See calculations in Appendix B.

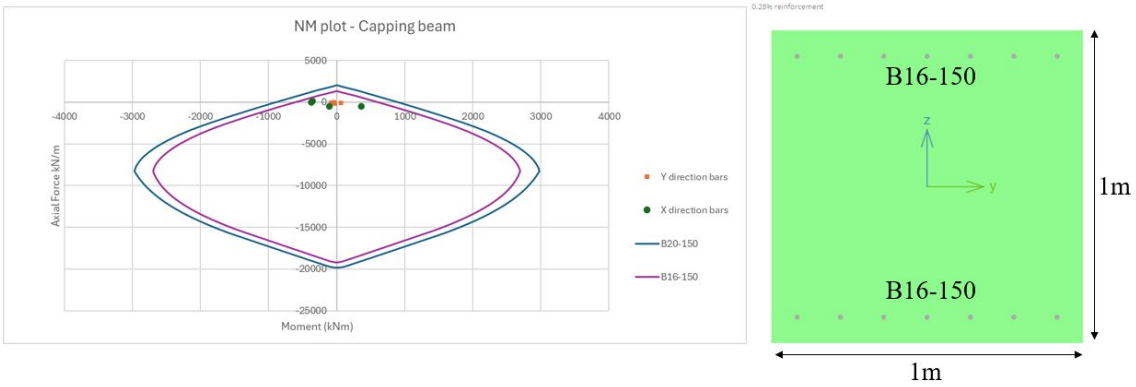
6.3.7 Capping beam design

6.3.7.1 Flexural

The fully reinforced capping beam is conservatively designed as a simply supported beam in accordance with STR limit state design.

For the governing construction stage, the maximum/minimum moments and forces with coexisting forces and moments were determined from the non-linear static analysis summarised in Section 6.3.1. They were used to determine the reinforcement arrangement for the capping beam section, using Oasys Adsec version 10.0. The forces and bending moments have been plotted on an interaction diagram against the section design capacity curves. The resulting NM plot is shown in Figure 6-12, and concludes B16 at 150mm spacing is sufficient in both directions.

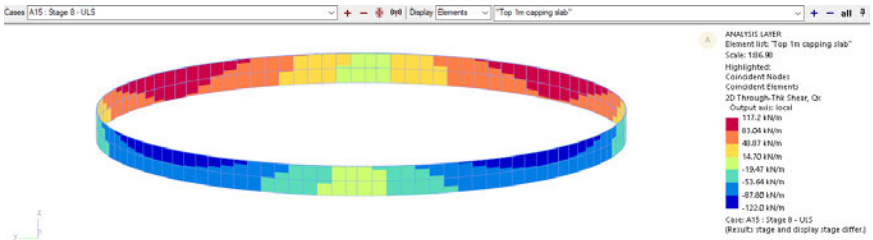
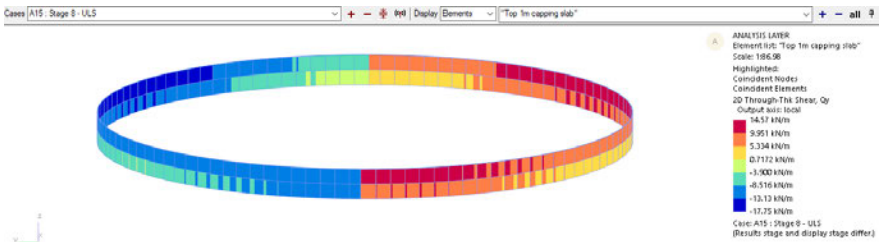
Figure 6-12 - NM plot for reinforced capping beam



6.3.7.2 Shear

Shear reinforcement requirements within the capping beam are calculated according to NEN EN 1992-1-1, considering the concrete as unreinforced ($V_{Rd,c}$). For the governing construction stage, the design shear is less than the design shear resistance as shown in Table 6-10. Therefore, shear reinforcement in the capping beam is not required, see calculations in Appendix B.

Table 6-10 – Capping beam 2d shear diagrams

x direction	
y-direction	

6.3.7.3 Early/late thermal cracking

The RC capping beam in-situ concrete element is designed to Ciria C766 to identify the cracking potential and the minimum reinforcement required to prevent uncontrolled cracking. The concrete is designed according to the following mix parameters:

- Limestone aggregate (thermal conductivity of 1.8 W/m°C, as per Ciria 766 Table A);
- 0% of CEM replacement;
- A placing temperature of 15°C;
- A specific heat of 1 kJ/kg°C (in the absence of concrete trials);
- No effective wind speed (conservative assumption which results in a higher temperature drop);
- Ambient humidity of 50%;
- Coefficient of thermal expansion of concrete of 9.5με/°C;
- Early age creep factor of 0.65;
- A maximum restraint of 0.42 (Internal restraint);

Ciria C766 acknowledges that the restraint factor is a significant contributing factor for the calculation of the likelihood for cracking and the resulting crack widths. As recommended in Ciria C766, the restraint factor of 0.42 has been used given internal restraint.

The resulting early age/long term crack width is less than the 0.2mm limit according to NEN EN 1992-1-1. See calculations in Appendix B.

6.3.7.4 SLS crack width

A SLS crack width check to NEN EN 1992-1-1, using a limit of 0.2mm for flexural cracking, has been completed using SLS bending moments. The maximum crack width, assuming B25 reinforcement at 150mm spacing, is 0.16mm which is within the limit. See calculations in Appendix B.

6.3.8 Summary of reinforcement

In summary, Table 6-11 summarises the proposed reinforcement ratios for the launch shaft structural elements. Please note these reinforcement ratios are presented based on the required reinforcement only and do not include any additional allowance for fixings, laps, etc.

Table 6-11 – Reinforcement ratios for launch shaft design

Structural element	Required reinforcement [Bar diameter(mm) - spacing(mm)]	Governing check	Reinforcement ratio (average) (kg/m^3)
Base slab	B25-150	ULS flexural	135
Diaphragm wall	B40-150	SLS and early thermal crack width	285
Thrust wall	B32-150	SLS crack width	170
Tunnel eye	B20-150	Minimum reinforcement	140
Capping beam	B25-150	SLS crack width	205

7 RECEPTION PIT DESIGN

7.1 Introduction

7.1.1 Scope of design

This section considers the design of the reception pit required for TBM retrieval and pipeline installation. The reception pit design has been developed considering multiple options for TBM retrieval – initially assuming a base case with a “monopile” sleeve to retrieve the TBM, and subsequently updated on Company preference for “internal strengthening” (secondary lining) of the tunnel near to the reception pit prior to TBM retrieval. For full details of the TBM retrieval method, see Section 8. Corresponding variations of the reception pit are included here.

The reception pit takes the form of an open excavation in the seabed, with the geometry and setting out of the pit based on several factors. The tunnel alignment shown in drawing NL-ARM-040-ARU1-110323 primarily determines the setting out of the pit and proximity to existing assets (such as the TenneT cables) and hazards (such as UXO).

The depth of the tunnel at the retrieval location has a primary influence on the depth of the pit. The TBM recovery methodology has a secondary influence on the required dredge depth, as (if monopile option is adopted) this may need to accommodate activities such as placement of levelling gravel, concrete slabs etc. The geometry of the pit is also influenced by:

- the dredged slope gradients that can remain stable for the required timings of the works;
- any additional slope gradient requirements not related to stability (e.g. as may arise through EPC Contractor preferences or future dredging contractor recommendations);
- the footprint at the base of the excavation (TBM geometry plus allowances for access etc.)

Note this section was initially developed in consideration of completing the tunnel in segmental lining mode; for implications if completing the tunnel in pipe jack mode, refer to section 7.5.

7.1.2 Basis of design

The Tunnel FEED Basis of Design [1] was prepared when multiple options were being considered for the TBM recovery methodology and reception pit. These were appraised during preliminary design and the open pit solution preferred. The BOD outlines requirements for the open pit solution to consider:

- buried hazards & risks (archaeology, UXO)
- working space (working windows, exclusion zones etc.)
- TBM cover and drive angle.
- Type of dredging equipment, slope stability and rate of accretion.

The BOD highlights input from others being required for certain aspects of the open pit:

- Local dredging contractor experience with respect to achievable side slopes and rates of excavation and backfilling, limitations on dredge plant (given the pit depth), if it would be feasible for different dredge plant to be used for different stages of the project, access to licensed offshore excavation and disposal sites etc.;
- Company advice from regulator discussions on potential working limitations for dredging methodology, confirmation whether temporary storage is available on land, strategy for UXO removal etc.

Since BoD issue, the following changes in requirements are relevant to the reception pit:

- Arup were requested to consider potential loading from vessels (due to anchor drop, anchor dragging and / or vessels sinking). The assumed loads are included in section 5.2.

7.1.3 Development since preliminary design

The PDR [2] appraised options for TBM recovery and recommended a dredged open pit was carried forward, as described within this FEED report. The design of the reception pit has since been developed through multiple workshops between Consultant, Company, Contractor, as well as workshops with specialist dredging contractors. Thus, additional narrative is included to describe the development including envisaged geometry and dredging methodology. Note:

- The PDR made several recommendations for Company attention. Recommendations included sharing more information with Arup, including any other pre-FEED studies on unexploded ordnance (UXO) and strategy for UXO removal, as well as any consenting aspects that may impact the reception pit works, so that these matters could be considered when developing the reception pit methodology. No further information has been received and the recommendation remains valid.
- The reception pit design was initially developed based on the “monopile” solution, before Company instruction on 11/06/24 to adopt the “internal strengthening” as preferred while maintaining the “monopile solution” as alternative. This section discusses both options.
- The reception pit methodology has been developed through discussion with 2 dredging Contractors as detailed within this section of the report.

Figure 6-13 of the PDR showed a 20m x 10m excavated base area, set approx. 17m below bed level (based on the tunnel crown being 13m below the seabed). A 134m x 124m plan area was envisaged. The dredged side slopes of the pit were 1v:5h in the upper loose sand layer and 1v:3h slope in the lower sand layer. A 5m flat bench was included at the transition between slopes.

The tunnel alignment has been developed since PDR, with technical justification being produced for raising the crown to a minimum of 7.2m below seabed level. The reception pit size has subsequently been refined as described in this chapter. The refinements affect the bottom of the dredged pit, and the overall dredge volume:

- For the internal strengthening option, the pit is dredged to approx. 10.6m below seabed level, which partially exposes the TBM.

- The monopile option requires the pit to be dredged to approx. 14.5m below seabed level, which includes dredging 3m to 4m below the tunnel to accommodate the monopile and supporting structures (concrete slab & gravel bed).

Section 7.2.3 describes input received from 2No dredging contractors, including advice on the side slopes of the pit, subsequent to preliminary design. Arup PLAXIS analysis has also been undertaken suggesting that the slopes shown at PDR stage (based on research papers) should be slightly shallower as now described in Section 7.2.

In service and extreme environmental conditions (wind, wave, current) were assessed in the Preliminary Design Report. Detailed numerical modelling of these actions transformed to reception pit level and the resulting effect this has on sediment transport are outside the scope of the FEED design. Dredging Contractor advice was sought on their experience with sediment transport into similar excavations during regular tides, with a rate of "*5-10 cm/ week during a normal summer period. No details can be given during the winter period*" being advised. The probability of severe storms during the period the pit is open and sediment transport during this event is more complex to predict with reasonable certainty. Consequences from storms may include bed material being carried into the reception pit, above typical seasonal sedimentation rates. Should these loadings have severe consequence for any aspects of the design, this may require further study at EPC stage.

7.2 Description of the reception pit

7.2.1 Monopile option

The monopile retrieval option was developed first during FEED.

Within the pit, the monopile option requires deeper excavation to install the monopile sleeve, levelling gravel, concrete slabs etc. Conceptual details for these items are developed as described in Section 8. Further development of the details would be required by the EPC Contractor, including TBM recovery contractor and / or the contractor responsible for follow-on pipeline laying activities. Temporary fill may be required within the pit for protection of the tunnel (and any operatives within) from vessel related hazards such as dropped anchors.

Figure 7-1 shows the envisaged pit geometry on plan for the monopile option, which results in a dredge volume of approximately 166,000 m3. Figure 7-2 shows the envisaged pit geometry in section for the monopile option. The figures show the maximum extents of the reception pit relative to the final tunnel location, providing allowance for foundations and temporary overburden described in Section 8.4.

Figure 7-1 - Plan view on reception pit – monopile option (from NL-ARM-040-110329)

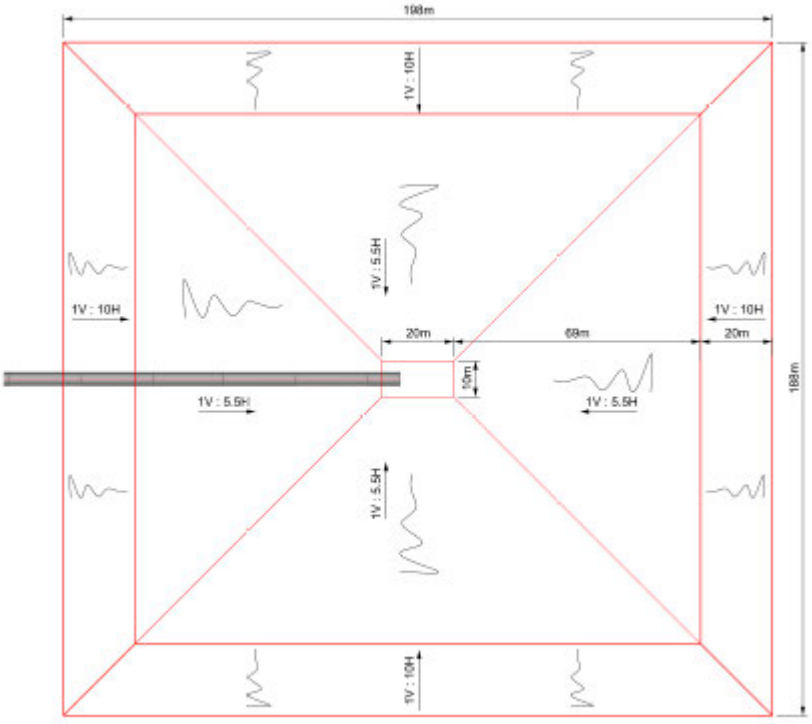
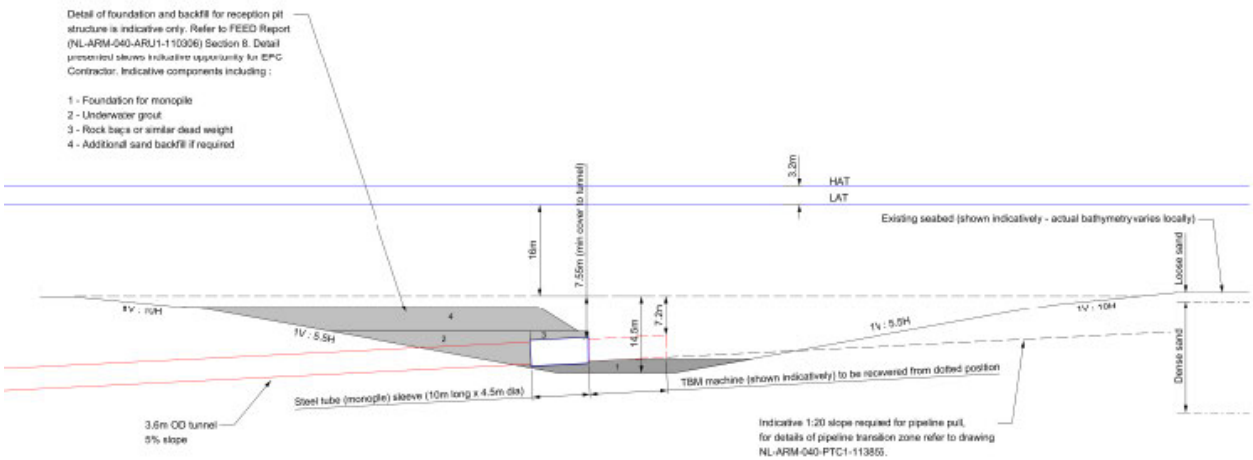


Figure 7-2 - Section view through reception pit – monopile option (from NL-ARM-040-110329)



7.2.2 Internal strengthening (secondary lining)

The internal strengthening option would not likely require additional structures within the reception pit and therefore a shallower dredge depth (and reduced dredge volume) is possible.

Figure 7-3 shows the envisaged pit geometry on plan for the internal strengthening option, which results in a dredge volume of approximately 73,500 m3. Figure 7-4 shows the envisaged pit geometry in section for the internal strengthening option. In this option, the top of the TBM (only) is exposed to allow recovery and it seems feasible for no operatives to be in the tunnel at this time, which reduces the risk from vessel related hazards such as dropped anchors.

Figure 7-3 - Plan view on reception pit – internal strengthening option (from NL-ARM-040-110329)

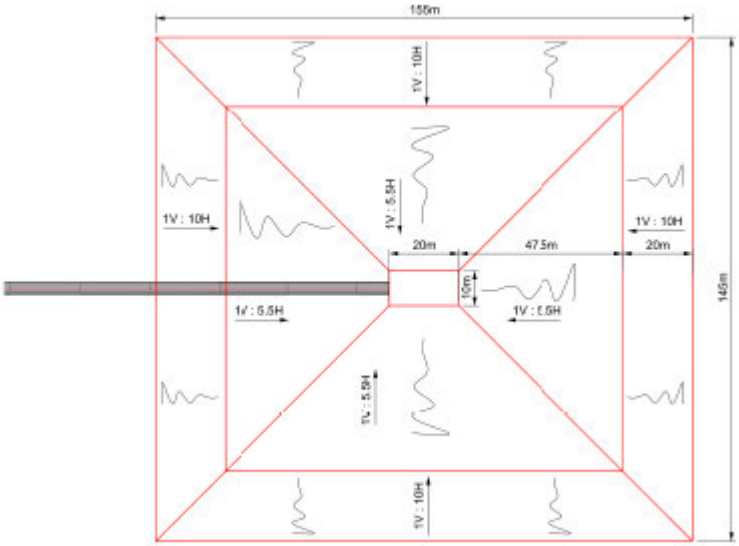
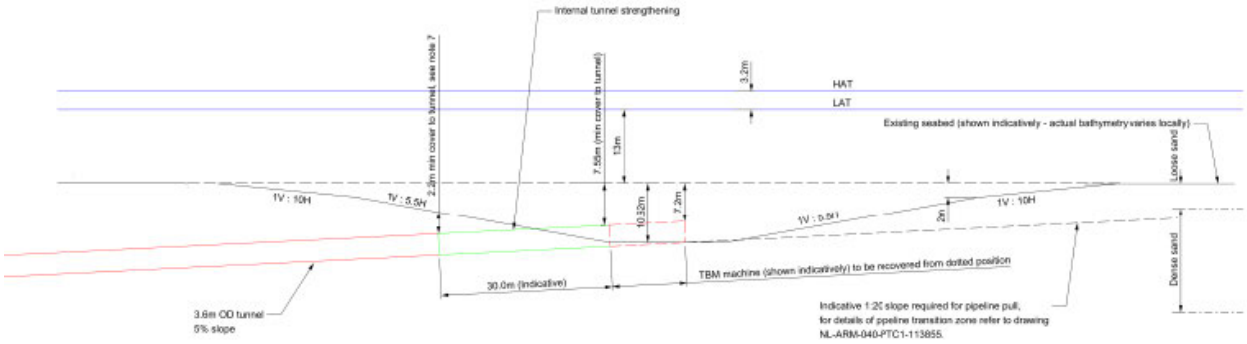


Figure 7-4 - Section view through reception pit – internal strengthening option (from NL-ARM-040-110329)

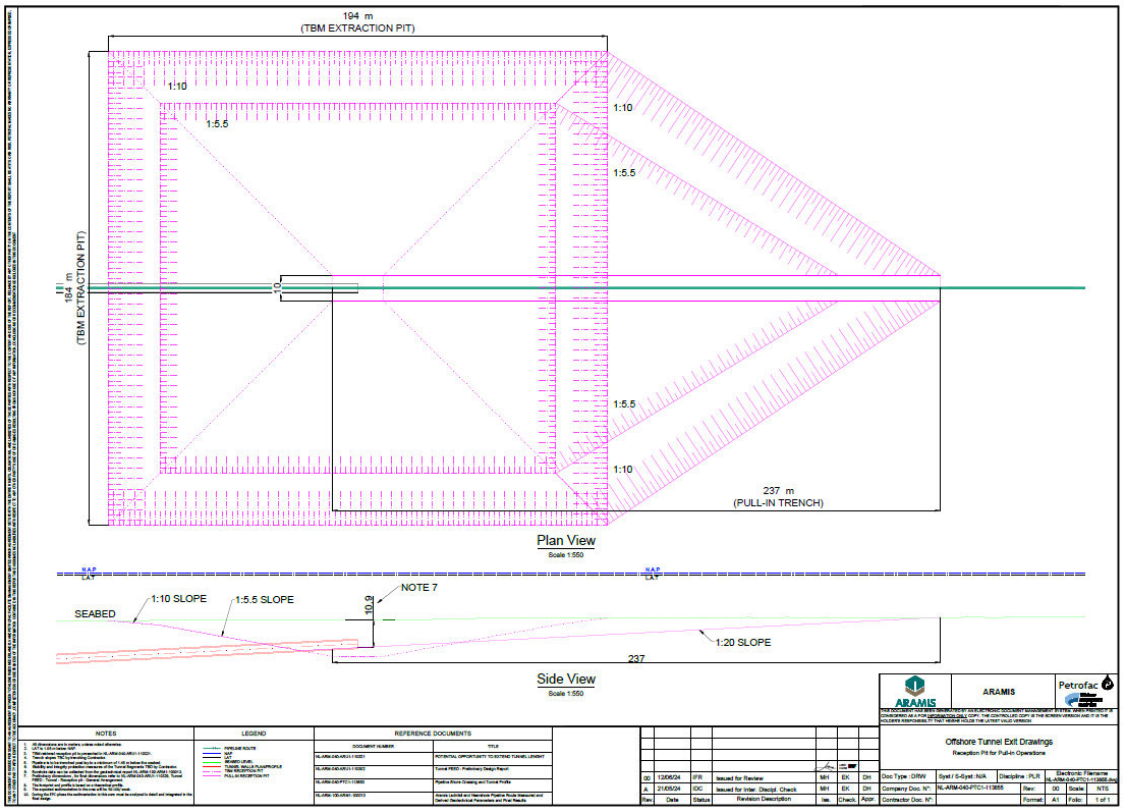


There are additional slope gradient requirements not related to stability, which are dependent on the EPC Contractor methodologies for pipe laying operations. Localised areas of shallower slopes may be required (e.g. see Figure 7-5 from drawing shared informally by the FEED pipeline design) but as these are not required for the TBM recovery, they are not shown on the reception pit geometry required for TBM retrieval.

Dredging extends down approximately 3.07m from the crown of the tunnel, leaving sufficient material to support the TBM and prevent rotation prior to recovery operations. Once the TBM is removed, surrounding material will settle into the void left. Reprofiling of this area will be required locally to achieve the levels required to support pipe pulling operations.

Whilst the general principle is to have a 1 in 5.5 slope within the sand layer and 1 in 10 within the soft silt layer, the bed levels vary over the footprint and change over time (due to sediment transfer into / away from the reception pit area through tidal effects or storms), the stratigraphy levels will also not be horizontal, meaning the extent of the actual dredged slopes may vary in the final dredged profile. There are risks associated with these uncertainties, though also opportunities to better categorize the materials within the dredge area and reduce volumes.

Figure 7-5 – Extract from pipeline designer drawing showing potential requirements associated with pipe laying operations



7.2.3 Dredging interface with other packages

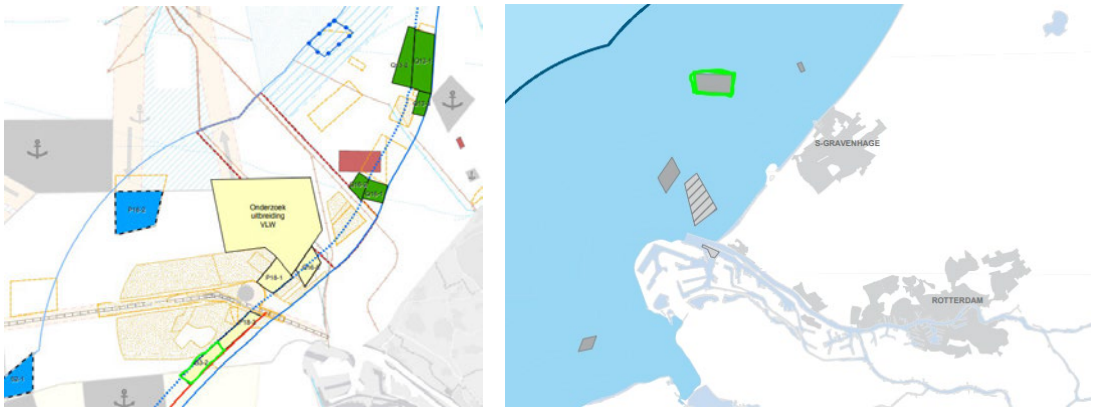
As recommended in the preliminary design report, there is likely economy in considering procurement options for all dredging activities on the project including any UXO clearance, preparatory dredging (removing peaks of sand waves), any sea-bed trench (if required) etc., with a view to reducing the number of mobilizations, maximizing favourable weather windows, whilst also having sufficient redundancy in equipment.

7.3 Excavation methodology – incorporating input from dredging contractors

Representatives from Company (Aramis) and Contractor (including Petrofac, Arup, and Offshore Independents) met with dredging contractors **Van Oord** and **Boskalis** separately to seek advice around the feasibility of the dredging profile, potential dredging equipment, risk mitigation, expected tolerances, spoil and borrow pit locations, and working patterns. The full set of responses can be found in meeting minutes, with key extracts summarised below.

- Both contractors reported a side slope of 1:10 in the upper 2m soft clay would be feasible. Boskalis suggested that a slope of 1:5 could be maintained when excavating the Sand, which is steeper than the 1 in 5.5 proposed by Arup in this report. Van Oord did not comment adversely on Arup's proposed slopes, though indicated more optimal slopes may be justifiable and this should be reviewed at detailed design. Both contractors selected a Trail Hopper Suction Dredger as their preferred choice of dredging equipment and the method of excavation would require changing when operating close to the tunnel and TBM to avoid accidental impact and damage to either the structure, machine, or dredging equipment; Van Oord suggested a DOP submersible dredge pump (a multifunctional dredging tool) for work close to the tunnel and any other maintenance dredging works (to remove sediment transported into the pit whilst it remains open).
- When asked for local experiences regarding rates of siltation into similar excavations, Van Oord suggested the South side of the channel see siltation of up to 1m/year (~2cm/week) though could not advise with confidence for the North side, Boskalis suggested a siltation rate of 5-10cm/week based on experience, and suggested they could do a quantitative assessment at detailed design stage to better estimate this.
- For the trailing suction dredger, Van Oord suggest a vertical dredging accuracy of +/-0.5m however Boskalis suggest +/-1m for the main slopes. Other tolerances associated with alignment of monopile sleeve, concrete slab construction etc. were also provided.
- For borrow & disposal sites, Boskalis reference a current project of theirs (Porthos) for which they will provide follow up information. Van Oord have specified both a spoil and borrow site as shown in Figure 7-6 below.
- Both contractors suggested that the reception pit be backfilled with gravel to take advantage of closer tolerances in level (Boskalis quote +/-30cm).
- Both contractors confirmed that 24/7 working would be possible for the dredge works

Figure 7-6 - Van Oord Extraction Area (S3-2, green outline, right image) and Disposal Area (Loswal NW, green outline, left image)



On production rates, both dredging contractors suggested figures assuming no clay within the reception pit footprint (clean sand only). Boskalis suggested a dredging rate of 10,000m3 per day meaning that the 150,000 to 200,000m3 volume could be achieved between 4 and 5 weeks including 2 weeks of more careful (and slower) dredging close to the TBM. Van Oord suggested the following schedule: “Indicative timing for Dredging 3-6 weeks and backfill 1-2 weeks. For re-dredge after the TBM completed the works and needs to be retrieved, the bulk with a TSHD and the area’s around the TBM with massflow/DOP pump; 3-5 weeks. Backfill after the pipe is installed is probably another 1-2 weeks.”. For comparison, Arup estimated dredging works for the monopile option would take up to 4-6 weeks depending on the size of Trailing Suction Hopper Dredger (TSHD).

7.4 Design calculations

At FEED stage, the following calculations are relevant to the reception pit:

- The reception pit geometry is based on slope stability analysis only, with the calculation methodology (PLAXIS runs using load factors and combinations as per Eurocode 7) as described in the Geotechnical Design Report (NL-ARM-040-ARU1-110305).
- Dredge volumes were estimated for the geometry described within this report (see Section 7.2). There is variation of the bed levels over the footprint of the reception pit, which changes over time due to the dynamic offshore environment. Reported dredge volumes are without bulking factors and based on a high bed level (which may overestimate volumes slightly).
- Estimation of potential loadings from vessel related hazards (anchor drop, anchor drag, vessel sinking) and the overburden thicknesses required to resist these, as considered as described in section 5.

7.5 TBM retrieval / reception pit requirements if completing in pipe jack mode

Note that if successfully completing the tunnel in pipe jacking mode, then limited strengthening works are required prior to TBM retrieval, for example limited to ensuring a connection between adjacent ultra-heavy pipes used at the tunnel exit location (for example by bolting adjacent pipes together using pre-installed flanges). Reception pit design would be similar to that required for the internal strengthening option (see Section 8).

8 TBM RETRIEVAL METHOD

The aim of the TBM retrieval activity is to retrieve the TBM main body from the seabed once it has reached its reception point. The TBM retrieval method and its execution duration directly impact the programme and thus the pipe pulling operations which are on the critical path of the overall Aramis project. The TBM retrieval method and sequence have both direct and indirect interface with the next activities such as pipe pulling and the tunnel grouting works. Thus, it is important to develop the most beneficial TBM retrieval method for the project. This section explains the details of the proposed preferred TBM retrieval method and its details, together with the alternative method studied during the FEED design.

Note this section was initially developed in consideration of completing the tunnel in segmental lining mode; for implications if completing the tunnel in pipe jack mode, refer to section 8.6.

Note in this section that any references to cost, schedule, etc are purely indicative as used for option selection and comparison.

8.1 Minimum required overburden

8.1.1 Introduction

The depth of TBM below the seabed level (overburden above the tunnel) directly influences the required dredging volume at the reception. To determine the minimum overburden required for the TBM retrieval, two checks have been carried out to meet the below two conditions:

- Minimum overburden required for safe control of the TBM face pressure.
- Minimum overburden required to prevent flotation of tunnel.

TBM face pressure related check is completed using industry standards, such as the DAUB guide, and the flotation check is carried out using Eurocode7.

8.1.2 Face pressure check

The methodology that has been followed to assess the minimum overburden at the TBM retrieval location can be found in the DAUB guide. The guideline details several sets of calculations that could be used in order to determine whether the face of the TBM has adequate pressure for safe construction and prevent failures such as blowouts from occurring.

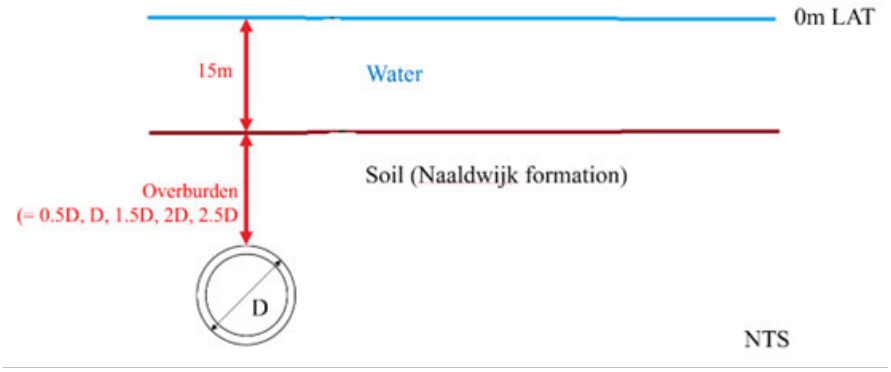
Input parameters include details of the tunnel alignment as well as the surrounding geology.

These inputs include details of the excavated tunnel diameter (D) = 3.775m, water elevation above ground = 15m, and tunnel overburden. The tunnel overburden is what is being assessed in this calculated, therefore a range of values will be tested from 0.5D to 2.5D increasing in 0.5D intervals.

Ground parameters are also required. The reception pit is to be constructed in strata consisting of Kreftenheye Formation, therefore the geotechnical inputs specific to this formation have been used in this calculation, such as saturated unit weight, effective friction angle etc. It has been assumed that the ground has no cohesion.

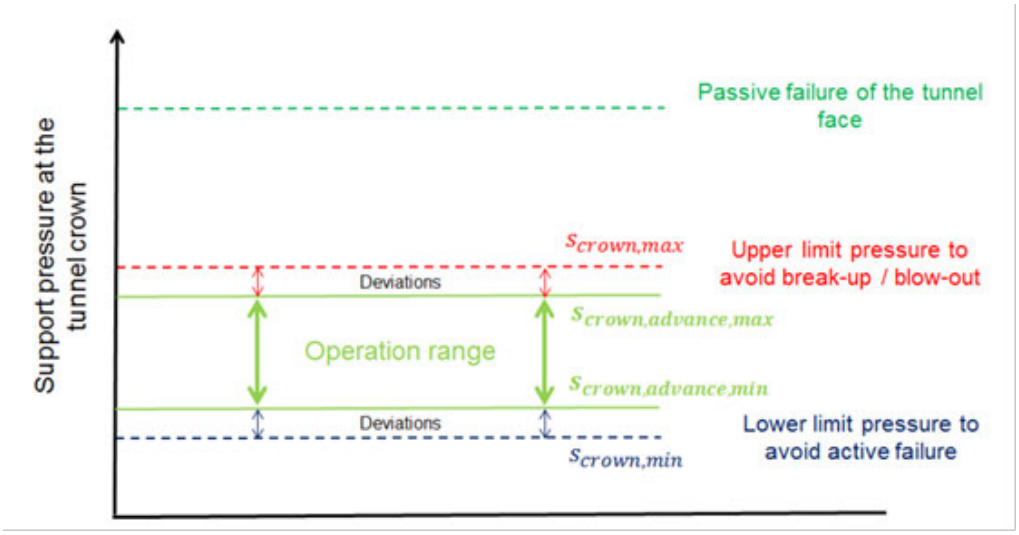
See Figure 8-1, below, for a schematic of the TBM cross section and the parameters being tested.

Figure 8-1– Schematic of TBM cross section and surrounding geological inputs



To determine whether the overburden secures an acceptable operation range of the TBM pressure, the upper and lower limit of support pressure at the tunnel crown corresponds to the selected overburden must be assessed. If the upper limit pressure at the crown is greater than the lower limit pressure, it means that the TBM should be able to find a face pressure that falls within the operation range. If the upper limit pressure at the crown becomes smaller than the lower limit pressure, it means that the TBM has a risk of blow out or loss of face stability at any applied face pressure. See Figure 8-2, below, for a graph of the allowable operation pressures at the tunnel crown of a shield wall.

Figure 8-2 – Allowable operational pressures at the tunnel crown of a shield machine (extract from Figure 9 of DAUB guide)



The DAUB guideline specifies that there are two safety factors that are applied to the destabilising forces in the calculation. These safety factors are Earth Pressure Safety Factor, η_E and Water Pressure Safety Factor, η_W, where the values of these are provided as 1.5 and 1.05, respectively.

After a risk review of the surrounding site area, it has been considered acceptable to reduce both safety factors to 1.0 near the TBM retrieval location. This is due to the surrounding area near the retrieval lacking in 3rd party structures or inhabitants. Thus, the impact of a potential blowout or subsidence is low and therefore risk safety factors can be reduced due to the reduction in overall risk. It should be noted that the global safety factor of 0.9 proposed in DAUB guideline is kept unchanged.

The results of this study can be found in Table 8-1, below. As can be seen from Table 8-1, when the overburden depth is greater than 2D, or 7.55m, TBM is able to be maintain stable face stability condition.

Table 8-1 – Face pressure summary results table

Overburden dimension [expressed in terms of Excavated Tunnel Diameter (D)]	Overburden dimension [m]	Earth Pressure Safety Factor, η_E	Water Pressure Safety Factor, η_W	Maximum Support Pressure at Tunnel Crown for Regular Advance, scr,adv,max. [kN/m^2]	Minimum Support Pressure at Tunnel Crown for Regular Advance, scr,adv,min. [kN/m^2]	Excavation possible? (Pass/Fail)
0.5D	1.89	1.0	1.0	154	178	Fail

Overburden dimension [expressed in terms of Excavated Tunnel Diameter (D)]	Overburden dimension [m]	Earth Pressure Safety Factor, η_E	Water Pressure Safety Factor, η_W	Maximum Support Pressure at Tunnel Crown for Regular Advance, scr,adv,max. [kN/m^2]	Minimum Support Pressure at Tunnel Crown for Regular Advance, scr,adv,min. [kN/m^2]	Excavation possible? (Pass/Fail)
1.0D	3.78	1.0	1.0	185	200	Fail
1.5D	5.66	1.0	1.0	217	221	Fail
2.0D	7.55	1.0	1.0	248	243	Pass
2.5D	9.44	1.0	1.0	280	251	Pass

8.1.3 Flotation check

A hand calculation has been conducted to determine the minimum overburden required to overcome the buoyant uplifting forces acting on the tunnel lining and prevent flotation occurring.

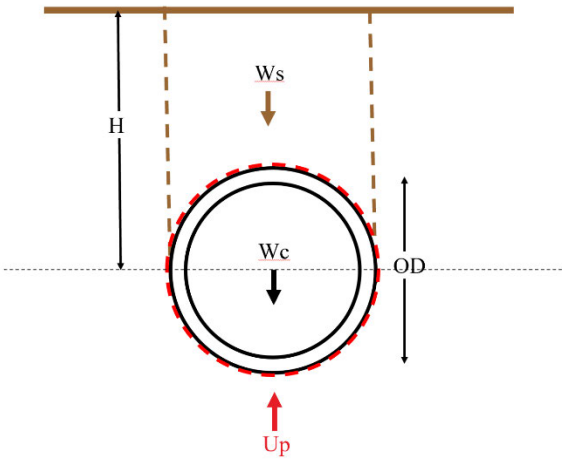
To begin the calculation, geometrical parameters of the tunnel lining were set, these being a 3.0m ID and a lining thickness of 250mm, and thus an External Diameter (OD) = 3.5m. The fluid volume that is expelled due to the size of the tunnel lining is what determines the total uplifting (buoyant) force, U_p.

The downward stabilising force, D_s, is a combination of the self-weight of the lining, W_c, and the ground loading acting on the tunnel lining, W_s. Friction resistance along the vertical plane of the ground block W_s is ignored. These favourable actions are applied with a safety factor of 0.9.

The ground loading is a function of the height from the ground surface to the tunnel axis, H. Therefore, as this H increases so does the ground loading. This flotation test has been set up to determine what H is required so that the uplifting actions are less than the stabilising downward actions.

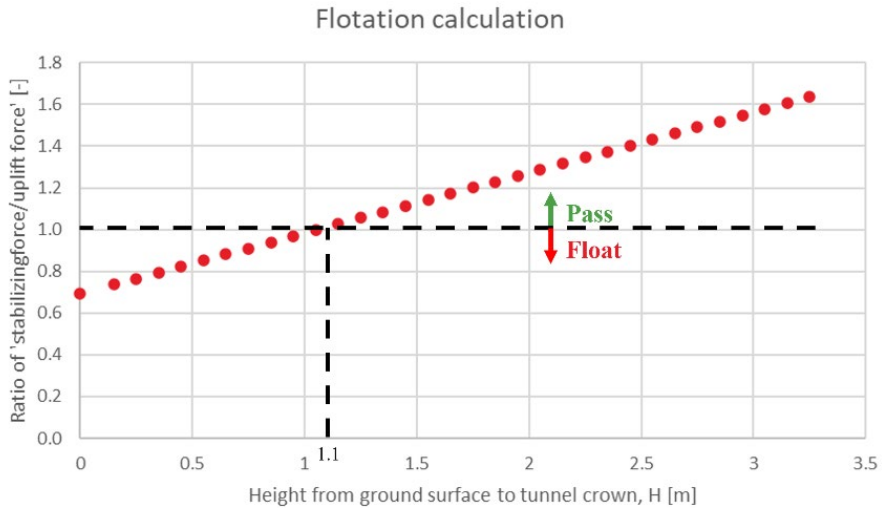
A simple schematic of the flotation check parameters and forces can be seen in Figure 8-3, below.

Figure 8-3 – Geometric parameters and forces acting on the tunnel lining that are considered in a flotation check



The results of this flotation check have been formatted for ease of analysis into the graph that can be seen in Figure 8-4. On this graph, the X axis shows the height of the ground surface to the tunnel crown. The Y axis, 'Ratio of stabilizing force/Uplift force' has been considered after the application of safety factors (0.9 to favourable permanent actions). The value of 1.0 on this Y axis determines the level where, values above pass the flotation test and for values below the test fails/floats.

Figure 8-4 – Flotation check calculation graph.



In summary, it has been determined that for the design of the tunnel to be constructed in a safe and stable manner, as well as adhere to both the face pressure check and flotation check, a minimum overburden of 7.55m from the tunnel crown to the ground surface is required at the TBM retrieval location.

8.2 Retrieval options studied

At the beginning of the FEED design, a concept was studied that strengthens the precast concrete segment lining using heavy-duty longitudinal dowel connectors together with the installation of temporary steel structural props inside. Flotation is assumed to be controlled by placing additional weight such as sandbags inside of the tunnel. However, after the introduction of the secondary lining and the monopile options, the dowel + steel frame option became less preferred as the other two options are considered to provide greater robustness in structural stability. Thus, the secondary lining option and the monopile option details are presented in this report.

While developing the TBM retrieval methods details, the below two technical risks are identified as key risks:

- Flotation of the tunnel during the dredging above/around the tunnel.
- Structural damage/instability of segment lining by the loss of support medium around the tunnel because of dredging and accidental impact load from pipe pulling activity.

Considering the above key risks, together with the level of robustness desired by the project, two TBM retrieval methods are selected for further study, namely the internal lining option and the monopile option.

The secondary lining option considers the construction of cast in place lining inside of the already constructed segment lining so as to mitigate the risk of tunnel flotation and the damage from accidental impact load during the pipe pulling. This option is relatively straightforward when compared to the monopile option in terms of the construction activities involved. However, the dredging of the reception pit in this instance could only be started after finishing the TBM drive.

The monopile option considers the installation of a grout-filled large diameter monopile prior to the TBM's arrival to the retrieval position, so as to mitigate flotation and tunnel damage risks near the end section of the tunnel during the pipe pulling. This option involves various marine activities, but most of the activities could be started and completed concurrently with the TBM tunnel drive. Thus, the two studied options each have distinctive benefits and disbenefits to the project.

The following sections detail the study results of each option and summarise rational behind how the secondary lining option is selected as the preferred option. Reference has been made, where relevant, to workshop meeting minutes 'Tunnel FEED – TBM Retrieval follow up session' (document number : ARM-ARC01-PTC1-ARM1-MOM-000083, 22/05/2024), that summarises the full detail of the TBM retrieval option selection history.

8.3 Secondary lining option (Preferred option for segmental lining)

8.3.1 Introduction

This section of the report will review the preferred TBM retrieval study option which proposes the use of a secondary lining option, as opposed to the alternative monopile option to be discussed in the next section. Details of the secondary lining option are to be further assessed in this section, where the cast in place tunnel lining is to be installed from inside the already constructed segmental lining tunnel. Further details of the methodology behind the decision making of this decision are detailed in Section 8.5.

The construction material of the internal reinforcement lining was first considered. A steel lining was initially assessed as to whether the precast concrete lining could be replaced. As the steel lining could be designed to have a strong structural connection between the segments and rings, if the steel segment is found to be stable against the risk of flotation, the sole use of steel segment for the last 20m-ish long tunnel section would mitigate both the flotation and damage risks. This means that the secondary lining construction inside of the segment lining is not anymore necessary. However, flotation calculations determined that the steel lining thickness required to prevent flotation would be 117mm, this is very thick and not practical. Therefore, the steel segment option not further considered, and the construction of cast in place secondary lining inside of the precast concrete segmental lining is further studied.

8.3.2 Minimum performance requirements

The cast in place secondary concrete lining has been initially designed to adhere to three minimum performance requirements. These checks will ensure that the design is practical and feasible. The minimum performance requirement checks that will be detailed in the following sections include a flotation check, a structural capacity check, and a space-proofing study.

8.3.2.1 Flotation check

The first performance requirement being assessed of the cast in place concrete internal secondary lining is that of a flotation check. By balancing the upward buoyant force with the favourable downward self-weight of both the inner and outer lining, it was calculated that to prevent buoyancy, the required thickness of the secondary, inner cast in place lining is 207mm. This thickness was rounded up to the nearest quarter to allow for redundancies and standardised construction tolerances, therefore the thickness of the internal lining being considered is 225mm.

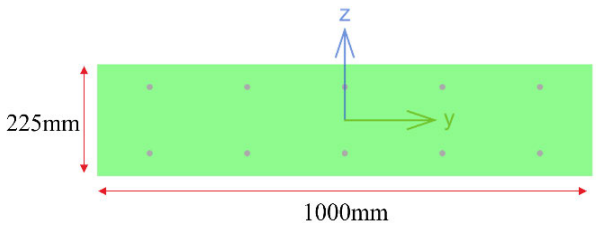
8.3.2.2 Structural capacity check

The second performance requirement is to determine the required thickness of the inner lining so that its structural capacity is that where it has adequate stiffness and strength properties so that the structure can withstand loading without failure and also not to deform excessively. The structure should also have the ability to withstand accidental loading. The value of this accidental load, as well as the minimum point load that the structure should be able to withstand, has yet to be determined.

The methodology that has been carried out to determine the value of the required secondary lining thickness considers designing the secondary lining to resist a certain level of deformation which corresponds to the segment lining's ULS design limit. Applying industry best practices, a tunnel's deformation limit is often defined as where its ovalisation is limited to 1%. Therefore, for the 3.5m OD segment lining in the design, the limiting deformation diameter change is 35mm. Thus, the secondary lining's acceptable thickness and corresponding reinforcement has been determined so that the structure does not reach ULS failure up to the ring deformation of 35mm.

From previous buoyancy checks, a secondary lining thickness of 225mm is the minimum value of secondary lining thickness to prevent buoyancy, where a standardised 25mm increment lining thickness sizes has been applied. For a lining thickness of 225mm, if a 35mm deformation occurs, then the bending moment acting on this lining is 37.5kNm. The reinforcement details for a 225mm thick lining to withstand 37.5kNm and therefore not deform greater than 35mm, at an 80% load utilisation, are as follows: cover depth of 40mm, rebar spacing of 200mm, and rebar size of 12mm. These details fall within the practical reinforcement detail range. See Figure 8-5 for a schematic depicting this reinforcement detail. These reinforcement details are not fixed and can be adjusted at the next stage of the design as the design detail evolves.

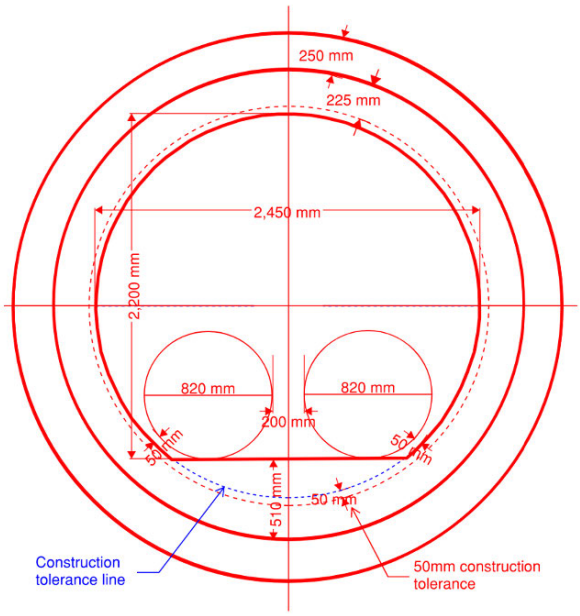
Figure 8-5 – Schematic of the reinforcement details required



8.3.2.3 Space-proofing study

A space-proofing study has been conducted to ensure that there is sufficient space for the CO_2 pipe(s) after the construction of the secondary lining. Figure 8-6, below, demonstrates that there will be sufficient space for the placement of two 820mm pipes after the construction of the secondary lining (note: at the time of this FEED report submission, only one CO_2 pipe installation is considered necessary). The invert level of the secondary lining has been adjusted to have a headroom of 2,200mm to ensure worker's safe access for the secondary lining construction.

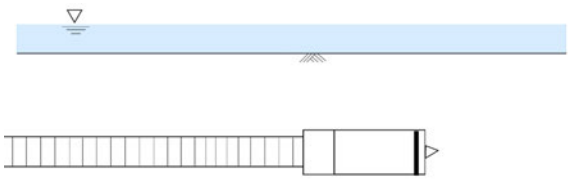
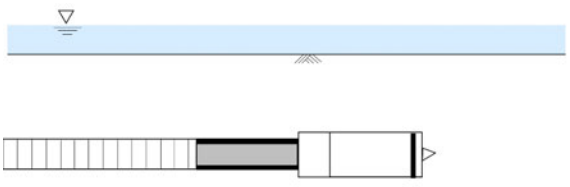
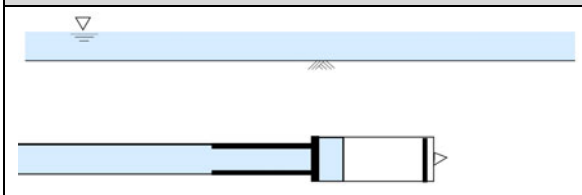
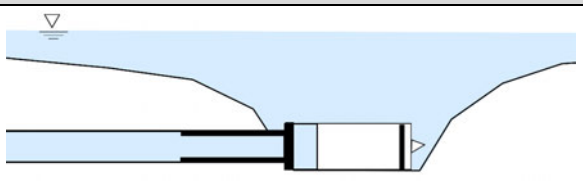
Figure 8-6 – Initial space-proofing design concept for tunnel

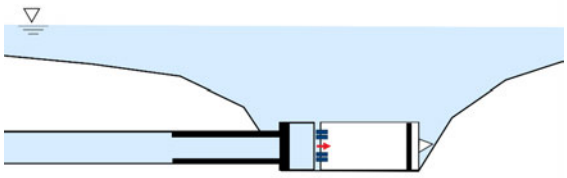
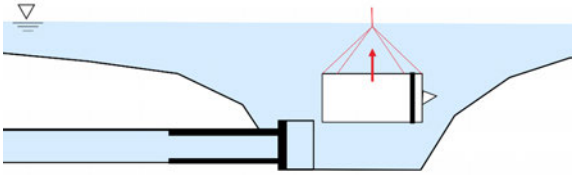
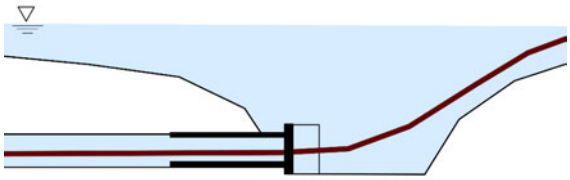
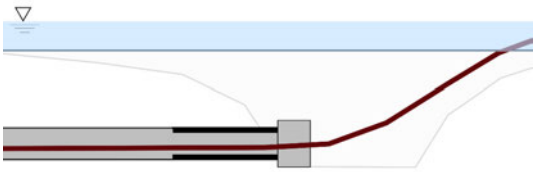


8.3.3 TBM retrieval sequence

A step-by-step annotated sequence detailing the envisaged construction sequence of the cast in place secondary lining construction and its respective TBM retrieval sequence can be found in Table 8-2.

Table 8-2 – Cast in place secondary lining and TBM retrieval construction sequence

Step 1	Step 2
	
TBM reaches its termination point. Seal joints between segments and TBM. Internal TBM parts removed.	Secondary lining (cast-in-place) construction in approx. 30m long section of the tunnel.
Step 3	**Step 4**
	
Install tearable or removable sand screen.	Seabed dredged to expose TBM.

Note: Tunnel could be flooded at this step if water pumped into the shaft.	Note: Tunnel could be flooded at this step if the flooding is carried out by open valve on TBM skin.
Step 5	**Step 6**
	
Jack push detaching main TBM body from the sacrificial TBM skin left in place.	TBM barge lifted and removed from reception pit.
Step 7	**Step 8**
	
CO_2 pipe laid within tunnel.	Dredge pit backfilled with sand and tunnel infilled with grout (or sand).

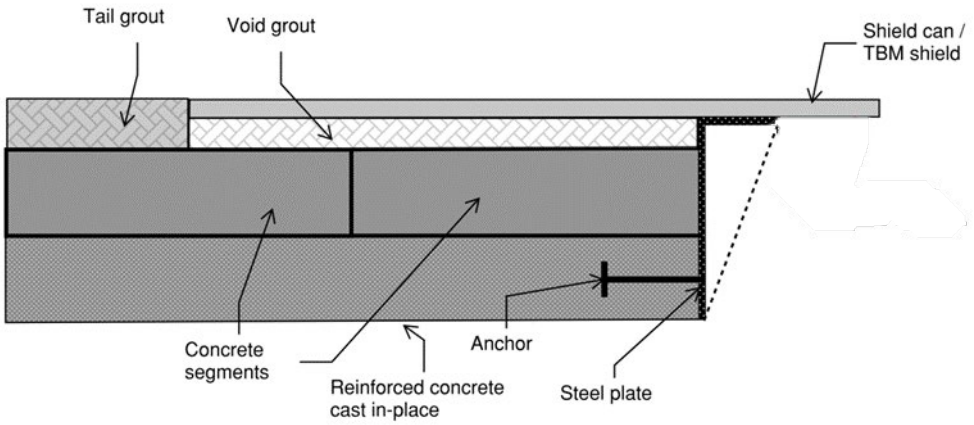
8.3.3.1 Bellmouth feature

The bellmouth feature, located on the inside face of the tunnel lining, is discussed within this report as it is deemed a necessary feature for protecting the tunnel lining from the potential impact load of the pipe which could be induced during installation, particularly if terminating the tunnel in segmental lining mode. However, full details of this bellmouth feature are yet to be determined, and the bellmouth feature should be developed during detailed design. Note that any secondary lining termination details will first need to be finalised to suit the arrangements of the actual TBM selected during EPC phase, if terminating in segmental lining mode.

In the case where the TBM tail skin is left in place after the TBM's retrieval, a conceptual sketch of the termination construction details of the bellmouth is shown in Figure 8-7. If an alternative TBM solution is proposed where the entire body could be retrieved (i.e. no TBM skin left over), then the end of the secondary lining could be cast to suit the EPC contractor's preference e.g. tapered edge.

The design of the bellmouth feature, shown in Figure 8-7, should consider minimising the protruding section of the TBM wall where the pipe nose is at risk of collision from a corner angle. This design detail should be considered and minimised by the tunnel designer at the next design stage. It is critical that the bellmouth design is sufficiently watertight, both during construction, and during lifting operations, to suit the EPC Contractor's proposed tunnel construction and TBM retrieval methodology.

Figure 8-7 – Conceptual schematic of bellmouth construction details



8.3.4 Reception pit details

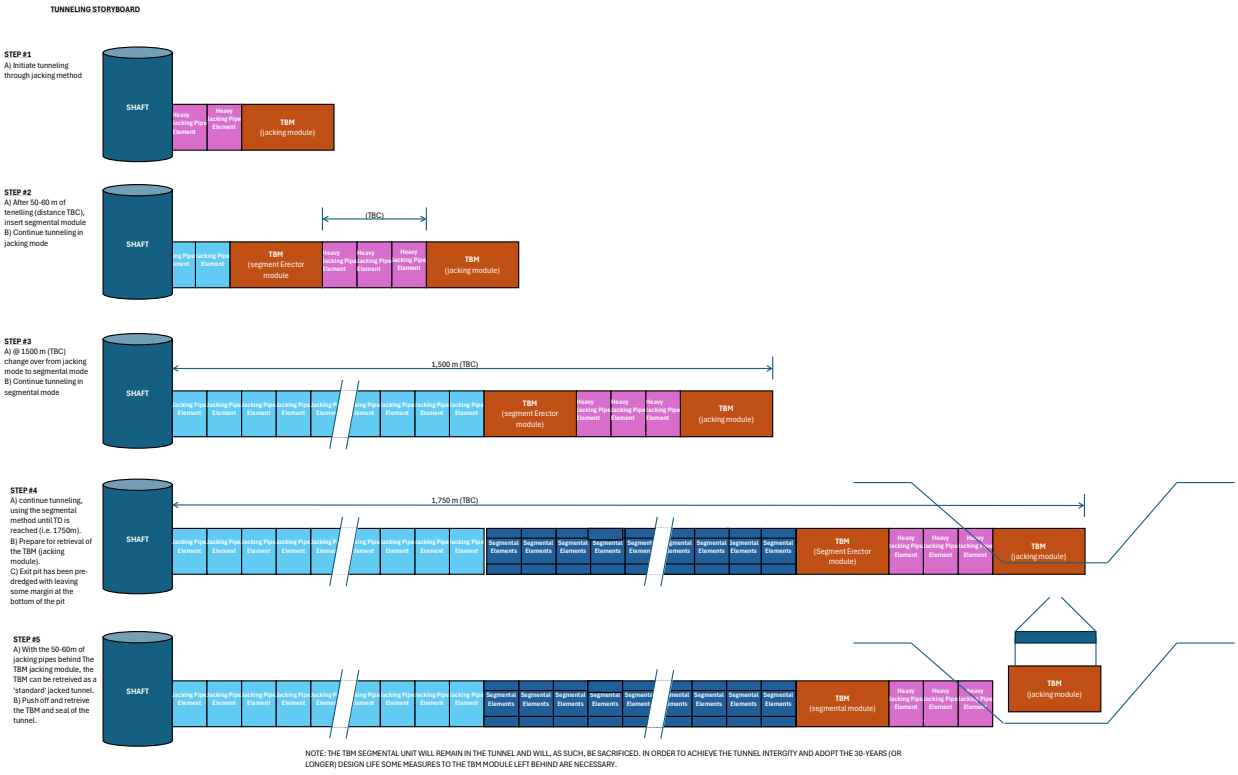
See Section 7 of this report for the details. Note that a dredging contractor is assumed to reprofile the pit for pipeline installation as per OI drawing “NL-ARM-040-PTC1-113855 - Offshore Tunnel Exit Drawings”, after the TBM retrieval has taken place.

8.3.5 Potential variation using pipe rings instead of internal strengthening

An out-of-the-box idea has been explored during the FEED design. This idea considers the use of a bespoke heavy-weight precast concrete pipe for the pre-defined length of the tunnel right behind the TBM, followed by the segment erector module (only) of the TBM machine. The potential key benefit of this option is saving of the overall tunnel construction/retrieval programme due to (1) TBM conversion from the jacking mode to the segment mode is not required, and (2) no need of secondary lining installation as the heavy weight concrete pipe could be prefabricated to have sufficient structural robustness and weight against the flotation. This concept is illustrated in Figure 8-8.

At the time of this FEED report submission, no precedent project case is known to the authors. This new concept requires in-depth study and discussion with the TBM manufacturers to assess its feasibility, benefits, and risks to conclude this concept’s applicability to Aramis project. In the next stage of the project, it is suggested that the EPC contractor/designer perform an in-depth study about this option and submit their findings and suggestions.

Figure 8-8 - Potential alteration to the secondary lining option



8.4 Monopile option (Alternative option for segmental lining)

8.4.1 Introduction

The steel tube monopile option has been proposed as an alternative construction option to the secondary lining option. This option proposes the construction of a large concrete tubular structure (the monopile) which the TBM will grind through at its end of drive. This would eliminate the issues of flotation and reinforce the end of the lining within the reception pit.

Monopile option allows the concurrent construction of the reception pit and minimised the time duration from the TBM drive completion to the TBM body retrieval, but this option involves higher complexity marine activities.

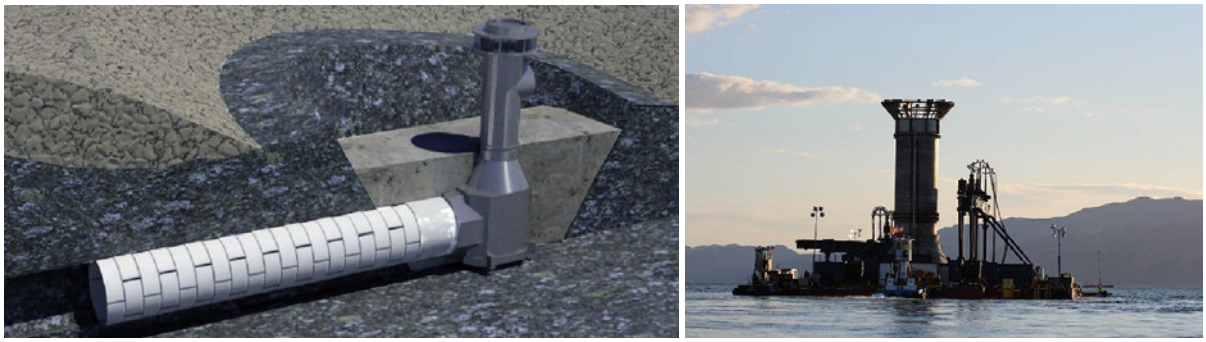
This option involves pre-installation of the heavy reception structure below >30m sea. Early desk study showed that the positioning tolerance control is important to optimise the size/weight of the grout filled monopile which direct influence the size of floating sheerleg required for the construction.

The Lake Mead project is known to have successfully constructed the large intake structure at depths below approximately 100m water. Depictions of the Lake Mead intake structure can be seen in Figure 8-9. The construction sequence of the intake structure was desk studied and the following concepts are adopted for Aramis project:

- Installation of steel frame as a guide before the main structure to control the positioning tolerance of the main structure. The steel frame is equipped with three hydraulic jacks that can self-level the steel frame after it is positioned under the water.
- Tremie concrete the steel frame to constrain the installed position of the steel frame.

Further details about the Lake Mead project's intake structure installation sequence is presented in Appendix D.

Figure 8-9 - Lake Mead intake structure



8.4.2 Minimum performance requirements

The steel monopile option has been preliminarily assessed to ensure that it adheres to two minimum performance criteria: (1) the ability to resist the torque force applied from the TBM, and (2) the grout-infilled monopile's weight is within an acceptable handling range.

Each of the design performance requirements have opposite preference for the weight of the monopile structure. Where, for check 1, a heavier structure has a greater ability to resist torque and, for check 2, a lighter structure is easier to handle and may allow for smaller lifting machinery (cranes).

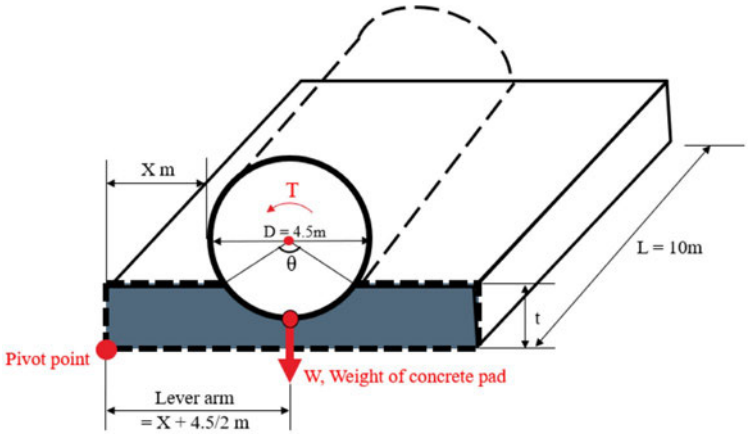
Therefore, due to the counteracting preferences, there is a balance which should be found between the design requirements. The following sections will detail the design considerations that have been conducted to ensure the minimum performance requirements are adhered to.

8.4.2.1 Resistance to TBM torque

When the TBM grinds through the grout-infilled monopile, the TBM's torque will try to overturn the monopile structure, hence there must be a sufficient weight in the structure that could generate the counter torque. The worst timing is when the TBM has grinded most of the grout infill as the monopile's weight will be minimum. For a simpler and conservative check, the monopile and the grout infill material's weight is ignored and the concrete pad that supports the monopile is assumed to be the only solid mass that shall generate required counter torque against TBM's maximum torque. This required concrete slab thickness (t) has been determined for a range of monopile positions.

A steel tube monopile diameter of 4.5m has been assumed for the design. This value has been determined by considering the following values and combining where appropriate: tunnel internal diameter of 3.0m, TBM outside diameter of 3.625m, TBM driving tolerance of +/- 100mm in radius, structure positioning tolerance of monopile structure under the dredged sea bed of +/- 200mm, monopile wall thickness of 45mm (thickness to diameter ratio of 1:100), and risk of tube deformation by self-weight of 50mm in radius. This makes the monopile OD = 3.625 + (0.1+0.2+0.045+0.05) x 2 = 4.415m, say 4.5m. A schematic of the monopile layout can be seen in Figure 8-10.

Figure 8-10 - Schematic of monopile layout



A maximum TBM torque of 1600kN/m has been considered, as per the information available from the product brochure from Herrenknecht AVN model. An additional safety factor of 1.2 has been applied to the adverse torque force to overcome. Therefore, the concrete slab has been designed to resist a factored torque of 1920kN/m. Figure 8-10 illustrates a schematic of the monopile and concrete slab configuration used to ensure structural resistance to the torque applied by the TBM.

The following summary table, Table 8-3, has been calculated determining the structural solution for the required concrete slab thickness to prevent the torque applied by the TBM. From observing the table, the working base case assumption going forward will be a concrete slab thickness of 0.75m. This is considering a minimum thickness of 0.49m for X = 1.5m case and securing minimum 0.25m space at the thinnest slab section directly under the monopile.

Table 8-3 – Required concrete slab thickness to prevent torque

Distance from edge of concrete pad to monopile, X [m]	Required concrete slab thickness, t [m]
1	0.65
1.5	0.49
2	0.38

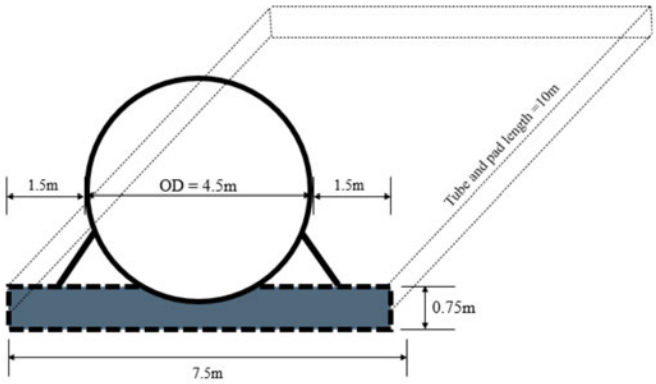
Distance from edge of concrete pad to monopile, X [m]	Required concrete slab thickness, t [m]
2.5	0.30

8.4.2.2 Lifting weight

The second design performance requirement check is to ensure that the steel monopile structure is light enough to be lifted and handled using lifting machinery.

With the use of a lightweight grout of 12kN/m3 and considering a concrete slab thickness of 0.75m and edge to monopile distance of 1.5m, the total weight of the monopile reception structure was determined to be 3567kN. This weight would require a 600t floating sheerleg to lift the structure (apply 60% utilization). See Figure 8-11 for a schematic of the reception structure geometry, consisting of the steel tube monopile and concrete slab.

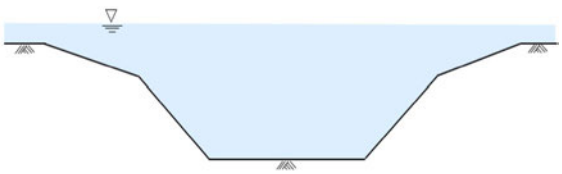
Figure 8-11- Reception structure geometry.

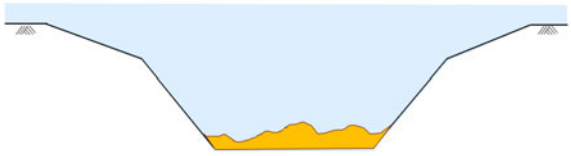
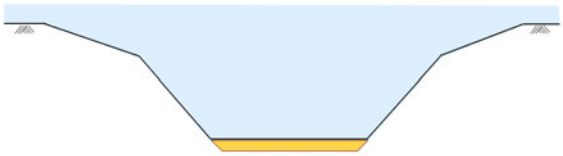
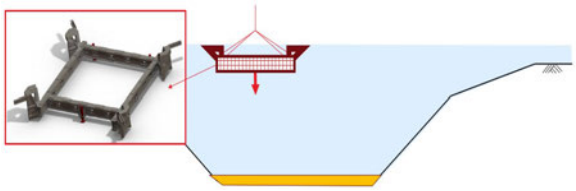
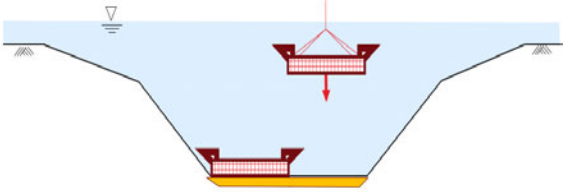
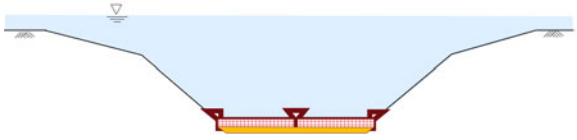
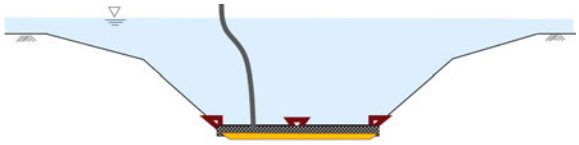
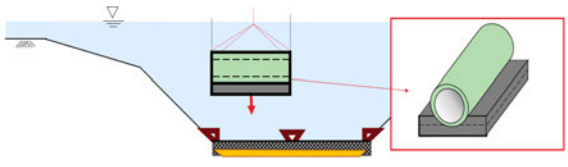
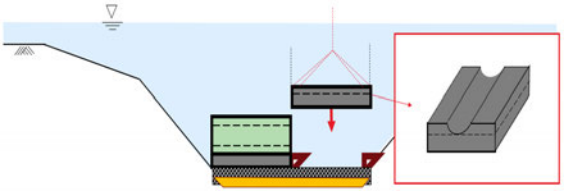


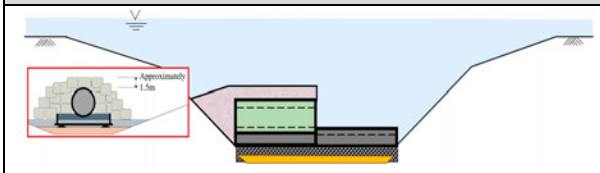
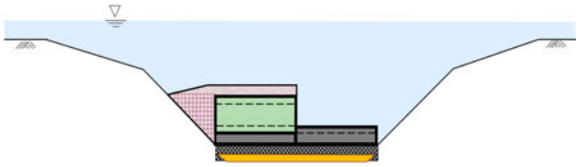
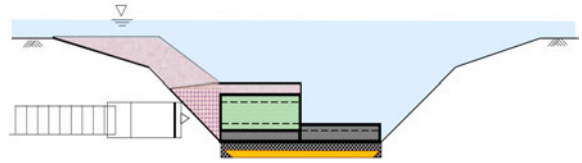
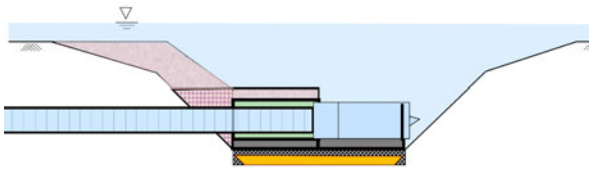
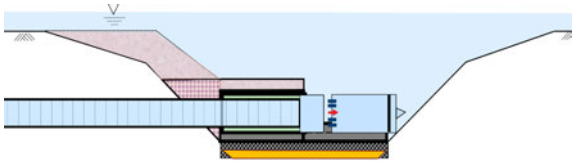
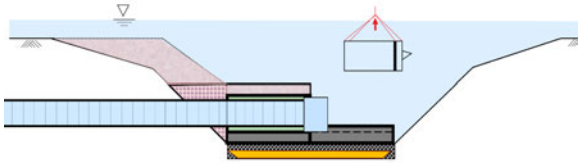
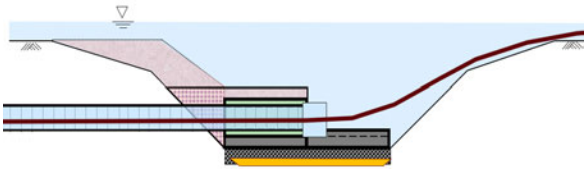
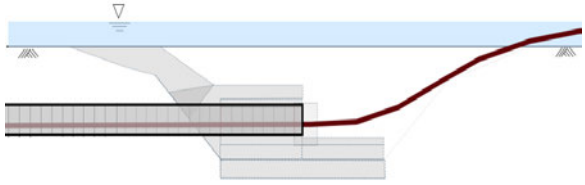
8.4.3 TBM retrieval sequence

The TBM retrieval sequence for the monopile option is illustrated in Table 8-4, where each stage of the construction works has been annotated and detailed for further understanding.

Table 8-4 – Steel tube monopile reception pit construction sequence

Step 1	Step 2
	
Existing undisturbed seabed.	Dredged seabed.

Step 3	Step 4
	
Gravel placed at bottom of excavation (telescopic pipe from the barge).	Gravel bed layer flattened and prepared as a foundation for the steel framing structures (grader or jack up platform).
Step 5	**Step 6**
	
Levelled guiding frame with hydraulic jacks, for the 'monopile + concrete pad' section, lowered to bottom of excavation.	Second guiding frame lowered into the 'concrete pad only' position at the bottom of excavation. (Note: Step 5 and Step 6 could be merged if two steel frames could be combined into one structure).
Step 7	**Step 8**
	
Self-levelling frame with hydraulic jacks equipped in the steel frame.	Steel frame filled with tremie concrete to permanently secure foundation. Columns remain extruding ready for concrete pads.
Step 9	**Step 10**
	
Monopile + concrete pad lowered onto the steel frame.	Second concrete pad only structure lowered onto steel frame.

Step 11	Step 12
	
Rock boulders/stone bags placed above/around monopile + concrete pad TBM reception structure. Gravel bags placement area excludes future TBM entrance corridor.	Underwater grouting installed at TBM entrance corridor. Backfill with sand above the grout if required. Reception pit ready for TBM.
Step 13	**Step 14**
	
As drive enters the underwater grout block, TBM stops, and the supporting fluid changes from slurry to water.	TBM completes drive and stops.
Step 15	**Step 16**
	
Seal joints between segment and the shield. Isolated section of tunnel. Flood tunnel.	Jack push detaching TBM from the sacrificial TBM skin left in place.
Step 17	**Step 18**
	
TBM barge lifted and removed from reception pit.	CO_2 pipe laid within tunnel.
Step 19	
	
Dredge pit backfilled with sand. Tunnel infilled with grout/sand.	

8.4.4 Potential further optimisation

Further study was conducted to consider the possibility of optimizing the sequence of the monopile option, that could be considered at EPC stage. Two elements were proposed for removal from the proposed baseline sequence, these being the gravel bed, and the steel frame.

Idea 1 proposes the idea of removing the gravel bed. It is assumed that a grader could flatten the dredging surface to a tolerance of +/-750mm. Then steel frame is place then tremie concreted. However, removing the gravel bed imposes a risk of larger vertical settlement when the monopile structure is placed due to the loose/disturbed surface of dredged ground. A potential solution for this is to use a larger monopile tube to accommodate for the settlement induced. This idea is feasible but will require a larger monopile and therefore a heavier crane.

An extension of idea 1 proposes still removing the gravel bed, however here the steel frame sits over the top of the pre-installed large pre-cast concrete tile. Then the steel frame is tremie concreted. This further extension is also deemed feasible, with reduced risk in facing larger vertical settlement when the monopile structure is placed.

Idea 2 proposes the removal of the steel frame. Removing the streel frame would eliminate the need for tremie concrete. There are two sub-ideas: 1) If the monopile is directly laid over the gravel bed, vertical settlement risk is less of an issue, but the positioning tolerance and the levelling tolerance will be much greater than the monopile baseline sequence. 2) If the monopile is laid over the dredged soil i.e. remove gravel bed, then the vertical settlement risk will be added to the previous no.1 case. Both implications impact the design by requiring a bigger monopile to compensate for the impacted tolerances.

Above mentioned optimization ideas are technically considered all feasible, but those ideas give a direct impact to the size of the monopile hence require bigger floating sheerleg. The availability of heavy capacity floating sheerleg sometimes constrained, thus the increase of the monopile size could add risk in construction programme control. The adoption of the monopile option and its sequence optimization shall be determined by the ECI contractor to suit their preferred means of method for the construction.

8.4.5 Reception pit details

See Section 7 of this report for the details. Note that a dredging contractor will reprofile the pit for pipeline installation as per OI drawing “NL-ARM-040-PTC1-113855 - Offshore Tunnel Exit Drawings”.

8.5 Options comparison

8.5.1 Assessment criteria

A quantitative assessment has been conducted to select the most beneficial TBM retrieval option for the project. Three TBM retrieval options were assessed. Two of these options are the secondary lining option and the monopile option which are described in previous sections. The other option is a scenario where the precast concrete segment lining is strengthened using heavy-duty longitudinal dowels together with the temporary steel structural props inside. This third option was initially considered at the early stage of the FEED design, but soon after the introduction of the secondary lining and monopile options, this option became less preferred than the other two options. Although the dowel + steel frame option became naturally less preferred option after the introduction of the other two options, all three options are assessed using the comparison matrix for the completeness. Further details of this comparison matrix have been documented in the following sections.

The three TBM retrieval options are assessed against 9 differentiator parameters which have been specifically established for the Aramis project and they are listed below:

- Flotation safety.
- Tunnel lining instability.
- TBM rolling/dropping risk.
- Impact to overall construction programme of Aramis.
- Complexity of marine works.
- Environmental impact.
- Health & Safety.
- Overall construction cost.
- Impact to the pipe installation and tunnel backfill works.

Each assessment criteria have been scored either a 1, 2, or 3 based on its beneficial impact to the project, with 1 being the least beneficial and 3 being the most beneficial. To consider the different levels of importance/criticality, each assessment criteria has a separate weighting factor of 1, 5, or 10, with 10 being the important/critical issue. Then, the benefit score and the weighting score are multiplied together to calculate the final score for each assessment criteria.

8.5.2 Construction programme risks comparison

The programme has been considering in light of the wider aim advised by Contractor to complete the TBM retrieval in summer 2027 while ideally allowing for the pipeline pull operations to be completed prior to the start of the offshore storm season, assumed to be around October 2027. This is a key programme constraint considered when assessing the TBM retrieval options.

The two TBM retrieval options have two distinctive differences in programme perspectives. First difference is the time required for the strengthening of the segmental lining. The secondary lining option requires the construction of secondary lining after the TBM's drive finish, and it requires reinstating of the construction worker's access route after the removal of the TBM's internal parts removal (but prior to the secondary lining installation).

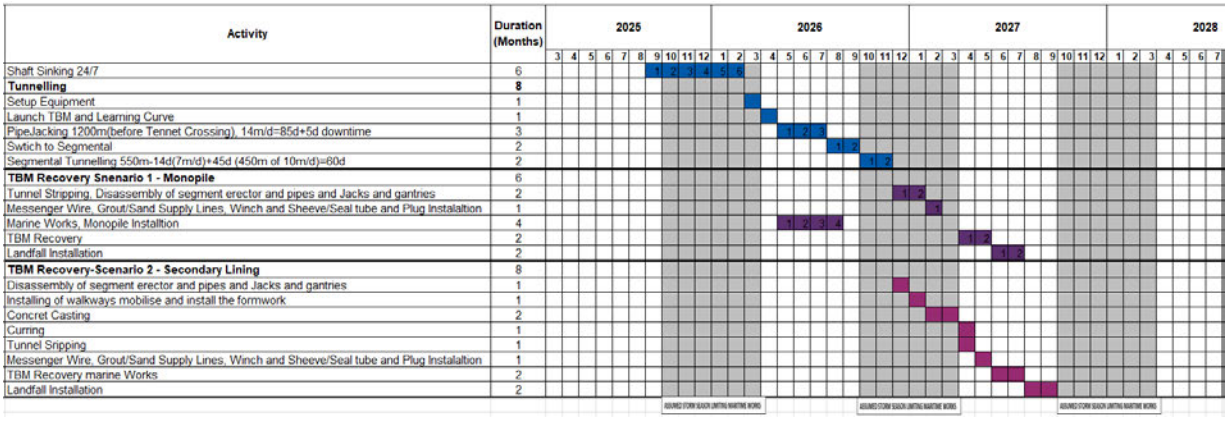
Second difference is the timing of the dredging works. In the secondary lining option, dredging works can only be started after the secondary lining installation is completed, but the monopile option could start the dredging works and the monopile structure installation works concurrently with the TBM driving.

Together with this, the offshore weather condition related restriction (i.e. storm season limiting maritime works for several months during the winter) also gives a significant impact to programme as an independent source of risk.

A very high-level programme comparison is shown in Figure 8-12. (Note that the programme shown in this Figure was developed to illustrate the difference between the two retrieval options. These are not to be considered the final programme.) The programme has been agreed as an appropriately accurate depiction of each of the construction programmes for both construction method. For further details of how the programme comparison was developed including input from Company, refer to meeting minutes 'Tunnel FEED – TBM Retrieval follow up session', document number: ARM-ARC01-PTC1-ARM1-MOM-000083, 22/05/2024.

Both the secondary lining and monopile options could meet the pipeline installation time constraint although secondary lining option's overall duration could be a couple of months longer than the other. However, the monopile option involves a lot of weather sensitive marine activities, therefore the programme uncertainty of the monopile option is greater. It is noted that these comparisons assume allowance for an EPC award and adequate engineering prior to the start of shaft sinking, which is governed by factors outside the control of tunnel FEED design (including for example confirmation of the EPC award date, and commencement of engineering and detailed design by the EPC Contractor).

Figure 8-12– TBM retrieval option programme high level comparison



8.5.3 Results

Considering the afore mentioned programme related risks, a simplified comparative assessment was carried out (see Figure 8-13). The assessment result shows that the secondary lining option scores marginally better than the monopile option, thus the FEED design adopted the secondary lining option as the preferred option.

The study history of TBM retrieval option is summarised in this document : ‘Tunnel FEED – TBM Retrieval follow up session’, ARM-ARC01-PTC1-ARM1-MOM-000083, 22/05/2024.

Figure 8-13 - Post workshop: TBM retrieval options comparative assessment matrix

Changes from previous version (22/05/24 Workshop) in red text.

Weight factor to be agreed with client in scale of 1 to 10
10 - Critical importance
5 - High importance
1 - Moderate importance

Preference (scale of 1 to 3)
3 - Better + most advantageous
2- Medium
1 - Worse + least advantageous

No.	Assessment Title	Description	Assessment weighting factor	UN-weighted preference order: Use of monopile	UN-weighted preference order: Cast in place secondary lining	UN-weighted preference order: Shear dowel + steel frame	UN-weighted preference order: Steel lining	Weighted preference order: Use of monopile	Weighted preference order: Cast in place secondary lining	Weighted preference order: Shear dowel + steel frame	Weighted preference order: Steel lining
1	Flotation	Tunnel floats as a result of the shallow overburden due to dredging around the tunnel reception. Lining joints are opened. Soil and water leak into the tunnel (quick sand). Uncontrolled flooding of tunnel and potential collapse.	1	2	2	1	discarded	2	2	1	discarded
2	Tunnel lining instability	Tunnel lining loses stability due to the loss of surrounding medium (soil) during the dredging and potential accidental load exerted to the lining during the pipe installation. Local damage/collapse of tunnel. Risk of uncontrolled flooding of tunnel.	5	3	3	1	discarded	15	15	5	discarded
3	TBM rolling/dropping	TBM rolls/drops due to the loss of soil support around TBM because of the excessive dredging under the TBM. Segment lining's joint is damaged and opened. Result in uncontrolled flooding of tunnel and potential local collapse.	1	2	2	1	discarded	2	2	1	discarded
4	Overall construction programme	If dredging could be done before TBM's arrival, retrieval duration is shorter. If a large amount of in-tunnel works are required, then this will add additional pressure to the construction programme. Due to seasonal restrictions, any delay will result in a year long delay to construciton. Therefore, the weighting factor has been adjusted to a '10' for the most imporant issues, to give the 'Critical important' issues greater weighting.	10	3	2	1	discarded	30	20	10	discarded
5	Complexitiy of marine works - Construction programme uncertainty	Proposed retrieval sequence is sensitive to the weather condition. Adds uncertainty to the construction programme.	1	1	3	3	discarded	1	3	3	discarded
6	Envromental impact	Environmental impact associated with the dredging and marine works	5	2	3	3	discarded	10	15	15	discarded
7	H&S	The implication of more diver workers is that there is a greater level of H&S risk during construction.	5	2	2	2	discarded	10	10	10	discarded
8	Overall construction cost - materials only (qualitative assessment)	Higher cost less preferrable.	5	1	3	3	discarded	5	15	15	discarded
9	Impact to the pipe installation and tunnel backfilling works	Whether option has sufficient space for the installation of single pipe.	1	3	3	3	discarded	3	3	3	discarded
		Total		19	23	18	0	**78**	**85**	**63**	0

= preference score * weighting factor

8.6 TBM retrieval / reception pit requirements if completing in pipe jack mode

Note that if successfully completing the tunnel in pipe jacking mode, then limited strengthening works are required prior to TBM retrieval, for example limited to ensuring a connection between adjacent ultra-heavy pipes used at the tunnel exit location (for example by bolting adjacent pipes together using pre-installed flanges). Reception pit design would be similar to that required for the internal strengthening option.

9 CONSTRUCTION SEQUENCE

This section describes high level considerations developed to inform the shaft and tunnel FEED calculations presented in this report, and is not intended to present a full end to end sequence which will include further aspects such as site setup and plant mobilisation etc. Further details of construction sequencing are provided in the Preliminary Construction Execution Plan [18].

9.1 Shaft construction sequence

The launch shaft will be the first structure to be constructed, located at the Sharkfin site. The following main construction stages have been identified:

- Construct 91No. 150mm diameter tension piles before dewatering from ground level, with 50mm diameter Dywidag or Macalloy, installed from approximately the middle of the middle of the base slab level to the toe of the piles;
- Construct temporary piling platform and guide wall;
- Construct 1.2m diaphragm wall and capping beam before dewatering;
- Excavate shaft in the wet to the bottom of the shaft in advances ranging from 1m to 1.2m, and install steel plates at the top of the tension piles;
- Cast 1st stage, unreinforced base slab in the before dewatering;
- Dewater the shaft down to the top of the 1st stage base slab.
- Cast 2nd stage, reinforced base slab in the dry;
- Construct tunnel eye and thrust wall.

The shaft construction sequence is shown on drawing NL-ARM-040-ARU1-110332.

9.2 Tunnel construction sequence

A hybrid excavation method has been proposed for the Aramis tunnel with allowance for pipejacking and segmental lining as the two main lining types along the alignment. Design of tunnel rings, segments and retrieval method is based on considering different “switchover” scenarios, as summarised in drawing NL-ARM-040-ARU1-110323. Details for the switchover between the pipe jacking and segmental lining sections of tunnel are presented in drawing NL-ARM-040-ARU1-110328.

- Hybrid ‘Flying start’: a start-up of tunnelling in pipe jack mode and switch to segmental tunnelling after approx. 100-150m.
- Hybrid solution with a planned switch from pipe jacking to segmental tunnelling between 900m and 1300m considering for upper and lower ground condition, respectively. Considering the location of the TenneT cables, a planned switchover could occur around 1200m , to avoid switchover below the TenneT cables, minimising the risk of any potential disturbance.

- Switch from pipe jacking to segmental tunnelling based on the observational method during construction. This scenario provides flexibility to EPC contractor to select the location of switch-over taking into consideration the actual ground condition, site logistics and potential risks during construction.
- Similar to the switch based on observational method, it is considered feasible to plan to pipejack the entire tunnel length on the condition of retaining a contingency to switch if required), based on EPC Contractor review and monitoring of actual ground conditions encountered. This approach assumes that enough interjack stations are incorporated by EPC contractor to reduce the risk of the jacking pipes become stuck, and that full planning is maintained for a switchover to segmental lining as a contingency method, due to the unacceptable risk of not completing the tunnel. This methodology will rely heavily on EPC Contractor's assessment of actual data gained during pipejacking operations.

For the purpose of developing the FEED schedule and cost estimate, a plan to pipejack the entire length of tunnel has been assumed (on the basis of full planning for a contingency switchover around 1200m chainage). Further discussion is included in the Preliminary Construction Execution Plan [18].

9.3 Decommissioning

Once all works on site have been completed all temporary structures shall be decommissioned, dismantled and removed from site. Site remediation and rehabilitation works will then be undertaken to areas disturbed during construction, to be agreed with Company. Once completed, post decommissioning monitoring will be put in place to ensure the effectiveness of remediation works, if required by the remedial works undertaken.

It is noted that different requirements may be provided by Company for remedial works / reinstatement across the site depending on the strategy for post-construction land ownership and maintenance. For example, area retained by Company may have different requirements compared to area returned to Port of Rotterdam. In general, it is assumed that all areas of the site that are to be returned to third parties must be returned to their original pre-construction condition.

Once the tunnel has been grouted, it is anticipated that no future access will be possible; and that the tunnel will be left in-situ indefinitely.

10 TBM INTERVENTIONS AND STOPS

10.1 Overview

There is a potential need to stop the TBM and carry out an intervention in the face in order to repair or replace the cutting tools. It was recommended in the pre-FEED design reports to allow for two cutterhead interventions either side of the shipping channel due to the high abrasivity of the ground. The feasibility of carrying out cutterhead interventions is highly dependent on the localised ground conditions and therefore three locations have been selected along the alignment and a range of design parameters have been considered.

The cutterhead intervention (CHI) locations are selected to provide a range of differing depths, ground conditions and access potential for ground improvement works.

10.2 Cutterhead Intervention Locations

For the planned tunnel drive, three locations have been selected to check the feasibility of carrying out a TBM intervention.

Table 10-1 - Cutterhead Intervention Locations

Model	Approx. Tunnel Chainage (m)	Comment	Ground Conditions
1	0+500	South of Shipping Channel prior to tunnel drive	KR-Sand & Silty Sand incl., (GI works possible)
2	1+000	Below shipping channel	KR-Sand & Silty Sand incl., (GI works not possible due to shipping channel)
3	1+500	North of Shipping Channel after tunnel drive	KR-Sand & Silty Sand incl., (GI works possible)

The ground conditions at each location are defined as per the GIR and the parameters selected for sensitivity include LB, LM and UM parameter sets.

10.3 Modelling Approach

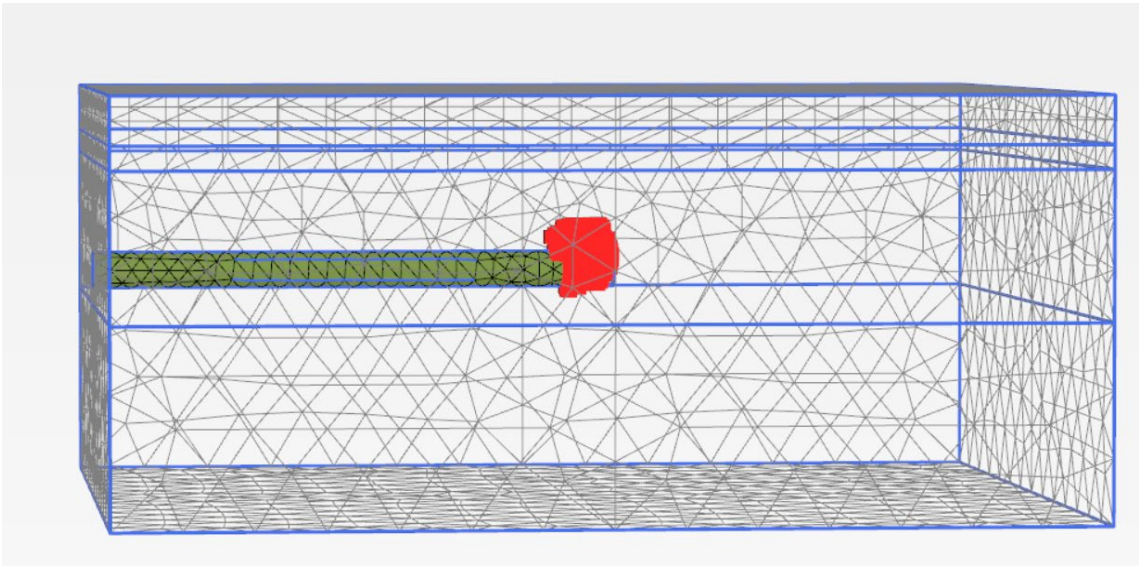
The cutterhead interventions were modelled in finite element analyses using PLAXIS 3D. The following assumptions were used in developing the model:

- The compressed air pressures were modelled as a uniform water pressure within the unsupported length of the CHI and the ground (soil stratum) around the tunnel has been modelled using the Mohr-Coulomb model.

- Annular gap between the TBM and the ground would be sealed during CHI works and potential water ingress contribution due to flow of ground water along the annular gap is not considered.
- Unsupported length of 1m at the cutterhead for CHI works
- Seepage into the tunnel is targeted to be less than 100l/min

Initial assessments of the required compressed air pressure for the CHI, in the absence of a GI (ground improvement) block, was based on an analytical approach at the Ultimate Limit State (ULS) condition. Further assessments to verify the adequacy of the compressed air pressures and proposed GI block design were carried out via 3D finite element analysis using PLAXIS 3D (See Figure 10-1).

Figure 10-1 – View of Plaxis 3D model 1 (Ch0+500)



Initial model set up was carried out using compressed air at an equivalent pressure to the hydrostatic pressure. Additional models considered the presence of a ground improvement block (for Model 1 and 3).

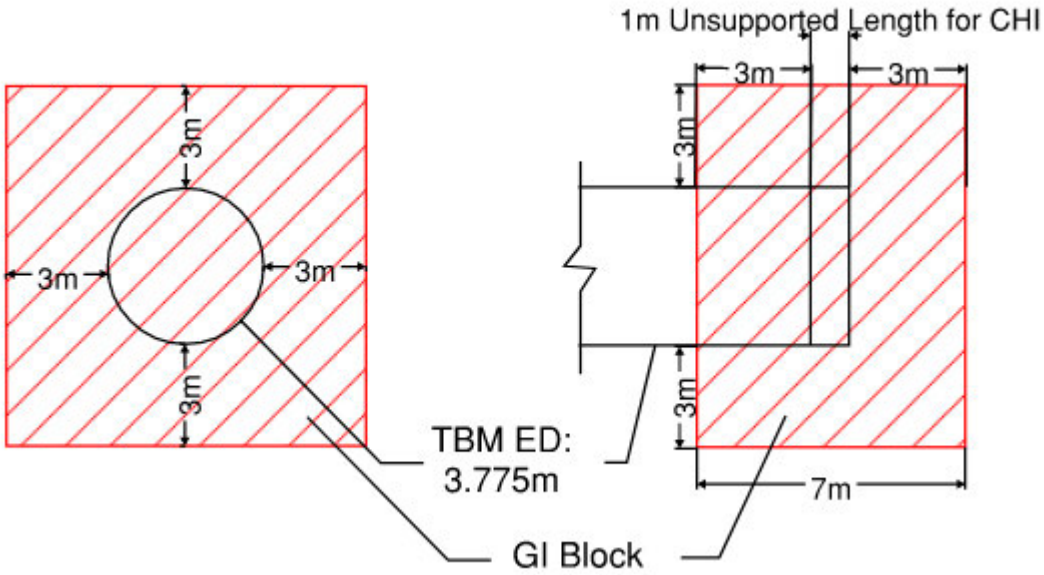
Additionally, a sensitivity assessment was carried out considering Upper Mean (UM) parameters with lower bound permeabilities, for the Kreftenheye Formation Sand & Silty Sand Inclusions Strata, in the absence of a GI block for Scenario 1 to assess the potential of omitting the GI block under ideal conditions.

The ground improvement adopted in the analysis has the following parameters as shown in Table 10-2 and Figure 10-2.

Table 10-2 - Proposed properties for GI block at CHI locations

Property	Value
Cu (kPa)	150
E' (MPa)	75
k (m/s)	1×10^{-7}

Figure 10-2- Geometry of GI Block



10.4 Results

10.4.1 Model 1 (Ch0+500)

The results, in Table 10-3, showed that for LB and LM parameters without ground improvement, there is a potential collapse of the tunnel face. For UM parameters, the model remained stable with a minimum pressure of 1.7 bars, however this leads to a large inflow of water in excess of 600 litres per minute. A higher pressure, equivalent to hydrostatic, has also been applied which reduced the inflow to approximately 133 litres per minute.

With the addition of a 3m ground improvement block, the inflow can be significantly reduced, and the model shows that the compressed air can be reduced to atmospheric.

Table 10-3 - CHI results for model 1 (Ch 0+500)

Tunnel face permeability, k (m/s)	**Strength and stiffness parameters (Seepage)**		
	LB Silty Sand	LM Silty Sand	UM Silty Sand
LB Sand ($3e^{-5}$)	Collapse	Collapse	1.7* bars (626 l/min) 3.7* bars (133 l/min)
3m GI block ($1e^{-7}$)	Free Air (6 l/min) 1.7 bars (3.6 l/min)	Free Air (6 l/min)	-

**Equivalent to hydrostatic pressure at tunnel crown*

10.4.2 Model 2 (Ch1+000) – No Ground Improvement considered

The results, in Table 10-4 show for the first model that for LM parameters without ground improvement, there is a collapse of the tunnel face. For UM parameters, the model remained stable. However, the inflows range from 97 litres per minute to 676 litres per minute. This is highly sensitive to the permeability of the ground and any CHI must be located within a section of ground with permeability of $3x10^{-6}$ m/s or lower to meet the seepage requirements.

Table 10-4 - CHI results for Model 2 (Beneath Shipping Channel)

Tunnel face permeability, k (m/s)	**Strength and stiffness parameters (Seepage)**		
	LB Silty Sand	LM Silty Sand	UM Silty Sand
LB Sand ($3e^{-5}$)	-	Collapse	4.2* bars (250 l/min)
LM Silty Sand ($3e^{-6}$)	-	-	3.2 bars (97 l/min)
UM Silty Sand ($1e^{-4}$)	-	-	4.2* bars (675 l/min)

**Equivalent to hydrostatic pressure at tunnel crown*

10.4.3 Model 3 (Ch1+500)

Similar to model 1, the results, in Table 10-5, showed that for LB and LM parameters without ground improvement, there is a collapse of the tunnel face. For UM parameters, the model remained stable with a minimum pressure of 1.8 bars, however this leads to a large inflow of water in excess of 1000 litres per minute. A higher pressure, equivalent to hydrostatic, has also been applied which reduced the inflow to approximately 181 litres per minute.

With the addition of a 3m ground improvement block, the inflow can be significantly reduced, and the model shows that the compressed air can be reduced to atmospheric.

Table 10-5 - CHI results for model 3 (Ch 1+500)

Tunnel face permeability, k (m/s)	**Strength and stiffness parameters (Seepage)**		
	LB Silty Sand	LM Silty Sand	UM Silty Sand
LB Sand ($3e^{-5}$)	Collapse	Collapse	1.8 bars (1080 l/min) 4.1* bars (181 l/min)
3m GI block (1^{e-7})	Free Air (7 l/min) 1.7 bars (4.2 l/min)	Free Air (7 l/min)	-

**Equivalent to hydrostatic pressure at tunnel crown*

10.5 Summary of Cutterhead Interventions

In the event of a cutterhead intervention being required during the tunnelling works, the following compressed air pressures are recommended in. Excessive water ingress is observed even when compressed air pressure is set to the hydrostatic pressure at the tunnel crown when without GI block for all three design sections. Hence, compressed air pressure set to the hydrostatic pressure at the tunnel invert is proposed in the absence of GI block (assuming full discharge of slurry from the excavation chamber).

Table 10-6 - Recommended Compressed air pressures during CHI

Ground Improvement	**CH. 0+500**	**Under Shipping Channel**	**CH. 1+500**
3m Ground Improvement Block	Free air	-	Free air
No Ground Improvement	4.2 bars*	4.7 bars*	4.5 bars*

**Equivalent to hydrostatic pressure at tunnel invert*

11 DURABILITY

11.1 General

All concrete elements shall follow the deemed-to-satisfy provisions of NEN EN 206 and the National Annex NEN 8005 as listed in table Table 4-2. If alternative cement types or combinations are proposed these shall be evaluated in accordance with CROW-CUR Aanbeveling 48:2022.

In terms of chemical attack from natural soil and groundwater, exposure class XA2 is considered most relevant for both shaft and tunnel elements. The characteristic values identified during ground investigation are summarized in Table 11-1

Table 11-1 - Relevant ground and groundwater chemistry information

Formation	pH	Chloride 2:1 extract [mg/L]	SO_4^{2-} 2:1 extract [mg/L]
Made ground and Naaldwijk Formation	≈ 5.5	≈ 2500	≈ 2500
Kreftenheye Formation	≈ 7.4	≈ 500	≈ 2400 - 3000
EN 206 design values, XA	5.5	2500	3000

Regardless of the exposure conditions all concrete shall adopt the prevention measures to avoid alkali-silica reaction and therefore meet CUR 89. The following strategies shall be adopted by order of preference:

1. Cement selection compliant with section 5.3.1 and Table 1 of CUR 89: CEM II/B-V (min. 25% fly ash), CEM III/A (min 50% slag), CEM III/B (min. 66% slag)
2. Equivalent combinations compliant with section 5.3.2 and Table 11-2 of CUR 89: CEM I + min. 25% fly ash, CEM I + min 50% slag.
3. Alternative cements or combinations to be tested in accordance with the accelerated mortar bar test (Appendix E) or concrete prism expansion test (Appendix F).

Table 11-2 Recommended limiting values for the durability of concrete containing reinforcing steel or prestressed elements with intended working life of at least 50 years.

Requirements/ design parameters	XC2	XC3	XC4	XS1	XS2	XS3	XF1	XF3	XF4	XA1	XA2	XA3
Strength class Note 1	C35/45	C35/45	C40/50	C40/50	C40/50	C45/55	C35/45	C35/45	C45/55	C35/45	C35/45	C35/45
@Max. w/c-ratio [-]	0.60	0.55	0.50	0.50	0.45	0.45	0.55	0.50	0.45	0.55	0.50	0.45
Permitted cement types	CEM II/B-V, CEM III/A, CEM III/B or equivalent combinations										a)	a) b)
Min. cement content [kg/m3]	280	280	320	300	320	340	300	320	340	300	320	340
Min. cover to reinf. $c_{min.}$ [mm]	25	25	30	35	40	40	30	30	30	25	25	25
Deviation, Δc [mm]	5 mm precast 10 mm to 15 mm in-situ											
Nominal cover, c_{nom}	$c_{min} + \Delta c$											
Notes	Note 1 If quality control of concrete is specified, reduction by 1 strength class applies. a) For concrete in this environmental class, the specific suitability of the cement must be demonstrated in accordance with CROW-CUR Recommendation 48. The specific suitability per environmental class is limited to the examined cement with the same or a higher strength class. Comparable cements may only be used without examination after the specific suitability of at least five cements of one cement type has been demonstrated and these cements have been available on the market and have been used for five years. b) For concrete in these environmental classes that is exposed to solutions containing more than 600 mg SO42-/l or to soil with a sulphate content greater than 3,000 mg/kg, cement with a high resistance to sulphates (SR) should be used and compliant with NEN-EN 197-1, with the exception of CEM I-SR 5, pozzolanic cement CEM IV/A-SR and CEM IV/B-SR. For these and other cements, the high resistance to sulphates must be demonstrated as described in CROW-CUR Recommendation 48.											

11.2 Shaft

The governing exposure class and corresponding minimum concrete cover for durability for the shaft, as adopted for FEED design, is summarised in as follows.

Sections of the shaft are also subject to:

- XA2, moderately aggressive chemical attack from the ground
- freezing and thawing, top section (XF1 or XF2 if near roadways)

Table 11-3 Shaft - recommended limiting values and additional performance requirements for the durability of concrete for intended working life of at least 50 years

Exposure class (**prevailing** + additional)	**XS2** + XC2 + XA2 + XF1/2 (extrados) XS2 (intrados)
Exposure class during construction	XS2 + XC2 + XA2 (extrados) XC3 (intrados)
Min. strength class	C40/50
@Max. w/c-ratio [-]	0.45
Permitted cement types or equivalent combinations	CEM II/B-V, CEM III/A+SF, CEM III/B
Min. cement content	320
Min. cover to reinforcement, c_{min} [mm]	45
Deviation, Δc [mm]	5 (minimum 50mm if cast against soil)
Nominal cover, c_{nom} [mm]	50 (minimum 100mm if cast against soil)
Additional performance requirements	
Sulfate resistance	CUR 46, Annex E
Chloride resistance [$\cdot 10^{-12}$ m^2/s] NT Build 492 or NEN EN 12390-11	<table><tr><td>Cement type</td><td>Average</td><td>Max.</td><td>Prob. corrosion initiation</td></tr><tr><td>CEM II/B-V</td><td>≤8.0</td><td>≤10</td><td>10% @ 50 years</td></tr><tr><td>CEM III/A+SF</td><td>≤4.5</td><td>≤5.8</td><td>7% @ 50 years</td></tr><tr><td>CEM III/B</td><td>≤2.5</td><td>≤3.2</td><td>4% @ 50 years</td></tr></table>
Critical crack width, $w_{k,crit}$ [mm]	0.20
Sacrificial layer, Δh [mm]	10 mm

11.3 Tunnel

The environmental load exposure values acting on the section of the tunnel is associated with the respective environmental loads. The tunnel segments will be subject to corrosion induced by chlorides from seawater (XS), carbonation induced corrosion (XC) and chemical attack (XA). Freezing and thawing (XF) is considered for segment storage conditions.

Service life verification in accordance with *fib*-34 is performed to confirm the expected reliability of the NEN 206+NEN 8005 for the prevailing deterioration mechanism of chloride-induced corrosion (XS2). This will result in the confirmation of concrete cover values and any other performance requirements e.g. chloride migration coef. In addition to the verifications above, for segments steel fibre-reinforced the following requirements apply:

Table 11-4 Tunnel segments - recommended limiting values and additional performance requirements for the durability of concrete for intended working life of at least 50 years

<table>
<tr><td>Exposure class (prevailing + additional)</td><td>XS2 + XC2 + XA2 (extrados)
XS2 (intrados)</td></tr>
<tr><td>Exposure class during construction</td><td>XC4 + XS1 + XF3 (storage)
XS2 + XC2 + XA2 (extrados)
XC3 (intrados)</td></tr>
<tr><td>Min. strength class</td><td>C40/50</td></tr>
<tr><td>@Max. w/c-ratio [-]</td><td>0.45</td></tr>
<tr><td>Permitted cement types or equivalent combinations</td><td>R/C: CEM II/B-V, CEM III/A, CEM III/B
SFRC: CEM II/B-V, CEM III/A + SF, CEM II/B-V + SF, CEM III/B</td></tr>
<tr><td>Min. cement content</td><td>320</td></tr>
<tr><td>Min. cover to reinforcement, c_{min} [mm]</td><td>45</td></tr>
<tr><td>Deviation, Δc [mm]</td><td>5</td></tr>
<tr><td>Nominal cover, c_{nom} [mm]</td><td>50</td></tr>
<tr><td>Sacrificial layer, Δh [mm]</td><td>10 mm</td></tr>
<tr><td>Critical crack width, $w_{k,crit}$ [mm]</td><td>0.20</td></tr>
<tr><td colspan="2">Additional performance requirements</td></tr>
<tr><td>Sulfate resistance</td><td>CUR 46, Annex E</td></tr>
<tr><td>Chloride resistance [$\cdot 10^{-12}$ m^2/s]
NT Build 492 or NEN EN 12390-11</td><td>R/C segments
<table>
<tr><td>Cement type</td><td>Average</td><td>Max.</td><td>Prob. corrosion initiation</td></tr>
<tr><td>CEM II/B-V</td><td>≤8.0</td><td>≤10</td><td>10% @ 50 years
4% @ 30 years</td></tr>
<tr><td>CEM III/A</td><td>≤3.8</td><td>≤4.9</td><td>11% @ 30 years</td></tr>
<tr><td>CEM III/B</td><td>≤2.5</td><td>≤3.2</td><td>4% @ 50 years
1% @ 30 years</td></tr>
</table>
SFRC segments
<table>
<tr><td>Cement type</td><td>Average</td><td>Max.</td><td>Prob. corrosion initiation</td></tr>
<tr><td>CEM II/B-V</td><td>≤5.6</td><td>≤7.2</td><td>15% @ 50 years
11% @ 30 years</td></tr>
<tr><td>CEM II/B + SF
CEM III/A + SF</td><td>≤2.5</td><td>≤3.2</td><td>15% @ 50 years
10% @ 30 years</td></tr>
<tr><td>CEM III/B</td><td>≤1.5</td><td>≤2.0</td><td>14% @ 50 years
10% @ 30 years</td></tr>
</table></td></tr>
</table>

The introduction of Δh is necessary to discount from the original cross-section and $w_{k,crit}$ thus ensuring that mechanical capacity at ULS is not compromised due to corrosion of the fibres.

Since the tunnel will be permanently submerged or backfilled, corrosion induced by carbonation is not the prevailing deterioration mechanism.

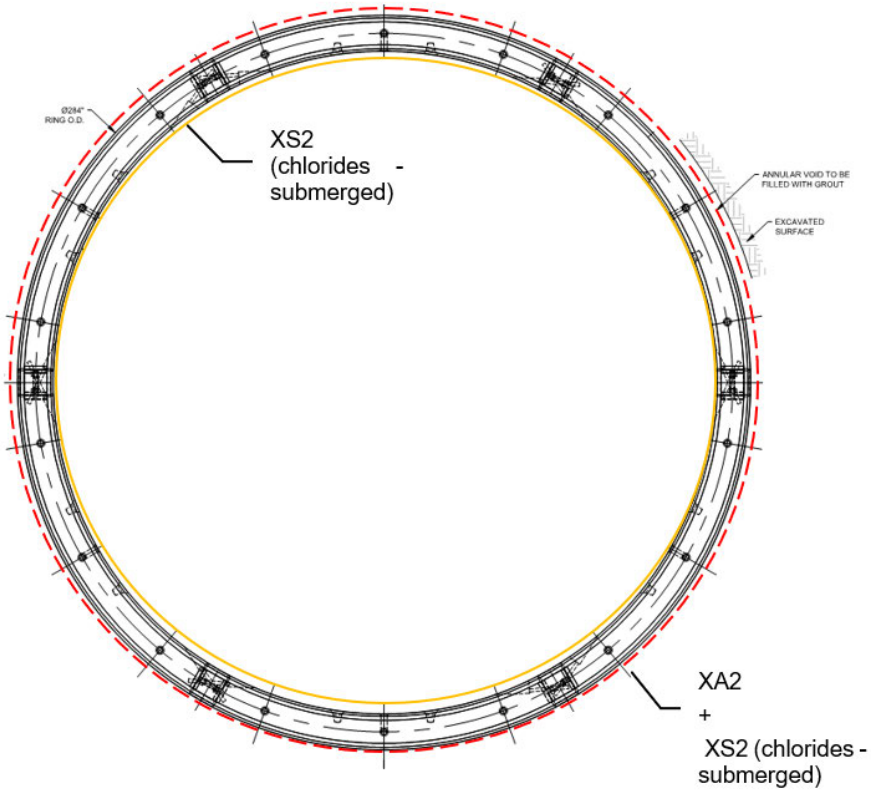
The models used to verify the durability design are only applicable in the cases when only one mechanism is dominating the deterioration of the structure e.g. chloride-induced corrosion over carbonation-induced corrosion for instance.

The representation of these exposure classes within a typical cross-section are shown in Figure 11-1.

Table 11-5 Generalized exposure levels for the tunnel sections

Exposure class NEN EN 206	Representative Elements	Environmental Loads		
		Cs [%/wt.cem]	Chemical attack (soil) [-]	Freeze-thaw
XS2 + XC2 + XA2	Precast tunnel segments and jacking pipes	2.5	3000 mg/L SO_4^{2-}	Yes, XF3 (during storage)

Figure 11-1 - Project specific exposure environments – representation of the tunnel environment (neglecting annulus grouting region)



During EPC Detailed Design, EPC Contractor should consider specifying of polypropylene fibres within the tunnel concrete to provide passive fire resistance, noting that the tunnel will be grouted in the long term..

11.4 Tunnel annulus grouting

The purpose of the annulus grouting is to:

- Minimize surface settlements and ground deformations
- Lock the segmental lining into position and transfer loads
- Ensure uniform contact between the ground and the lining
- Complement the waterproofing system of the tunnel

In terms of durability considerations it is very important to obtain groundwater and soil chemistry information to enable the classification of the ground and groundwater in accordance with Table 2 of NEN EN 206. Additional points to note:

- Sulfate Attack:
  - Geological formations may contain sulfates that react with grout components leading to expansion, cracking, and loss of strength.
  - Mitigation: Use sulfate-resistant cements or combinations of cement with fly ash or GGBS at suitably low w/c-ratios.
- Saline Groundwater:
  - Adverse effects on grout properties due to contact with saline groundwater
  - Required measures: Special formulations and protective measures to ensure stability and performance.
- Ground Humidity:
  - Influence of natural ground humidity on grout durability
  - Requirement: Grout must remain un-deformed and stable, achievable with maintained water content and low ground permeability
- Chemical Contaminants:
  - When detected, the presence of acids or alkalis in the ground can impact grout durability
  - Mitigation: The use of combinations of cement with fly ash or GGBS at suitably low w/c-ratios that result in chemical-resistant grouts.

11.5 Backfill grouting

Underwater grouting using controlled low-strength cementitious materials (CLSM) and foamed underwater grouts is a specialized technique designed to address the challenges of underwater construction. The primary objective is to create a low-strength, self-compacting grout that can be pumped over long distances, effectively displace water, and resist washout.

The mix design approach should make use of CEM I, fly ash or ggbs, foaming agents and, if necessary, anti-washout agents.

Typical performance requirements of underwater foamed grouts:

- Flowability: High flowability allows CLSM to fill voids and hard-to-reach areas effectively.
- Self-Leveling: Settles into place without the need for additional compaction.
- Low Permeability: Prevents water infiltration and maintains structural integrity.
- Strength: 0.70 (7-day) to 1.5 MPa (28-day)
- w/c ratios: 0.40 – 0.60
- Resistivity: 5-10 kOhm.cm (50-100 Ohm.m)
- Density: 320 – 1600 kg/m3

Representative mix designs (proprietary or designed) should be prepared in advance of the works to confirm fresh, hardened state properties during laboratory tests. Mock-up tests should also be executed to simulate application and test the limitations of the material under underwater conditions:

- Laboratory Trial Mixes:
  - Flowability: Mini-slump test.
  - Compressive Strength: Test at 1, 7, 28, and 90 days.
  - Electrical Resistivity: Measure for durability and corrosion resistance.
- Mock-Ups:
  - Simulate real-life underwater placement using a plexiglass model.
  - Observe grout behaviour, water displacement, and washout resistance.

For detailed information on the equipment and techniques for underwater grouting using controlled low-strength cementitious materials and foamed underwater grouts, including the use of colloidal grout mixers, foam concrete mixers, containerized grout plants, and mobile batch plants, please consult with a specialist in this field.

11.6 Shaft backfilling

Long term, the shaft is assumed to be backfilled with cementitious grout or sand. The specific mode of backfill should be considered in EPC detailed design, although it is not anticipated that the shaft structure would be particularly sensitive to the choice of backfill. While future re-excavation of the shaft has not been explicitly considered or analysed during FEED (and would need to be subject to design and verification based on the as-built structure and its condition at the time of any proposed re-excavation), it is suggested that the use of sand as backfill could retain greater flexibility for future re-excavation as well as enabling potential re-use of site-won material (subject to the appropriate PLANC considerations which are outside the scope of this report).

11.7 Commentary on design working life and consequence classes

This section provides narrative on design working life and consequence class, as defined in the Eurocodes referenced in the Tunnel FEED Basis of Design [1]. It also includes commentary on the potential implications of adopting Rijkswaterstaat's Guidelines for the Design of Structures (ROK) [19] should this be required at a future stage of the project.

11.7.1 Consideration of Rijkswaterstaat Guidelines for the Design of Structures (ROK)

Due to the proximity of the Aramis tunnel with Rijkswaterstaat (RWS) assets (sea defence and shipping lane (Maasgeul)), a high-level comparison has been undertaken between the standards in the Tunnel FEED Basis of Design (including Eurocodes) and the RWS Guidelines for the Design of Structures (ROK) version 2.0 [19]. As described in its foreword, the ROK guidelines are to be applied to the execution of RWS's own civil engineering works (i.e. may not directly applicable to Aramis), and are supplementary to Eurocodes. The following considerations (not exhaustive) are included for information only in case of any potential future requirement for the project to comply with ROK guidelines for tunnels:

- ROK-0010 requires that tunnels have at least a 100-year design life for all parts of the main load-bearing structure.
- ROK-0012 requires that "tunnels in and under main roads and under main waterways must be classified in consequence class 3." (NOTE: It is understood that the shipping lane (Maasgeul) is considered a main waterway.)
- ROK-0633 (including tables T0633-1 & T0633-2) provides specific load factors to be used in design for different consequence classes; although it is noted that these appear to be the same as the factors already included in the Dutch Eurocodes for the corresponding consequence class.

11.7.2 Potential to extend the design working life to 100 years

In accordance with the Tunnel FEED Basis of Design, a minimum required design working life of 30 years has been assumed for the FEED design of the shaft and tunnel. Eurocodes typically provide basic parameters corresponding to a 50 year design working life and reference periods, with options to increase the design working life where required. In order to avoid deviating from standard Eurocode recommendations during FEED, factors relating to a 50 year design life have been adopted to evidence a minimum 30 year design working life.

Key impacts of extending the design working life to 100 years have been identified as follows, based on comparison with requirements for a 50 year design working life:

- An increase in structural classification for concrete structures (refer to NEN-EN-1992-1-1 section 4.4.1.2), which is used to calculate the minimum concrete cover for durability ($c_{min,dur}$). The increase in $c_{min,dur}$ to achieve a 100 year design life is an additional 10mm cover per face. Where durability governs concrete cover, this increase may be accommodated by increasing overall element thicknesses, or by maintaining the same thickness with a slight reduction in structural capacity arising from increasing the cover.
- Standard values for environmental actions (such as snow, wind and temperature) are provided based on a recurrence time of 50 years. For a longer design working life, longer return periods may need to be considered, resulting in an increase to environmental actions. It is noted that the shaft and tunnel (being buried) are not particularly sensitive to environmental actions.
- Consideration of specific geotechnical factors is discussed in the Geotechnical Design Report NL-ARM-040-ARU1-110305, in particular relating to:
  - Potential increased influence of soil liquefaction in seismic scenarios on design.
  - Potential increase in loading associated with differential settlements over time.
  - Potential impact on corrosion factors and sacrificial thickness assumed to be required within tension pile reinforcement.

11.7.3 Selection of Consequence Class

Consequence Classes are defined qualitatively in NEN-EN 1990, Table NB.23, with examples of structures provided in Table NB.24 (reproduced together as Table 11-6). The examples do not include tunnels, and the qualitative descriptions can be interpreted subjectively and are thus often defined explicitly by client and / or stakeholder requirements (and it is noted that the consequence class required for the Aramis project has not been explicitly defined).

Given the lack of public access to the Aramis tunnel, the relatively small workforce that may be required to access the tunnel, and in consideration of the proposal to grout the tunnel in the long term (reducing the impact of potential failure of the tunnel structure itself and thus reducing the safety, economic, social and / or environmental consequences), FEED has been undertaken on the assumption of Consequence Class 2, which correlates with a 50 year design life when using the standard partial factors within EN 1990 (as per NEN-EN 1990 Table B.2).

Table 11-6 - Consequence Class descriptions and examples (extracts from NEN-EN 1990 Tables NB.23 and NB.24)

Table NB.23 - B1 - Definition of consequence classes.

Consequence class [a]	Description
CC3	**Major** consequences with respect to loss of life [b], or **very large** economic or social or environmental consequences
CC2	**Medium** impact with respect to loss of life, or **significant** economic, social or environmental consequences
CC1 further subdivided into: CC1b CC1a	 **Minor** impacts with respect to loss of life and **minor** or **negligible** economic or social or environmental impacts **Virtually excluded** loss of life and **very minor** or **negligible** economic or social or environmental impacts

[a] Project-specific classification may be done on the basis of a risk analysis, according to 2.2 of NEN-EN 1990+A1+A1/C2:2019. Examples for the application of consequence classes are given in Table NB.24 - B1.

[b] Refers to situations of structures, in which many people may be present at the same time and where, if an essential part collapses, a large number of people may suddenly be affected.

Table NB.24 - B1 - Examples of applications consequence classes for structures

Consequence class	Examples of applications for buildings and structures except bridges and marine structures	Examples of applications for bridges	Examples of applications for marine structures
CC3	Tall buildings, whose highest floor of an area of use, as defined in the 2012 Building Code, is more than 70 m above ground level at the point of entry Structures, where the span of the structure in a bearing direction exceeds 50 m and where in case of collapse of that span more than 500 persons would be at risk simultaneously (such as large exhibition and station halls) Structures with the intended public function (e.g. educational buildings stadiums, concert halls, grandstands), where in the event of a collapse more than 500 persons would be at risk simultaneously Buildings intended for a group of less self-sufficient persons such as hospitals, cellular buildings, nursing homes, with 4 or more floors Buildings where highly vital processes are controlled, e.g., Schiphol control tower, traffic control building Industrial buildings for hazardous substances and/or processes being part of an establishment for which an environmental permit is required and whose collapse of the building may result in the relevant undesirable environmental effect	Bridges in and over main roads, main waterways and national railroads This concerns bridges: — in and over the main infrastructure or over main waterways with intensive industrial (marine) navigation or — where collapse has society-disrupting consequences;	Maritime structures, Quay walls that are part of a primary water barrier or a .main waterway Bollards with primary nautical functions, such as marking or lighting a harbor entrance, lock or bridge

Consequence class	Examples of applications for buildings and structures except bridges and marine structures	Examples of applications for bridges	Examples of applications for marine structures
CC2 [a] [b]	To the extent not included in CC1 or CC3: Single-family houses [c] with 4 or more building levels Residential buildings, hotels and office buildings Educational buildings Shops Buildings intended for a group of reduced self-sufficient persons such as hospitals, cellular buildings, nursing homes, with a maximum of 3 building layers Public buildings Industrial buildings where the number of persons inside is not limited or with 3 or more building levels Parking garages	Bridges not classified in CC 1 or CC 3	Maritime structures, such as quay walls, jetties, breakwaters and mooring posts, without functional redistribution capacity [d] Jetty structures, fenders and mooring posts part of the equipment of high-threshold BRZO companies (Major Accident Hazards Decree)

11.7.4 Potential to increase the Consequence Class

Where required to achieve a higher Consequence Class than CC2 (e.g. CC3), it is necessary to consider several additional requirements within the Eurocodes:

- Increased requirements for Design Supervision:
  - For CC2, Design Supervision Level 2 requires design checking to be undertaken by different persons in accordance with the procedure of the organisation.
  - For CC3, Design Supervision Level 3 requires an accredited third party checking (i.e. checking to be performed by an organisation different from that which has prepared the design).
- Increased requirements for Inspection Levels during execution:
  - For CC2, Inspection Level 2 requires inspection in accordance with the procedures of the organisation.
  - For CC3, Inspection Level 3 requires an accredited third party inspection during execution.
- Partial factors for design are to be increased by a factor (K_{FI} = 1.1) i.e. leading to a 10% increase in design loads to be accounted for in design.

11.7.5 Conclusion on potential to increase the design working life and Consequence Class

FEED design has been undertaken using partial factors in accordance with the Tunnel FEED Basis of Design [1], which presented partial factors correlated to Consequence Class 2 and a 50 year design working life.

- If required to adopt a consequence class of CC3, there would be an increase in design loading of around 10%. Based on review of the FEED analysis undertaken, this could likely be accommodated within the shaft design with minimal impact; while the impact on the tunnel design is estimated to be limited to a potential increase to localised bursting reinforcement, and steel fibre dosage.
- Further requirements arising from a potential requirement to adopt Consequence Class 3, i.e. the requirements for Design Supervision Level 3 and Inspection Level 3, do not directly impact the FEED design verifications undertaken, as they are related to the supervision of design and inspection requirements during execution. These would need to be considered as additional requirements during EPC design and execution.
- Increasing the design working life to 100 years would lead to some further impact in relation to concrete durability and environmental loading assumptions, which would need to be considered during detailed design.

12 SETTLEMENTS AND IMPACT ANALYSIS

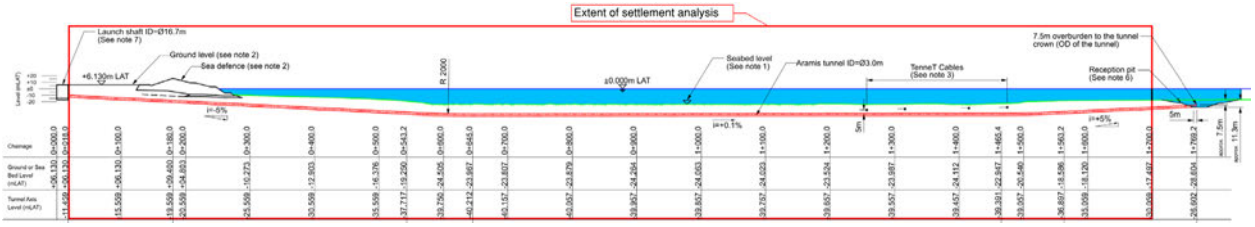
12.1 Summary

Settlement analysis along the alignment has been performed using Oasys Xdisp – an asset damage assessment analysis software. Xdisp estimates three-dimensional displacements caused by different geotechnical engineering activities and their potential impact on nearby structures. The software uses well established empirical methods to provide analytical solutions. These methods are based on the understanding whereby ground movements associated with tunnel excavation commonly resemble an inverted normal Gaussian distribution curve.

The analysis methodology proposed by O'Reilly & New (1982) [20] was adopted to determine vertical settlement along the alignment for 1%, 2% and 3% volume loss to address typical operational range of volume loss for a slurry TBM. A volume loss of 2% has been considered as the baseline for tunnel excavation – this is based on industry experience of typically achievable values for TBMs and is further supported by Vu (2016) [21] stating a maximum volume loss of less than 1.5% can be expected when the cover-to-diameter C/D ratio is greater than 1. The parameters adopted are considered appropriate for tunnelling in soft ground and are conservative when compared with recorded parameters for similar tunnel projects.

As indicated in Figure 12-1, the settlement analysis covers Ch0+018m to ch1+700m. This excludes the Tennet Cables and the sea defence which are addressed in section 12.3. The ground movements at the reception pit have not been included due to associated dredging and excavation works. The ground movements related to the launch shaft are available in NL-ARM-040-ARU1-110381.

Figure 12-1 - Extent of settlement analysis



Plan view settlement contours along the surface of the seabed are available in Appendix C. The maximum expected settlement occurs at the end of the alignment where the tunnel is closest to the seabed. The maximum values of settlement at the seabed are 13mm for 1% VL, 26mm for 2% VL, 39mm for 3% VL at Ch1+700.

Further calculations considering analytical and numerical methods have been performed for third party assets. These have been addressed in Section 12.3.

12.2 Model assumptions and properties

The methodology considers a greenfield ground movement profile and determines horizontal and vertical ground settlements due to the proposed tunnel excavation. The approach has considered a simplification of the seabed.

The XDisp model utilises a “wished into place” construction resulting in maximum values to be calculated and does not consider phases of construction. Furthermore, a limitation of the analysis is the basis of greenfield calculations whereby displacements are calculated solely on the modelled excavation, independent of influence by other existing structures.

The trough width parameter, K, has been calculated within the software using relative distance between the tunnel axis and the location of the analysis. This K parameter is an input dependent on ground conditions, and in this instance is based on the equations presented by O’Reilly and New (1982) [20] under the assumed condition of granular soil.

12.3 Third party assets

Settlement calculations, impact analysis, and recommendations for instrumentation and monitoring in relation to the impact on shaft and tunnel construction on third party impacts is considered in the following standalone report:

- NL-ARM-040-ARU1-110381: Tunnel FEED - Settlement Impact Analysis and Monitoring Plan - Sea Defence and Shipping Lane
- NL-ARM-040-ARU1-110382: Tunnel FEED - Settlement Impact Analysis and Monitoring Plan - TenneT Cables

12.4 General requirements for instrumentation and monitoring

As above, the instrumentation and monitoring related to third party assets has been addressed separately. The general recommendations for instrumentation and monitoring of the tunnel during construction stage are discussed in the following section.

Note, as the tunnel has been designed to be non-maintained, back-grouted and non-accessible, permanent instrumentation for operational stage is not deemed as necessary.

Instrumentation and monitoring of the tunnel during construction will aid to:

- Control and provide confidence in the design and construction process.
- Verify that both ground and structures are behaving as predicted.
- Identify any unexpected trends, anomalies, and responses.

During construction stage, adoption of lining ovalisation monitoring is recommended to inform structural performance is as expected and provide indication of potential ground movements associated with tunnel construction.

Table 12-1 – Outline of proposed construction stage monitoring

Instrumentation	Extent of Monitoring	Frequency of Monitoring	Monitored parameters
Array of reflective prisms (to measure ovalisation)	Maximum 50m spacing along the completed TBM tunnel lining.	Refer to Table 12-2	use absolute positions (x, y, z) to determine tunnel ovalisation.

Lining ovalisation can be monitored using arrays of reflective prisms around the tunnel lining whereby x,y,and z positions are used to determine the tunnel ovalisation. The monitoring arrays for ovalisation shall be placed at maximum 50m spacing along the completed TBM tunnel lining in addition to intervention locations.

Figure 12-2 - Indicative sketch of ovalisation monitoring points

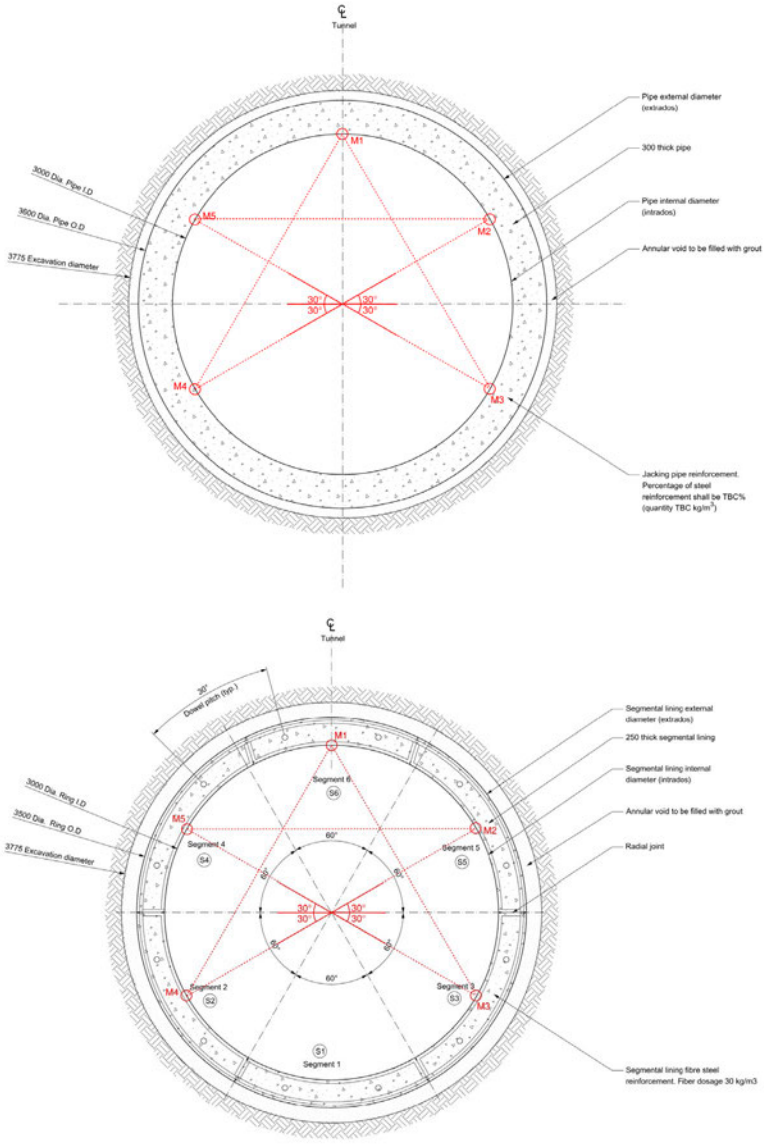


Figure 12-3 - illustration of monitoring distance from TBM face

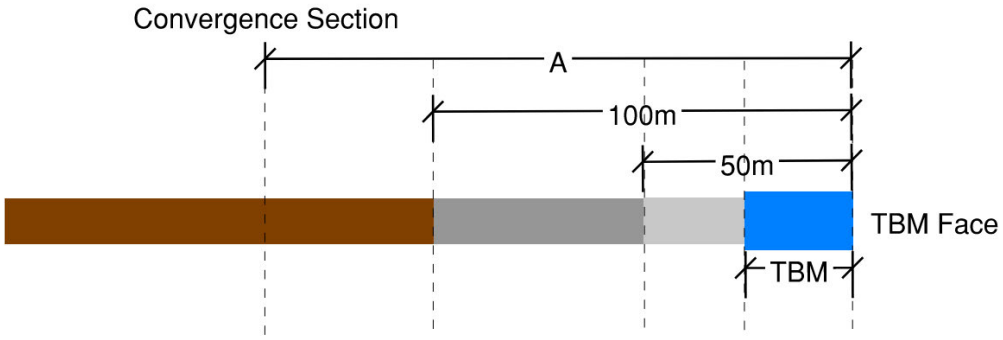


Table 12-2 - Suggested monitoring frequencies

Instrumentation	Frequency			
Array of reflective prisms	Distance from the TBM face to the convergence section (A) [m]	Green	Amber	Red
	A > 100m	1 per week	2 per week	3 per week
	50m < A < 100m	2 per week	3 per week	1 per day
	A < 50m	1 per day	2 per day	3 per day

1. First convergence section reading when possible due to operative issues.
2. Refer to Figure 12-3 for illustration of distance A.

12.4.1 Trigger levels

The following trigger levels have been proposed for the ovalisation arrays based on 1% allowable ovalisation of the tunnel.

Table 12-3 – Table of Trigger levels for ovalisation arrays

Location	Measurement	Trigger values [mm]			Max. Distortion [mm]
		Green[1]	Amber[2]	Red[3]	
Alignment	Change of ovalisation array	18	29	36	36

1. Green trigger set to 50% of SLS value.
2. Amber trigger set to 80% of SLS value.
3. Red trigger set to SLS value.

Note – values are expressed in terms of inner diameter dimension.

Note 2 – trigger levels are change in absolute distance between monitoring points after ring build and annular grouting and after installation of the measuring system.

Table 12-4- Table of actions linked to trigger levels

Location	Trigger Level	Observation / measurements	Actions
As per proposed location in Figure 12-2	Green	absolute positions (x, y, z) to determine tunnel ovalisation.	- Inform the project manager within 24 hours. - Examine the TBM segments / rings. - Prepare and submit a written report reviewing absolute movements / distortions to date. Assessing the effects of the movements on monitored elements and predicting further movements and their effect on monitored elements based on trend to date. - Where it is considered and agreed by the project manager that movement trends indicate that “Amber level” may be reached during the course of works, submit proposal for remedial measures to limit further movement for the approval of the project manager.

Location	Trigger Level	Observation / measurements	Actions
As per proposed location in Figure 12-2	Amber	absolute positions (x, y, z) to determine tunnel ovalisation.	- Inform the project manager immediately. - Examine the TBM segments / rings. - Immediately implement the approved remedial measures in accordance with the contractor's method of construction and temporary works agreed at the "Green level" status. - Work may only proceed if the remedial measures have been implemented and are in the opinion of the project manager to be effective. - Submit an updated report to reviewing the movements including absolute movements and distortion. - Assess the effects on monitored elements and predict further movement and subsequent effects on monitored elements. - Submit revised "Green level" and "Amber level" values which have taken into account the implemented remedial works to project manager for approval before works are resumed.
As per proposed location in Figure 12-2	Red	absolute positions (x, y, z) to determine tunnel ovalisation.	- Inform the project manager immediately. - Immediate cessation of work. - Immediately implement requirements for any emergency measures. - Submit a report detailing full history of movements and remedial measures adopted in relation to actual construction sequence. The report shall include interpretation of events with recommendations for enabling work to proceed. - Work may resume upon written instruction of the project manager.

13 CONCLUSION AND RECOMMENDATIONS

This report has detailed the design calculations and verifications carried out during FEED stage, as relevant to a FEED level of maturity, in order to justify the design decisions taken and presented on the FEED drawings. These drawings as well as further aspects not explicitly discussed in this report (including risk assessments and decisions / mitigations informed by Health, Safety & Environmental aspects) are covered in separate deliverables as described in Section 2.

It is noted that the design has been developed with input from multiple contractors, however the future EPC Contractor is likely to have their own preferences with regard to specific design choices, methods of construction, and sequencing. Adequate opportunity should be provided to the EPC Contractor to undertaken engineering in order to validate the adoption of the FEED design, and to be developed and refined by the EPC Contractor.

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APPENDIX A TUNNEL LINING DESIGN

A.1 Tunnel lining design: Face Stability Calculations

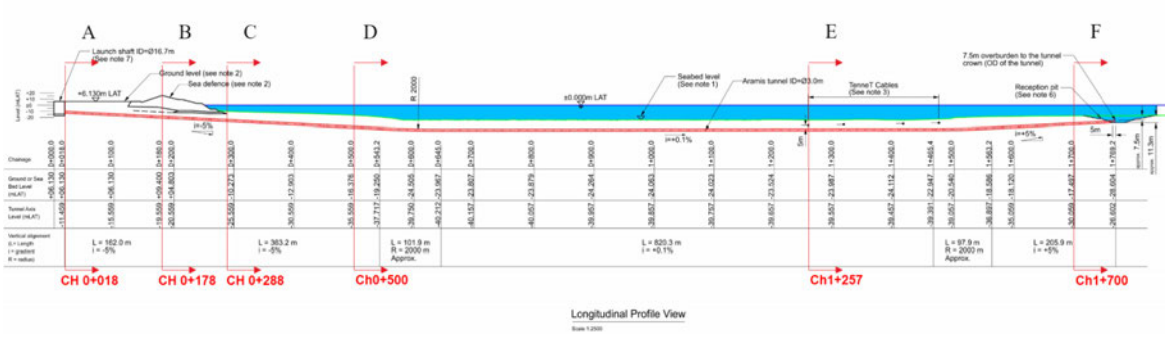
ARUP	Job No.	Sheet No.	Rev.
	603258-53	1	1
	Member/Location		
Job Title: Aramis Tunnel FEED Design	Drg. Ref.		
Calculation: Face Pressure DAUB 2016 - Jancsecz & Steiner, 1994 - Arching	Made by [redacted]	Date 16/05/2024	Chd. [redacted]

(1) Purpose of spreadsheet

This spreadhseet aims to offer a calculation tool for the tunnel face stability assessment in mechanized tunneelling and to estimated the operation range of the face pressure.

(2) Problem Description

Preliminary assesment of the face pressure for Aramis Tunnel, identifying the minimum cover.



(3) Assumptions

- Slurry machine- no water inside the tunnel/excavation chamber
- Soil strength parameters for granular grounds
- Min and Max pressure estimated with the average weight unit weight of the stratigraphy column
- Arching phenomena contribution when Depth > 2D tunnel diameters - sensitivity analysis considered
- Water level at sea level
- Critical angle estimated with an iteration process
- Face pressure deviation ±10kpa, granular ground
- Max pressure > Min pressure +0.2bar (operation range 20kPa)
- Water level has been considered at +4mLAT and 0mLAT (Upper and lower levels)

ARUP	Job No.	Sheet No.	Rev.
	603258-53	2	1
	Member/Location		
Job Title: Aramis Tunnel FEED Design	Drg. Ref.		
Calculation: Face Pressure DAUB 2016 - Jancsecz & Steiner, 1994 - Arching	Made by [redacted]	Date: 16/05/2024	Chd.: CH

(5) Tunnel Aligment

Section No.	Chainage	Internal Tunnel Diameter	Excavation Tunnel Diameter	Grond Elevation	Water Elevation	Tunnel Crown Elevation	Tunnel Sping line elevation
-	m	m	m	m (LAT)	m (LAT)	m (LAT)	m (LAT)
A1	0+18.00	3.00	3.775	6.1	4.0	-9.6	-11.5
B1	0+178.00	3.00	3.775	15.5	4.0	-17.6	-19.5
C1	0+288.00	3.00	3.775	-10.0	4.0	-23.1	-25.0
D1	0+500.00	3.00	3.775	-16.3	4.0	-33.6	-35.5
E1	1+257.00	3.00	3.775	-24.0	4.0	-37.7	-39.6
F1	1+700.00	3.20	3.775	-17.5	4.0	-28.1	-30.0
A2	0+18.00	3.20	3.775	6.1	0.0	-9.6	-11.5
B2	0+178.00	3.20	3.775	15.5	0.0	-17.6	-19.5
C2	0+288.00	3.20	3.775	-10.0	0.0	-23.1	-25.0
D2	0+500.00	3.20	3.775	-16.3	0.0	-33.6	-35.5
E2	1+257.00	3.50	3.775	-24.0	0.0	-37.7	-39.6
F2	1+700.00	3.50	3.775	-17.5	0.0	-28.1	-30.0

ARUP	Job No.	Sheet No.	Rev.
	603258-53	3	1
	Member/Location	Member/Location	
Job Title: Aramis Tunnel FEED Design	Drg. Ref.	Drawing reference	
Calculation: Face Pressure DAUB 2016 - Jancsecz & Steiner, 1994 - Arching	Made by [redacted]	Date 16/05/2024	Chd. CH

(6) Geotechnical design parameters Lower

Secton A	Description	Material behavior	Thickness	Satured unit weight	Effective Friction Angle	Effective Cohesion	Undrained Shear Strength	K_{OCR}
			m	kN/m3	degree	kPa	kPa	
Made Ground (MGR)	MGR - Sand LB	Non-cohesive	13.7	18.0	30.0	0.0	0.0	0.53
Naaldwijk Formation (NLD)	NLD - Clay LM	Non-cohesive	2.0	17.0	19.0	0.0	49.0	0.67
Naaldwijk Formation (NLD)	NLD - Silty Sand LB	Non-cohesive	3.0	17.0	28.0	0.0	0.0	0.53
Naaldwijk Formation (NLD)	NLD - Clay LM	Non-cohesive	-	17.0	19.0	0.0	72.0	0.67

Secton B	Description	Material behavior	Thickness to tunnel spring line	Satured unit weight	Effective Friction Angle	Effective Cohesion	Undrained Shear Strength	K_{OCR}
			m	kN/m3	degree	kPa	kPa	
Made Ground (MGR)	MGR - Sand LM	Non-cohesive	13.1	18.5	31.0	0.0	0.0	0.52
Naaldwijk Formation (NLD)	NLD - Silty Sand LM	Non-cohesive	20.0	17.5	30.0	0.0	0.0	0.52
Naaldwijk Formation (NLD)	NLD - Clay LM	Non-cohesive	2.0	17.0	19.0	0.0	86.0	0.67
Naaldwijk Formation (NLD)	NLD - Silty Sand LM	Non-cohesive	3.0	17.5	30.0	0.0	0.0	0.52
Naaldwijk Formation (NLD)	NLD - Peat	Non-cohesive	2.0	17.5	30.0	0.0	0.0	0.52

Secton C	Description	Material behavior	Thickness to tunnel spring line	Satured unit weight	Effective Friction Angle	Effective Cohesion	Undrained Shear Strength	K_{OCR}
			m	kN/m3	degree	kPa	kPa	
Kreftenheye Formation (KR)	KR - Clay LM	Non-cohesive	11.5	17.0	19.0	0.0	98.0	0.67
Kreftenheye Formation (KR)	KR - Peat	Non-cohesive	2.0	15.0	15.0	0.0	20.0	0.67
Kreftenheye Formation (KR)	KR - Silty Sand LM	Non-cohesive	3.0	18.0	30.0	0.0	0.0	0.52
Kreftenheye Formation (KR)	KR - Sand LB	Non-cohesive	-	19.0	31.0	0.0	0.0	0.53

Secton D	Description	Material behavior	Thickness to tunnel spring line	Satured unit weight	Effective Friction Angle	Effective Cohesion	Undrained Shear Strength	K_{OCR}
			m	kN/m3	degree	kPa	kPa	
Kreftenheye Formation (KR)	KR - Silty Sand LM	Non-cohesive	17.3	18.0	30.0	0.0	0.0	0.52
Kreftenheye Formation (KR)	KR - Silty Sand LB	Non-cohesive	3.0	17.0	28.0	0.0	0.0	0.53
Winterton Shoal Formation (WN)	KR - Clay LB	Non-cohesive	-	16.0	24.0	0.0	120.0	0.81

Secton E	Description	Material behavior	Thickness to tunnel spring line	Satured unit weight	Effective Friction Angle	Effective Cohesion	Undrained Shear Strength	K_{OCR}
			m	kN/m3	degree	kPa	kPa	
Kreftenheye Formation (KR)	KR - Sand LB	Non-cohesive	13.7	19.0	31.0	0.0	0.0	0.53
Kreftenheye Formation (KR)	KR - Silty Sand LM	Non-cohesive	3.0	18.0	30.0	0.0	0.0	0.52
Kreftenheye Formation (KR)	KR - Sand LB	Non-cohesive	-	19.0	31.0	0.0	0.0	0.53

Secton f	Description	Material behavior	Thickness to tunnel spring line	Satured unit weight	Effective Friction Angle	Effective Cohesion	Undrained Shear Strength	K_{OCR}
			m	kN/m3	degree	kPa	kPa	
Kreftenheye Formation (KR)	KR - Sand LB	Non-cohesive	10.6	19.0	31.0	0.0	0.0	0.53
Kreftenheye Formation (KR)	KR - Silty Sand LM	Non-cohesive	3.0	18.0	30.0	0.0	0.0	0.52
Kreftenheye Formation (KR)	KR - Sand LB	Non-cohesive	-	19.0	31.0	0.0	0.0	0.53

ARUP	Job No.	Sheet No.	Rev.
	603258-53	4	1
	Member/Location	Member/Location	
Job Title: Aramis Tunnel FEED Design	Drg. Ref.	Drawing reference	
Calculation: Face Pressure DAUB 2016 - Jancsecz & Steiner, 1994 - Arching	Made by [redacted]	Date 16/05/2024	Chd. CH

(7) Limit equilibrium method (granular material) [Jancsecz & Steiner, 1994]

Additional Input paremeters

Unit weight of support fluid (γs)	12.0	**kN/m^3**
Unit weight of water (γw)	9.8	**kN/m^3**
Operation Range	0.2	**bar**
Surcharge on surface q	0.0	**kPa**
Suport pressure deviation (Based on the DAUB raccomendation)	10.0	**kPa**
Earth Pressure Safety Factor, η_E	1.5	
Water Pressure Safety Factor, η_W	1.05	
Coefficient of the lateral earth pressure at the lateral planes of the silo k1, (DAUB raccomendation)	0.8	-

Tunnel				**Longitudinal Profile**														
Section No.	**Chainage**	**Internal Tunnel Diameter**	**Excavation Tunnel Diameter**	**Grond Elevation**	**Water Elevation**	**Tunnel Crown Elevation**	**Tunnel Sping line elevation**	**Ground surface to ground water table (artisan)**	**Ground surface to tunnel crown (t)**	**Ground Surface to Tunnel Springline**	**Design groundwater level to tunnel crown**	**Design groundwater level to tunnel springline**	**Formation at tunnel location**	**γ_{sat} (avg)**	**ϕ'**	**c'**	**Ka**	**K_0**
-	m	m	m	m (LAT)	m (LAT)	m (LAT)	m (LAT)	m	m	m	m	m	-	kN/m^3	degree	kPa	-	-
A1	0+18	3.00	3.78	6.1	4.0	-9.6	-11.5	**2.10**	**15.71**	**17.60**	**13.61**	**15.50**	NLD - Silty Sand LB	17.9	28.0	0.0	0.4	0.53
B1	0+178	3.00	3.78	15.5	4.0	-17.6	-19.5	**11.50**	**33.11**	**35.00**	**21.61**	**23.50**	NLD - Silty Sand LM	18.9	30.0	0.0	0.3	0.52
C1	0+288	3.00	3.78	-10.0	4.0	-23.1	-25.0	**-14.00**	**13.11**	**15.00**	**27.11**	**29.00**	KR - Silty Sand LM	17.2	30.0	0.0	0.3	0.52
D1	0+500	3.00	3.78	-16.3	4.0	-33.6	-35.5	**-20.30**	**17.31**	**19.20**	**37.61**	**39.50**	KR - Silty Sand LB	18.0	28.0	0.0	0.4	0.53
E1	01+257	3.00	3.78	-24.0	4.0	-37.7	-39.6	**-28.00**	**13.71**	**15.60**	**41.71**	**43.60**	KR - Silty Sand LM	19.0	30.0	0.0	0.3	0.52
F1	01+700	3.20	3.78	-17.5	4.0	-28.1	-30.0	**-21.50**	**10.61**	**12.50**	**32.11**	**34.00**	KR - Silty Sand LM	19.0	30.0	0.0	0.3	0.52
A2	0+18	3.20	3.78	6.1	0.0	-9.6	-11.5	**6.10**	**15.71**	**17.60**	**9.61**	**11.50**	NLD - Silty Sand LB	17.9	28.0	0.0	0.4	0.53
B2	0+178	3.20	3.78	15.5	0.0	-17.6	-19.5	**15.50**	**33.11**	**35.00**	**17.61**	**19.50**	NLD - Silty Sand LM	18.9	30.0	0.0	0.3	0.52
C2	0+288	3.20	3.78	-10.0	0.0	-23.1	-25.0	**-10.00**	**13.11**	**15.00**	**23.11**	**25.00**	KR - Silty Sand LM	17.2	30.0	0.0	0.3	0.52
D2	0+500	3.20	3.78	-16.3	0.0	-33.6	-35.5	**-16.30**	**17.31**	**19.20**	**33.61**	**35.50**	KR - Silty Sand LB	18.0	28.0	0.0	0.4	0.53
E2	01+257	3.50	3.78	-24.0	0.0	-37.7	-39.6	**-24.00**	**13.71**	**15.60**	**37.71**	**39.60**	KR - Silty Sand LM	19.0	30.0	0.0	0.3	0.52
F2	01+700	3.50	3.78	-17.5	0.0	-28.1	-30.0	**-17.50**	**10.61**	**12.50**	**28.11**	**30.00**	KR - Silty Sand LM	19.0	30.0	0.0	0.3	0.52

ARUP		Job No.	Sheet No.	Rev.
		603258-53	5	1
Job Title	Aramis Tunnel FEED Design	Member/Location	Member/Location	
Calculation	Face Pressure DAUB 2016 - Jancsecz & Steiner, 1994 - Arching	Drg. Ref.	Drawing reference	
		Made by	Date 16/05/2024	Chd. CH

$$\sigma_v(z) = \frac{\frac{A}{U}\cdot\gamma_{1,av} - c'_1}{K_1\cdot\tan(\varphi'_1)}\cdot\left(1 - e^{-\frac{U}{A}\cdot K_1\cdot z\cdot\tan(\varphi_1)}\right) + \sigma_s\cdot e^{-\frac{U}{A}\cdot K_1\cdot z\cdot\tan(\varphi'_1)} \qquad (12)$$

for $t_{crown} > 2\cdot D$

$$T_{R,2} = \underset{x1}{tan(\varphi'_2)}\cdot K_2\cdot\left(\frac{\overset{x2}{D^2\cdot\sigma_v(t)}}{\underset{x3}{2\cdot tan(\vartheta_{crit})}} + \frac{\overset{x4}{D^3\cdot\gamma_2}}{\underset{x5}{6\cdot tan(\vartheta_{crit})}}\right)$$

									DAUB (Drained)								
Chainage	Arching/ No Arching	Total Vertical Pressure at Crown, No Arching,σv	Efective Vertical Pressure at Crown, No Arching, σv,c'	k2, Coefficient of the lateral earth pressure acting on the vertical triangular planes of the wedge (DAUB raccomendation)	Critical Failure Angle at TBM Face, θcrit	Cross-Sectional Area of the Silo, A	Circumference of the Silo, U	Total Vertical Pressure at Crown. Arching, σv	x1	x2	x3	x4	x5	Shear Resistance due to Friction, Tr	Shear Resistance due to Cohesion, Tc	Total Shear Resistance, T	Self-Weight of the Wedge, G
m	-	kPa	kPa	-	deg	m2	m	kPa	-	-	-	-	-	kN	kN	kN	kN
0+18	Arching	280.78	147.24	0.45	60.47	8.07	11.83	12.93	0.24	184.32	3.53	433.59	10.59	22.1	0.0	22.1	122.8
0+178	Arching	626.35	414.33	0.43	61.93	7.60	11.58	12.94	0.25	184.43	3.75	489.86	11.25	22.8	0.0	22.8	130.6
0+288	Arching	362.84	96.87	0.43	61.93	7.60	11.58	10.50	0.25	149.66	3.75	397.41	11.25	18.5	0.0	18.5	106.0
0+500	Arching	510.72	141.74	0.45	60.47	8.07	11.83	13.14	0.24	187.23	3.53	440.45	10.59	22.4	0.0	22.4	124.8
01+257	Arching	535.17	125.97	0.43	61.93	7.60	11.58	13.06	0.25	186.09	3.75	494.20	11.25	23.0	0.0	23.0	131.8
01+700	Arching	412.32	97.29	0.43	61.91	7.60	11.58	13.04	0.25	185.87	3.75	493.18	11.24	23.0	0.0	23.0	131.6
0+18	Arching	280.78	186.48	0.45	60.47	8.07	11.83	12.93	0.24	184.32	3.53	433.59	10.59	22.1	0.0	22.1	122.8
0+178	Arching	626.35	453.57	0.43	61.93	7.60	11.58	12.94	0.25	184.43	3.75	489.86	11.25	22.8	0.0	22.8	130.6
0+288	Arching	323.60	96.87	0.43	61.93	7.60	11.58	10.50	0.25	149.66	3.75	397.41	11.25	18.5	0.0	18.5	106.0
0+500	Arching	471.48	141.74	0.45	60.47	8.07	11.83	13.14	0.24	187.23	3.53	440.45	10.59	22.4	0.0	22.4	124.8
01+257	Arching	495.93	125.97	0.43	61.93	7.60	11.58	13.06	0.25	186.09	3.75	494.20	11.25	23.0	0.0	23.0	131.8
01+700	Arching	373.08	97.29	0.43	61.91	7.60	11.58	13.04	0.25	185.87	3.75	493.18	11.24	23.0	0.0	23.0	131.6

ARUP	Job No.	Sheet No.	Rev.
	603258-53	6	1
	Member/Location	Member/Location	
Job Title: Aramis Tunnel FEED Design	Drg. Ref.	Drawing reference	
Calculation: Face Pressure DAUB 2016 - Jancsecz & Steiner, 1994 - Arching	Made by [redacted]	Date: 16/05/2024	Chd.: CH

0

$$E_{re}(\vartheta) = \frac{(G + P_V) \cdot (\sin(\vartheta) - \cos(\vartheta) \cdot \tan(\varphi'_2)) - 2 \cdot T - c'_2 \cdot \frac{D^2}{\sin(\vartheta)}}{\sin(\vartheta) \cdot \tan(\varphi'_2) + \cos(\vartheta)}$$

Destab. forces — *Shear resistance*

Lower support.: $S_{ci} = \eta_E \cdot E_{max,ci} + \eta_W \cdot W_{ci}$ — *Earth pressure* — *Ground water pressure*

Upper support.: $1 \leq \frac{0.9 \cdot \sigma_{v,crown,min}}{s_{crown,max}}$ — *To be <90 % of the total vertical stress at the tunnel crown.*

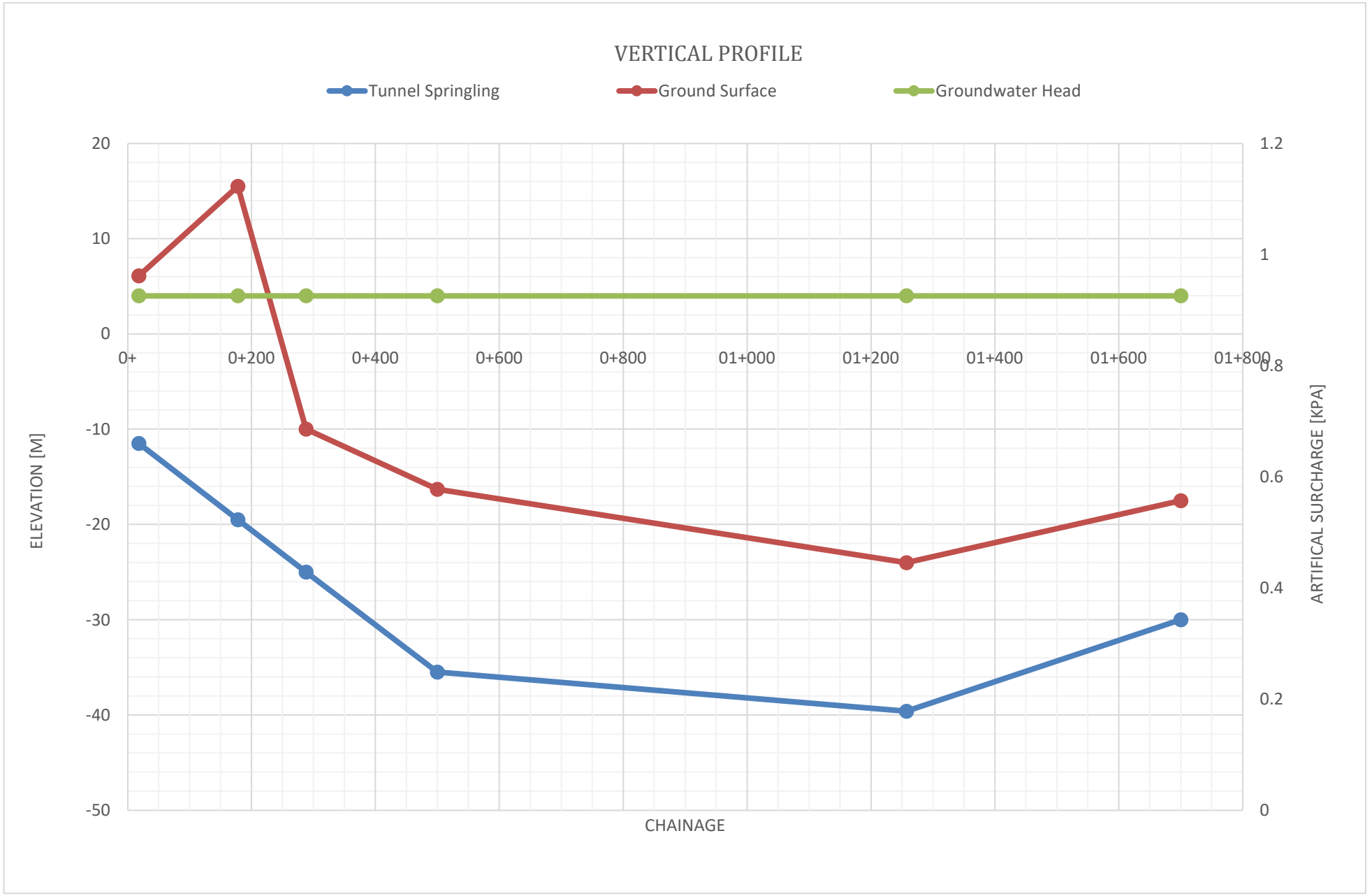
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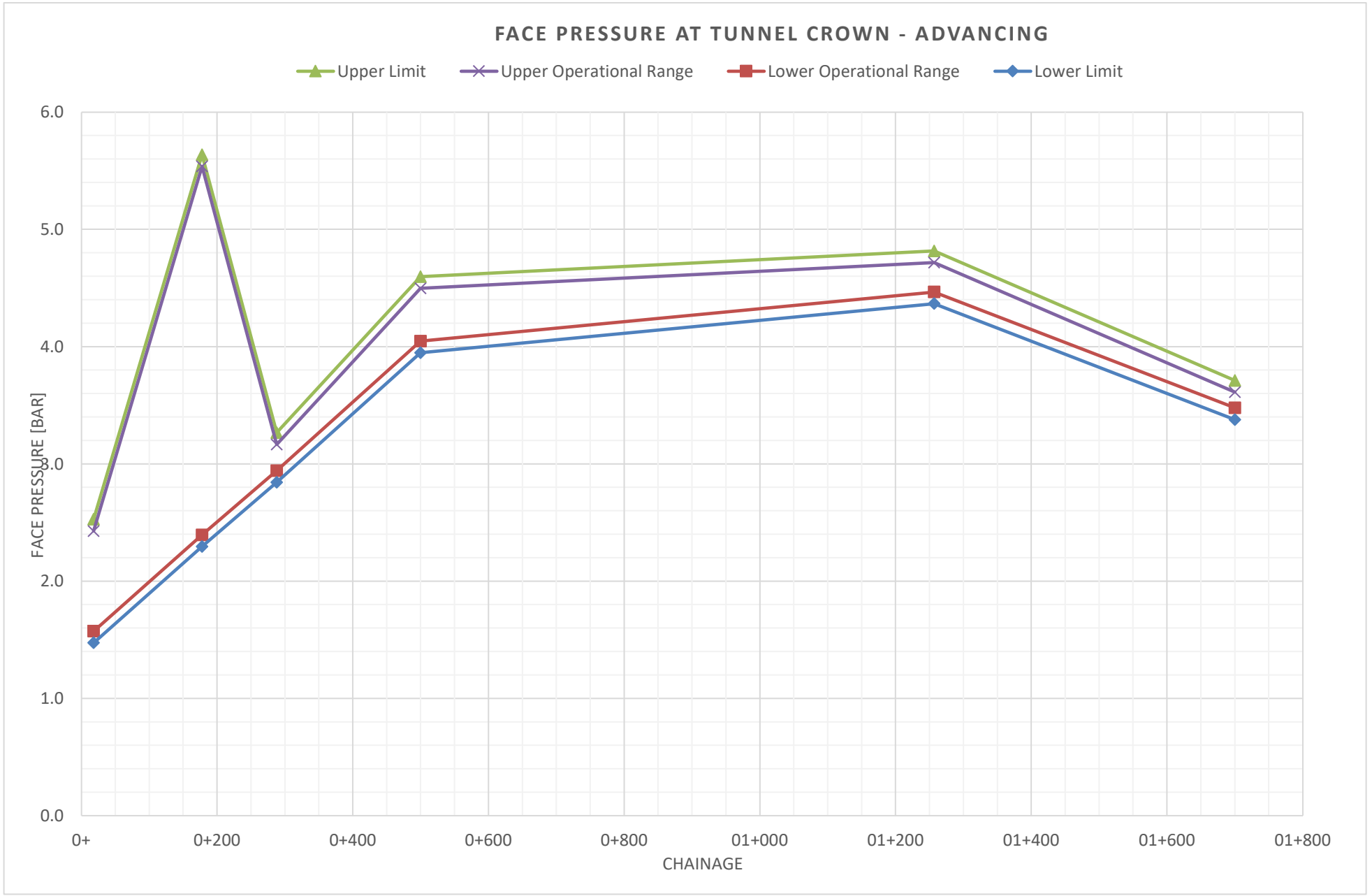
											Summary				Crown check
Chainage	Vertical Load from the Silo on the Wedge, Pv	Support Load due to Earth Pressure, Ere	Support Load for a Circular Face, Ere ci	Acting Water Load, Rectangular Face, Wre	Acting Water Load, Circular Face, Wre ci	Total Support Force, Factored, S_{Ci}	Minimum Support Pressure at Tunnel Crown, $s_{crown,min}$	Minimum Support Pressure at Tunnel Crown for Regular Advance, $s_{cr,adv,min}$	Maximum Allowable Pressure at Tunnel Crown due to Break-Up Safety, $s_{crown,max}$	Maximum Support Pressure at Tunnel Crown for Regular Advance, $s_{cr,adv,max}$	Minimum Support Pressure at Tunnel Crown, $s_{crown,min}$	Minimum Support Pressure at Tunnel Crown for Regular Advance, $s_{cr,adv,min}$	Hydro Static pressure at Springline	Check	Minimum Support Pressure at Tunnel Crown, scrown,min
m	kN	kN	kN	kN	kN	kN	bar	bar	bar	bar	bar	bar	bar		Drained
0+18	104.4	98.4	77.3	2166.9	1701.9	1902.88	1.47	1.57	2.53	2.43	1.47	1.57	1.52	Excavation is possible	1.47
0+178	98.4	96.1	75.5	3285.3	2580.2	2822.44	2.30	2.40	5.64	5.54	2.30	2.40	2.31	Excavation is possible	2.30
0+288	79.8	77.9	61.2	4054.2	3184.1	3435.17	2.84	2.94	3.27	3.17	2.84	2.94	2.84	Excavation is possible	2.84
0+500	106.1	100.0	78.5	5522.0	4337.0	4671.62	3.95	4.05	4.60	4.50	3.95	4.05	3.87	Excavation is possible	3.95
01+257	99.2	96.9	76.1	6095.2	4787.2	5140.73	4.37	4.47	4.82	4.72	4.37	4.47	4.28	Excavation is possible	4.37
01+700	99.2	96.7	76.0	4753.2	3733.1	4033.75	3.38	3.48	3.71	3.61	3.38	3.48	3.34	Overburden has been reduced	3.38
0+18	104.4	98.4	77.3	1607.7	1262.7	1441.73	1.06	1.16	2.53	2.43	1.06	1.16	1.13	Excavation is possible	1.06
0+178	98.4	96.1	75.5	2726.1	2141.1	2361.29	1.88	1.98	5.64	5.54	1.88	1.98	1.91	Excavation is possible	1.88
0+288	79.8	77.9	61.2	3495.0	2744.9	2974.02	2.43	2.53	2.91	2.81	2.43	2.53	2.45	Excavation is possible	2.43
0+500	106.1	100.0	78.5	4962.9	3897.8	4210.47	3.54	3.64	4.24	4.14	3.54	3.64	3.48	Excavation is possible	3.54
01+257	99.2	96.9	76.1	5536.0	4348.0	4679.58	3.95	4.05	4.46	4.36	3.95	4.05	3.88	Excavation is possible	3.95
01+700	99.2	96.7	76.0	4194.0	3293.9	3572.60	2.97	3.07	3.36	3.26	2.97	3.07	2.94	Overburden has been reduced	2.97

ARUP		Job No.	Sheet No.	Rev.
		603258-53	7	1
		Member/Location	Member/Location	
Job Title	Aramis Tunnel FEED Design	Drg. Ref.	Drawing reference	
Calculation	Face Pressure DAUB 2016 - Jancsecz & Steiner, 1994 - Arching	Made by	[redacted] Date 16/05/2024	Chd. CH

(8) Summary face Pressures for all sections and both water levels

								Draw Down			
Critical Sections	**Chainage**	**Water Level**	**Check**	**Minimum Support Pressure at Tunnel Crown for Regular Advance, scr,adv,min**	**Maximum Support Pressure at Tunnel Crown for Regular Advance, scr,adv,max**	**Operation Range**	**Ground Surface to Crown,(t)**	**Minimum Support Pressure at Tunnel Crown, scrown,min**	**1/3-Drawdown**	**1/2-Drawdown**	**full Drawdown**
		mLAT		**bar**	**bar**	**bar**	**m**	Drained	Max + half front material	Max + half front material	Max + half front material
A1	+18.00	4.0	Excavation is possible	1.57	2.43	0.85	15.71	1.57	1.72	1.80	2.03
B1	+178.00	4.0	Excavation is possible	2.40	5.54	3.14	33.11	2.40	2.55	2.62	2.85
C1	+288.00	4.0	Excavation is possible	2.94	3.17	0.22	13.11	2.94	3.09	3.17	3.40
D1	+500.00	4.0	Excavation is possible	4.05	4.50	0.45	17.31	4.05	4.20	4.27	4.50
E1	1+257.00	4.0	Excavation is possible	4.47	4.72	0.25	13.71	4.47	4.62	4.69	4.92
F1	1+700.00	4.0	Overburden has been reduced	3.48	3.61	0.13	10.61	3.48	3.63	3.70	3.93
A2	+18.00	0.0	Excavation is possible	1.16	2.43	1.27	15.71	1.16	1.31	1.39	1.61
B2	+178.00	0.0	Excavation is possible	1.98	5.54	3.55	33.11	1.98	2.13	2.21	2.44
C2	+288.00	0.0	Excavation is possible	2.53	2.81	0.28	13.11	2.53	2.68	2.76	2.98
D2	+500.00	0.0	Excavation is possible	3.64	4.14	0.51	17.31	3.64	3.79	3.86	4.09
E2	1+257.00	0.0	Excavation is possible	4.05	4.36	0.31	13.71	4.05	4.21	4.28	4.51
F2	1+700.00	0.0	Overburden has been reduced	3.07	3.26	0.19	10.61	3.07	3.22	3.29	3.52


VERTICAL PROFILE
Tunnel Springling
Ground Surface
Groundwater Head
20
10
0
-10
-20
-30
-40
-50
0+
0+200
0+400
0+600
0+800
01+000
01+200
01+400
01+600
01+800
1.2
1
0.8
0.6
0.4
0.2
0
ELEVATION [M]
ARTIFICAL SURCHARGE [KPA]
CHAINAGE


FACE PRESSURE AT TUNNEL CROWN - ADVANCING
Upper Limit
Upper Operational Range
Lower Operational Range
Lower Limit
FACE PRESSURE [BAR]
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0+
0+200
0+400
0+600
0+800
01+000
01+200
01+400
01+600
01+800
CHAINAGE

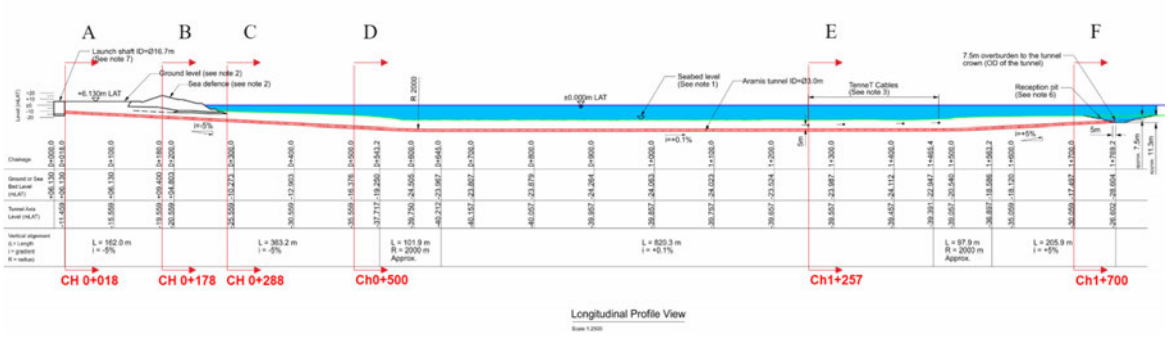
ARUP	Job No.	Sheet No.	Rev.
	603258-53	1	1
	Member/Location		
Job Title: Aramis Tunnel FEED Design	Drg. Ref.		
Calculation: Face Pressure DAUB 2016 - Jancsecz & Steiner, 1994 - Arching	Made by [redacted]	Date 16/05/2024	Chd. CH

(1) Purpose of spreadsheet

This spreadhseet aims to offer a calculation tool for the tunnel face stability assessment in mechanized tunneelling and to estimated the operation range of the face pressure.

(2) Problem Description

Preliminary assesment of the face pressure for Aramis Tunnel, identifying the minimum cover.



(3) Assumptions

- Slurry machine- no water inside the tunnel/excavation chamber
- Soil strength parameters for granular grounds
- Min and Max pressure estimated with the average weight unit weight of the stratigraphy column
- Arching phenomena contribution when Depth > 2D tunnel diameters - sensitivity analysis considered
- Effective ground cohesion has been neglected
- Critical angle estimated with an iteration process
- Face pressure deviation ±10kpa, granular ground
- Max pressure > Min pressure +0.2bar (operation range 20kPa)
- Water level has been considered at +4mLAT and 0mLAT (Upper and lower levels)

ARUP	Job No.	Sheet No.	Rev.
	603258-53	2	1
	Member/Location		
Job Title: Aramis Tunnel FEED Design	Drg. Ref.		
Calculation: Face Pressure DAUB 2016 - Jancsecz & Steiner, 1994 - Arching	Made by [redacted]	Date 16/05/2024	Chd. CH

(5) Tunnel Aligment

Section No.	Chainage	Internal Tunnel Diameter	Excavation Tunnel Diameter	Grond Elevation	Water Elevation	Tunnel Crown Elevation	Tunnel Sping line elevation
-	m	m	m	m (LAT)	m (LAT)	m (LAT)	m (LAT)
A1	0+18.00	3.00	3.775	6.1	4.0	-9.6	-11.5
B1	0+178.00	3.00	3.775	15.5	4.0	-17.6	-19.5
C1	0+288.00	3.00	3.775	-10.0	4.0	-23.1	-25.0
D1	0+500.00	3.00	3.775	-16.3	4.0	-33.6	-35.5
E1	1+257.00	3.00	3.775	-24.0	4.0	-37.7	-39.6
F1	1+700.00	3.20	3.775	-17.5	4.0	-28.1	-30.0
A2	0+18.00	3.20	3.775	6.1	0.0	-9.6	-11.5
B2	0+178.00	3.20	3.775	15.5	0.0	-17.6	-19.5
C2	0+288.00	3.20	3.775	-10.0	0.0	-23.1	-25.0
D2	0+500.00	3.20	3.775	-16.3	0.0	-33.6	-35.5
E2	1+257.00	3.50	3.775	-24.0	0.0	-37.7	-39.6
F2	1+700.00	3.50	3.775	-17.5	0.0	-28.1	-30.0

ARUP	Job No.	Sheet No.	Rev.
	603258-53	3	1
	Member/Location	Member/Location	
Job Title: Aramis Tunnel FEED Design	Drg. Ref.	Drawing reference	
Calculation: Face Pressure DAUB 2016 - Jancsecz & Steiner, 1994 - Arching	Made by [redacted]	Date 16/05/2024	Chd. CH

(6) Geotechnical design parameters Upper

Secton A	Description	Material behavior	Thickness	Satured unit weight	Effective Friction Angle	Effective Cohesion	Undrained Shear Strength	K_{OCR}
			m	kN/m3	degree	kPa	kPa	
Made Ground (MGR)	MGR - Sand LB	Non-cohesive	15.7	20.0	38.0	0.0	0.0	0.67
Naaldwijk Formation (NLD)	NLD - Sand UB	Non-cohesive	3.0	20.0	42.0	0.0	0.0	0.65
Naaldwijk Formation (NLD)	NLD - Sand UM	Non-cohesive	-	20.0	38.0	0.0	0.0	0.67

Secton B	Description	Material behavior	Thickness to tunnel spring line	Satured unit weight	Effective Friction Angle	Effective Cohesion	Undrained Shear Strength	K_{OCR}
			m	kN/m3	degree	kPa	kPa	
Made Ground (MGR)	MGR - Sand LM	Non-cohesive	33.1	20.0	38.0	0.0	0.0	0.67
Naaldwijk Formation (NLD)	NLD - Sand UB	Non-cohesive	3.0	20.0	42.0	0.0	0.0	0.65
Naaldwijk Formation (NLD)	NLD - Sand UM	Non-cohesive	-	20.0	38.0	0.0	0.0	0.67

Secton C	Description	Material behavior	Thickness to tunnel spring line	Satured unit weight	Effective Friction Angle	Effective Cohesion	Undrained Shear Strength	K_{OCR}
			m	kN/m3	degree	kPa	kPa	
Kreftenheye Formation (KR)	KR - Silty Sand UM	Non-cohesive	13.1	20.0	37.0	0.0	0.0	0.67
Kreftenheye Formation (KR)	KR - Sand UB	Non-cohesive	3.0	22.0	43.0	0.0	0.0	0.65
Kreftenheye Formation (KR)	KR - Sand UM	Non-cohesive	-	21.0	39.0	0.0	0.0	0.67

Secton D	Description	Material behavior	Thickness to tunnel spring line	Satured unit weight	Effective Friction Angle	Effective Cohesion	Undrained Shear Strength	K_{OCR}
			m	kN/m3	degree	kPa	kPa	
Kreftenheye Formation (KR)	KR - Sand UM	Non-cohesive	17.3	21.0	39.0	0.0	0.0	0.67
Kreftenheye Formation (KR)	KR - Sand UB	Non-cohesive	3.0	22.0	43.0	0.0	0.0	0.65
Kreftenheye Formation (KR)	KR - Sand UM	Non-cohesive	-	21.0	39.0	0.0	0.0	0.67

Secton E	Description	Material behavior	Thickness to tunnel spring line	Satured unit weight	Effective Friction Angle	Effective Cohesion	Undrained Shear Strength	K_{OCR}
			m	kN/m3	degree	kPa	kPa	
Kreftenheye Formation (KR)	KR - Sand UM	Non-cohesive	13.7	21.0	39.0	0.0	0.0	0.67
Kreftenheye Formation (KR)	KR - Sand UB	Non-cohesive	3.0	22.0	43.0	0.0	0.0	0.65
Kreftenheye Formation (KR)	KR - Sand UM	Non-cohesive	-	21.0	39.0	0.0	0.0	0.67

Secton f	Description	Material behavior	Thickness to tunnel spring line	Satured unit weight	Effective Friction Angle	Effective Cohesion	Undrained Shear Strength	K_{OCR}
			m	kN/m3	degree	kPa	kPa	
Kreftenheye Formation (KR)	KR - Sand UM	Non-cohesive	10.6	21.0	39.0	0.0	0.0	0.67
Kreftenheye Formation (KR)	KR - Sand UB	Non-cohesive	3.0	22.0	43.0	0.0	0.0	0.65
Kreftenheye Formation (KR)	KR - Sand UM	Non-cohesive	-	21.0	39.0	0.0	0.0	0.67

ARUP	Job No.	Sheet No.	Rev.
	603258-53	4	1
	Member/Location	Member/Location	
Job Title: Aramis Tunnel FEED Design	Drg. Ref.	Drawing reference	
Calculation: Face Pressure DAUB 2016 - Jancsecz & Steiner, 1994 - Arching	Made by [redacted]	Date 16/05/2024	Chd. CH

(7) Limit equilibrium method (granular material) [Jancsecz & Steiner, 1994]

Additional Input paremeters

Unit weight of support fluid (γs)	12.0	**kN/m^3**
Unit weight of water (γw)	9.8	**kN/m^3**
Operation Range	0.2	**bar**
Surcharge on surface q	0.0	**kPa**
Suport pressure deviation (Based on the DAUB raccomendation)	10.0	**kPa**
Earth Pressure Safety Factor, η_E	1.5	
Water Pressure Safety Factor, η_W	1.05	
Coefficient of the lateral earth pressure at the lateral planes of the silo k1, (DAUB raccomendation)	0.80	-

Tunnel				Longitudinal Profile															
Section No.	**Chainage**	**Internal Tunnel Diameter**	**Excavation Tunnel Diameter**	**Grond Elevation**	**Water Elevation**	**Tunnel Crown Elevation**	**Tunnel Sping line elevation**	**Ground surface to ground water table (artisan)**	**Ground surface to tunnel crown (t)**	**Ground Surface to Tunnel Springline**	**Design groundwater level to tunnel crown**	**Design groundwater level to tunnel springline**	**Formation at tunnel location**	**γ_{sat} (avg above crown)**	**ϕ'**	**c'**	**Ka**	**K_0**
-	m	m	m	m (LAT)	m (LAT)	m (LAT)	m (LAT)	m	m	m	m	m	-	kN/m^3	degree	kPa	-	-
A1	0+18	3.00	3.78	6.1	4.0	-9.6	-11.5	**2.10**	**15.71**	**17.60**	**13.61**	**15.50**	NLD - Sand UB	20.0	42.0	0.0	0.2	0.65
B1	0+178	3.00	3.78	15.5	4.0	-17.6	-19.5	**11.50**	**33.11**	**35.00**	**21.61**	**23.50**	NLD - Sand UB	20.0	42.0	0.0	0.2	0.65
C1	0+288	3.00	3.78	-10.0	4.0	-23.1	-25.0	**-14.00**	**13.11**	**15.00**	**27.11**	**29.00**	KR - Sand UB	20.0	43.0	0.0	0.2	0.65
D1	0+500	3.00	3.78	-16.3	4.0	-33.6	-35.5	**-20.30**	**17.31**	**19.20**	**37.61**	**39.50**	KR - Sand UB	21.0	43.0	0.0	0.2	0.65
E1	01+257	3.00	3.78	-24.0	4.0	-37.7	-39.6	**-28.00**	**13.71**	**15.60**	**41.71**	**43.60**	KR - Sand UB	21.0	43.0	0.0	0.2	0.65
F1	01+700	3.20	3.78	-17.5	4.0	-28.1	-30.0	**-21.50**	**10.61**	**12.50**	**32.11**	**34.00**	KR - Sand UB	21.0	43.0	0.0	0.2	0.65
A2	0+18	3.20	3.78	6.1	0.0	-9.6	-11.5	**6.10**	**15.71**	**17.60**	**9.61**	**11.50**	NLD - Sand UB	20.0	42.0	0.0	0.2	0.65
B2	0+178	3.20	3.78	15.5	0.0	-17.6	-19.5	**15.50**	**33.11**	**35.00**	**17.61**	**19.50**	NLD - Sand UB	20.0	42.0	0.0	0.2	0.65
C2	0+288	3.20	3.78	-10.0	0.0	-23.1	-25.0	**-10.00**	**13.11**	**15.00**	**23.11**	**25.00**	KR - Sand UB	20.0	43.0	0.0	0.2	0.65
D2	0+500	3.20	3.78	-16.3	0.0	-33.6	-35.5	**-16.30**	**17.31**	**19.20**	**33.61**	**35.50**	KR - Sand UB	21.0	43.0	0.0	0.2	0.65
E2	01+257	3.50	3.78	-24.0	0.0	-37.7	-39.6	**-24.00**	**13.71**	**15.60**	**37.71**	**39.60**	KR - Sand UB	21.0	43.0	0.0	0.2	0.65
F2	01+700	3.50	3.78	-17.5	0.0	-28.1	-30.0	**-17.50**	**10.61**	**12.50**	**28.11**	**30.00**	KR - Sand UB	21.0	43.0	0.0	0.2	0.65

ARUP	Job No. 603258-53	Sheet No. 5	Rev. 1
Job Title: Aramis Tunnel FEED Design	Member/Location: Member/Location		
Calculation: Face Pressure DAUB 2016 - Jancsecz & Steiner, 1994 - Arching	Drg. Ref.: Drawing reference		
	Made by: [redacted]	Date: 16/05/2024	Chd.: CH

$$\sigma_v(z) = \frac{\frac{A}{U}\cdot\gamma_{1,av} - c'_1}{K_1\cdot\tan(\varphi'_1)}\cdot\left(1 - e^{-\frac{U}{A}\cdot K_1\cdot z\cdot\tan(\varphi_1)}\right) + \sigma_s\cdot e^{-\frac{U}{A}\cdot K_1\cdot z\cdot\tan(\varphi'_1)} \qquad (12)$$

for $t_{crown} > 2\cdot D$

$$T_{R,2} = \tan(\varphi_2')^{x1}\cdot K_2\cdot\left(\frac{D^{2\,(x2)}\cdot\sigma_v(t)}{2\cdot\tan(\vartheta_{crit})^{(x3)}} + \frac{D^{3\,(x4)}\cdot\gamma_2}{6\cdot\tan(\vartheta_{crit})^{(x5)}}\right)$$

	DAUB (Drained)																
Chainage	Arching/ No Arching	Total Vertical Pressure at Crown, No Arching,σv	Efective Vertical Pressure at Crown, No Arching, σv,c'	k2, Coefficient of the lateral earth pressure acting on the vertical triangular planes of the wedge (DAUB raccomendation)	Critical Failure Angle at TBM Face, θcrit	Cross-Sectional Area of the Silo, A	Circumference of the Silo, U	Total Vertical Pressure at Crown. Arching, σv	x1	x2	x3	x4	x5	Shear Resistance due to Friction, Tr	Shear Resistance due to Cohesion, Tc	Total Shear Resistance, T	Self-Weight of the Wedge, G
m	-	kPa	kPa	-	deg	m2	m	kPa	-	-	-	-	-	kN	kN	kN	kN
0+18	Arching	314.20	180.66	0.42	71.18	4.86	10.12	6.79	0.38	96.70	5.87	548.01	17.60	18.2	0.0	18.2	93.4
0+178	Arching	662.20	450.18	0.42	71.18	4.86	10.12	6.79	0.38	96.71	5.87	548.10	17.60	18.2	0.0	18.2	93.4
0+288	Arching	399.54	133.57	0.42	71.79	4.69	10.03	6.38	0.39	90.92	6.08	547.98	18.24	17.6	0.0	17.6	90.1
0+500	Arching	562.65	193.67	0.42	71.79	4.69	10.03	7.01	0.39	99.85	6.08	601.82	18.24	19.3	0.0	19.3	99.0
01+257	Arching	562.59	153.39	0.42	71.79	4.69	10.03	7.01	0.39	99.84	6.08	601.77	18.24	19.3	0.0	19.3	99.0
01+700	Arching	433.52	118.49	0.42	71.79	4.69	10.03	6.99	0.39	99.66	6.08	600.65	18.24	19.3	0.0	19.3	98.8
0+18	Arching	314.20	219.90	0.42	71.18	4.86	10.12	6.79	0.38	96.70	5.87	548.01	17.60	18.2	0.0	18.2	93.4
0+178	Arching	662.20	489.42	0.42	71.18	4.86	10.12	6.79	0.38	96.71	5.87	548.10	17.60	18.2	0.0	18.2	93.4
0+288	Arching	360.30	133.57	0.42	71.79	4.69	10.03	6.38	0.39	90.92	6.08	547.98	18.24	17.6	0.0	17.6	90.1
0+500	Arching	523.41	193.67	0.42	71.79	4.69	10.03	7.01	0.39	99.85	6.08	601.82	18.24	19.3	0.0	19.3	99.0
01+257	Arching	523.35	153.39	0.42	71.79	4.69	10.03	7.01	0.39	99.84	6.08	601.77	18.24	19.3	0.0	19.3	99.0
01+700	Arching	394.28	118.49	0.42	71.79	4.69	10.03	6.99	0.39	99.66	6.08	600.65	18.24	19.3	0.0	19.3	98.8

ARUP		Job No.	Sheet No.	Rev.
		603258-53	6	1
		Member/Location	Member/Location	
Job Title	Aramis Tunnel FEED Design	Drg. Ref.	Drawing reference	
Calculation	Face Pressure DAUB 2016 - Jancsecz & Steiner, 1994 - Arching	Made by	[redacted] Date 16/05/2024	Chd. CH

0

$$E_{re}(\vartheta) = \frac{(G + P_V) \cdot (\sin(\vartheta) - \cos(\vartheta) \cdot \tan(\varphi'_2)) - 2 \cdot T - c'_2 \cdot \frac{D^2}{\sin(\vartheta)}}{\sin(\vartheta) \cdot \tan(\varphi'_2) + \cos(\vartheta)}$$

Destab. forces *Shear resistance*

Lower support.: $S_{ci} = \eta_E \cdot E_{max,ci} + \eta_W \cdot W_{ci}$

Earth pressure *Ground water pressure*

Upper support.: $1 \leq \frac{0.9 \cdot \sigma_{v,crown,min}}{s_{crown,max}}$

To be <90 % of the total vertical stress at the tunnel crown.

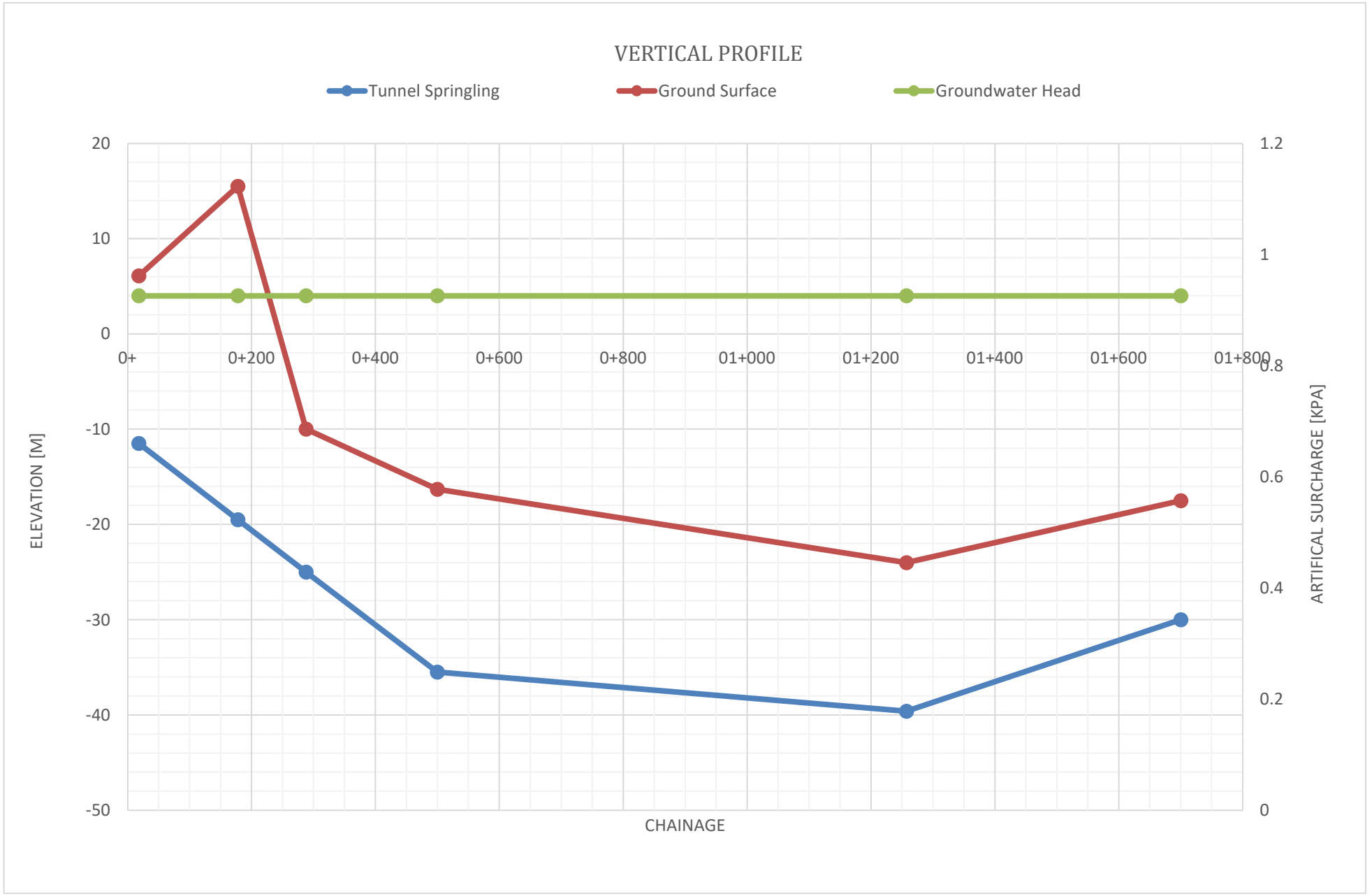
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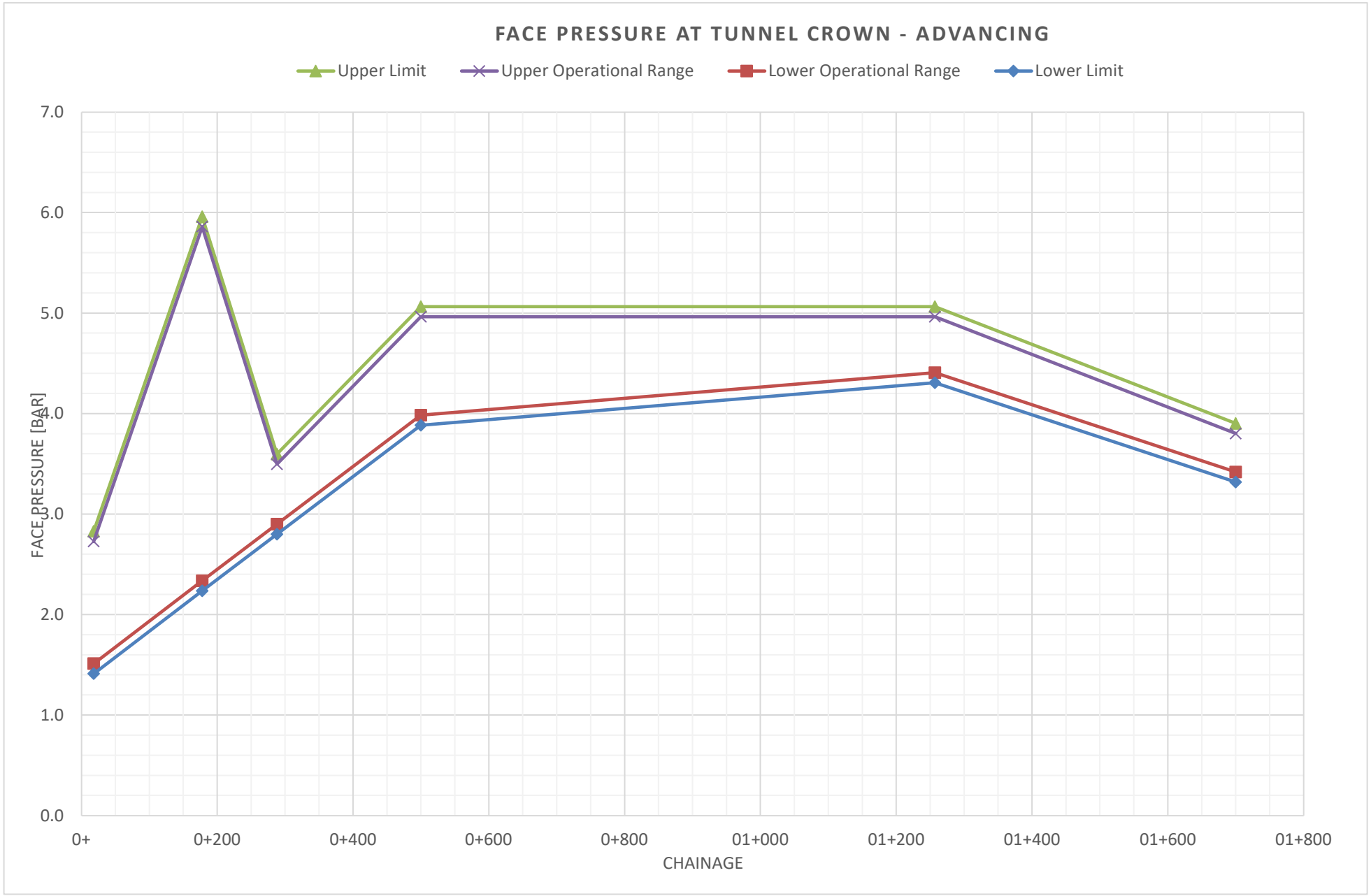
											Summary				Crown check
Chainage	Vertical Load from the Silo on the Wedge, Pv	Support Load due to Earth Pressure, Ere	Support Load for a Circular Face, Ere ci	Acting Water Load, Rectangular Face, Wre	Acting Water Load, Circular Face, Wre ci	Total Support Force, Factored, S_{Ci}	Minimum Support Pressure at Tunnel Crown, $s_{crown,min}$	Minimum Support Pressure at Tunnel Crown for Regular Advance, $s_{cr,adv,min}$	Maximum Allowable Pressure at Tunnel Crown due to Break-Up Safety, $s_{crown,max}$	Maximum Support Pressure at Tunnel Crown for Regular Advance, $s_{cr,adv,max}$	Minimum Support Pressure at Tunnel Crown, $s_{crown,min}$	Minimum Support Pressure at Tunnel Crown for Regular Advance, $s_{cr,adv,min}$	Hydro Static pressure at Springline	Check	Minimum Support Pressure at Tunnel Crown, scrown,min
m	kN	kN	kN	kN	kN	kN	bar	bar	bar	bar	bar	bar	bar		Drained
0+18	33.0	39.6	31.1	2166.9	1701.9	1833.61	1.41	1.51	2.83	2.73	1.41	1.51	1.52	Excavation is possible	1.41
0+178	33.0	39.6	31.1	3285.3	2580.2	2755.92	2.24	2.34	5.96	5.86	2.24	2.34	2.31	Excavation is possible	2.24
0+288	29.9	36.6	28.7	4054.2	3184.1	3386.44	2.80	2.90	3.60	3.50	2.80	2.90	2.84	Excavation is possible	2.80
0+500	32.8	40.2	31.6	5522.0	4337.0	4601.19	3.88	3.98	5.06	4.96	3.88	3.98	3.87	Excavation is possible	3.88
01+257	32.8	40.2	31.6	6095.2	4787.2	5073.87	4.31	4.41	5.06	4.96	4.31	4.41	4.28	Excavation is possible	4.31
01+700	32.8	40.1	31.5	4753.2	3733.1	3967.02	3.32	3.42	3.90	3.80	3.32	3.42	3.34	Excavation is possible	3.32
0+18	33.0	39.6	31.1	1607.7	1262.7	1372.46	1.00	1.10	2.83	2.73	1.00	1.10	1.13	Excavation is possible	1.00
0+178	33.0	39.6	31.1	2726.1	2141.1	2294.77	1.82	1.92	5.96	5.86	1.82	1.92	1.91	Excavation is possible	1.82
0+288	29.9	36.6	28.7	3495.0	2744.9	2925.29	2.39	2.49	3.24	3.14	2.39	2.49	2.45	Excavation is possible	2.39
0+500	32.8	40.2	31.6	4962.9	3897.8	4140.04	3.47	3.57	4.71	4.61	3.47	3.57	3.48	Excavation is possible	3.47
01+257	32.8	40.2	31.6	5536.0	4348.0	4612.72	3.89	3.99	4.71	4.61	3.89	3.99	3.88	Excavation is possible	3.89
01+700	32.8	40.1	31.5	4194.0	3293.9	3505.87	2.91	3.01	3.55	3.45	2.91	3.01	2.94	Excavation is possible	2.91

ARUP		Job No.	Sheet No.	Rev.
		603258-53	7	1
		Member/Location	Member/Location	
Job Title	Aramis Tunnel FEED Design	Drg. Ref.	Drawing reference	
Calculation	Face Pressure DAUB 2016 - Jancsecz & Steiner, 1994 - Arching	Made by	[redacted] Date 16/05/2024	Chd. CH

(8) Summary face Pressures for all sections and both water levels

								Draw Down			
Critical Sections	**Chainage**	**Water Level**	**Check**	**Minimum Support Pressure at Tunnel Crown for Regular Advance, scr,adv,min**	**Maximum Support Pressure at Tunnel Crown for Regular Advance, scr,adv,max**	**Operation Range**	**Ground Surface to Crown,(t)**	**Minimum Support Pressure at Tunnel Crown, scrown,min**	**1/3-Drawdown**	**1/2-Drawdown**	**full Drawdown**
		mLAT		**bar**	**bar**	**bar**	**m**	Drained	Max + half front material	Max + half front material	Max + half front material
A1	+18.00	4.0	Excavation is possible	1.51	2.73	1.22	15.71	1.51	1.66	1.74	1.96
B1	+178.00	4.0	Excavation is possible	2.34	5.86	3.52	33.11	2.34	2.49	2.56	2.79
C1	+288.00	4.0	Excavation is possible	2.90	3.50	0.60	13.11	2.90	3.05	3.13	3.35
D1	+500.00	4.0	Excavation is possible	3.98	4.96	0.98	17.31	3.98	4.14	4.21	4.44
E1	1+257.00	4.0	Excavation is possible	4.41	4.96	0.56	13.71	4.41	4.56	4.63	4.86
F1	1+700.00	4.0	Excavation is possible	3.42	3.80	0.38	10.61	3.42	3.57	3.64	3.87
A2	+18.00	0.0	Excavation is possible	1.10	2.73	1.63	15.71	1.10	1.25	1.33	1.55
B2	+178.00	0.0	Excavation is possible	1.92	5.86	3.94	33.11	1.92	2.07	2.15	2.38
C2	+288.00	0.0	Excavation is possible	2.49	3.14	0.66	13.11	2.49	2.64	2.71	2.94
D2	+500.00	0.0	Excavation is possible	3.57	4.61	1.04	17.31	3.57	3.72	3.80	4.03
E2	1+257.00	0.0	Excavation is possible	3.99	4.61	0.62	13.71	3.99	4.15	4.22	4.45
F2	1+700.00	0.0	Excavation is possible	3.01	3.45	0.44	10.61	3.01	3.16	3.23	3.46


VERTICAL PROFILE
Tunnel Springling
Ground Surface
Groundwater Head
20
10
0
-10
-20
-30
-40
-50
0+
0+200
0+400
0+600
0+800
01+000
01+200
01+400
01+600
01+800
1.2
1
0.8
0.6
0.4
0.2
0
ELEVATION [M]
ARTIFICAL SURCHARGE [KPA]
CHAINAGE


FACE PRESSURE AT TUNNEL CROWN - ADVANCING
Upper Limit
Upper Operational Range
Lower Operational Range
Lower Limit
FACE PRESSURE [BAR]
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0+
0+200
0+400
0+600
0+800
01+000
01+200
01+400
01+600
01+800
CHAINAGE

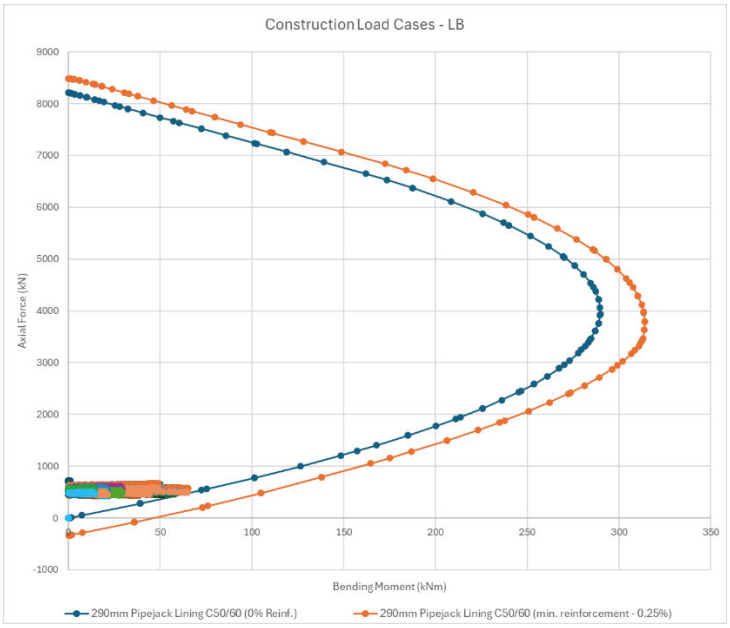
A.2 Tunnel lining design: Concrete flexural design checks

ARUP

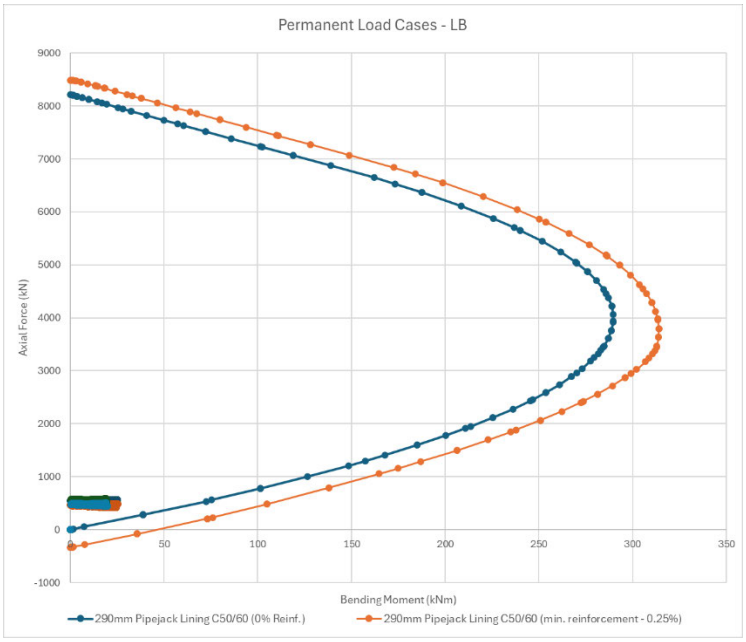
Subject	Aramis - FEED Appendix - Tunnel Lining Design - Flexural Checks
Job No/Ref	303154-00
Date	20 September 2024

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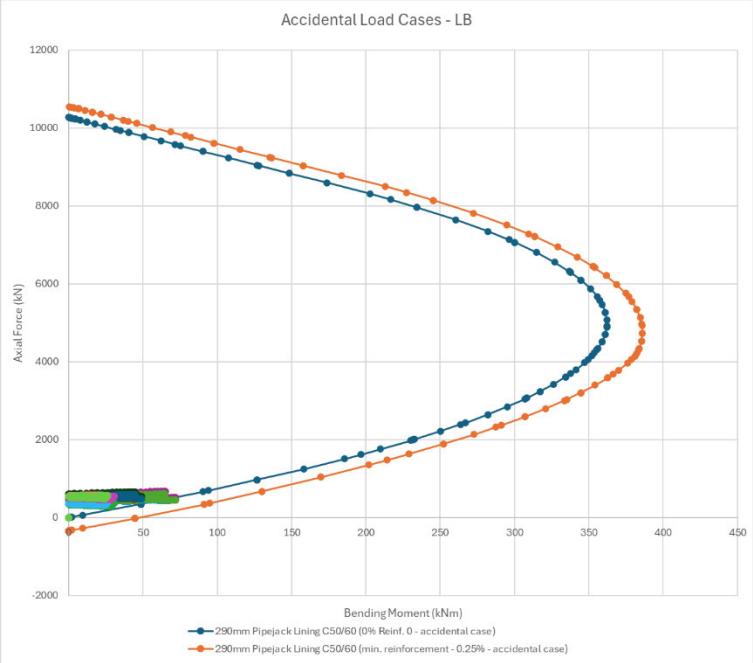
1. **Section B, lower bound Pipejack Lining Results – Terzaghi Soil Arching – Construction Load Cases – Flexural Checks**



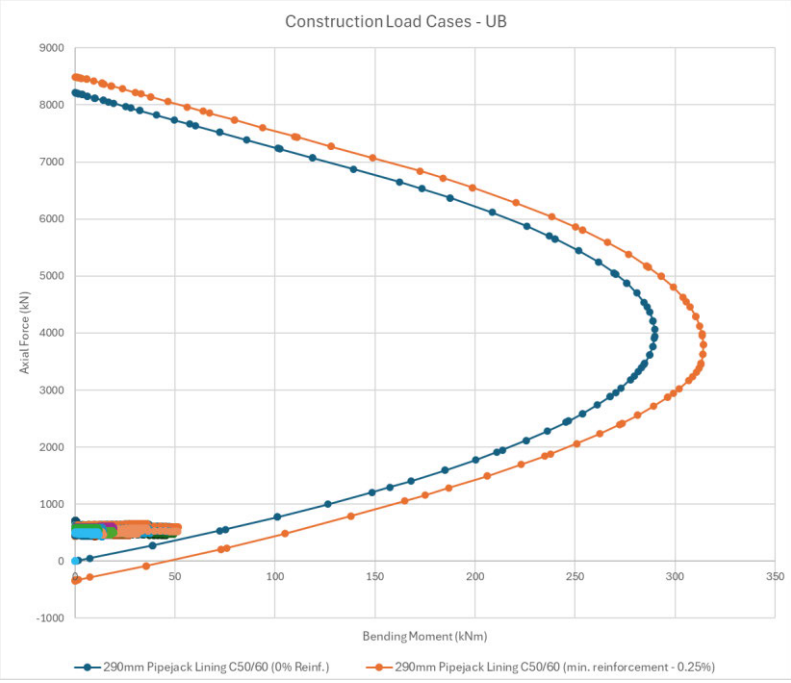
2. **Section B, lower bound Pipejack Lining Results – Terzaghi Soil Arching – Permanent Load Cases – Flexural Checks**



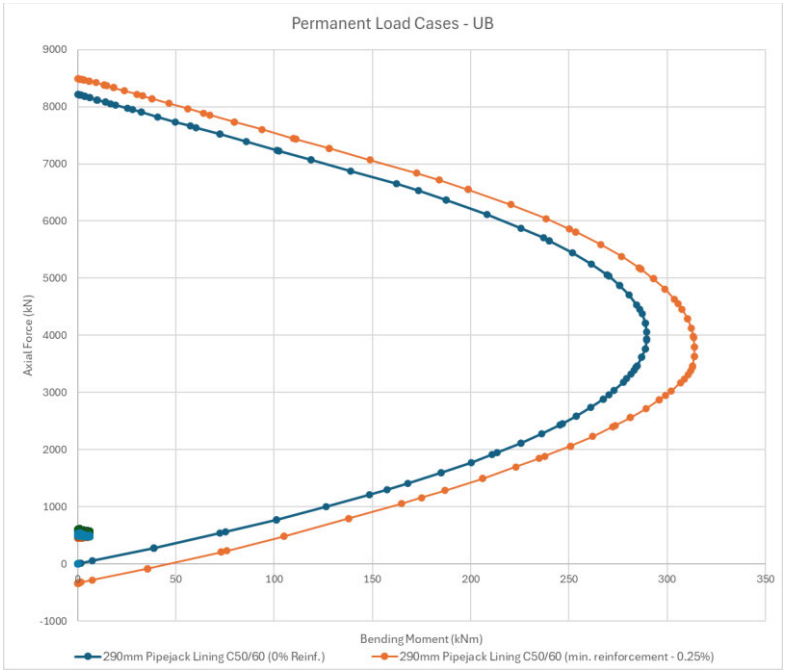
3. Section B, lower bound Pipejack Lining Results – Terzaghi Soil Arching – Accidental Load Cases – Flexural Checks



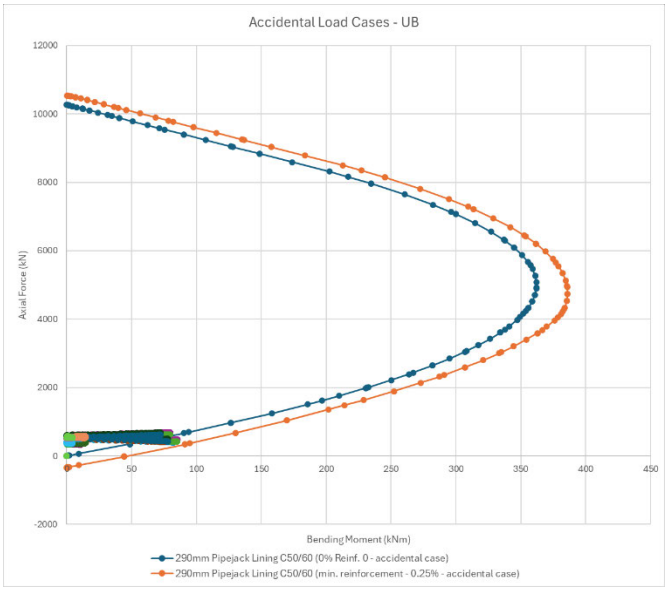
4. Section B, upper bound Pipejack Lining Results – Terzaghi Soil Arching – Construction Load Cases – Flexural Checks



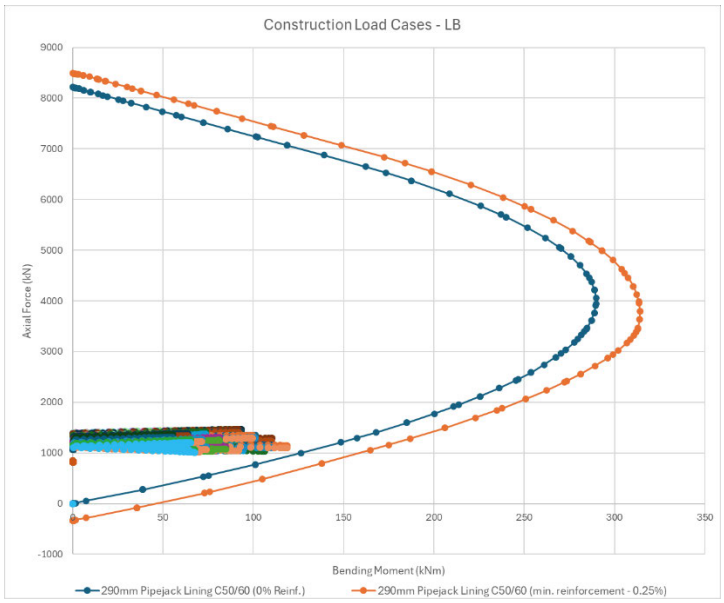
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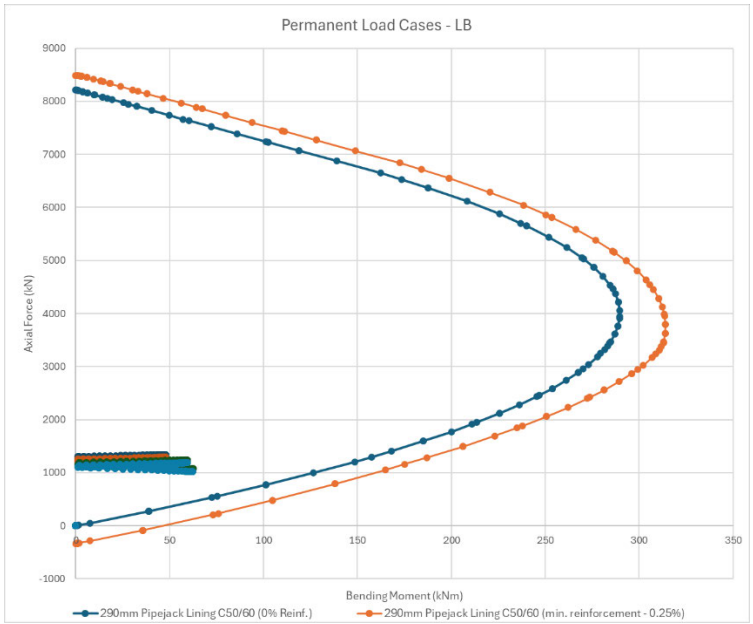
6. **Section B, upper bound Pipejack Lining Results – Terzaghi Soil Arching – Accidental Load Cases – Flexural Checks**



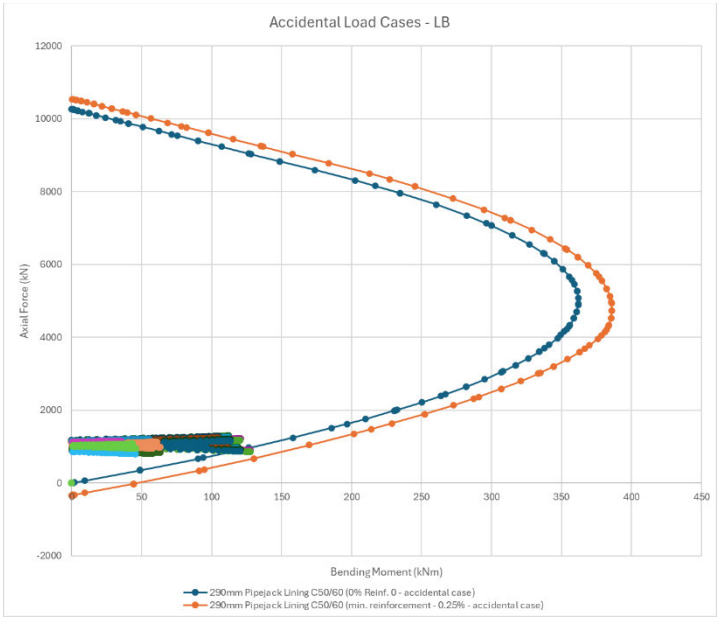
7. **Section E, lower bound Pipejack Lining Results – Full Overburden – Construction Load Cases – Flexural Checks**



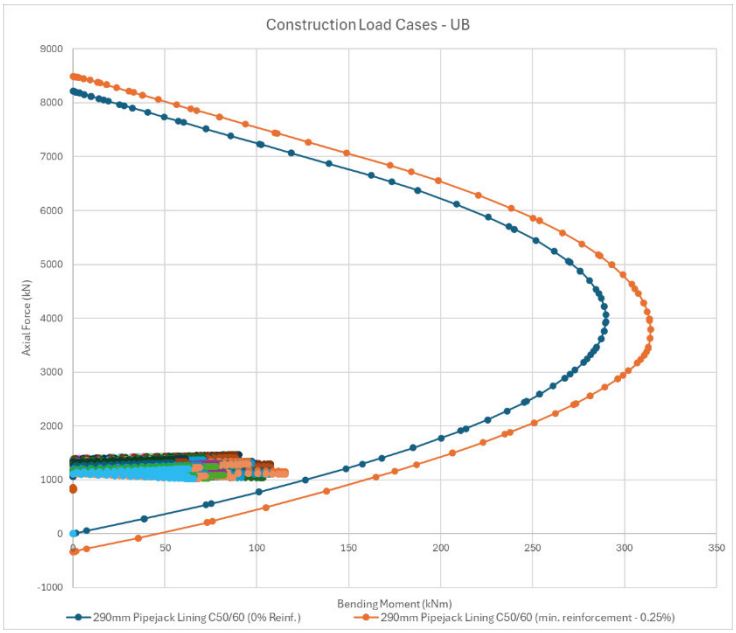
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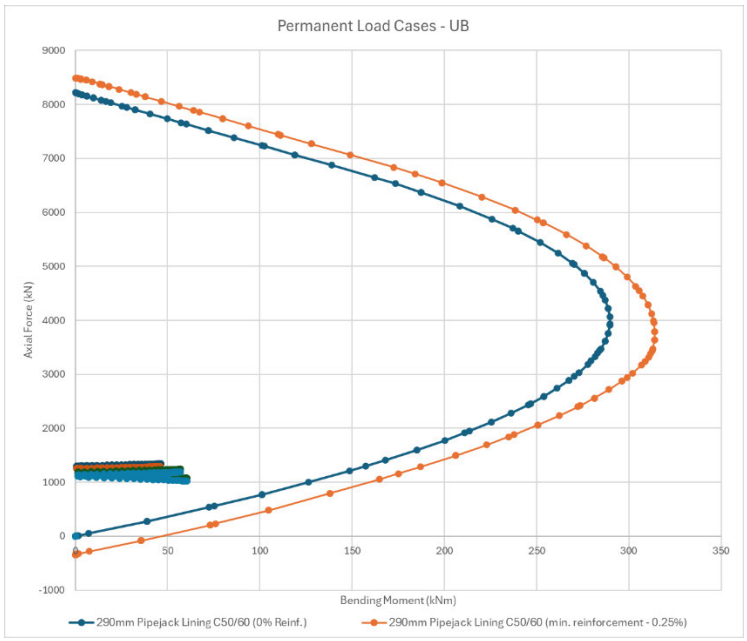
9. Section E, lower bound Pipejack Lining Results – Full Overburden – Accidental Load Cases – Flexural Checks



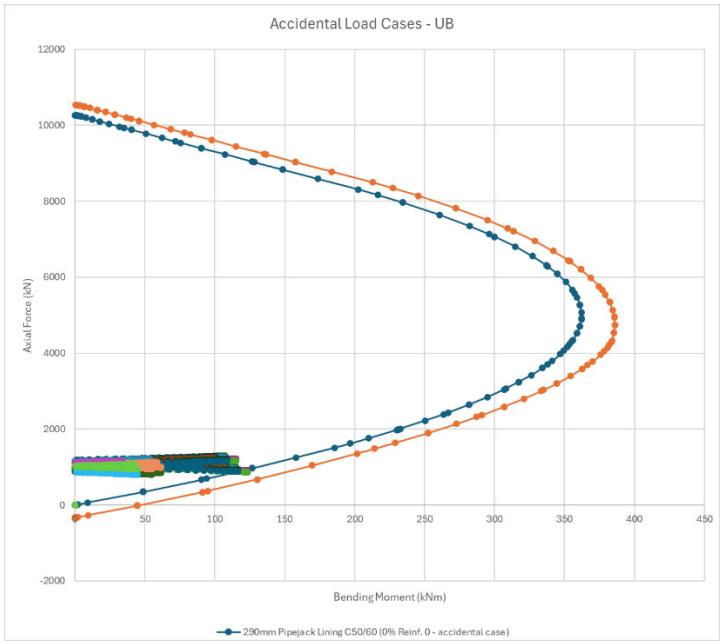
10. Section E, upper bound Pipejack Lining Results – Full Overburden – Construction Load Cases – Flexural Checks



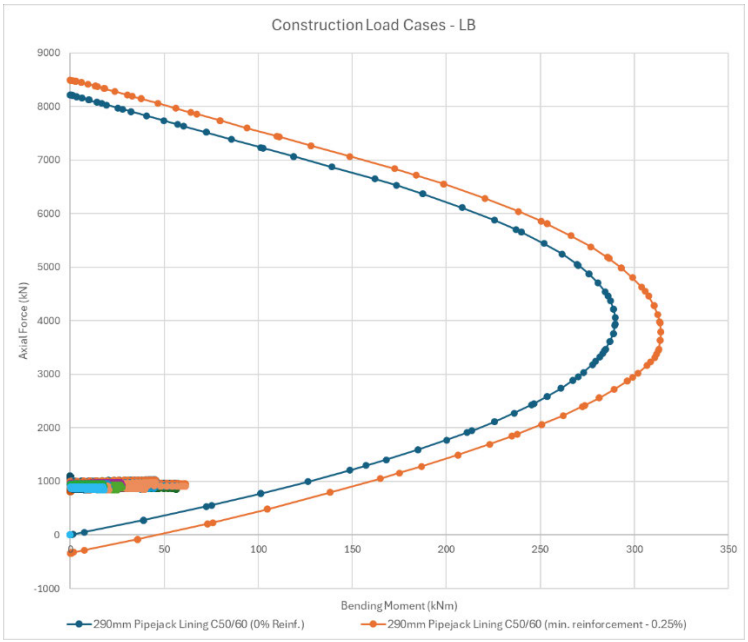
11. Section E, upper bound Pipejack Lining Results – Full Overburden – Permanent Load Cases – Flexural Checks



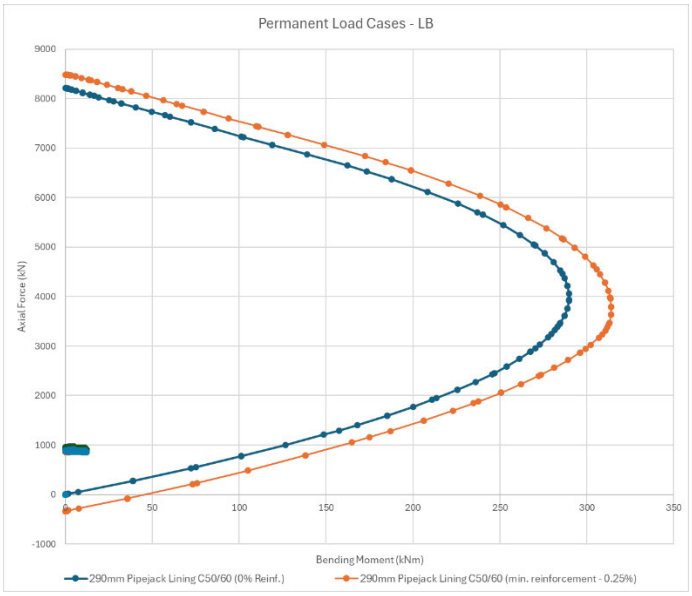
12. Section E, upper bound Pipejack Lining Results – Full Overburden – Accidental Load Cases – Flexural Checks



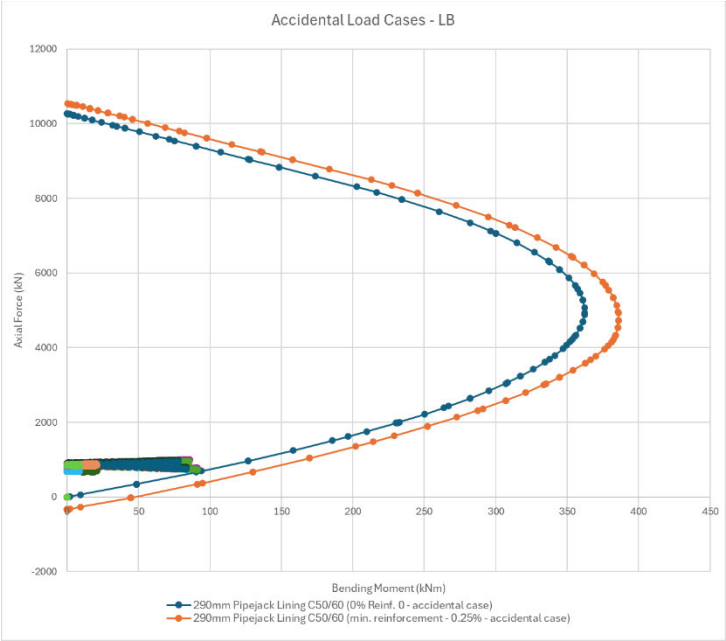
13. Section E, lower bound Pipejack Lining Results – Terzaghi Soil Arching – Construction Load Cases – Flexural Checks



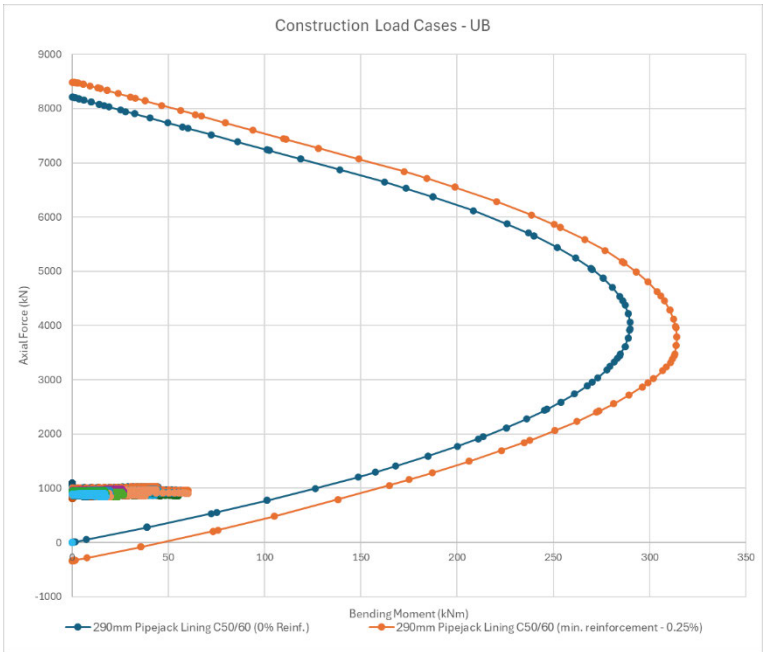
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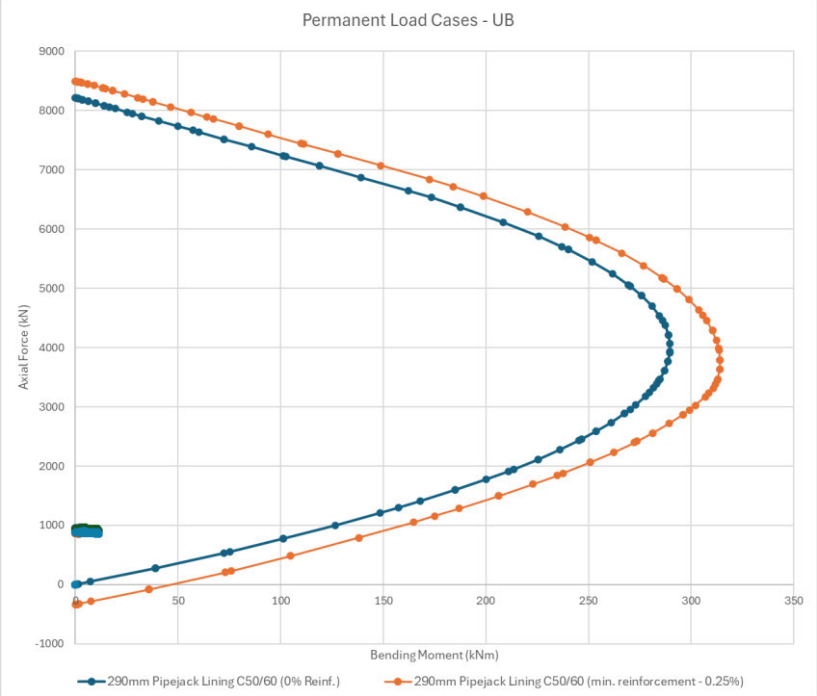
15. Section E, lower bound Pipejack Lining Results – Terzaghi Soil Arching – Accidental Load Cases – Flexural Checks



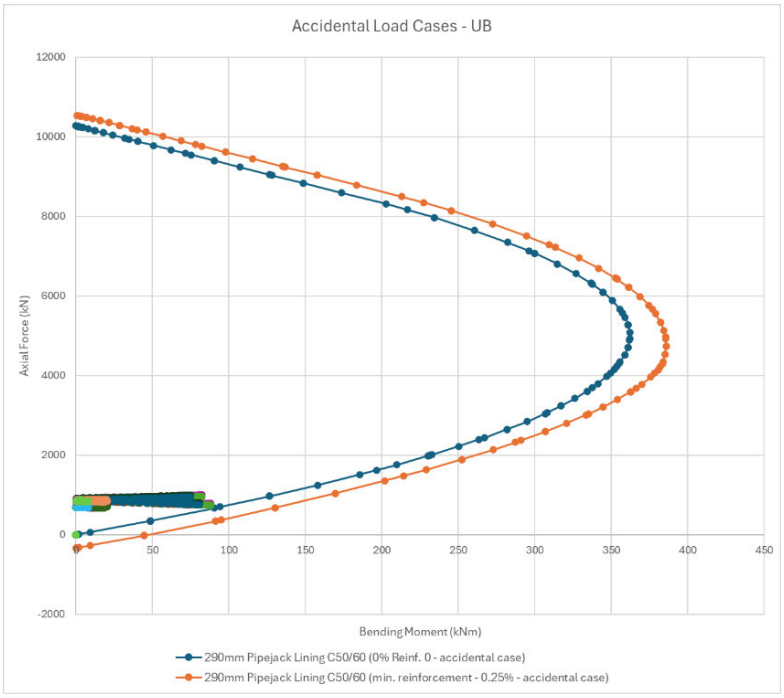
16. Section E, upper bound Pipejack Lining Results – Terzaghi Soil Arching – Construction Load Cases – Flexural Checks



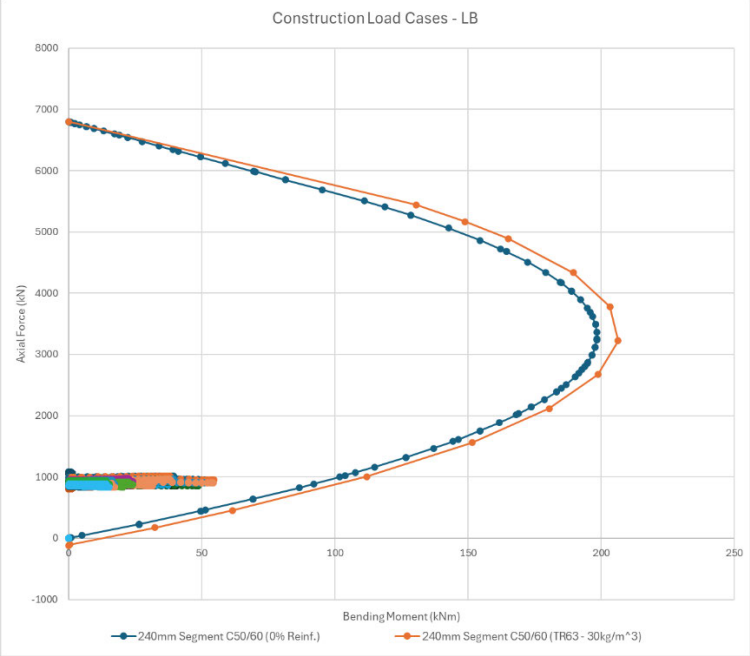
17. Section E, upper bound Pipejack Lining Results – Terzaghi Soil Arching – Permanent Load Cases – Flexural Checks



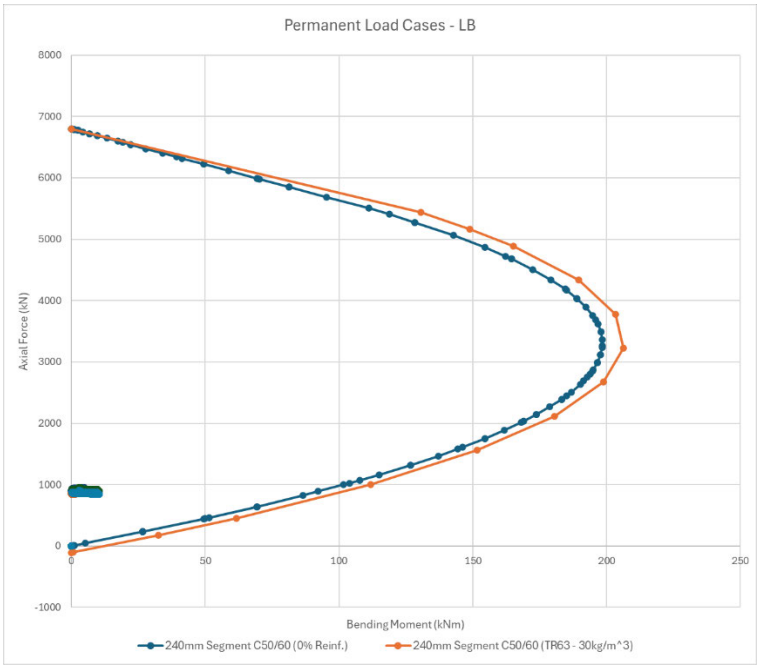
18. Section E, upper bound Pipejack Lining Results – Terzaghi Soil Arching – Accidental Load Cases – Flexural Checks



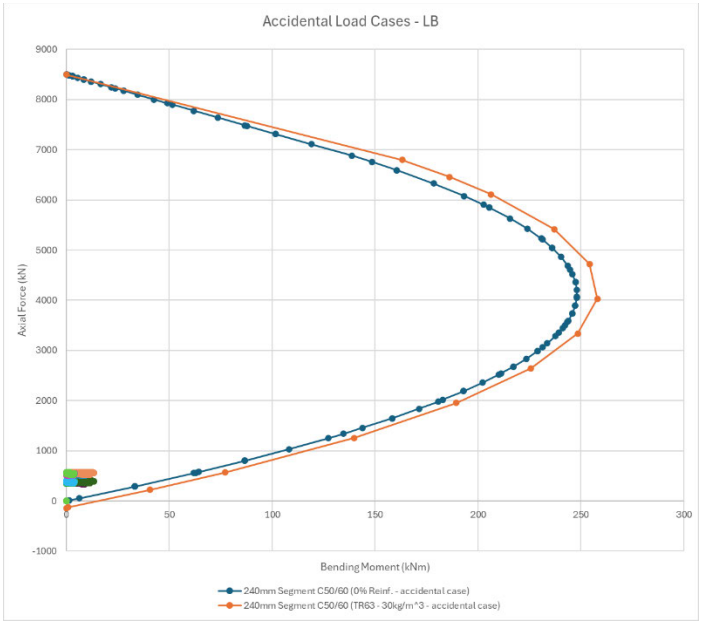
19. Section B, lower bound Segmental Lining Results – Terzaghi Soil Arching – Construction Load Cases – Flexural Checks



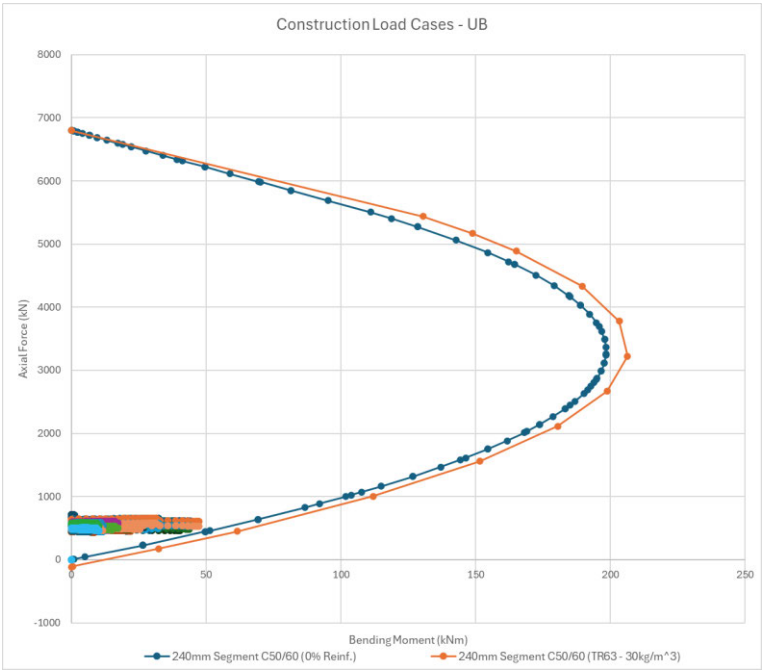
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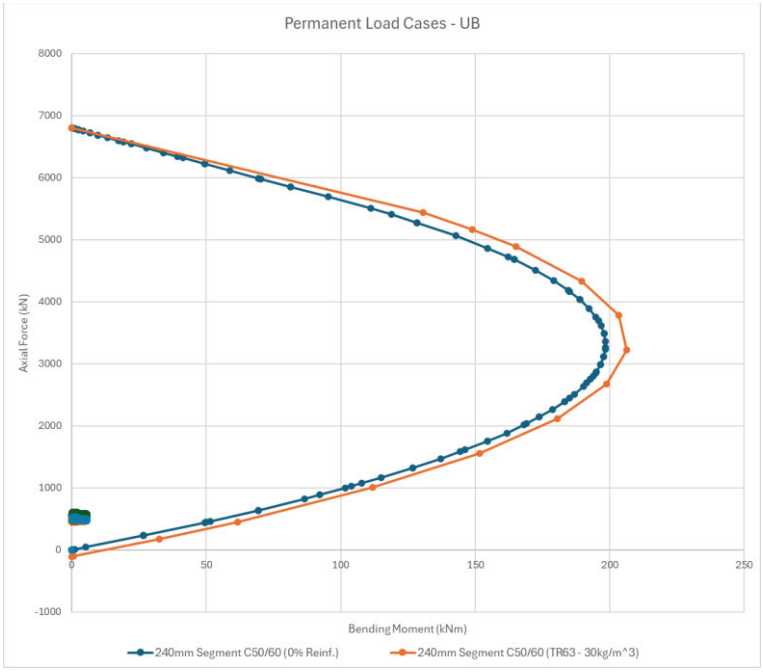
21. Section B, lower bound Segmental Lining Results – Terzaghi Soil Arching – Accidental Load Cases – Flexural Checks



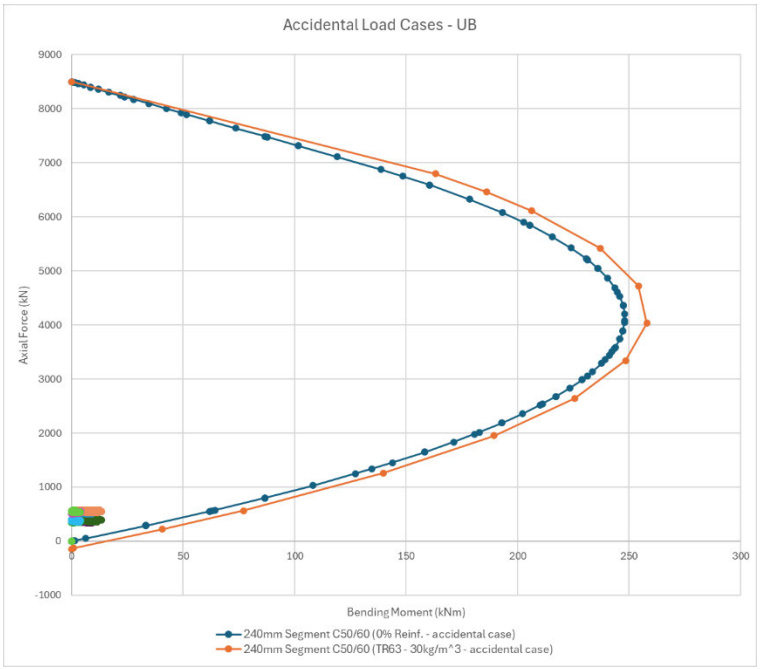
22. Section B, upper bound Segmental Lining Results – Terzaghi Soil Arching – Construction Load Cases – Flexural Checks



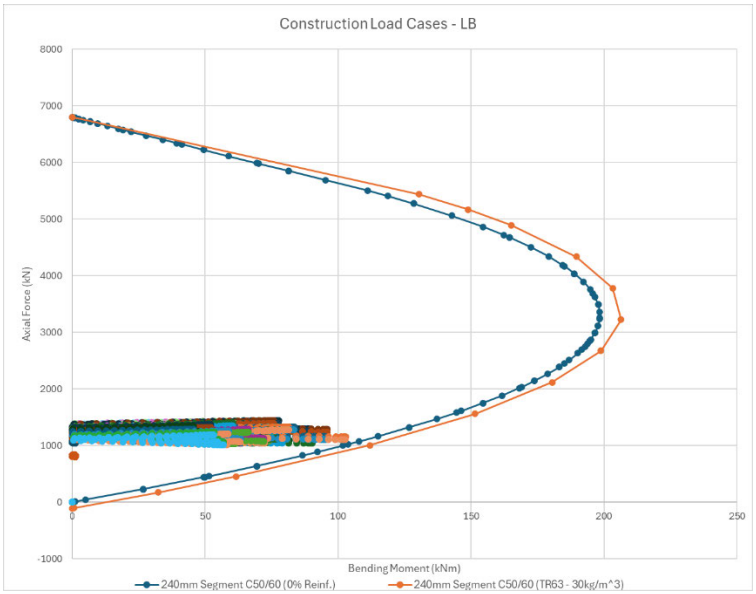
23. Section B, upper bound Segmental Lining Results – Terzaghi Soil Arching – Permanent Load Cases – Flexural Checks



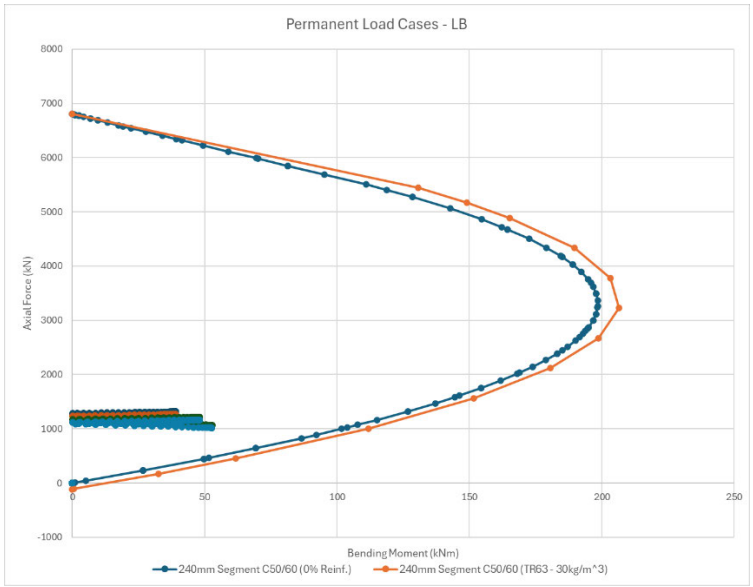
24. Section B, upper bound Segmental Lining Results – Terzaghi Soil Arching – Accidental Load Cases – Flexural Checks



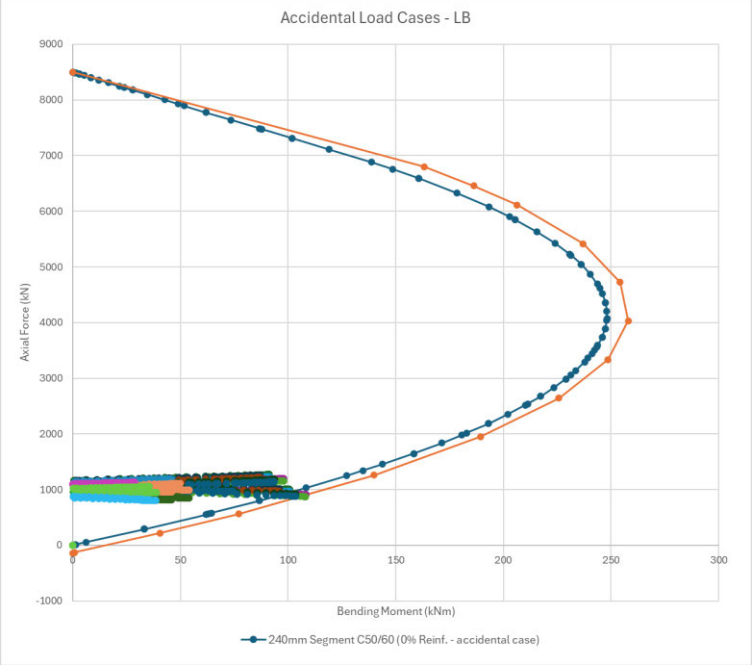
25. Section E, lower bound Segmental Lining Results – Full Overburden – Construction Load Cases – Flexural Checks



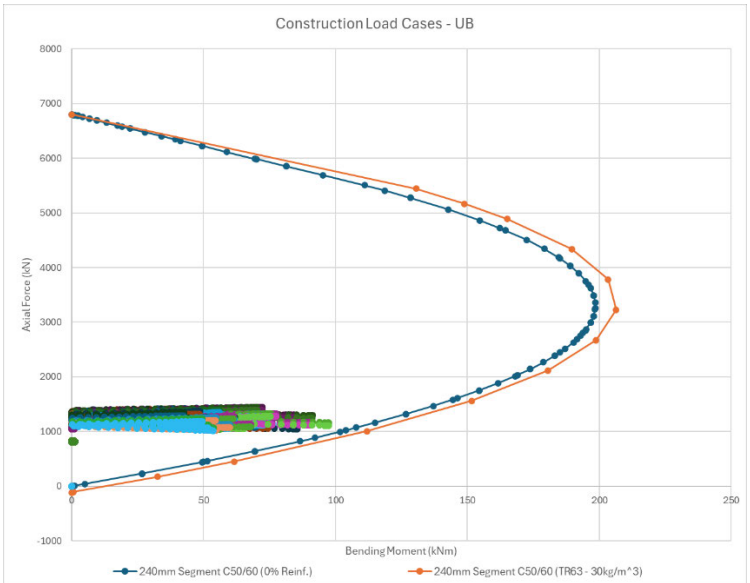
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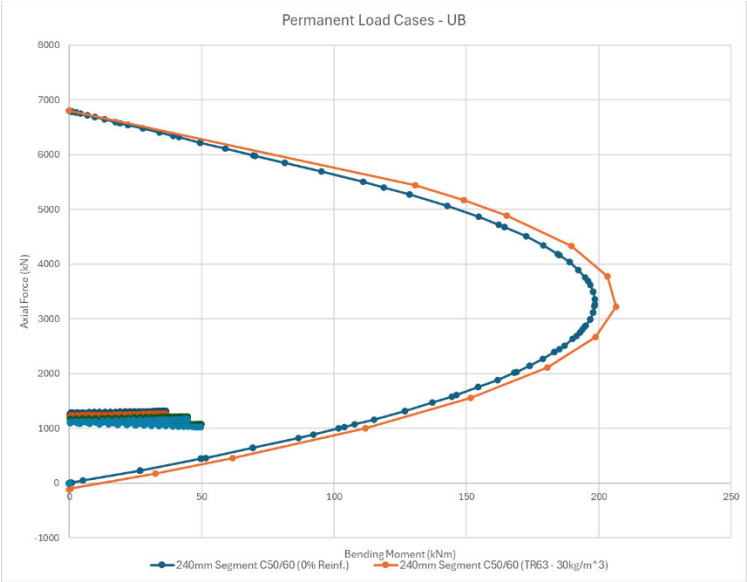
27. Section E, lower bound Segmental Lining Results – Full Overburden – Accidental Load Cases – Flexural Checks



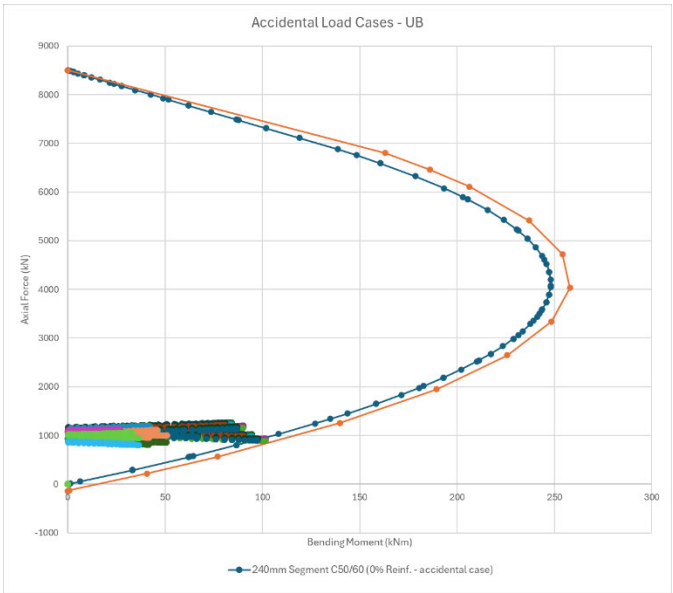
28. Section E, upper bound Segmental Lining Results – Full Overburden – Construction Load Cases – Flexural Checks



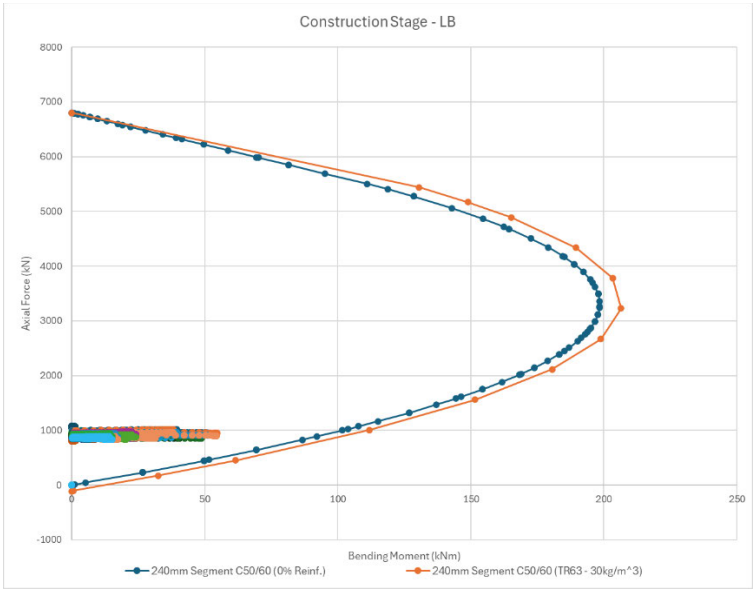
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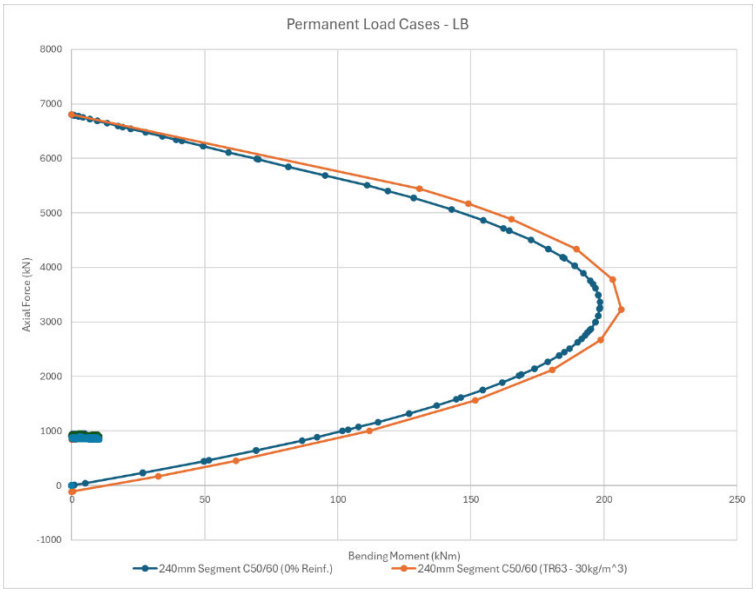
30. Section E, upper bound Segmental Lining Results – Full Overburden – Accidental Load Cases – Flexural Checks



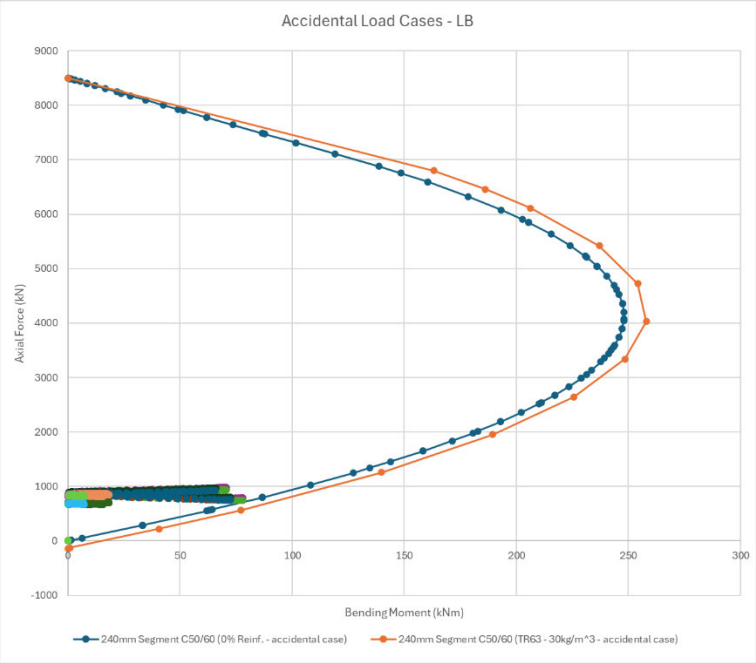
31. Section E, lower bound Segmental Lining Results – Terzaghi Soil Arching – Construction Load Cases – Flexural Checks



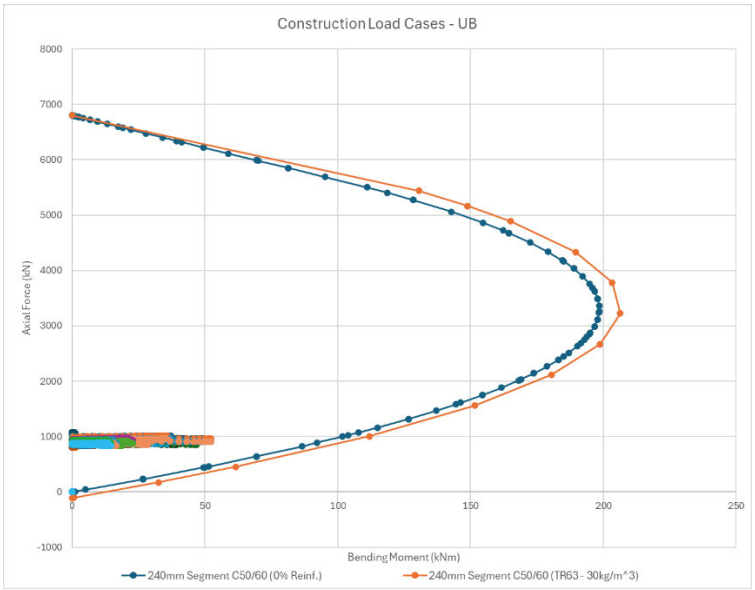
32. Section E, lower bound Segmental Lining Results – Terzaghi Soil Arching – Permanent Load Cases – Flexural Checks



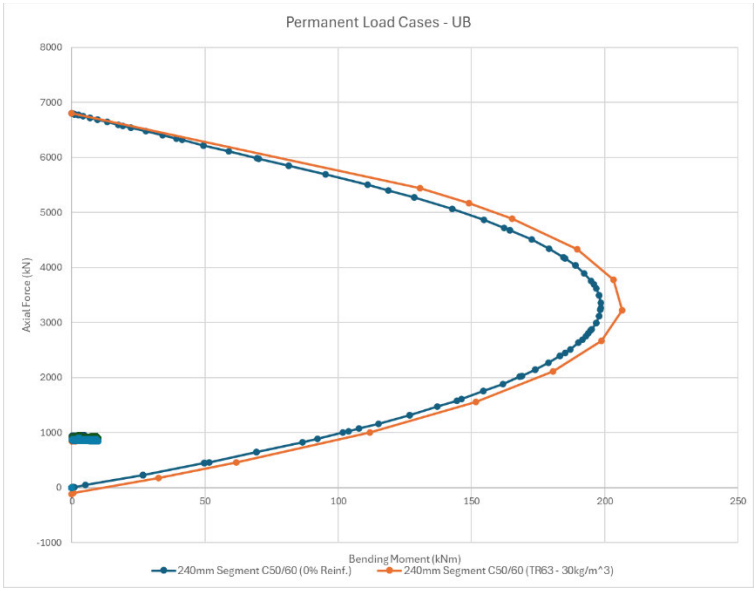
33. Section E, lower bound Segmental Lining Results – Terzaghi Soil Arching – Accidental Load Cases – Flexural Checks



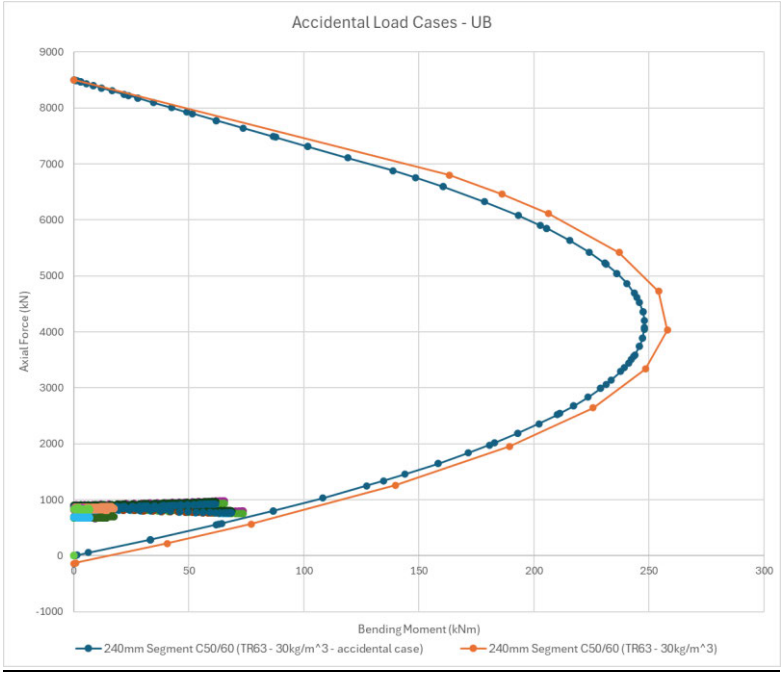
34. Section E, upper bound Segmental Lining Results – Terzaghi Soil Arching – Construction Load Cases – Flexural Checks



35. Section E, upper bound Segmental Lining Results – Terzaghi Soil Arching – Permanent Load Cases – Flexural Checks



36. Section E, upper bound Segmental Lining Results – Terzaghi Soil Arching – Accidental Load Cases – Flexural Checks



A.3 Tunnel lining design: Concrete shear design checks

ARUP	Job No.	Sheet No.	Rev.
	Member/Location		
Job Title: **Aramis Tunnel FEED**	Drg. Ref.		
Calculation: **Shear check EC2 - Pipejacking Section B LB**	Made by [redacted]	Date **03/06/2024**	Chd.

Shear resistance will be calculated in accordance with NEN-EN 1992-1-1+C2:2011 clause 6.2.2.

1 Linning geometry

Lining thickness	t=	300	mm
d dimension in section	d=	250	mm
Lining width	b_w =	1,000	mm

2. Concrete

Characteristic compressive strength of concrete

f_{ck} = 50 MPa

Design compressive strength of concrete

f_{cd} = 50/1.2*1 = 41.67 MPa

3. Shear contribution of the ring (no shear reinforcement)

Shear coefficient factor	k=	MIN(1+((200/250)^0.5),2) =	1.89	-
Min. shear resistance	vmin=	0.035*(1.89^(3/2))*(50^(0.5)) =	0.65	MPa
Nomal force in section	N_{ED}=	618	kN/m	
Cross section	Ac=	300,000	mm²	
Compression stress	σ_{cp}=	(+995*1000)/300,000.00 =	2.06	MPa
Shear coeffcient	k_1=	0.15	-	
$k_1*\sigma_{cp}$=		+MIN(3.32,8.33)*0.15 =	0.31	MPa

$k = 1+\sqrt{\frac{200}{d}} \le 2{,}0$ with d in mm

$v_{min} = 0{,}035\, k^{3/2} \cdot f_{ck}^{1/2}$

$\sigma_{cp} = N_{Ed}/A_c < 0.2\, f_{cd}$ (MPa)

< 0.2 fcd = 8.33 MPa

Shear force resistance	∴ $V_{Rd,C}$ =	(0.65+0.50)*1,000*250/1000 =	238.58	kN/m

$V_{Rd,c} = (v_{min} + k_1 \sigma_{cp})\, b_w d$

4. Shear check

Shear force in section	V_{Ed} =	91	kN/m	[Structural analysys - ULS maximum shear condition]
Utlisation factor	$UR_{concrete}$=	94/285.70 =	0.38	
			OK	

ARUP	Job No.	Sheet No.	Rev.
	Member/Location		
Job Title: **Aramis Tunnel FEED**	Drg. Ref.		
Calculation: **Shear check EC2 - Pipejacking Section B UB**	Made by [redacted]	Date **03/06/2024**	Chd.

Shear resistance will be calculated in accordance with NEN-EN 1992-1-1+C2:2011 clause 6.2.2.

1 Linning geometry

Lining thickness	t=	300	mm
d dimension in section	d=	250	mm
Lining width	b_w =	1,000	mm

2. Concrete

Characteristic compressive strength of concrete

f_{ck} = 50 MPa

Design compressive strength of concrete

f_{cd} = 50/1.2*1 = 41.67 MPa

3. Shear contribution of the ring (no shear reinforcement)

Shear coefficient factor	k=	MIN(1+((200/250)^0.5),2) =	1.89	-
Min. shear resistance	vmin=	0.035*(1.89^(3/2))*(50^(0.5)) =	0.65	MPa

$k = 1+\sqrt{\frac{200}{d}} \le 2{,}0$ with d in mm

$v_{min} = 0{,}035\, k^{3/2} \cdot f_{ck}^{1/2}$

Nomal force in section	N_{ED}=	624	kN/m	
Cross section	Ac=	300,000.00	mm²	

$\sigma_{cp} = N_{Ed}/A_c < 0.2\, f_{cd}$ (MPa)

Compression stress	σ_{cp}=	(+995*1000)/300,000.00 =	2.08	MPa	< 0.2 fcd =	8.33 MPa
Shear coeffcient	k_1=	0.15	-			
$k_1*\sigma_{cp}$=	+MIN(3.32,8.33)*0.15 =	0.31	MPa			

Shear force resistance	∴ $V_{Rd,C}$ =	(0.65+0.50)*1,000*250/1000 =	239.33	kN/m

$V_{Rd,c} = (v_{min} + k_1 \sigma_{cp})\, b_w d$

4. Shear check

Shear force in section	V_{Ed} =	77	kN/m	[Structural analysys - ULS maximum shear condition]
Utlisation factor	$UR_{concrete}$=	94/285.70 =	0.32	
			OK	

ARUP	Job No.	Sheet No.	Rev.
	Member/Location		
Job Title: **Aramis Tunnel FEED**	Drg. Ref.		
Calculation: **Shear check EC2 - Pipejacking Section E LB**	Made by [redacted]	Date **03/06/2024**	Chd.

Shear resistance will be calculated in accordance with NEN-EN 1992-1-1+C2:2011 clause 6.2.2.

1 Linning geometry

Lining thickness	t=	300	mm
d dimension in section	d=	250	mm
Lining width	b_w =	1,000	mm

2. Concrete

Characteristic compressive strength of concrete

f_{ck} = 50 MPa

Design compressive strength of concrete

f_{cd} = 50/1.2*1 = 41.67 MPa

3. Shear contribution of the ring (no shear reinforcement)

Shear coefficient factor	k=	MIN(1+((200/250)^0.5),2) =	1.89	-
Min. shear resistance	vmin=	0.035*(1.89^(3/2))*(50^(0.5)) =	0.65	MPa

$k = 1 + \sqrt{\frac{200}{d}} \leq 2{,}0$ with d in mm

$v_{min} = 0{,}035\, k^{3/2} \cdot f_{ck}^{1/2}$

Nomal force in section	N_{ED}=	995	kN/m	
Cross section	Ac=	300,000.00	mm²	

$\sigma_{cp} = N_{Ed} / A_c < 0.2\, f_{cd}$ (MPa)

Compression stress	σ_{cp}=	(+995*1000)/300,000.00 =	3.32	MPa	< 0.2 fcd =	8.33 MPa
Shear coeffcient	k_1=	0.15	-			
$k_1*\sigma_{cp}$=	+MIN(3.32,8.33)*0.15 =	0.50	MPa			

Shear force resistance	∴ $V_{Rd,C}$ =	(0.65+0.48)*1,000*250/1000 =	285.64	kN/m

$V_{Rd,c} = (v_{min} + k_1 \sigma_{cp})\, b_w d$

4. Shear check

Shear force in section	V_{Ed} =	95	kN/m	[Structural analysys - ULS maximum shear condition]
Utlisation factor	$UR_{concrete}$=	95/281.22 =	0.33	
			OK	

ARUP	Job No.	Sheet No.	Rev.
	Member/Location		
Job Title: **Aramis Tunnel FEED**	Drg. Ref.		
Calculation: **Shear check EC2 - Pipejacking Section E UB**	Made by [redacted]	Date **03/06/2024**	Chd.

Shear resistance will be calculated in accordance with NEN-EN 1992-1-1+C2:2011 clause 6.2.2.

1 Linning geometry

Lining thickness	t=	300	mm
d dimension in section	d=	250	mm
Lining width	b_w =	1,000	mm

2. Concrete

Characteristic compressive strength of concrete

f_{ck} = 50 MPa

Design compressive strength of concrete

f_{cd} = 50/1.2*1 = 41.67 MPa

3. Shear contribution of the ring (no shear reinforcement)

Shear coefficient factor	k=	MIN(1+((200/250)^0.5),2) =	1.89	-
Min. shear resistance	vmin=	0.035*(1.89^(3/2))*(50^(0.5)) =	0.65	MPa

$k = 1 + \sqrt{\frac{200}{d}} \leq 2{,}0$ with d in mm

$v_{min} = 0{,}035\, k^{3/2} \cdot f_{ck}^{1/2}$

Nomal force in section	N_{ED}=	995	kN/m	
Cross section	Ac=	300,000.00	mm²	

$\sigma_{cp} = N_{Ed} / A_c < 0.2\, f_{cd}$ (MPa)

Compression stress	σ_{cp}=	(+995*1000)/300,000.00 =	3.32	MPa	< 0.2 fcd =	8.33 MPa
Shear coeffcient	k_1=	0.15	-			
$k_1*\sigma_{cp}$=	+MIN(3.32,8.33)*0.15 =	0.50	MPa			

Shear force resistance	∴ $V_{Rd,C}$ =	(0.65+0.50)*1,000*250/1000 =	285.70	kN/m

$V_{Rd,c} = (v_{min} + k_1 \sigma_{cp})\, b_w d$

4. Shear check

Shear force in section	V_{Ed} =	94	kN/m	[Structural analysys - ULS maximum shear condition]
Utlisation factor	$UR_{concrete}$=	94/285.70 =	0.33	
			OK	

ARUP	Job No.	Sheet No.	Rev.
	Member/Location		
Job Title: **Aramis Tunnel FEED**	Drg. Ref.		
Calculation: **Shear check EC2 - Segmental Section B LB**	Made by [redacted]	Date **03/06/2024**	Chd.

Shear resistance will be calculated in accordance with NEN-EN 1992-1-1+C2:2011 clause 6.2.2.

1 Linning geometry

Lining thickness	t=	250	mm
d dimension in section	d=	200	mm
Lining width	b_w =	1,000	mm

2. Concrete

Characteristic compressive strength of concrete

f_{ck} = 50 MPa

Design compressive strength of concrete

f_{cd} = 50/1.2*1 = 41.67 MPa

3. Shear contribution of the ring (no shear reinforcement)

$k = 1 + \sqrt{\frac{200}{d}} \leq 2{,}0$ with d in mm

$v_{min} = 0{,}035\, k^{3/2} \cdot f_{ck}^{1/2}$

Shear coefficient factor	k=	MIN(1+((200/200)^0.5),2) =	2.00	-
Min. shear resistance	vmin=	0.035*(2.00^(3/2))*(50^(0.5)) =	0.70	MPa
Nomal force in section	N_{ED}=	482	kN/m	
Cross section	Ac=	250,000.00	mm²	

$\sigma_{cp} = N_{Ed} / A_c < 0.2\, f_{cd}$ (MPa)

Compression stress	σ_{cp}=	(+482*1000)/250,000.00 =	1.93	MPa	< 0.2 fcd =	8.33 MPa
Shear coeffcient	k_1=	0.15	-			
$k_1*\sigma_{cp}$=	+MIN(1.93,8.33)*0.15 =	0.29	MPa			

$V_{Rd,c} = (v_{min} + k_1 \sigma_{cp})\, b_w d$

Shear force resistance	∴ $V_{Rd,C}$ =	(0.70+0.29)*1,000*200/1000 =	197.84	kN/m

4. Shear check

Shear force in section	V_{Ed} =	113	kN/m	[Structural analysys - ULS maximum shear condition]
Utlisation factor	$UR_{concrete}$=	113/197.84 =	0.57	
			OK	

ARUP	Job No.	Sheet No.	Rev.
	Member/Location		
Job Title: **Aramis Tunnel FEED**	Drg. Ref.		
Calculation: **Shear check EC2 - Segmental Section B UB**	Made by [redacted]	Date **03/06/2024**	Chd.

Shear resistance will be calculated in accordance with NEN-EN 1992-1-1+C2:2011 clause 6.2.2.

1 Linning geometry

Lining thickness	t=	250	mm
d dimension in section	d=	200	mm
Lining width	b_w =	1,000	mm

2. Concrete

Characteristic compressive strength of concrete

f_{ck} = 50 MPa

Design compressive strength of concrete

f_{cd} = 50/1.2*1 = 41.67 MPa

3. Shear contribution of the ring (no shear reinforcement)

$k = 1 + \sqrt{\frac{200}{d}} \leq 2{,}0$ with d in mm

Shear coefficient factor	k=	MIN(1+((200/200)^0.5),2) =	2.00	-
Min. shear resistance	vmin=	0.035*(2.00^(3/2))*(50^(0.5)) =	0.70	MPa

$v_{min} = 0{,}035\, k^{3/2} \cdot f_{ck}^{1/2}$

Nomal force in section	N_{ED}=		631.2	kN/m
Cross section	Ac=		250,000.00	mm²

$\sigma_{cp} = N_{Ed} / A_c < 0.2\, f_{cd}$ (MPa)

Compression stress	σ_{cp}=	(+482*1000)/250,000.00 =	2.52	MPa	< 0.2 fcd =	8.33 MPa
Shear coeffcient	k_1=		0.15	-		
$k_1*\sigma_{cp}$=		+MIN(1.93,8.33)*0.15 =	0.38	MPa		

Shear force resistance	∴ $V_{Rd,C}$ =	(0.70+0.29)*1,000*200/1000 =	215.74	kN/m

$V_{Rd,c} = (v_{min} + k_1 \sigma_{cp})\, b_w d$

4. Shear check

Shear force in section	V_{Ed} =	75	kN/m	[Structural analysys - ULS maximum shear condition]
Utlisation factor	$UR_{concrete}$=	113/197.84 =	0.35	
			OK	

ARUP	Job No.	Sheet No.	Rev.
	Member/Location		
Job Title: **Aramis Tunnel FEED**	Drg. Ref.		
Calculation: **Shear check EC2 - Segmental Section E LB**	Made by [redacted]	Date **03/06/2024**	Chd.

Shear resistance will be calculated in accordance with NEN-EN 1992-1-1+C2:2011 clause 6.2.2.

1 Linning geometry

Lining thickness	t=	250	mm
d dimension in section	d=	200	mm
Lining width	b_w =	1,000	mm

2. Concrete

Characteristic compressive strength of concrete

f_{ck} = 50 MPa

Design compressive strength of concrete

f_{cd} = 50/1.2*1 = 41.67 MPa

3. Shear contribution of the ring (no shear reinforcement)

Shear coefficient factor	k=	MIN(1+((200/200)^0.5),2) =	2.00	-
Min. shear resistance	vmin=	0.035*(2.00^(3/2))*(50^(0.5)) =	0.70	MPa

$k = 1 + \sqrt{\frac{200}{d}} \leq 2{,}0$ with d in mm

$v_{min} = 0{,}035\, k^{3/2} \cdot f_{ck}^{1/2}$

Nomal force in section	N_{ED}=	986	kN/m	
Cross section	Ac=	250,000.00	mm²	

$\sigma_{cp} = N_{Ed} / A_c < 0.2\, f_{cd}$ (MPa)

Compression stress	σ_{cp}=	(+482*1000)/250,000.00 =	3.95	MPa	< 0.2 fcd =	8.33 MPa
Shear coeffcient	k_1=	0.15	-			
$k_1 * \sigma_{cp}$=	+MIN(1.93,8.33)*0.15 =	0.59	MPa			

Shear force resistance	∴ $V_{Rd,C}$ =	(0.70+0.29)*1,000*200/1000 =	258.37	kN/m

$V_{Rd,c} = (v_{min} + k_1 \sigma_{cp})\, b_w d$

4. Shear check

Shear force in section	V_{Ed} =	90	kN/m	[Structural analysys - ULS maximum shear condition]
Utlisation factor	$UR_{concrete}$=	113/197.84 =	0.35	
			OK	

ARUP	Job No.	Sheet No.	Rev.
	Member/Location		
Job Title: **Aramis Tunnel FEED**	Drg. Ref.		
Calculation: **Shear check EC2 - Segmental Section E UB**	Made by [redacted]	Date **03/06/2024**	Chd.

Shear resistance will be calculated in accordance with NEN-EN 1992-1-1+C2:2011 clause 6.2.2.

1 Linning geometry

Lining thickness	t=	250	mm
d dimension in section	d=	200	mm
Lining width	b_w =	1,000	mm

2. Concrete

Characteristic compressive strength of concrete

f_{ck} = 50 MPa

Design compressive strength of concrete

f_{cd} = 50/1.2*1 = 41.67 MPa

3. Shear contribution of the ring (no shear reinforcement)

Shear coefficient factor	k=	MIN(1+((200/200)^0.5),2) =	2.00	-
Min. shear resistance	vmin=	0.035*(2.00^(3/2))*(50^(0.5)) =	0.70	MPa
Nomal force in section	N_{ED}=	988	kN/m	
Cross section	Ac=	250,000.00	mm²	
Compression stress	σ_{cp}=	(+482*1000)/250,000.00 =	3.95	MPa
Shear coeffcient	k_1=	0.15	-	
$k_1*\sigma_{cp}$=		+MIN(1.93,8.33)*0.15 =	0.59	MPa

$k = 1+\sqrt{\frac{200}{d}} \le 2{,}0$ with d in mm

$v_{min} = 0{,}035\, k^{3/2} \cdot f_{ck}^{1/2}$

$\sigma_{cp} = N_{Ed}/A_c < 0.2\, f_{cd}$ (MPa)

< 0.2 fcd = 8.33 MPa

Shear force resistance $\therefore V_{Rd,C}$ = (0.70+0.29)*1,000*200/1000 = 258.56 kN/m

$V_{Rd,c} = (v_{min} + k_1 \sigma_{cp})\, b_w d$

4. Shear check

Shear force in section V_{Ed} = 88 kN/m [Structural analysys - ULS maximum shear condition]

Utlisation factor $UR_{concrete}$= 113/197.84 = 0.34

OK

APPENDIX B SHAFT DESIGN CALCULATIONS

B.1. Shaft loading summary

ARUP	Job No.	Sheet No.	Rev.
	303154-00	01	P01
	Member/Location		
Job Title: Aramis	Drg. Ref.		
Calculation: Launch shaft loading - GSA	Made by [redacted]	Date: 04/06/2024	Chd.

Load Case Specification

L1	Lateral earth pressure (outside - full height)
L2	Lateral earth pressure (inside - full height)
L3	Lateral earth pressure (inside - below BS)
L4	Pore water pressure (outside - full height)
L5	Pore water pressure (inside - full height)
L6	Pore water pressure (inside -below BS)
L7	Self-weight
L8	Construction loading
L9	Base slab buoyancy (bottom slab)
L10	Base slab buoyancy (top slab)
L11	Winch loads (x)
L12	Winch loads (y)
L13	Winch loads (z)
L14	Jacking on thrust wall
L15	Mobile crrane
L16	Rail mounted gantry crane
L17	Jacking on tunnel eye

f

Analysis stages

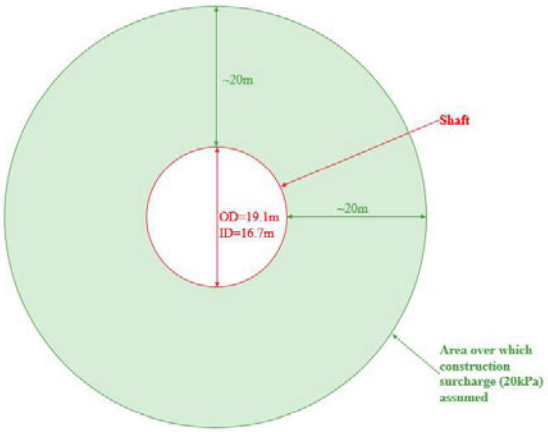
D-walls	PA2 PS3 to PS527
Capping slab	PA2 PS3 to PS527 PB2
Tension piles & excavate shaft	PA2 PS3 to PS527 PB2
Cast base plug (wet)	PA2 PS5 to PS529 PB2 PA1 PS1
Cast RC slab (dry)	PA2 PS5 to PS529 PB2 PA1 PS1 PA3 PS530 PB3
Cast thrust wall	PA2 PS5 to PS529 PB2 PA1 PS1 PA3 PS530 PA4 PS528 PB3
Cast tunnel eye	PA2 PS5 to PS529 PB2 PA1 PS1 PA3 PS530 PA4 PS528 PA5 PS529 PB3
Jacking	PA2 PS5 to PS529 PB2 PA1 PS1 PA3 PS530 PA4 PS528 PA5 PS529 PB3
Winching	PA2 PS5 to PS529 PB2 PA1 PS1 PA3 PS530 PA4 PS528 PA5 PS529 PB3

Analysis tasks

Stage 1 - ULS	1.35L1+1.35L2+1.35L4+1.35L5+1.35L7+1.5L8 + 1.5L15
Stage 1 - SLS	1.L1+1L2+1L4+1L5+1L7+1L8 + 1L15
Stage 2 - ULS	1.35L1+1.35L2+1.35L4+1.35L5+1.35L7+1.5L8 + 1.5L15
Stage 2 - SLS	1.L1+1L2+1L4+1L5+1L7+1L8 + 1L15
Stage 3 - ULS	1.35L1+1.35L3+1.35L4+1.35L5+1.35L7+1.5L8 + 1.5L15
Stage 3 - SLS	1.L1+1L3+1L4+1L5+1L7+1L8 + 1L15
Stage 4 - ULS	1.35L1+1.35L3+1.35L4+1.35L5+1.35L7+1.5L8 + 1.35L9 + 1.5L15
Stage 4 - SLS	1.L1+1L3+1L4+1L5+1L7+1L8+1L9 + 1L15
Stage 5 - ULS	1.35L1+1.35L3+1.35L4+1.35L6+1.35L7+1.5L8+1.35L10 + 1.5L15
Stage 5 - SLS	1.L1+1L3+1L4+1L6+1L7+1L8+1L10 + 1L15
Stage 6 - ULS	1.35L1+1.35L3+1.35L4+1.35L6+1.35L7+1.5L8 + 1.35L10 + 1.5L15
Stage 6 - SLS	1.L1+1L3+1L4+1L6+1L7+1L8+1L10 + 1L15
Stage 7 - ULS	1.35L1+1.35L3+1.35L4+1.35L6+1.35L7+1.5L8 + 1.35L10 + 1.5L15
Stage 7 - SLS	1.L1+1L3+1L4+1L6+1L7+1L8+1L10 + 1L15
Stage 8 - ULS	1.35L1+1.35L3+1.35L4+1.35L6+1.35L7+1.5L8 + 1.35L10 +1.5L14 + 1.5L15+1.5L16 + 1.5L17
Stage 8 - SLS	1.L1+1L3+1L4+1L6+1L7+1L8+1L10 +1L14 +1L15 + 1L16 +1L17
Stage 9 - ULS	1.35L1+1.35L3+1.35L4+1.35L6+1.35L7+1.5L8 + 1.35L10 + 1.5L11 + 1.5L12 + 1.5L13
Stage 9 - SLS	1.L1+1L3+1L4+1L6+1L7+1L8+1L10 + 1L11 + 1L12 + 1L13

ARUP	Job No.	Sheet No.	Rev.
	303154-00	01	P01
	Member/Location		
Job Title: Aramis	Drg. Ref.		
Calculation: Construction surchage	Made by	Date: 04/06/2024	Chd.

Intoduction Summary of construction loading application to GSA model.



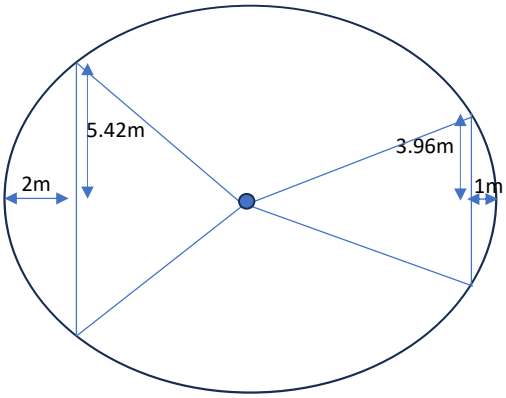
Surcharge 20 kPa

Depth (m)	K0	Surcharge x K0	Load (kPa)
0	0.52	10.4	-10.4
1	0.52	10.4	-10.4
2	0.52	10.4	-10.4
3	0.52	10.4	-10.4
4	0.52	10.4	-10.4
5	0.52	10.4	-10.4
6	0.52	10.4	-10.4
7	0.52	10.4	-10.4
8	0.52	10.4	-10.4
9	0.52	10.4	-10.4
10	0.52	10.4	-10.4
11	0.52	10.4	-10.4
12	0.52	10.4	-10.4
13	0.52	10.4	-10.4
14	0.67	13.4	-13.4
15	0.67	13.4	-13.4
16	0.52	10.4	-10.4
17	0.52	10.4	-10.4
18	0.67	13.4	-13.4
19	0.67	13.4	-13.4
20	0.52	10.4	-10.4
21	0.52	10.4	-10.4
22	0.52	10.4	-10.4
23	0.52	10.4	-10.4
24	0.67	13.4	-13.4
25	0.8	16	-16
26	0.8	16	-16

ARUP	Job No.	Sheet No.	Rev.
	303154-00	01	P01
	Member/Location		
Job Title: Aramis	Drg. Ref.		
Calculation: Jacking force	Made by	Date: 04/06/2024	Chd.

Maximum jacking force	3208	tonnes	*Email from Ben Castle 04.06.2024 - 4980tonnes, updated to 3208 on 11.06.24*
	31470.5	kN	

Thrust wall
Chord length
11.80 m



Tunnel eye
Chord length
8.25 m

Distribution area of thrust wall

Height thrust wall	5	m	
Width thrust wall	5	m	total width of thrust wall = 10.8m
Area thrust wall	25.0	m2	
Pressure = F/A	1258.8	kN/m2	

Distribution area of tunnel eye

Rubber compression friction coeff	0.5		taken from 'Fabric BULLFLEX' data sheet.
Total shear force	15735.2	kN	BULLFLEX® | Segmental Seals and Sealing Strips (tunnellingacces
Divided by 20 nodes	786.8	kN	per node

	Thrust wall		Tunnel eye	
Angle	40.5		28.3	
Diameter of shaft	16.7	m	16.7	m
Area of circle	219.0	m2	219.0	m2
Area of sector	49.3	m2	34.5	m2
Area of triangle	34.4	m2	29.106	m2
Area of segment	14.8	m2	5.3	m2
Height	6.6		5.5	m
Volume segment	98.0	m3	29.4	m3
Chord length	11.80	m	8.25	m

Beton- und Monierbau GmbH

Data Sheet for the Fabric BULLFLEX® ff

Material:	Polyamide 6.6 (Nylon)	
Weight [g/m²]:	approximately 660	
Thickness of the fabric [mm]	approximately 1.0	

Tensile Strength and maximum Elongation according DIN 53857-1

Tensile Strength (at 100 mm width) [N]:	weft:	min 12,000
	warp:	min 24,000
Maximum Elongation [%]:	weft:	approximately 20
	warp:	approximately 20

Friction factor BULLFLEX®$_{\text{grout filled (wet and dry)}}$ / Steel µ:	0.5

Airflow through fabric at pressure [mbar]:	10	20	30
[l/min x 100cm²]:	approx. 6.5	approx. 13	approx. 19

Water flow through fabric:	= airflow -5% to -10%
Remaining tensile strength after 1 year of light exposure in Florida [%]:	20 – 30
Solubility:	concentrated inorganic acids and Phenol

Creep elongation:

> Off: -“Geotextiles and Geomembranes in civil Engineering”, page 156 ff., published by A.A. Balkema/Rotterdam/Boston/1986, ISBN 90 6191 6240

Polyamide 6.6 has got a creep elongation with a constant stress of 60 % of the maximum within one year of approximately 16.5 % (approx. 6 % stress constant at 20 %).

For the installation of pre-stressed Polyamide fabrics for a longer period (10 to 100 years) a pre-stress of maximally 40 % of the maximum stress force is recommended. This corresponds at a round woven Bullflex®-Hose to an effective overpressure of 1.75 bar (10.9 psi).

Subject to change on technical reasons!

ARUP	Job No.	Sheet No.	Rev.
	303154-00	1 of 1	0
	Member/Location		
Job Title: Aramis	Drg. Ref.		
Calculation: Diaphragm wall - weight of reinforcement cage	Made by	Date: 04/06/2024	Chd.: MP

Introduction

Calculation to determine weight of one diaphragm wall reinforcement cage, to determine lifting requirements.

Inputs

No. panels	12		
D-wall panel length	4.5	m	
Height panels	26.6	m	
Width panel	1	m	
Diameter vertical bars	32	mm	
Area one vertical bar	0.00080	m2/m	
Spacing vertical bars	150	mm	
No. vertical bars per face	30		
Vertical bars volume per face	0.64	m3	
x2 faces	1.28	m3	
Diameter horizontal bars	32	mm	
Area one horizontal bar	0.00080	m2	
Spacing horizontal bars	150	mm	
No. horizontal bars per face	177		
Horizontal bars volume per face	0.64	m3	
x2 faces	1.28	m3	
Total rebar volume per panel	2.57	m3	
Concrete volume	119.70	m3	2%
Density reinforcement	7850	kg/m3	
Mass=density x volume	20152	kg	
	20	tonnes	per reinforcement panel

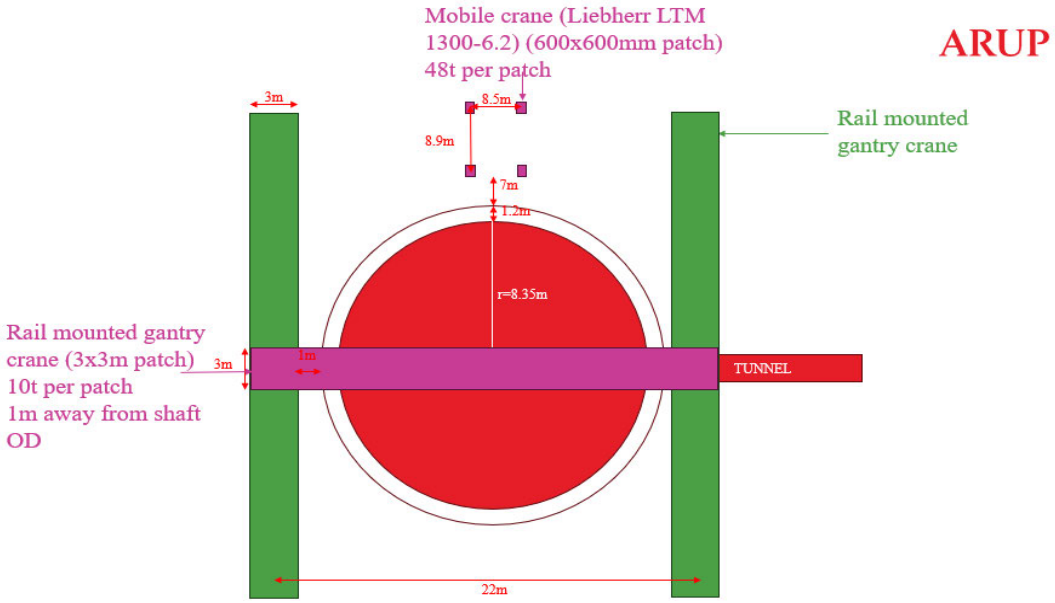
ARUP	Job No.	Sheet No.	Rev.
	303154-00	01	P02
	Member/Location		
Job Title: Aramis	Drg. Ref.		
Calculation: Crane loading	Made by	Date: 17/10/2024	Chd.

Introduction Calculation to determine crane types required for all lifting operations

Weights to lift

TBM	95	tonnes	Machine can, major component - see Preliminary Construction Execution Plan (NL-ARM-040-ARU1-110315)
One 3.0mID tunnel ring	5.3	tonnes	FP McCann data sheet
One d-wall reinforcement cage	20	tonnes	

Type	Lifting operation and capacity	Required lifting capacity	Crane SWL	
Mobile crane	For lifting TBM and d-wall reinforcement cages	TBM =95 tonnes (assumed could be lifted in 2 pieces):	Assume Liebherr LTM 1300-6.2 (6 axle) - 96 tonne	600x600mm patch loads
			18m from shaft: SWL = ~43.8t	96t + 95 tonnes /4 = 48t per patch
Rail mounted gantry crane	For lifting pipe segments	One 3.0mID tunnel ring: 5.3 tonnes	Liebherr Single: 40.6-50t, Twin: 50-65t – no info about load application?	3x3m patch load
		Shaft reinforcement panels: 20 tonnes		



Patch loads to be converted to stresses on the shaft by geotechnical team.

B.2. Diaphragm wall design calculations

ARUP	Job No.	Sheet No.	Rev.
	303154-00	1 of 1	0
	Member/Location		
Job Title: Aramis Launch Shaft	Drg. Ref.		
Calculation: Flexural - Diaphragm wall 0-5m	Made by	Date: 31/05/2024	Chd.

Introduction A calculation to determine the minimum requirements for flexural reinforcement, using Oasys GSA and Adsec

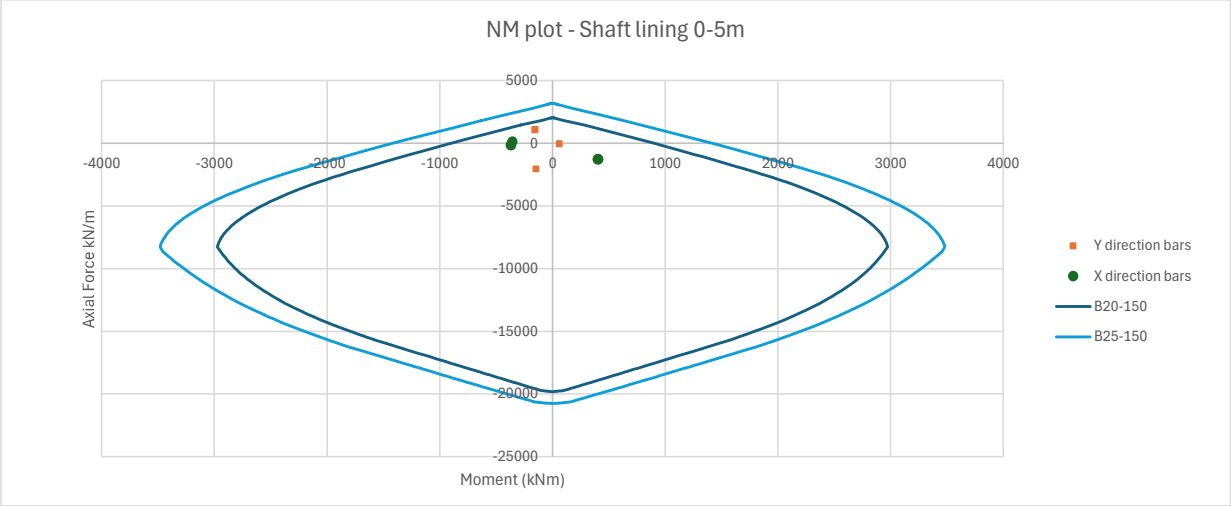
Oasys GSA

				[kNm/m]	[kN/m]	
	Elem	**Case**	**Node**	**My + sgn(My)|Myx|**	**Ny**	
B : y-direction	60	A17	119	59.88	-51.8	*Max moment with coexisting force*
	395	A17	395	-155.7	1082	*Min moment with coexisting force*
	395	A17	395	-155.7	1082	*Max axial T(+ve) with coexisting M*
	507	A17	507	-144.9	-2051	*Min axial C (-ve) with coexisting M*

-ve downwards = sagging moment, so rebar at bottom
+ve upwards = hogging moment, so rebar at top

				[kNm/m]	[kN/m]	
	Elem	**Case**	**Node**	**Mx + sgn(Mx)|Mxy|**	**Nx**	
A : x-direction	1119	A15	1231	404.5	-1271	*Max moment with coexisting force*
	251	A15	251	-368.4	-146.8	*Min moment with coexisting force*
	83	A15	168	-356	146.3	*Max axial T(+ve) with coexisting M*
	1009	A15	1121	402.7	-1275	*Min axial C (-ve) with coexisting M*

-ve downwards = sagging moment, so rebar at bottom
+ve upwards = hogging moment, so rebar at top



Minimum reinforcement required for flexural B20-150

ARUP	Job No.	Sheet No.	Rev.
	303154-00	1 of 1	0
	Member/Location		
Job Title: Aramis Launch Shaft	Drg. Ref.		
Calculation: Flexural - Diaphragm wall 5-10m	Made by [redacted]	Date: 31/05/2024	Chd.

Introduction A calculation to determine the minimum requirements for flexural reinforcement, using Oasys GSA and Adsec

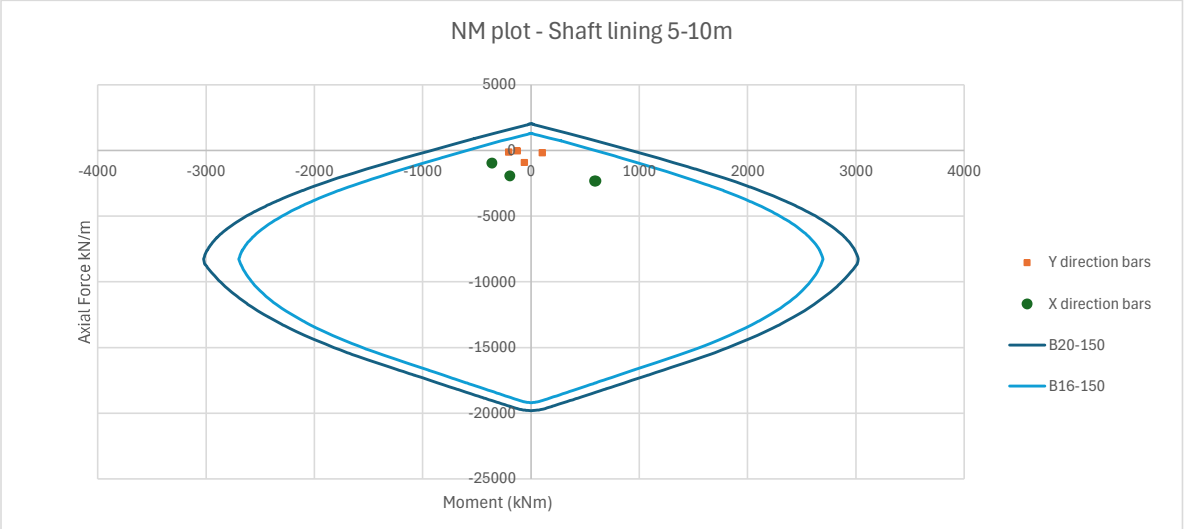
Oasys GSA

	Elem	Case	Node	[kNm/m] My + sgn(My)|Myx|	[kN/m] Ny	
B : y-direction	2231	A15	2343	105	-165.9	*Max moment with coexisting force*
	2236	A15	2349	-205.6	-123.7	*Min moment with coexisting force*
	2183	A15	2296	-124.7	-42.51	*Max axial T(+ve) with coexisting M*
	2215	A15	2215	-59.24	-912.9	*Min axial C (-ve) with coexisting M*

-ve downwards = sagging moment, so rebar at bottom
+ve upwards = hogging moment, so rebar at top

	Elem	Case	Node	[kNm/m] Mx + sgn(Mx)|Mxy|	[kN/m] Nx	
A : x-direction	2130	A15	2243	598.8	-2304	*Max moment with coexisting force*
	1146	A15	1146	-360.6	-943.1	*Min moment with coexisting force*
	2226	A15	2339	-194.6	-1914	*Max axial T(+ve) with coexisting M*
	2129	A15	2241	585.7	-2310	*Min axial C (-ve) with coexisting M*

-ve downwards = sagging moment, so rebar at bottom
+ve upwards = hogging moment, so rebar at top



Minimum reinforcement required for flexural B16-150

ARUP	Job No.	Sheet No.	Rev.
	303154-00	1 of 1	0
	Member/Location		
Job Title: Aramis Launch Shaft	Drg. Ref.		
Calculation: Flexural - Diaphragm wall 10-15m	Made by	Date: 31/05/2024	Chd.

Introduction A calculation to determine the minimum requirements for flexural reinforcement, using Oasys GSA and Adsec

Oasys GSA

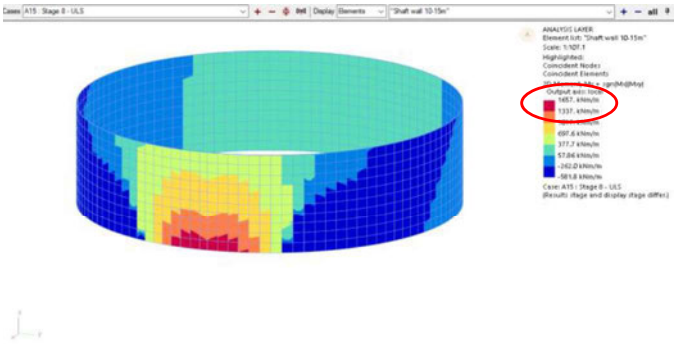
	Elem	Case	Node	[kNm/m] My + sgn(My)|Myx|	[kN/m] Ny	
B : y-direction	3355	A15	3467	617.6	223.4	*Max moment with coexisting force*
	3132	A15	3245	-573.4	184.6	*Min moment with coexisting force*
	3252	A15	3252	460.1	427.2	*Max axial T(+ve) with coexisting M*
	3341	A15	3341	-389.4	-1270	*Min axial C (-ve) with coexisting M*

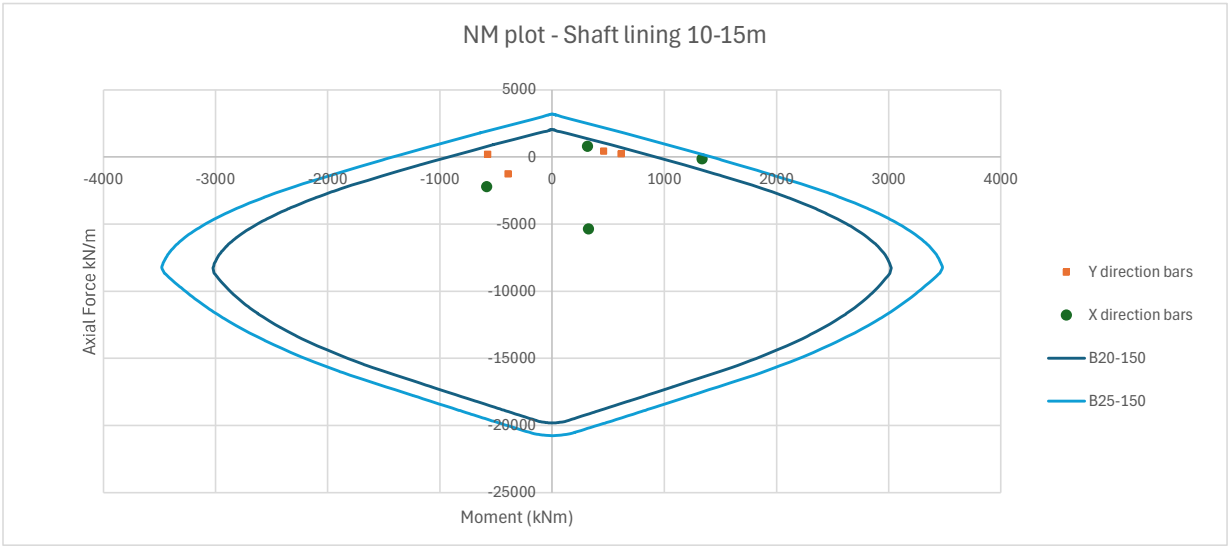
-ve downwards = sagging moment, so rebar at bottom
+ve upwards = hogging moment, so rebar at top

	Elem	Case	Node	[kNm/m] Mx + sgn(Mx)|Mxy|	[kN/m] Nx	
A : x-direction	3358	A15	3470	1337	-152.7	*Max moment with coexisting force*
	3346	A15	3458	-581.8	-2215	*Min moment with coexisting force*
	3304	A15	3417	314.8	804.6	*Max axial T(+ve) with coexisting M*
	5777	A11	3472	325.4	-5365	*Min axial C (-ve) with coexisting M*

-ve downwards = sagging moment, so rebar at bottom
+ve upwards = hogging moment, so rebar at top

Mx spike of 1657kNm/m discounted as localised around tunnel adit
Mx taken as 1337kNm/m





Minimum reinforcement required for flexural B25-150

ARUP	Job No.	Sheet No.	Rev.
	303154-00	1 of 1	0
	Member/Location		
Job Title: Aramis Launch Shaft	Drg. Ref.		
Calculation: Flexural - Diaphragm wall 15-20m	Made by	Date: 31/05/2024	Chd.

Introduction A calculation to determine the minimum requirements for flexural reinforcement, using Oasys GSA and Adsec

Oasys GSA

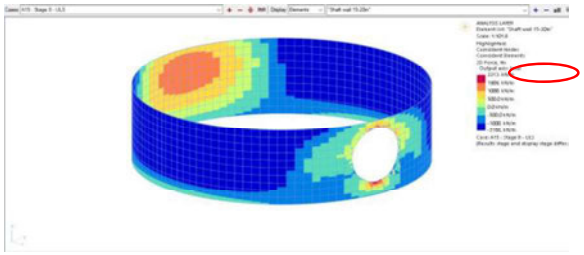
				[kNm/m]	[kN/m]	
	Elem	Case	Node	My + sgn(My)|Mxy|	Ny	
B : y-direction	6355	A15	6341	981.2	859.1	*Max moment with coexisting force*
	6368	A15	5904	-800	1095	*Min moment with coexisting force*
	6278	A15	5896	981.2	560.7	*Max axial T(+ve) with coexisting M*
	6345	A11	6341	398.6	-6306	*Min axial C (-ve) with coexisting M*

-ve downwards = sagging moment, so rebar at bottom
+ve upwards = hogging moment, so rebar at top

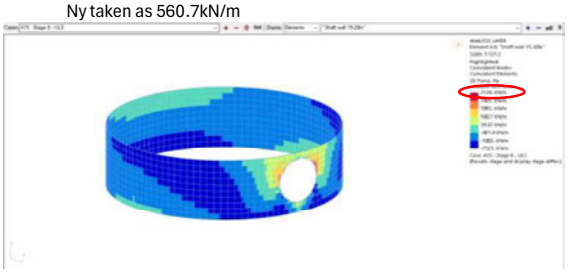
				[kNm/m]	[kN/m]	
	Elem	Case	Node	Mx + sgn(Mx)|Mxy|	Nx	
A : x-direction	6353	A15	6463	937.8	500	*Max moment with coexisting force*
	6368	A15	5901	-819	338.8	*Min moment with coexisting force*
	6267	A15	5909	937.8	500	*Max axial T(+ve) with coexisting M*
	6354	A11	6460	2265	-9537	*Min axial C (-ve) with coexisting M*

-ve downwards = sagging moment, so rebar at bottom
+ve upwards = hogging moment, so rebar at top

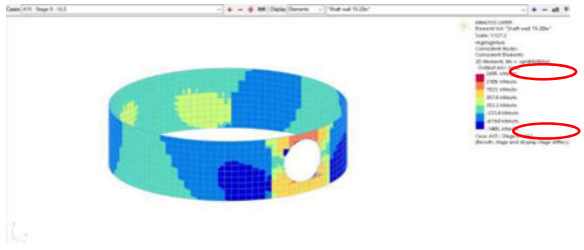
Nx spike of 2213kN/m discounted as localised around tunnel adit
Nx taken as 500kN/m



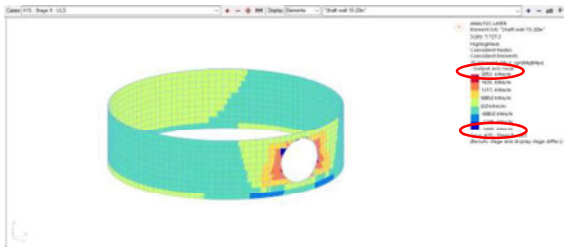
Ny spike of 2124kN/m discounted as localised around tunnel adit
Ny taken as 560.7kN/m

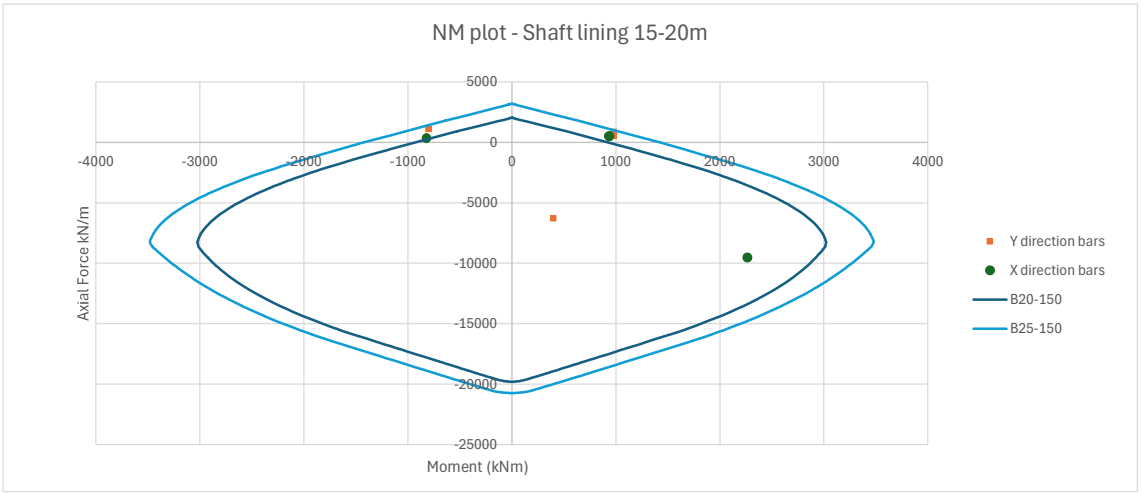


Mx spikes of 2695kNm/m and -1405kNm/m discounted
Mx taken as 937.8kNm/m and -819kNm/m



My spikes of 2052kNm/m and -1697kNm/m discounted
My taken as 981.2kNm/m and -800kNm/m





Minimum reinforcement required for flexural B25-150

ARUP	Job No.	Sheet No.	Rev.
	303154-00	1 of 1	0
	Member/Location		
Job Title: Aramis Launch Shaft	Drg. Ref.		
Calculation: Flexural - Diaphragm wall 20-26m	Made by	Date: 31/05/2024	Chd.

Introduction A calculation to determine the minimum requirements for flexural reinforcement, using Oasys GSA and Adsec

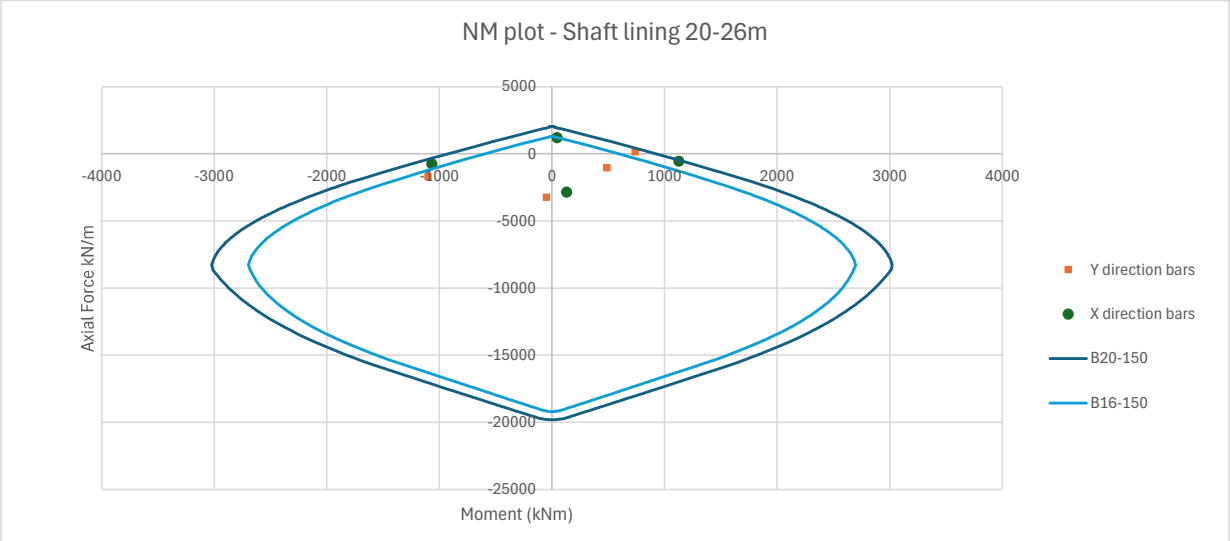
Oasys GSA

				[kNm/m]	[kN/m]	
	Elem	**Case**	**Node**	**My + sgn(My)|Myx|**	**Ny**	
B : y-direction	4779	A15	4796	739.8	156.3	*Max moment with coexisting force*
	4535	A15	4552	-1099	-1689	*Min moment with coexisting force*
	4519	A15	4535	490.1	-1029	*Max axial T(+ve) with coexisting M*
	5762	A15	5781	-47.11	-3238	*Min axial C (-ve) with coexisting M*

-ve downwards = sagging moment, so rebar at bottom
+ve upwards = hogging moment, so rebar at top

				[kNm/m]	[kN/m]	
	Elem	**Case**	**Node**	**Mx + sgn(Mx)|Mxy|**	**Nx**	
A : x-direction	4528	A15	4544	1127	-548.2	*Max moment with coexisting force*
	4521	A15	4538	-1067	-756.9	*Min moment with coexisting force*
	4779	A15	4909	45.47	1219	*Max axial T(+ve) with coexisting M*
	4419	A11	4433	130	-2840	*Min axial C (-ve) with coexisting M*

-ve downwards = sagging moment, so rebar at bottom
+ve upwards = hogging moment, so rebar at top



Minimum reinforcement required for flexural B20-150

ARUP	Job No.	Sheet No.	Rev.
	303154-00	1 of 1	0
	Member/Location		
Job Title: Aramis Launch Shaft	Drg. Ref.		
Calculation: Shear - RC diaphragm wall	Made by	Date: 31/05/2024	Chd.

Source/Ref

Checks to undertake:

1. Material properties
2. Minimum reinforcement requirements
3. Shear Capacity

1. Material properties

Tensile strength of steel rebar, fyk (assume B500)	f_{yk} =	500	N/mm^2
Partial factor for steel, γ_s		1.15	
$f_{ywd} = f_{yk}/\gamma_s$	fywd =	435	N/mm^2
Compressive strength of concrete, fck	f_{ck} =	32	N/mm^2
	$f_{ck,cub}$ =	40	N/mm^2
Partial factor for concrete, γ_c		1.5	
a_{cw}		1	NA to Cl 3.1.6
$f_{cd} = a_{cw}f_{ck}/\gamma_c$		21	N/mm^2

2. Minimum reinforcement requirements

Eurocode 2 part 1-1, 9.2.1.1 (1)

Minimum a

C32/40

fctm	3	Mpa
fyk	500	MPa

Element	Depth	$A_{s,min} = 0.26\frac{f_{ctm}b_t d}{f_{yk}}$	*A s,min* = 0.0013btd	*A s,max* = 0.04Ac
	mm	mm2 / m	mm2 / m	mm2/m
D-wall	1000	1560	1300	400000

Bar diamete	20	mm
Area	314	mm2
No. bars	5	
Spacing	201	

3. Shear Capacity in diaphragm wall

Check if shear reinforcement is required:

EC1992-1-1 Cl 6.2.2

$$V_{Rd,c} = [C_{Rd,c}k(100\,\rho_l f_{ck})^{1/3} + k_1\,\sigma_{cp}]\,b_w d \qquad (6.2.a)$$

with a minimum of

$$V_{Rd,c} = (v_{min} + k_1\sigma_{cp})\,b_w d \qquad (6.2.b)$$

where:

f_{ck} is in MPa

$k \quad = 1+\sqrt{\frac{200}{d}} \le 2{,}0$ with d in mm

$\rho_l \quad = \frac{A_{sl}}{b_w d} \le 0{,}02$

A_{sl} is the area of the tensile reinforcement, which extends $\ge (l_{bd} + d)$ beyond the section considered (see Figure 6.3).

b_w is the smallest width of the cross-section in the tensile area [mm]

σ_{cp} $= N_{Ed}/A_c < 0{,}2\,f_{cd}$ [MPa]

N_{Ed} is the axial force in the cross-section due to loading or prestressing [in N] (N_{Ed}>0 for compression). The influence of imposed deformations on N_{Ed} may be ignored.

A_C is the area of concrete cross section [mm^2]

$V_{Rd,c}$ is [N]

where

f_{ck} =	32	N/mm^2	
d =	1000	mm	
k = 1+(200/d)0.5	1.45		<2.0 ∴ OK
b_w =	1000	mm	per m strip
A_{sl} =	3436	mm^2	Assuming B25 bars at 150mm spacing
ρ_1 =Asl/bwd	0.0034		<0.02 ∴ OK
σ_{cp} =	0.0	N/mm^2	Assumed
$C_{Rd,c}$ = 0.18/γc	0.12		
v_{min} =	0.34	N/mm^2	$v_{min} = 0{,}035\,k^{3/2}\cdot f_{ck}^{1/2}$
k_1 =	0.15		

Design she: $V_{Rd,c}$ =	386177	N	(6.2a)		
	386	kN	**USE THIS AS BIGGER**		
$V_{Rd,cmin}$ =	344701	N	(6.2b)		
	345	kN			
Max V_{Ed} 0-5m	517.1	kN	From GSA	1000	taken as max. Ved ignoring spikes
5-10m	112.7	kN	From GSA	kN	
10-15m	927.2	kN	From GSA		
15-20m	1988	kN	From GSA		
20-26m	858.2	kN	From GSA		

As VEd > VRd,c shear reinforcement in the slab is required

Check if maximum allowable shear force is exceeded — EC1992-1-1 Cl 6.2.2

The shear force V_{Ed}, calculated without reduction by β, should however always satisfy the condition

$$V_{Ed} \le 0{,}5\, b_w d\, \nu f_{cd} \qquad (6.5)$$

where ν is a strength reduction factor for concrete cracked in shear

Note: The value ν for use in a Country may be found in its National Annex. The recommended value follows from:

$$\nu = 0{,}6\left[1 - \frac{f_{ck}}{250}\right] \quad (f_{ck} \text{ in MPa}) \qquad (6.6N)$$

where

v = 0.523 N/mm²

$V_{Ed,max}$ =	5580800	N
	5581	kN

As VEd < VEd,max shear in the slab is acceptable

If shear reinforcement required:

Shear resistance will be calculated in accordance with BS EN 1992-1-1:2004 clause 6.2.3.

$V_{Rd,s}$ = $[A_{sw} / s] * z * f_{ywd} * \cot\theta$

				outer bar	inner bar
A_{sw} =	402	mm²	Link dia =	16	
Spacing, s =	150	mm	No. legs =	2	
Lever arm, z =	900	mm (0.9d)			
f_{ywd} =	434.8	N/mm²			
θ =	45	degrees	≡	0.785	radians
α =	45	degrees	≡	0.785	radians

∴ $V_{Rd,s}$ = 1,049 kN

$V_{Rd,max}$ = $\alpha_{cw} * b_w * z * \nu_1 * f_{cd} / (\cot\theta + \tan\theta)$

α_{cw} =	1.0	(Non-prestressed)			
b_w =	1000.0	mm			
f_{ck} =	32.0	N/mm²	f_{cd} =	21.3	N/mm²
ν_1 = v =		0.5232 N/mm²			

∴ $V_{Rd,max}$ = 5,023 kN

$V_{Ed} < V_{Rd,s} < V_{Rd,max}$

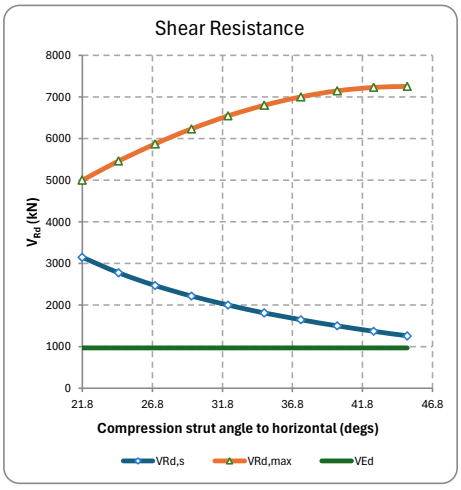
$A_{sw,max}$ = $0.5 * \alpha_{cw} * \nu_1 * f_{cd} * b_w * s / f_{ywd}$ for θ = 45.0 degrees

∴ $A_{sw,max}$ = 1925.4 mm² ---

θ (degs)	$V_{Rd,s}$ (kN)	$V_{Rd,max}$ (kN)	ΔF_{td} (kN)
21.8	2623	3464	1250
24.4	2313	3779	1102
27.0	2059	4063	981
29.6	1847	4314	880
32.2	1666	4530	794
34.8	1509	4708	719
37.4	1372	4847	654
40.0	1250	4946	596
42.6	1141	5005	544
45.0	1049	5023	500

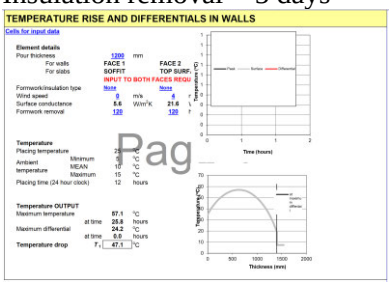
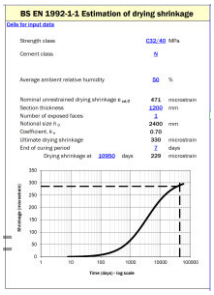
V_{Ed} = 1000 kN

$UR_{concrete}$=	0.20	
	OK	
UR_{links}=	0.95	
	OK	



21.8	1,000
45.0	1,000
2.6	step

RC Diaphragm Wall - Early Thermal Checks Summary

	RC Diaphragm wall **Assume: edge restraint, as restrained by soil behind**
Temperature Risk and Differentials in walls	Pour thickness = 1200mm Insulation type (top surface) = none Insulation type (bottom surface) = none Wind speed= 0m/s (inside shaft, effects of wind will be minimal) Insulation removal = 5 days 
Restraint	Edge restraint from the soil creates vertical tension, therefore cracking will be controlled by the vertical reinforcement R=0.8 (max possible restraint taking into account redistribution). No reinforcement between the soil and d-walls, the edge restraint factor is taken as **<u>0.7</u>** Long term restraint – generally reduces so assume an R2 of **0.5** for all pours
Drying shrinkage	229 microstrain (For long-term cracking) Design life = 30 year = 10950 days 
Crack width	Concrete strength = C32/40 Bar diameter = 32mm Bar spacing = 150mm Cover = 75mm top / 75mm bottom Aggregate type: Ecm kept as default Coefficient of thermal expansion of concrete = 0.5με/oC (table 4.6 C766) Early age crack width = 0.16mm < 0.3mm (Section 7.3.3, NEN EN 1992-1-1) Long term crack width = 0.25mm < 0.3mm (Section 7.3.3, NEN EN 1992-1-1) **<u>Also checked for 40mm diameter bars at 150mm spacing:</u>** Early age crack width = 0.14mm < 0.15mm (NEN EN 1992-3 Water Tightness Class 1 Criteria) Long term crack width = 0.22mm < 0.3mm (Section 7.3.3, NEN EN 1992-1-1)

C766 Temperature prediction in walls and slabs

This model provides a basis for the comparative assessment of early-age temperature rise and temperature differentials in concrete walls, suspended slabs and ground slabs.

The model is based on the numerical method of Ross and Bray (1949) and is described in detail in Appendix A2

Background data on heat generation were provided by the University of Dundee (Dhir *et al*, 2006) obtained using semi-adiabatic testing to BS EN 196-9:2003

Data for the concrete are input on PAGE 2

Data specific to walls and suspended slabs are input on PAGE 3

Data specific to ground slabs are input on PAGE 4

Each page of this spreadsheet is password protected to prevent accidental deletion of equations. For those wishing to modify or copy the spreadsheet, the password is C766

PAGE 2 ADIABATIC TEMPERATURE			
Concrete mix details	Binder content	kg/m^3	The binder is the total content of cementitious material, including CEM I and CEM I in combination with either fly ash or ground granulated blast-furnace slag, ggbs
	Binder type		Three binder options are available from a drop-down menu CEM I, fly ash, ggbs
	Percent addition	%	The percent of either fly ash or ggbs by weight of binder
	Density	kg/m^3	The wet density of the concrete
	Specific heat	kJ/kg°C	The early-age value when the concrete is only partially hydrated. A value of 1 kJ/kg°C should be used unless more reliable data are available
	Thermal conductivity	W/m°C	Assumed to be 2.0 W/m°C (see Appendix A2 for values for concrete using difference aggregates)
	Multiplier		If results are available for a specific mix, the multiplier may be used to adjust the adiabatic curve to match the data. The default value is 1,
Temperature input	Placing temperature	°C	The temperature of the fresh concrete when poured. As a general rule, the placing temeprature be 5°C above the mean ambinebt temeprature.
	Minimum ambient temperature	°C	These values determine the variable ambient temperature which is assumed to vary on a sinusoidal basis. As a eneral rule the ambient temeprature will vary by ±5°C from the mean value.
	Mean ambient temperature	°C	
	Maximmum ambient temperatur	°C	
	Constant or variable ambient temperature		There is the option to assume either a variable sinusoidal ambient temperature variation or a constant ambient temeprature variation. In the latter case this is assumed to be the MEAN ambient temperature,
	Pacing time	Hours	This assumes a 24 hour clock
Output	Temperature drop T_1		Values are calculated in PAGE 3 and PAGE 4 but are shown on PAGE 2 to enable the effect of changes in the concrete mix and placing temperature to be observed without changing pages. Input on section geometry, formwork etc, is on PAGE 3 and PAGE 4
GRAPHS	Reference curve for the cement		The reference adiabatic curve is estimated from the cement type and content and represents the value at a starting temperature of 20°C
	Adiabatic temperature rise for the concrete		The estimated adiabatic temperature rise is shown based on the reference curve at 20°C and the value obtained when a different placing temperature is used.
PAGE 3 TEMPERATURE CALCULATIONS FOR WALLS AND SUSPENDED SLABS			
Section and formwork details	Section thickness	mm	
	Formwork/insulation type		Formwork/insulation options are available for both faces from drop-down menus. Information for each face must be input. REMEMBER TO INPUT FOR BOTH FACES
	Wind speed	m/s	Assumed to be 4 m/s for typical UK conditions REMEMBER TO INPUT FOR BOTH FACES
	Formwork removal time	hours	The time after casting. Input must be made for both faces. There is the options to remove formwork from opposite faces at different times REMEMBER TO INPUT FOR BOTH FACES
Concrete mix details	Thermal conductivity	W/m°C	Copied from PAGE 2
	Density and specific heat		Copied from PAGE 2
PAGE 4 TEMPERATURE CALCULATIONS FOR GROUND SLABS			
Section and formwork details	Section thickness	mm	This is the minimum dimension through which heat loss is predominant
	Insulation type		Options are available from a drop-down menu for both the soffit formwork and insulation applied to the top surface (if required).
	Wind speed	m/s	Assumed to be 4 m/s for typical UK conditions
	Formwork removal time	hours	The time after casting. Input must be made for both the soffit and the top surface. There is the options to remove opposite faces at different times
Concrete mix details	Thermal conductivity	W/m°C	Copied from PAGE 2
	Density and specific heat		Copied from PAGE 2
Properties of the ground	Specific heat	kJ/kg°C	Default value is 1.2 kJ/kg°C
	Density	kg/m^3	Default value is 2000 kJ/kg°C
	Thermal conductivity	W/m°C	Default value is 1.1 W/m°C
	Temperature	°C	

TEMPERATURE RISE AND DIFFERENTIALS IN WALLS

Cells for input data

Element details

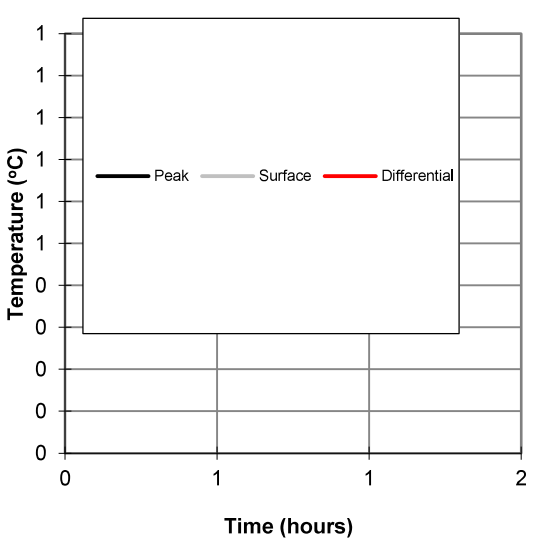
Pour thickness		1200	mm	
For walls		FACE 1		FACE 2
For slabs		SOFFIT		TOP SURF
		INPUT TO BOTH FACES REQU		
Formwork/insulation type		None		None
Wind speed		0	m/s	4
Surface conductance		5.6	W/m^{2}K	21.6
Formwork removal		120		120

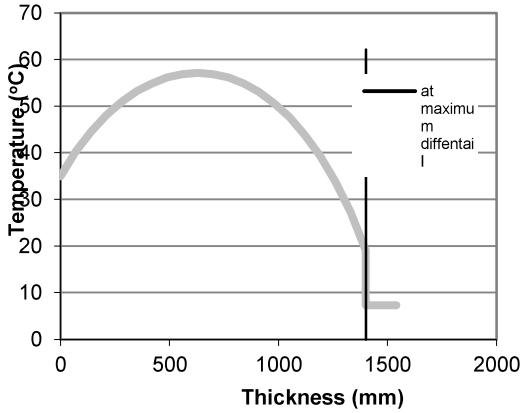
Temperature

Placing temperature		25	°C
Ambient temperature	Minimum	5	°C
	MEAN	10	°C
	Maximum	15	°C
Placing time (24 hour clock)		12	hours

Temperature OUTPUT

Maximum temperature		57.1	°C
	at time	25.8	hours
Maximum differential		24.2	°C
	at time	0.0	hours
Temperature drop	T_1	47.1	°C





C766 Estimation of drying shrinkage according to BS EN 1992-1-1

This calculator estimates the drying shrinkage using the expressions and relationships given in BS EN 1992-1-1, Section 3.1.4

The method is described in detail in Appendix A3

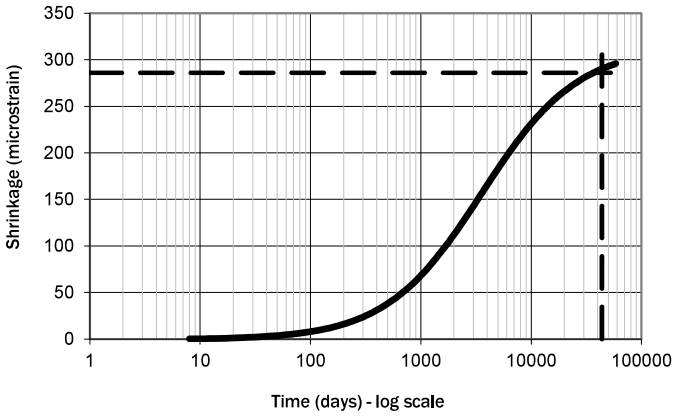
Each page of this spreadsheet is password protected to prevent accidental deletion of equations. For those wishing to modify or copy the spreadsheet, the password is C766

PAGE 2 DRYING SHRINKAGE			
Concrete mix details	Strength class		This is selected from a drop-down menu
	Cement class		Three options are available from a drop-down menu: **S (slow), N (normal) and R (rapid)**
Ambient conditions	Relative humidity	%	The average relative humidity over the period considered
Nominal unrestrained drying shrinkage $\varepsilon_{cd,0}$			The ultimate drying shrinkage achievable for the combination of concrete type and relative humidity
Section geometry	Section thickness	mm	
	Exposed faces		In most cases drying will be from either one of two of the faces. These options are given
	Notional thickness h_0	mm	The notional thickness $h_0 = A_c/u$ where A_c is the concrete cross sectional area and u is the perimeter of that part of the concrete that is exposed to drying. For walls or slabs the thickness h is normally small in relation to the other dimensions and h_0 approximates to h when drying is from both faces and $2h$ when drying is from one face only.
	Coefficient k_h		k_h is a function of h_0 and is given in Table 3.3 of BS EN 1992-1-1
Ultimate drying shrinkage = $k_h\ \varepsilon_{cd,0}$			The ultimate drying shrinkage achievable for the combination of concrete type, relative humidity and notional section thickness
	End of curing period t_s	days	It is assumed that shrinkage begins at the end of the curing period
Drying shrinkage at time t, $\varepsilon_{cd}(t) = \beta ds(t,t_s)\ k_h\ \varepsilon_{cd,0}$			The time function $\beta_{ds}(t, t_s) = (t - t_s)/\{(t - t_s) + 0.04\sqrt{h_0}^3\}$. BS EN 1992-1-1 notes that the estimated value of drying shrinkage has an expected coefficient of variation of 30%

BS EN 1992-1-1 Estimation of drying shrinkage

Cells for input data

Strength class	C32/40	MPa
Cement class	N	
Average ambient relative humidity	50	%
Nominal unrestrained drying shrinkage $\varepsilon_{cd,0}$	471	microstrain
Section thickness	1200	mm
Number of exposed faces	1	
Notional size h_0	2400	mm
Coefficient, k_h	0.70	
Ultimate drying shrinkage	330	microstrain
End of curing period	7	days
Drying shrinkage at 10950 days	229	microstrain



C766 Estimation of restraint

This calculator estimates restraint in a wall cast onto a rigid based. The method is based of the ACI approach in which the restraint at the joint is determined by the relative stiffness of the new section and the section against which it is cast. The variation in restraint with height is determined by the length to height ratio.

The method is described in detail in Appendix A5

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PAGE 2 RESTRAINT	
Wall geometry	The wall geometry is define by the length L , the height H and the thickness, h. When a new pour that is restrained along one edge abuts an exisiting pour at one end then for the purpose of calculating the restraint profile, the length of the edge restraaining joint should be assumed to be 2 x L, ie it is assumed that there is zero displacement at the joint between the new pour and the adjacent pour
Base geometry	The base geometry is defined by the width W and the thickness, h . The length is assumed to be the same as the length of wall being cast
Relative areas	The ratio of the cross-sectional area (c.s.a.) of the wall A_{new} and the base A_{old} is calculated. The relative areas of influence A_{old} and A_{new} may be difficult to define. For example, when a wall is cast onto a slab, what width of slab should be considered as providing effective restraint? In such circumstances it is recommended that the relative area is assumed to be in proportion to the relative thicknesses, h_{new} and h_{old} , of the wall and slab. The following simple rules may be applied: • For a high wall cast at the edge of a slab assume $A_{new}/A_{old} = h_{new}/h_{old}$ • For a wall cast remote from the edge of a slab assume $A_{new}/A_{old} = h_{new}/2h_{old}$ • For a slab cast against an existing slab assume $A_{new}/A_{old} = h_{new}/h_{old}$
Relative modulus of elasticity	Bamforth and Price (1995) recommends that the ratio E_{new}/E_{old} is assume to be in the range 0.7 to 0.8 at early age. The lower value gives most conservative result and should be used for conditions in which cool-down is most rapid. When estimating restraint to long term deformations, should there be any differential strains, it should be assumed that $E_{new}/E_{old} = 1$
Restraint at the joint R_j	$$R_j = \frac{1}{1 + \frac{A_{new}}{A_{old}} \frac{E_{new}}{E_{old}}}$$
Restraint at the location of the maximum crack width	The maximum restrained strain and crack width occur at a perpendicular distance from the joint of 0.1 times the length of the joint if the new pour is not cast against existing concrete at either end. If the new pour abuts an exisitng pour at one end then the maximum restrained strain and crack width occur at a perpendicular distance from the joint of 0.2 times the length of the joint

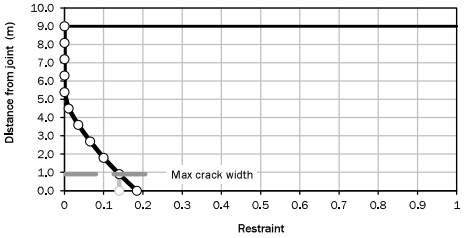
Estimation of restraint

Wall on a rigid base

Cells for input data

Dimensions	New Concrete Element			Old Concrete Element	
Length	16.70	m			
Height	16.70	m	Width	26.44	m
Thickness	1.00	m	Thickness	0.25	m
c.s.a. A_n	16.7	m^2	c.s.a A_o	6.61	m^2
L/H	1		Centroid	6.02	m
Ratio of areas		A_n/A_o	4.00		
Ratio of moduli		E_n/E_o	0.92		

Restraint factor at joint R_j	0.21	
Height of maximum crack width	1.67	m
Restraint at maximum crack width R_{wmax}	0.16	



C766 Predicting the risk of cracking and controlling crack widths

This calculator provides a basis for estimating the likelihood of cracking and for selecting or checking reinforcement to control crack widths

The approach is based on the method of BS EN 1992-1-1 with adjustments where considered appropriate (see Appendix A8)

Three conditions of restraint are dealt with

PAGE 2 Continuous EDGE restraint.

PAGE 3 END restraint

PAGE 4 INTERNAL restraint from temperature differentials

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Input data are as follows (all other parameters are calculated):

PAGE 2 CONTINUOUS EDGE RESTRAINT			
Section details and reinforcement	Section thickness	mm	Section thickness in mm
	Strength class	f_{ck} / $f_{ck,cube}$	Select from the drop-down menu
	Bar diameter	φ	Bar diameter in mm
	Bar spacing	S	Bar spacing in mm
	Cover to reinforcement	c	Minimum cover to reinforcement
	Yield strength of reinforcement	f_{yk}	The characteristic yield strength of the reinforcement in MPa
Early-age restrained strain	Coefficient of expansion	α_c	A value of 12 με / °C is recommended if there is no knowledge of the aggregate type. The recommended value in BS EN 1992-1-1 is 10 με / °C but many aggregates in the UK produce concrete with higher values
	Temperature drop	T_1	This may be obtained from data in C660, the temperature model (Appendix A2) or by independent validated modelling or measurement.
	Creep factor	Kc_1	BS EN 1992-1-1 does not define a creep factor but includes this in the value of restraint R = 0.5 which is the maximum value recommended by BS EN 1992-1-1 and so must cover the worst condition of restraint. When this value of R is used, K_1 should be assumed to be 1. When R is calculated K_1 may be assumed to be 0.65
	Restraint at the location of the maximum crack width	R	BS EN 1992-1-1 permits R to be calculated from the relative stiffness of the element and the member against which it is cast. Where it can be demonstrated that R < 0.8 the calculated value may be used with K_1 = 0.65. The the maximum restrained strain occurs at a distance of 0.1 x length of the wall above the joint with the base. When the wall abuts a previously cast length, the assumed length is calculating the location of the maximum restrained strain is double the actual length.
Risk of early-age cracking	Age at cracking	days	Assumed to be 3 days for early age cracking unless more reliable information is available
	Aggregate type		The tensile strain capacity is the ratio of the tensile strength to the elastic modulus and the latter is affected by the aggregate type. If the aggregate source is known this can be selected from the drop down menu. The default value for unknown aggregate is 1. This is the value dervied from the properties calculated according to BS EN 1992-1-1 for quartz aggregate. Where the elastic modulusat 28 days is known this may be entered and 'measured' should be selected from the drop down menu.
Minimum area of reimforcement $A_{s,min}$	Coefficient for effect edge restraint (Optional)	$k_{R,edge}$	A drop down menu (YES/NO)provides the option to take account of the contributon of the edge restraint in limiting crack opening and the amount of steel required to ensure that yield does not occur.
Crack spacing	Factor for bond	k_1	BS EN 1992-1-1 recommends k_1 = 0.8 but provides a factor of 0.7 where good bond cannot be guaranteed. k_1 = 0.8/0.7 = 1.14 for section >300 mm thick with cover < 50 mm. If cracking is not predicted at early age, then k_1 = 0.8.
Long term cracking	Age at cracking	days	For calculating long term cracking the 28-day properties of the concrete are adopted.
	Long term temperature change	$T2$	Where both the restrained and the restraining element are subject to a change in temperature, T2 is the differential change in temperature between the restrained and the restraining element unless there is external restraint to both elements.
	Drying shrinkage	ε_{cd}	Calculate using the method of BS EN 1992-1-1 unless more reliable information is available. Only apply when causing differential contraction between elements or when the elements acting integrally are subject to external restraint. The drying shrinkage is that occurring at the end of the design life of the structure or part thereof.
	Creep factor	K_{C2}	When restraint is calculated, the long-term creep coefficient is taken as 0.5
	Restraint to T_2	R_2	Long term restraint will be reduced as the ratio of stiffness of the elements reduces and E_n / E_o approaches 1. Consideration must also be given to the element acting integrally.
	Restraint to drying shrinkage	R_3	
Crack spacing	Coefficient for bond characteristics	k_1	If cracking has occurred at early age, the same carack spacing is assumed for long-term cracking. If cracking has not occurred at early age, the crack spacing is recalculated assuming that k_1 =0.8

PAGE 3 END RESTRAINT			
Section details	Section thickness	h	Section thickness in mm
	Strength class	f_{ck} / $f_{ck,cube}$	Select from the drop-down menu.
	Bar diameter	ϕ	Bar diameter in mm
	Bar spacing	s	Bar spacing in mm
	Cover	c	Minimum cover to reinforcement
Properties of reinforcement	Characteristic yield strength	f_{yk}	The characteristic yield strength of the reinforcement = 500 MPa
	Elastic modulus	E_s	200 GPa
Crack sacing	Coefficient for bond characteristics	k_1	See ref to k_1 above
Age at cracking	Early-age	*3 days*	Values of 3 days and 28 days are assumed for the estimation of early age and long-term properties. Values derived on this basis are have been shown to be safe in relation to the design of reinforcement for crack control
	Long term	*28 days*	
Early-age restrained strain	Coefficient of expansion	α_c	A value of 12 με / °C is recommended if there is no knowledge of the aggregate type. The recommended value in BS EN 1992-1-1 is 10 με / °C but many aggregates in the UK produce concrete with higher values
	Temperature drop	T_1	This may be obtained from data in C766, the temperature model (Appendix A2) or by independent validated modelling or measurement.
	Creep factor	Kc_1	BS EN 1992-1-1 does not define a creep factor but includes this in the value of restraint R = 0.5 which is the maximum value recommended by BS EN 1992-1-1 and must cover the worst condition of restraint. When this value of R is used, K_1 should be assumed to be 1. When R is calculated K_1 may be assumed to be 0.65
	Restraint	R	BS EN 1992-1-1 permits R to be calculated from the relative stiffness of the element and the member against which it is cast. Where it can be demonstrated that R < 0.8 the calculated value may be used with K_1 = 0.65. END restraint may be difficult to calculated, particulary when there is a combination of restraints acting on the member, eg a slab on piles with a combination of friction from the base and lateral resistance from piles. Some guidance is provided in Appendix A5, but the user may be required to undertake additional calculations where the arrangement of restraints is complex.
Risk of early-age cracking	Early-age	*Days*	Properties at 3 days are assumed for the estimation of early age cracking. However, when there is confience in the predicted time of cracking there is the option to use other ages.
	Aggregate type		The tensile strain capacity is the ratio of the tensile strength to the elastic modulus and the latter is affected by the aggregate type. If the aggregate source is known this can be selected from the drop down menu. The default value (coefficient = 1) is the value dervied from the properties calculated according to BS EN 1992-1-1 for quartz aggregate. Where the elastic modulus is known this may be entered and 'measured' should be selected from the drop down menu.
Long term cracking	Age at cracking	days	For calculating long-term cracking the 28-day properties of the concrete are adopted.
	Long term temperature change	*T2*	Where both the restrained and the restraining element are subject to a change in temperature, *T2* is the differential change in temperature between the restrained and the restraining element unless there is external restraint to both elements.
	Drying shrinkage	ε_{cd}	Calculate using the method of BS EN 1992-1-1 unless more reliable information is available.
	Creep factor	*Kc2*	
	Restraint to T_2	R_2	Long-term restraint will be reduced as the ratio of stiffness of the elements reduces and E_n / E_o approaches 1 and as elements act integrally.
	Restraint to drying shrinkage	R_3	
	Restraint required to cause cracking	R_{crack}	As restraint may be difficult to calculate, the restraint required to cause cracking is also estimated and the user may judge whether this level of restraint is likely to be achieved in practice.

PAGE 3 INTERNAL RESTRAINT
The input data for dealing with cracking caused by internal restraint is broadly the same as that required for edge restraint with the principal exception that the temperature differential between the centre and the surface ΔT replaces T_1. Other changes inherent in the design calculation are the restraint factor R assumed to be 0.42 and the coefficients k and kc which reflect the difference in the stress distribution within the section compared with the condition of external restraint.

Risk and control of cracking due to continuous edge restraint

Input parameters	Symbol	Unit	Value	Comments/guidance
Section details and reinforcement				
Section thickness	h	mm	1200	
Strength class	f_{ck} / $f_{ck,cube}$	MPa	C32/40	Select from drop down menu
Bar diameter	φ	mm	32	
Bar spacing	s	mm	150	
Cover	c	mm	100	Minimum cover to steel in the direction perpendicular to the crack
Characteristic yield strength of reinforcement	f_{yk}	MPa	500	500 MPa
Area of steel per face per m	A_s	mm^2	5362	
Early age cracking				
Restrained strain				
Coefficient of thermal expansion of concrete	α_c	με/°C	9.5	If aggregate is unknown use 12 με / °C. For limestone aggregate use table 4.6 C766
Temperature drop	T_1	°C	47.1	T_1 = Peak temperature - mean ambient temperature
Autogenous shrinkage	$\varepsilon_{ca}(ea)$	με	16	BS EN 1992-1-1 $\varepsilon_{ca}(ea) = 2.5 (f_{ck} - 10) \times (1-\exp(-0.2\, t_c^{0.5}))$
Free contraction	$\boldsymbol{\varepsilon_{free}(ea)}$	με	464	$\varepsilon_{free(ea)} = T_1 \alpha_c + \varepsilon_{ca}(ea)$
Creep factor - early age	K_{c1}		0.65	K_{c1} = 0.65 if R is calculated; K_{c1} = 1 if R is assumed to be 0.5 (including creep) according to BS EN 1992-1-1
Restraint	R_1		0.70	Use restraint calculator for walls or adjacent slabs; or historical data. The restrant at the point of maximum crack width is estimated at a distance of 0.1L from the joint, where L is the length of the joint
Early-age restrained contraction	$\boldsymbol{\varepsilon_r(ea)}$	με	211	$\varepsilon_r(ea) = R_1 K_{c1} (T_1 \alpha_c + \varepsilon_{ca}(ea))$
Risk of cracking and crack inducing strain				
Age at early cracking	t_{eac}	days	3	Assume 3 days unless more reliable information is available
Aggregate type. Use the 'selected' option and enter the E value if measured or obtained from historical results.	Limestone		0.90	Coefficient for stiffness of aggregate
			30	Enter value of E_{cm}(28) if 'measured' selected from menu for aggregate type
Elastic modulus	$E_{cm}(t_c)$	GPa	25.7	Mean value $E_{cm}(t_c)$. BS EN 1992-1-1 values are based on quartz aggregate
Tensile strength at cracking	$f_{ctk,0.05}(t_c)$	MPa	1.27	Characterisitic tensile strength $f_{ctk,0.05}(t_c)$
Tensile strain capacity	$\boldsymbol{\varepsilon_{ctu(ea)}}$	**με**	76	$\varepsilon_{ctu(ea)} = [f_{ctk,0.05}(t_{eac}) / [E_{cm}(t_{eac}) \times K_{c1}]$
Risk of early age cracking	$\varepsilon_{r(ea)}/\varepsilon_{ctu(ea)}$		2.79	Cracking predicted if $\varepsilon_{r(ea)}/\varepsilon_{ctu} > 1$.
Early-age crack-inducing strain	$\boldsymbol{\varepsilon_{cr(ea)}}$	με	173	$\varepsilon_{cr(ea)} = R_1 K_{c1} (T_1 \alpha_c + \varepsilon_{ca(ea)}) - 0.5\, \varepsilon_{ctu(ea)} - (\varepsilon_{sm} - \varepsilon_{cm})$
Minimum area of reinforcement $A_{s,min}$				
Coefficient (BS EN 1992-1-1)	k		0.65	k = 1.0 for $h \leq 300$ mm, k = 0.65 for $h \geq 800$ mm, intermediate values are interpolated
Coefficient (BS EN 1992-1-1)	k_c		1.00	For pure tension $k_c = 1$
Coefficient for effect of edge restraint (optional)	k_{Redge}	YES	0.65	Calculated as (1-0.5R_1) for R_1 at the location of the maximum crack width
Depth of surface zone representing A_{ct}	$h_{s,min}$	mm	390	$h_{s,min} = k\, k_c\, h/2$
Minimum area of steel per face per m	$A_{s,min}$	mm^2	642	$A_{s,min} = (k_{Redge}\, k\, k_c\, (f_{ctk,0.05}(t_c)/f_{yk})\, A_{ct}$ Highlighted if $A_s < A_{s,min}$
Crack spacing				
Depth of the surface zone defining the effective area of concrete in tension, $A_{c,eff}$	$h_{e,ef}$	mm	290	$h_{e,ef}$ = lesser of $h/2$ a nd 2.5 $(c + \varphi/2)$
Steel ratio for estimating crack spacing	$\rho_{p,eff}$		0.01849	$\rho_{p,eff} = A_s / A_{c,eff} = A_s / (h_{e,ef} \times 1000)$
Coefficient for bond characteristics.	k_1		0.8	BS EN 1992-1-1 recommends k_1 = 0.8, but provides a factor of 0.7 where good bond cannot be guaranteed. Recommended k_1 = 0.8/0.7 = 1.14 at early age for sections >300 mm thick with cover < 50 mm.
Crack spacing	$S_{r,max}$	mm	928	$S_{r,max} = 3.4c + 0.425\, k_1\, \varphi/\rho_{p,eff}$
Early age crack width	w_k	mm	0.16	$w_k = \varepsilon_{c(ea)} S_{r,max}$ (=0 if early-age cracking is not predicted)
Maximum stress in the reinforcement	σ_s	N/mm^2	84	$\sigma_s = 2(\varepsilon_{sm} - \varepsilon_{cm})E_s$
Long term cracking				
Long-term restrained strain				
Autogenous shrinkage at 28 days	$\boldsymbol{\delta\varepsilon_{ca}(lt)}$	με	36	
Long-term temperature change	T_2	°C	20	T_2 and ε_{cd} only apply when causing differential contraction between the restrained and the restraining member or when the members acting integrally are subject to external restraint. The drying shrinkage is that occurring at the end of the design life of the structure or part thereof.
Drying shrinkage	ε_{cd}	με	229	
Long-term free contraction	$\varepsilon_{free}(lt)$	με	902	$\alpha_c T_1 + \varepsilon_{ca}(lt) + \alpha_c T_2 + \varepsilon_{cd}$
Creep factor - long term	K_{c2}		0.50	K_{c2} = 0.5 if R is calculated, K_{c2} = 1 if R is assumed to be 0.5 (including creep to BS EN 1992-1-1)
Restraint to long-term thermal strains	R_2		0.50	Restraint of an individual member will reduce as E_n / E_o approaches 1 in the long term. However, consideration must be given to the fact that individually cast elements will act integrally in the partially or fully completed structure. R_2 and R_3 will generally have the same value
Restraint to drying shrinkage	R_3		0.50	

Long-term restrained strain	$\varepsilon_r(lt)$	με	336	$K_{c1}[\alpha_c T_1+\varepsilon_{ca(3)}]R_1+K_{c1}[(\varepsilon_{ca(28)}-\varepsilon_{ca(3)})+\alpha_c T_2]R_2+K_{c2}\varepsilon_{cd}R_3]$
Risk of cracking and crack-inducing strain				
Age at long term cracking	t_{ltc}	days	28	Assume 28 days unless more reliable information is available
Elastic modulus	$E_{cm}(lt)$	GPa	30.0	Mean 28-day value unless more reliable information is available
Tensile strength	$f_{ctk,0.05}(lt)$	MPa	2.12	Characterisitic tensile strength $f_{ctk,0.05}(28)$ unless more reliable information available
Tensile strain capacity	$\varepsilon_{ctu}(lt)$	με	141	$\varepsilon_{ctu}(lt) = f_{ctk,0.05}(t_{ltc})/(E_{cm} K_{c2})$
Risk of long term cracking	$\varepsilon_r(lt)/\varepsilon_{ctu}(lt)$		2.38	Cracking predicted if $\varepsilon_r(lt)/\varepsilon_{ctu}(lt) > 1$.
Long term crack-inducing strain	$\varepsilon_{cr}(lt)$	με	266	$K_{c1}[\alpha_c T_1+\varepsilon_{ca}(ea)]R_1+K_{c2}[(\varepsilon_{ca}(lt)-\varepsilon_{ca}(ea))R_2+\alpha_c T_2 R_2+\varepsilon_{cd}R_3]-0.5\varepsilon_{ctu}(lt)$
Minimum area of reinforcement $A_{s,min}$				
Coefficient for effect edge restraint (Optional)	k_{Redge}	YES	0.75	Calculated as (1-0.5R_3) for R_3 at the location of the maximum crack width
Minimum area of steel per face	$A_{s,min}$	mm^2	1238	$A_{s,min} = k_{Redge} k k_c \alpha_{ct} f_{ctm,0.05}(lt)/f_{yk})A_{ct}$ Highlighted if $A_s < A_{s,min}$
Crack spacing	$S_{r,max}$	mm	928	Recalculated using k_1 = 0.8 if no early-age cracking
Long term crack width	w_k	mm	0.25	$w_k = \varepsilon_{cr}(lt)S_{r,max}$
Maximum stress in the reinforcement	σ_s	N/mm^2	135	$\sigma_s = 2(\varepsilon_{sm} - \varepsilon_{cm})E_s$

ARUP	Job No.	Sheet No.	Rev.
	303154-00	1 of 1	0
	Member/Location		
Job Title: Aramis Launch Shaft	Drg. Ref.		
Calculation: SLS crack width - RC Diaphragm wall	Made by: [redacted]	Date: 10/06/2024	Chd.

Introduction A calculation to determine the minimum requirements for SLS crack widths, using Oasys GSA and Adsec

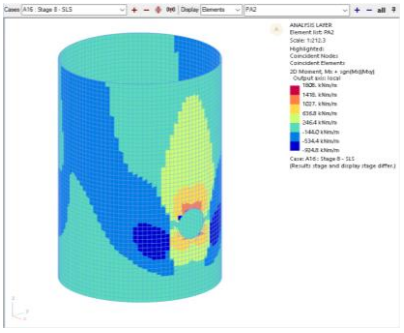
Oasys GSA

PA2 -d wall Cases: A2 A4 A6 A8 A10 A12 A14 A16 A18

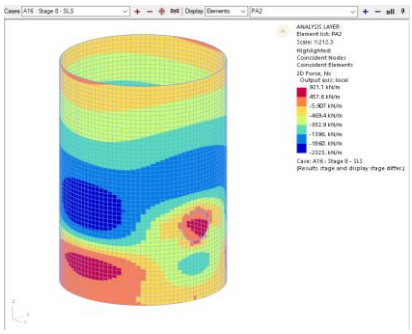
				[kNm/m]	[kN/m]	
	Elem	Case	Node	My + sgn(My)|Myx|	Ny	
B : y-direction	6355	A16	6341	689.2	248.4	*Max moment with coexisting force*
	6368	A16	5904	-401.9	631	*Min moment with coexisting force*
	6278	A16	5896	689.2	344.8	*Max axial T(+ve) with coexisting M*
	6345	A12	6341	292.6	-4643	*Min axial C (-ve) with coexisting M*

				[kNm/m]	[kN/m]	
	Elem	Case	Node	Mx + sgn(Mx)|Mxy|	Nx	
A : x-direction	6353	A16	6463	636.8	514.3	*Max moment with coexisting force*
	6368	A16	5901	-534.4	91.57	*Min moment with coexisting force*
	6267	A16	5909	350	364.2	*Max axial T(+ve) with coexisting M*
	6354	A12	6460	308.7	-7016	*Min axial C (-ve) with coexisting M*

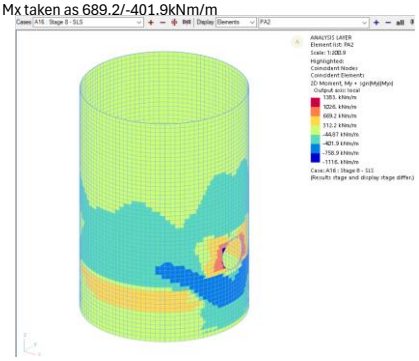
Mx spikes of 1808 and-925kNm/m discounted as localised around tunnel adit
Mx taken as 636.8/-534kNm/m



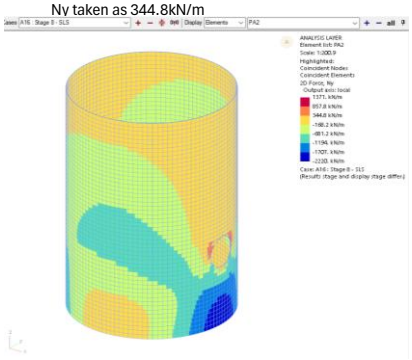
Nx spike of 921.1kN/m discounted as localised peak
Nx taken as 364.2kN/m



My spikes of 1383 and-1116kNm/m discounted as localised around tunnel adi
Mx taken as 689.2/-401.9kNm/m



Ny spike of 1371kN/m discounted as localised peal
Ny taken as 344.8kN/m

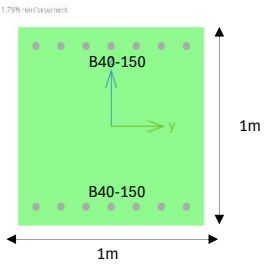


Adsec

SLS crack width check **Minimum reinforcement required for SLS crack width** B40-150 required

Case	Load F_x (kN)	Load M_{yy} (kNm)	Load M_{zz} (kNm)	Cracked?	Crack width (mm)
1	248.4	689.2	0	CRACKED	0.152
2	631	-401.9	0	UNCRACKED	0.125
3	344.8	689.2	0	CRACKED	0.163
4	-4643	292.6	0	UNCRACKED	
5	514.3	636.8	0	CRACKED	0.169
6	91.6	-534	0	UNCRACKED	0.108
7	364.2	350	0	UNCRACKED	0.094
8	-7016	308.7	0	UNCRACKED	

All <0.2mm



Limit 0.2 mm Table 7.1, NEN EN 1992-1-1 OK

B.3. Base slab design calculations

ARUP	Job No.	Sheet No.	Rev.
	303154-00	1 of 1	0
	Member/Location		
Job Title: Aramis Launch Shaft	Drg. Ref.		
Calculation: Flexural - RC base slab	Made by	Date: 31/05/2024	Chd.

Introduction A calculation to determine the minimum requirements for flexural reinforcement, using Oasys GSA and Adsec

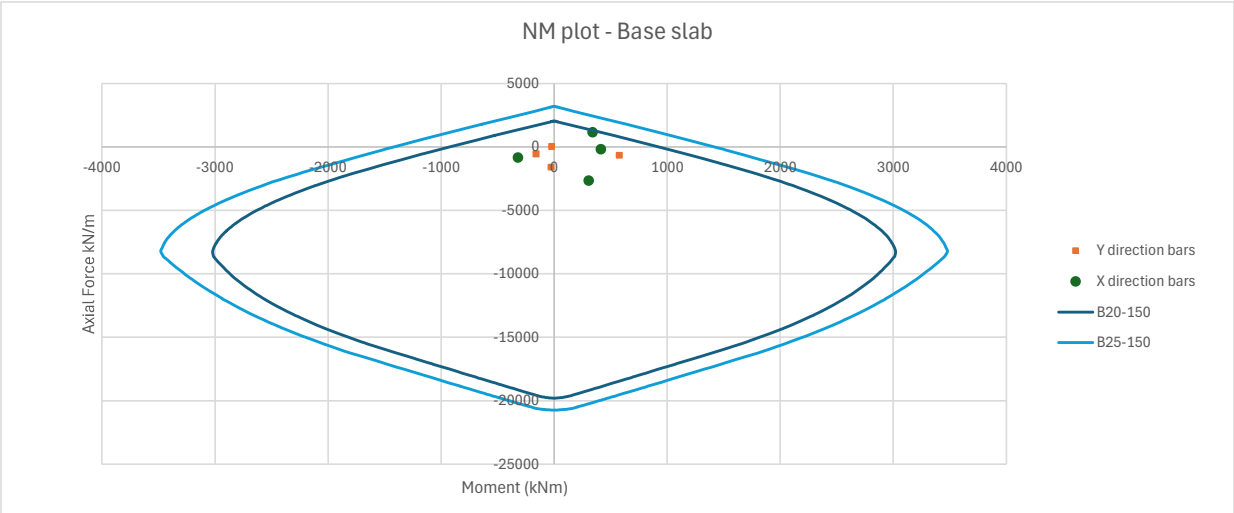
Oasys GSA

				[kNm/m]	[kN/m]	
	Elem	Case	Node	My + sgn(My)|Myx|	Ny	
B : y-direction	42274	A17	55183	578.8	-665	*Max moment with coexisting force*
	42250	A17	54678	-160.2	-553	*Min moment with coexisting force*
	42367	A15	37366	-20.59	31.84	*Max axial T(+ve) with coexisting M*
	42846	A15	55141	-27.03	-1589	*Min axial C (-ve) with coexisting M*

-ve downwards = sagging moment, so rebar at bottom
+ve upwards = hogging moment, so rebar at top

				[kNm/m]	[kN/m]	
	Elem	Case	Node	Mx + sgn(Mx)|Mxy|	Nx	
A : x-direction	42274	A17	55183	413.5	-177.2	*Max moment with coexisting force*
	42188	A17	53606	-321.6	-827.5	*Min moment with coexisting force*
	42247	A17	55188	342.2	1155	*Max axial T(+ve) with coexisting M*
	42582	A17	55188	305	-2642	*Min axial C (-ve) with coexisting M*

-ve downwards = sagging moment, so rebar at bottom
+ve upwards = hogging moment, so rebar at top



Minimum reinforcement required for flexural B25-150

ARUP	Job No.	Sheet No.	Rev.
	303154-00	1 of 1	0
	Member/Location		
Job Title: Aramis Launch Shaft	Drg. Ref.		
Calculation: Shear - RC base slab	Made by	Date: 31/05/2024	Chd.

Source/Ref

Checks to undertake:

1. Material properties
2. Minimum reinforcement requirements
3. Shear Capacity

1. Material properties

Tensile strength of steel rebar, fyk (assume B500)	f_{yk} =	500	N/mm^2
Partial factor for steel, γ_s		1.15	
$f_{ywd} = f_{yk}/\gamma_s$	fywd =	435	N/mm^2
Compressive strength of concrete, fck	f_{ck} =	32	N/mm^2
	$f_{ck,cub}$ =	40	N/mm^2
Partial factor for concrete, γ_c		1.5	
a_{cw}		1	NA to Cl 3.1.6
$f_{cd} = a_{cw}f_{ck}/\gamma_c$		21	N/mm^2

2. Minimum reinforcement requirements

Eurocode 2 part 1-1, 9.2.1.1 (1)

Minimum a

C32/40

fctm	3	Mpa
fyk	500	MPa

Element	Depth	$A_{s,min} = 0.26\frac{f_{ctm}b_t d}{f_{yk}}$	*A s,min* = 0.0013btd	*A s,max* = 0.04Ac
	mm	mm2 / m	mm2 / m	mm2/m
Base slab	1000	1560	1300	400000

Bar diamete	20	mm
Area	314	mm2
No. bars	5	
Spacing	201	

3. Shear Capacity in base slab

Check if shear reinforcement is required:

EC1992-1-1 Cl 6.2.2

$$V_{Rd,c} = [C_{Rd,c}k(100\,\rho_l f_{ck})^{1/3} + k_1\,\sigma_{cp}]\,b_w d \qquad (6.2.a)$$

with a minimum of

$$V_{Rd,c} = (v_{min} + k_1\sigma_{cp})\,b_w d \qquad (6.2.b)$$

where:

f_{ck} is in MPa

$k = 1+\sqrt{\frac{200}{d}} \le 2{,}0$ with d in mm

$\rho_l = \frac{A_{sl}}{b_w d} \le 0{,}02$

A_{sl} is the area of the tensile reinforcement, which extends $\ge (l_{bd} + d)$ beyond the section considered (see Figure 6.3).

b_w is the smallest width of the cross-section in the tensile area [mm]

$\sigma_{cp} = N_{Ed}/A_c < 0{,}2\,f_{cd}$ [MPa]

N_{Ed} is the axial force in the cross-section due to loading or prestressing [in N] (N_{Ed}>0 for compression). The influence of imposed deformations on N_{Ed} may be ignored.

A_C is the area of concrete cross section [mm^2]

$V_{Rd,c}$ is [N]

where

f_{ck} =	32	N/mm^2	
d =	1000	mm	
k = 1+(200/d)0.5	1.45		<2.0 ∴ OK
b_w =	1000	mm	per m strip
A_{sl} =	3436	mm^2	Assuming B25 bars at 150mm spacing
ρ_1 =Asl/bwd	0.0034		<0.02 ∴ OK
σ_{cp} =	0.0	N/mm^2	Assumed
$C_{Rd,c}$ = 0.18/γc	0.12		
v_{min} =	0.34	N/mm^2	$v_{min} = 0{,}035\,k^{3/2}\cdot f_{ck}^{1/2}$
k_1 =	0.15		

Design she: $V_{Rd,c}$ =	386177	N	(6.2a)	
	386	kN	**USE THIS AS BIGGER**	
$V_{Rd,cmin}$ =	344701	N	(6.2b)	
	345	kN		
V_{Ed} =	279.5	kN	From GSA	

As VEd < VRd,c shear reinforcement in the slab is NOT required

Check if maximum allowable shear force is exceeded — EC1992-1-1 Cl 6.2.2

The shear force V_{Ed}, calculated without reduction by β, should however always satisfy the condition

$$V_{Ed} \le 0{,}5\, b_w d\, \nu f_{cd} \qquad (6.5)$$

where ν is a strength reduction factor for concrete cracked in shear

Note: The value ν for use in a Country may be found in its National Annex. The recommended value follows from:

$$\nu = 0{,}6\left[1 - \frac{f_{ck}}{250}\right] \quad (f_{ck} \text{ in MPa}) \qquad (6.6N)$$

where

v =	0.523	N/mm²
$V_{Ed,max}$ =	5580800	N
	5581	kN

As VEd < VEd,max shear in the slab is acceptable

If shear reinforcement required:

Shear resistance will be calculated in accordance with NEN EN 1992-1-1:2004 clause 6.2.3.

$V_{Rd,s}$ = $[A_{sw} / s] * z * f_{ywd} * \cot\theta$

				outer bar	
A_{sw} =	402	mm²	Link dia =	16	
Spacing, s =	150	mm	No. legs =	2	
Lever arm, z =	900	mm (0.9d)			
f_{ywd} =	434.8	N/mm²			
θ =	45	degrees	≡	0.785	radians
α =	45	degrees	≡	0.785	radians

∴ $V_{Rd,s}$ = 1,049 kN

$V_{Rd,max}$ = $\alpha_{cw} * b_w * z * \nu_1 * f_{cd} / (\cot\theta + \tan\theta)$

α_{cw} =	1.0	(Non-prestressed)			
b_w =	1000.0	mm			
f_{ck} =	32.0	N/mm²	f_{cd} =	21.3	N/mm²
$\nu_1 = \nu$ =	0.5232	N/mm²			

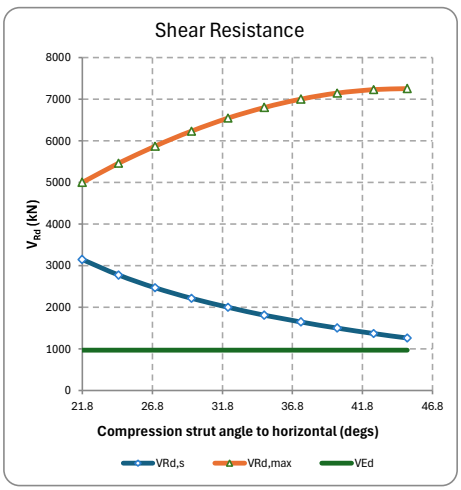
∴ $V_{Rd,max}$ = 5,023 kN

$V_{Ed} < V_{Rd,s} < V_{Rd,max}$

$A_{sw,max}$ = $0.5 * \alpha_{cw} * \nu_1 * f_{cd} * b_w * s / f_{ywd}$ for θ = 45.0 degrees

∴ $A_{sw,max}$ = 1925.4 mm² ---

θ (degs)	$V_{Rd,s}$ (kN)	$V_{Rd,max}$ (kN)	ΔF_{td} (kN)
21.8	2623	3464	349
24.4	2313	3779	308
27.0	2059	4063	274
29.6	1847	4314	246
32.2	1666	4530	222
34.8	1509	4708	201
37.4	1372	4847	183
40.0	1250	4946	167
42.6	1141	5005	152
45.0	1049	5023	140

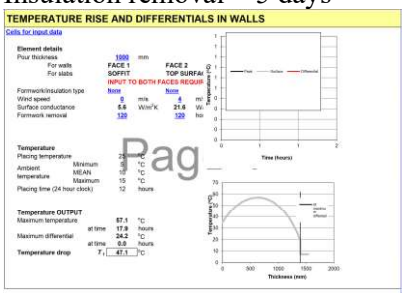
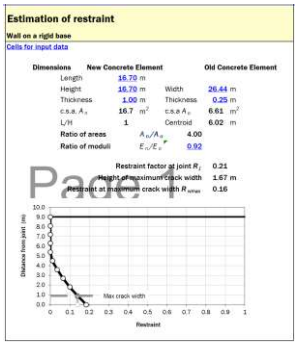
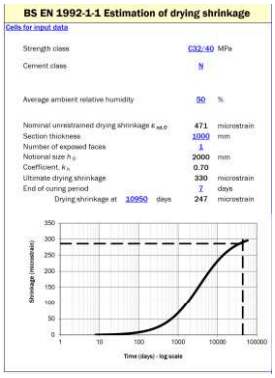


21.8	280	
45.0	280	
2.6	step	

V_{Ed} = 280 kN

$UR_{concrete}$=	0.06	
	OK	
UR_{links}=	0.27	
	OK	

RC base slab - Early Thermal Checks Summary

	RC base slab **Assume: edge restraint, as restrained by shaft lining**
Temperature Risk and Differentials in ground slabs	Pour thickness = 1000mm Insulation type (top surface) = none Insulation type (bottom surface) = none Wind speed= 0m/s (slab in base of shaft, effects of wind will be minimal) Insulation removal = 5 days 
Restraint	R=0.16 
Drying shrinkage	247 microstrain (For long-term cracking) 
Crack width	Concrete strength = C32/40 Bar diameter = 16mm Bar spacing = 150mm Cover = 75mm top / 75mm bottom Aggregate type: Ecm kept as default Early age crack width = 0.0mm < 0.3mm (Section 7.3.3, NEN EN 1992-1-1) Long term crack width = 0.15mm < 0.3mm (Section 7.3.3, NEN EN 1992-1-1)

C766 Temperature prediction in walls and slabs

This model provides a basis for the comparative assessment of early-age temperature rise and temperature differentials in concrete walls, suspended slabs and ground slabs.

The model is based on the numerical method of Ross and Bray (1949) and is described in detail in Appendix A2

Background data on heat generation were provided by the University of Dundee (Dhir *et al*, 2006) obtained using semi-adiabatic testing to BS EN 196-9:2003

Data for the concrete are input on PAGE 2

Data specific to walls and suspended slabs are input on PAGE 3

Data specific to ground slabs are input on PAGE 4

Each page of this spreadsheet is password protected to prevent accidental deletion of equations. For those wishing to modify or copy the spreadsheet, the password is C766

PAGE 2 ADIABATIC TEMPERATURE			
Concrete mix details	Binder content	kg/m^3	The binder is the total content of cementitious material, including CEM I and CEM I in combination with either fly ash or ground granulated blast-furnace slag, ggbs
	Binder type		Three binder options are available from a drop-down menu CEM I, fly ash, ggbs
	Percent addition	%	The percent of either fly ash or ggbs by weight of binder
	Density	kg/m^3	The wet density of the concrete
	Specific heat	kJ/kg°C	The early-age value when the concrete is only partially hydrated. A value of 1 kJ/kg°C should be used unless more reliable data are available
	Thermal conductivity	W/m°C	Assumed to be 2.0 W/m°C (see Appendix A2 for values for concrete using difference aggregates)
	Multiplier		If results are available for a specific mix, the multiplier may be used to adjust the adiabatic curve to match the data. The default value is 1,
Temperature input	Placing temperature	°C	The temperature of the fresh concrete when poured. As a general rule, the placing temeprature be 5°C above the mean ambinebt temeprature.
	Minimum ambient temperature	°C	These values determine the variable ambient temperature which is assumed to vary on a sinusoidal basis. As a eneral rule the ambient temeprature will vary by ±5°C from the mean value.
	Mean ambient temperature	°C	
	Maximmum ambient temperatur	°C	
	Constant or variable ambient temperature		There is the option to assume either a variable sinusoidal ambient temperature variation or a constant ambient temeprature variation. In the latter case this is assumed to be the MEAN ambient temperature,
	Pacing time	Hours	This assumes a 24 hour clock
Output	Temperature drop T_1		Values are calculated in PAGE 3 and PAGE 4 but are shown on PAGE 2 to enable the effect of changes in the concrete mix and placing temperature to be observed without changing pages. Input on section geometry, formwork etc, is on PAGE 3 and PAGE 4
GRAPHS	Reference curve for the cement		The reference adiabatic curve is estimated from the cement type and content and represents the value at a starting temperature of 20°C
	Adiabatic temperature rise for the concrete		The estimated adiabatic temperature rise is shown based on the reference curve at 20°C and the value obtained when a different placing temperature is used.
PAGE 3 TEMPERATURE CALCULATIONS FOR WALLS AND SUSPENDED SLABS			
Section and formwork details	Section thickness	mm	
	Formwork/insulation type		Formwork/insulation options are available for both faces from drop-down menus. Information for each face must be input. REMEMBER TO INPUT FOR BOTH FACES
	Wind speed	m/s	Assumed to be 4 m/s for typical UK conditions REMEMBER TO INPUT FOR BOTH FACES
	Formwork removal time	hours	The time after casting. Input must be made for both faces. There is the options to remove formwork from opposite faces at different times REMEMBER TO INPUT FOR BOTH FACES
Concrete mix details	Thermal conductivity	W/m°C	Copied from PAGE 2
	Density and specific heat		Copied from PAGE 2
PAGE 4 TEMPERATURE CALCULATIONS FOR GROUND SLABS			
Section and formwork details	Section thickness	mm	This is the minimum dimension through which heat loss is predominant
	Insulation type		Options are available from a drop-down menu for both the soffit formwork and insulation applied to the top surface (if required).
	Wind speed	m/s	Assumed to be 4 m/s for typical UK conditions
	Formwork removal time	hours	The time after casting. Input must be made for both the soffit and the top surface. There is the options to remove opposite faces at different times
Concrete mix details	Thermal conductivity	W/m°C	Copied from PAGE 2
	Density and specific heat		Copied from PAGE 2
Properties of the ground	Specific heat	kJ/kg°C	Default value is 1.2 kJ/kg°C
	Density	kg/m^3	Default value is 2000 kJ/kg°C
	Thermal conductivity	W/m°C	Default value is 1.1 W/m°C
	Temperature	°C	

TEMPERATURE RISE AND DIFFERENTIALS IN WALLS

Cells for input data

Element details

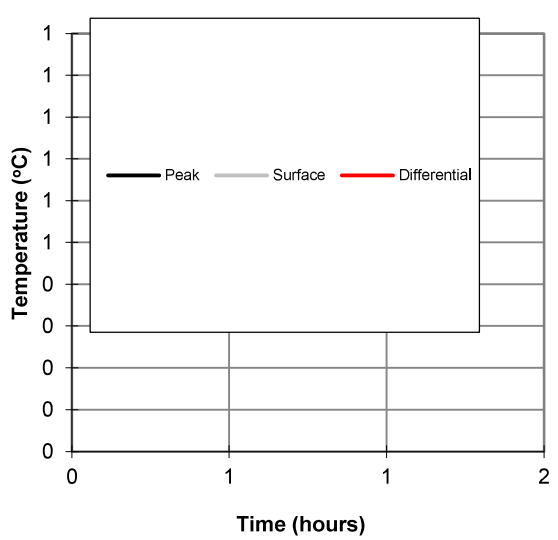
Pour thickness		1000	mm	
For walls		FACE 1		FACE 2
For slabs		SOFFIT		TOP SURF/
		INPUT TO BOTH FACES REQUI		
Formwork/insulation type		None		None
Wind speed		0	m/s	4 r
Surface conductance		5.6	W/m^{2}K	21.6 \
Formwork removal		120		120 h

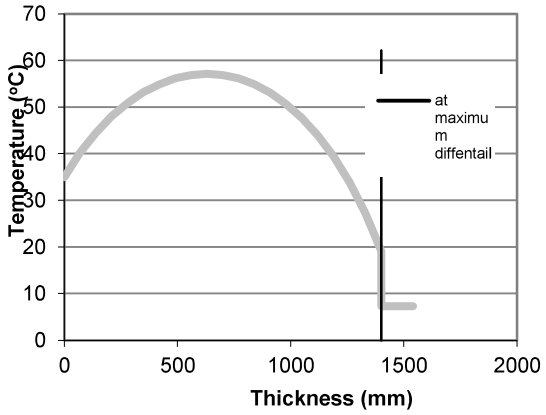
Temperature

Placing temperature		25	°C
Ambient temperature	Minimum	5	°C
	MEAN	10	°C
	Maximum	15	°C
Placing time (24 hour clock)		12	hours

Temperature OUTPUT

Maximum temperature		57.1	°C
	at time	17.9	hours
Maximum differential		24.2	°C
	at time	0.0	hours
Temperature drop	T_1	47.1	°C





C766 Estimation of drying shrinkage according to BS EN 1992-1-1

This calculator estimates the drying shrinkage using the expressions and relationships given in BS EN 1992-1-1, Section 3.1.4

The method is described in detail in Appendix A3

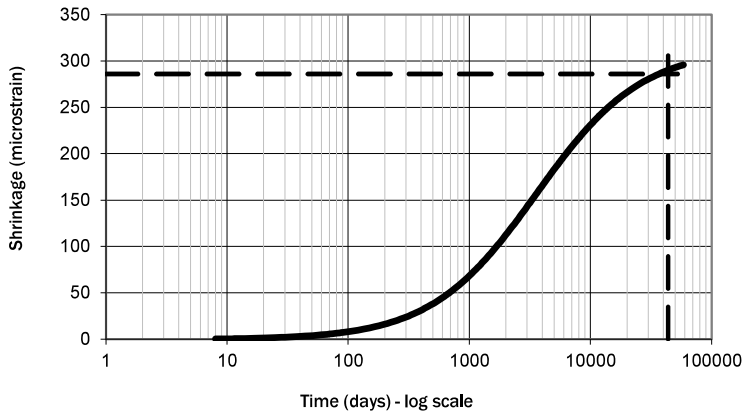
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PAGE 2 DRYING SHRINKAGE			
Concrete mix details	Strength class		This is selected from a drop-down menu
	Cement class		Three options are available from a drop-down menu: **S (slow), N (normal) and R (rapid)**
Ambient conditions	Relative humidity	%	The average relative humidity over the period considered
Nominal unrestrained drying shrinkage $\varepsilon_{cd,0}$			The ultimate drying shrinkage achievable for the combination of concrete type and relative humidity
Section geometry	Section thickness	mm	
	Exposed faces		In most cases drying will be from either one of two of the faces. These options are given
	Notional thickness h_0	mm	The notional thickness $h_0 = A_c/u$ where A_c is the concrete cross sectional area and u is the perimeter of that part of the concrete that is exposed to drying. For walls or slabs the thickness h is normally small in relation to the other dimensions and h_0 approximates to h when drying is from both faces and $2h$ when drying is from one face only.
	Coefficient k_h		k_h is a function of h_0 and is given in Table 3.3 of BS EN 1992-1-1
Ultimate drying shrinkage = $k_h\ \varepsilon_{cd,0}$			The ultimate drying shrinkage achievable for the combination of concrete type, relative humidity and notional section thickness
	End of curing period t_s	days	It is assumed that shrinkage begins at the end of the curing period
Drying shrinkage at time t, $\varepsilon_{cd}(t) = \beta ds(t,t_s)\ k_h\ \varepsilon_{cd,0}$			The time function $\beta_{ds}(t, t_s) = (t - t_s)/\{(t - t_s) + 0.04\sqrt{h_0}^3\}$. BS EN 1992-1-1 notes that the estimated value of drying shrinkage has an expected coefficient of variation of 30%

BS EN 1992-1-1 Estimation of drying shrinkage

Cells for input data

Strength class	C32/40	MPa
Cement class	N	
Average ambient relative humidity	50	%
Nominal unrestrained drying shrinkage $\varepsilon_{cd,0}$	471	microstrain
Section thickness	1000	mm
Number of exposed faces	1	
Notional size h_0	2000	mm
Coefficient, k_h	0.70	
Ultimate drying shrinkage	330	microstrain
End of curing period	7	days
Drying shrinkage at 10950 days	247	microstrain



C766 Estimation of restraint

This calculator estimates restraint in a wall cast onto a rigid based. The method is based of the ACI approach in which the restraint at the joint is determined by the relative stiffness of the new section and the section against which it is cast. The variation in restraint with height is determined by the length to height ratio.

The method is described in detail in Appendix A5

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PAGE 2 RESTRAINT	
Wall geometry	The wall geometry is define by the length L , the height H and the thickness, h. When a new pour that is restrained along one edge abuts an exisiting pour at one end then for the purpose of calculating the restraint profile, the length of the edge restraaining joint should be assumed to be 2 x L, ie it is assumed that there is zero displacement at the joint between the new pour and the adjacent pour
Base geometry	The base geometry is defined by the width W and the thickness, h . The length is assumed to be the same as the length of wall being cast
Relative areas	The ratio of the cross-sectional area (c.s.a.) of the wall A_{new} and the base A_{old} is calculated. The relative areas of influence A_{old} and A_{new} may be difficult to define. For example, when a wall is cast onto a slab, what width of slab should be considered as providing effective restraint? In such circumstances it is recommended that the relative area is assumed to be in proportion to the relative thicknesses, h_{new} and h_{old} , of the wall and slab. The following simple rules may be applied: • For a high wall cast at the edge of a slab assume $A_{new}/A_{old} = h_{new}/h_{old}$ • For a wall cast remote from the edge of a slab assume $A_{new}/A_{old} = h_{new}/2h_{old}$ • For a slab cast against an existing slab assume $A_{new}/A_{old} = h_{new}/h_{old}$
Relative modulus of elasticity	Bamforth and Price (1995) recommends that the ratio E_{new}/E_{old} is assume to be in the range 0.7 to 0.8 at early age. The lower value gives most conservative result and should be used for conditions in which cool-down is most rapid. When estimating restraint to long term deformations, should there be any differential strains, it should be assumed that $E_{new}/E_{old} = 1$
Restraint at the joint R_j	$$R_j = \frac{1}{1 + \frac{A_{new}}{A_{old}} \frac{E_{new}}{E_{old}}}$$
Restraint at the location of the maximum crack width	The maximum restrained strain and crack width occur at a perpendicular distance from the joint of 0.1 times the length of the joint if the new pour is not cast against existing concrete at either end. If the new pour abuts an exisitng pour at one end then the maximum restrained strain and crack width occur at a perpendicular distance from the joint of 0.2 times the length of the joint

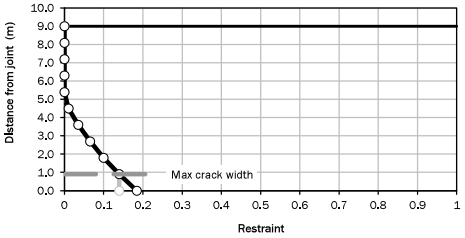
Estimation of restraint

Wall on a rigid base

Cells for input data

Dimensions	New Concrete Element			Old Concrete Element	
Length	16.70	m			
Height	16.70	m	Width	26.44	m
Thickness	1.00	m	Thickness	0.25	m
c.s.a. A_n	16.7	m^2	c.s.a A_o	6.61	m^2
L/H	1		Centroid	6.02	m
Ratio of areas		A_n/A_o	4.00		
Ratio of moduli		E_n/E_o	0.92		

Restraint factor at joint R_j	0.21	
Height of maximum crack width	1.67	m
Restraint at maximum crack width R_{wmax}	0.16	



C766 Predicting the risk of cracking and controlling crack widths

This calculator provides a basis for estimating the likelihood of cracking and for selecting or checking reinforcement to control crack widths

The approach is based on the method of BS EN 1992-1-1 with adjustments where considered appropriate (see Appendix A8)

Three conditions of restraint are dealt with

PAGE 2 Continuous EDGE restraint.

PAGE 3 END restraint

PAGE 4 INTERNAL restraint from temperature differentials

Each page of this spreadsheet is password protected to prevent accidental deletion of equations. For those wishing to modify or copy the spreadsheet, the password is C766

Input data are as follows (all other parameters are calculated):

PAGE 2 CONTINUOUS EDGE RESTRAINT			
Section details and reinforcement	Section thickness	mm	Section thickness in mm
	Strength class	f_{ck} / $f_{ck,cube}$	Select from the drop-down menu
	Bar diameter	φ	Bar diameter in mm
	Bar spacing	S	Bar spacing in mm
	Cover to reinforcement	c	Minimum cover to reinforcement
	Yield strength of reinforcement	f_{yk}	The characteristic yield strength of the reinforcement in MPa
Early-age restrained strain	Coefficient of expansion	α_c	A value of 12 με / °C is recommended if there is no knowledge of the aggregate type. The recommended value in BS EN 1992-1-1 is 10 με / °C but many aggregates in the UK produce concrete with higher values
	Temperature drop	T_1	This may be obtained from data in C660, the temperature model (Appendix A2) or by independent validated modelling or measurement.
	Creep factor	Kc_1	BS EN 1992-1-1 does not define a creep factor but includes this in the value of restraint R = 0.5 which is the maximum value recommended by BS EN 1992-1-1 and so must cover the worst condition of restraint. When this value of R is used, K_1 should be assumed to be 1. When R is calculated K_1 may be assumed to be 0.65
	Restraint at the location of the maximum crack width	R	BS EN 1992-1-1 permits R to be calculated from the relative stiffness of the element and the member against which it is cast. Where it can be demonstrated that R < 0.8 the calculated value may be used with K_1 = 0.65. The the maximum restrained strain occurs at a distance of 0.1 x length of the wall above the joint with the base. When the wall abuts a previously cast length, the assumed length is calculating the location of the maximum restrained strain is double the actual length.
Risk of early-age cracking	Age at cracking	days	Assumed to be 3 days for early age cracking unless more reliable information is available
	Aggregate type		The tensile strain capacity is the ratio of the tensile strength to the elastic modulus and the latter is affected by the aggregate type. If the aggregate source is known this can be selected from the drop down menu. The default value for unknown aggregate is 1. This is the value dervied from the properties calculated according to BS EN 1992-1-1 for quartz aggregate. Where the elastic modulusat 28 days is known this may be entered and 'measured' should be selected from the drop down menu.
Minimum area of reimforcement $A_{s,min}$	Coefficient for effect edge restraint (Optional)	$k_{R,edge}$	A drop down menu (YES/NO)provides the option to take account of the contributon of the edge restraint in limiting crack opening and the amount of steel required to ensure that yield does not occur.
Crack spacing	Factor for bond	k_1	BS EN 1992-1-1 recommends k_1 = 0.8 but provides a factor of 0.7 where good bond cannot be guaranteed. k_1 = 0.8/0.7 = 1.14 for section >300 mm thick with cover < 50 mm. If cracking is not predicted at early age, then k_1 = 0.8.
Long term cracking	Age at cracking	days	For calculating long term cracking the 28-day properties of the concrete are adopted.
	Long term temperature change	$T2$	Where both the restrained and the restraining element are subject to a change in temperature, T2 is the differential change in temperature between the restrained and the restraining element unless there is external restraint to both elements.
	Drying shrinkage	ε_{cd}	Calculate using the method of BS EN 1992-1-1 unless more reliable information is available. Only apply when causing differential contraction between elements or when the elements acting integrally are subject to external restraint. The drying shrinkage is that occurring at the end of the design life of the structure or part thereof.
	Creep factor	K_{C2}	When restraint is calculated, the long-term creep coefficient is taken as 0.5
	Restraint to T_2	R_2	Long term restraint will be reduced as the ratio of stiffness of the elements reduces and E_n / E_o approaches 1. Consideration must also be given to the element acting integrally.
	Restraint to drying shrinkage	R_3	
Crack spacing	Coefficient for bond characteristics	k_1	If cracking has occurred at early age, the same carack spacing is assumed for long-term cracking. If cracking has not occurred at early age, the crack spacing is recalculated assuming that k_1 =0.8

PAGE 3 END RESTRAINT			
Section details	Section thickness	h	Section thickness in mm
	Strength class	f_{ck} / $f_{ck,cube}$	Select from the drop-down menu.
	Bar diameter	ϕ	Bar diameter in mm
	Bar spacing	s	Bar spacing in mm
	Cover	c	Minimum cover to reinforcement
Properties of reinforcement	Characteristic yield strength	f_{yk}	The characteristic yield strength of the reinforcement = 500 MPa
	Elastic modulus	E_s	200 GPa
Crack sacing	Coefficient for bond characteristics	k_1	See ref to k_1 above
Age at cracking	Early-age	*3 days*	Values of 3 days and 28 days are assumed for the estimation of early age and long-term properties. Values derived on this basis are have been shown to be safe in relation to the design of reinforcement for crack control
	Long term	*28 days*	
Early-age restrained strain	Coefficient of expansion	α_c	A value of 12 με / °C is recommended if there is no knowledge of the aggregate type. The recommended value in BS EN 1992-1-1 is 10 με / °C but many aggregates in the UK produce concrete with higher values
	Temperature drop	T_1	This may be obtained from data in C766, the temperature model (Appendix A2) or by independent validated modelling or measurement.
	Creep factor	Kc_1	BS EN 1992-1-1 does not define a creep factor but includes this in the value of restraint R = 0.5 which is the maximum value recommended by BS EN 1992-1-1 and must cover the worst condition of restraint. When this value of R is used, K_1 should be assumed to be 1. When R is calculated K_1 may be assumed to be 0.65
	Restraint	R	BS EN 1992-1-1 permits R to be calculated from the relative stiffness of the element and the member against which it is cast. Where it can be demonstrated that R < 0.8 the calculated value may be used with K_1 = 0.65. END restraint may be difficult to calculated, particulary when there is a combination of restraints acting on the member, eg a slab on piles with a combination of friction from the base and lateral resistance from piles. Some guidance is provided in Appendix A5, but the user may be required to undertake additional calculations where the arrangement of restraints is complex.
Risk of early-age cracking	Early-age	*Days*	Properties at 3 days are assumed for the estimation of early age cracking. However, when there is confience in the predicted time of cracking there is the option to use other ages.
	Aggregate type		The tensile strain capacity is the ratio of the tensile strength to the elastic modulus and the latter is affected by the aggregate type. If the aggregate source is known this can be selected from the drop down menu. The default value (coefficient = 1) is the value dervied from the properties calculated according to BS EN 1992-1-1 for quartz aggregate. Where the elastic modulus is known this may be entered and 'measured' should be selected from the drop down menu.
Long term cracking	Age at cracking	days	For calculating long-term cracking the 28-day properties of the concrete are adopted.
	Long term temperature change	*T2*	Where both the restrained and the restraining element are subject to a change in temperature, *T2* is the differential change in temperature between the restrained and the restraining element unless there is external restraint to both elements.
	Drying shrinkage	ε_{cd}	Calculate using the method of BS EN 1992-1-1 unless more reliable information is available.
	Creep factor	*Kc2*	
	Restraint to T_2	R_2	Long-term restraint will be reduced as the ratio of stiffness of the elements reduces and E_n / E_o approaches 1 and as elements act integrally.
	Restraint to drying shrinkage	R_3	
	Restraint required to cause cracking	R_{crack}	As restraint may be difficult to calculate, the restraint required to cause cracking is also estimated and the user may judge whether this level of restraint is likely to be achieved in practice.

PAGE 3 INTERNAL RESTRAINT
The input data for dealing with cracking caused by internal restraint is broadly the same as that required for edge restraint with the principal exception that the temperature differential between the centre and the surface ΔT replaces T_1. Other changes inherent in the design calculation are the restraint factor R assumed to be 0.42 and the coefficients k and kc which reflect the difference in the stress distribution within the section compared with the condition of external restraint.

Risk and control of cracking due to continuous edge restraint

Input parameters	Symbol	Unit	Value	Comments/guidance
Section details and reinforcement				
Section thickness	h	mm	1000	
Strength class	f_{ck} / $f_{ck,cube}$	MPa	C32/40	Select from drop down menu
Bar diameter	φ	mm	16	
Bar spacing	s	mm	150	
Cover	c	mm	75	Minimum cover to steel in the direction perpendicular to the crack
Characteristic yield strength of reinforcement	f_{yk}	MPa	500	500 MPa
Area of steel per face per m	A_s	mm^2	1340	
Early age cracking				
Restrained strain				
Coefficient of thermal expansion of concrete	α_c	με/°C	12.0	If aggregate is unknown use 12 με / °C
Temperature drop	T_1	°C	47.1	T_1 = Peak temperature - mean ambient temperature
Autogenous shrinkage	$\varepsilon_{ca}(ea)$	με	16	BS EN 1992-1-1 $\varepsilon_{ca}(ea) = 2.5 (f_{ck} - 10) \times (1-\exp(-0.2\, t_c^{0.5}))$
Free contraction	$\varepsilon_{free}(ea)$	με	581	$\varepsilon_{free(ea)} = T_1 \alpha_c + \varepsilon_{ca}(ea)$
Creep factor - early age	K_{c1}		0.65	K_{c1} = 0.65 if R is calculated; K_{c1} = 1 if R is assumed to be 0.5 (including creep) according to BS EN 1992-1-1
Restraint	R_1		0.16	Use restraint calculator for walls or adjacent slabs; or historical data. The restrant at the point of maximum crack width is estimated at a distance of 0.1L from the joint, where L is the length of the joint
Early-age restrained contraction	$\varepsilon_r(ea)$	με	60	$\varepsilon_r(ea) = R_1 K_{c1} (T_1 \alpha_c + \varepsilon_{ca}(ea))$
Risk of cracking and crack inducing strain				
Age at early cracking	t_{eac}	days	3	Assume 3 days unless more reliable information is available
Aggregate type. Use the 'selected' option and enter the E value if measured or obtained from historical results.	Limestone		0.90	Coefficient for stiffness of aggregate
			30	Enter value of $E_{cm}(28)$ if 'measured' selected from menu for aggregate type
Elastic modulus	$E_{cm}(t_c)$	GPa	25.7	Mean value $E_{cm}(t_c)$. BS EN 1992-1-1 values are based on quartz aggregate
Tensile strength at cracking	$f_{ctk,0.05}(t_c)$	MPa	1.27	Characterisitic tensile strength $f_{ctk,0.05}(t_c)$
Tensile strain capacity	$\varepsilon_{ctu(ea)}$	**με**	76	$\varepsilon_{ctu(ea)} = [f_{ctk,0.05}(t_{eac}) / [E_{cm}(t_{eac}) \times K_{c1}]$
Risk of early age cracking	$\varepsilon_{r(ea)}/\varepsilon_{ctu(ea)}$		0.80	Cracking predicted if $\varepsilon_{r(ea)}/\varepsilon_{ctu} > 1$.
Early-age crack-inducing strain	$\varepsilon_{cr(ea)}$	με	23	$\varepsilon_{cr(ea)} = R_1 K_{c1} (T_1 \alpha_c + \varepsilon_{ca(ea)}) - 0.5\, \varepsilon_{ctu(ea)} = (\varepsilon_{sm} - \varepsilon_{cm})$
Minimum area of reinforcement $A_{s,min}$				
Coefficient (BS EN 1992-1-1)	k		0.65	k = 1.0 for $h \leq 300$ mm, k = 0.65 for $h \geq 800$ mm, intermediate values are interpolated
Coefficient (BS EN 1992-1-1)	k_c		1.00	For pure tension $k_c = 1$
Coefficient for effect of edge restraint (optional)	k_{Redge}	YES	0.92	Calculated as $(1-0.5R_1)$ for R_1 at the location of the maximum crack width
Depth of surface zone representing A_{ct}	$h_{s,min}$	mm	325	$h_{s,min} = k\, k_c\, h/2$
Minimum area of steel per face per m	$A_{s,min}$	mm^2	757	$A_{s,min} = (k_{Redge}\, k\, k_c\, (f_{ctk,0.05}(t_c)/f_{yk})\, A_{ct}$ Highlighted if $A_s < A_{s,min}$
Crack spacing				
Depth of the surface zone defining the effective area of concrete in tension, $A_{c,eff}$	$h_{e,ef}$	mm	207.5	$h_{e,ef}$ = lesser of $h/2$ a nd $2.5\,(c + \varphi/2)$
Steel ratio for estimating crack spacing	$\rho_{p,eff}$		0.00646	$\rho_{p,eff} = A_s / A_{c,eff} = A_s / (h_{e,ef} \times 1000)$
Coefficient for bond characteristics.	k_1		0.8	BS EN 1992-1-1 recommends k_1 = 0.8, but provides a factor of 0.7 where good bond cannot be guaranteed. Recommended k_1 = 0.8/0.7 = 1.14 at early age for sections >300 mm thick with cover < 50 mm.
Crack spacing	$S_{r,max}$	mm	1097	$S_{r,max} = 3.4c + 0.425\, k_1\, \varphi/\rho_{p,eff}$
Early age crack width	w_k	mm	0.00	$w_k = \varepsilon_{c(ea)}\, S_{r,max}$ (=0 if early-age cracking is not predicted)
Maximum stress in the reinforcement	σ_s	N/mm^2	24	$\sigma_s = 2(\varepsilon_{sm} - \varepsilon_{cm})E_s$
Long term cracking				
Long-term restrained strain				
Autogenous shrinkage at 28 days	$\delta\varepsilon_{ca}(lt)$	με	36	
Long-term temperature change	T_2	°C	20	T_2 and ε_{cd} only apply when causing differential contraction between the restrained and the restraining member or when the members acting integrally are subject to external restraint. The drying shrinkage is that occurring at the end of the design life of the structure or part thereof.
Drying shrinkage	ε_{cd}	με	247	
Long-term free contraction	$\varepsilon_{free}(lt)$	με	1088	$\alpha_c\, T_1 + \varepsilon_{ca}(lt) + \alpha_c\, T_2 + \varepsilon_{cd}$
Creep factor - long term	K_{c2}		0.50	K_{c2} = 0.5 if R is calculated, K_{c2} = 1 if R is assumed to be 0.5 (including creep to BS EN 1992-1-1)
Restraint to long-term thermal strains	R_2		0.50	Restraint of an individual member will reduce as E_n / E_o approaches 1 in the long term. However, consideration must be given to the fact that individually cast elements will act integrally in the partially or fully completed structure. R_2 and R_3 will generally have the same value
Restraint to drying shrinkage	R_3		0.50	

Long-term restrained strain	$\varepsilon_r(lt)$	με	207	$K_{c1}[\alpha_c T_1+\varepsilon_{ca(3)}]R_1+K_{c1}[(\varepsilon_{ca(28)}-\varepsilon_{ca(3)})+\alpha_c T_2]R_2+K_{c2}\varepsilon_{cd}R_3]$
Risk of cracking and crack-inducing strain				
Age at long term cracking	t_{ltc}	days	28	Assume 28 days unless more reliable information is available
Elastic modulus	$E_{cm}(lt)$	GPa	30.0	Mean 28-day value unless more reliable information is available
Tensile strength	$f_{ctk,0.05}(lt)$	MPa	2.12	Characterisitic tensile strength $f_{ctk,0.05}(28)$ unless more reliable information available
Tensile strain capacity	$\varepsilon_{ctu}(lt)$	με	141	$\varepsilon_{ctu}(lt) = f_{ctk,0.05}(t_{ltc})/(E_{cm}K_{c2})$
Risk of long term cracking	$\varepsilon_r(lt)/\varepsilon_{ctu}(lt)$		1.46	Cracking predicted if $\varepsilon_r(lt)/\varepsilon_{ctu}(lt) > 1$.
Long term crack-inducing strain	$\varepsilon_{cr}(lt)$	με	136	$K_{c1}[\alpha_c T_1+\varepsilon_{ca}(ea)]R_1+K_{c2}[(\varepsilon_{ca}(lt)-\varepsilon_{ca}(ea))R_2+\alpha_c T_2 R_2+\varepsilon_{cd}R_3]-0.5\varepsilon_{ctu}(lt)$
Minimum area of reinforcement $A_{s,min}$				
Coefficient for effect edge restraint (Optional)	k_{Redge}	YES	0.75	Calculated as (1-0.5R_3) for R_3 at the location of the maximum crack width
Minimum area of steel per face	$A_{s,min}$	mm^2	1032	$A_{s,min} = k_{Redge}\, k\, k_c\, \alpha_{ct}\, f_{ctm,0.05}(lt)/f_{yk})A_{ct}$ Highlighted if $A_s < A_{s,min}$
Crack spacing	$S_{r,max}$	mm	1097	Recalculated using k_1 = 0.8 if no early-age cracking
Long term crack width	w_k	mm	0.15	$w_k = \varepsilon_{cr}(lt)S_{r,max}$
Maximum stress in the reinforcement	σ_s	N/mm^2	83	$\sigma_s = 2(\varepsilon_{sm}-\varepsilon_{cm})E_s$

ARUP	Job No.	Sheet No.	Rev.
	303154-00	1 of 1	0
	Member/Location		
Job Title: Aramis Launch Shaft	Drg. Ref.		
Calculation: SLS crack width - RC base slab	Made by	Date: 10/06/2024	Chd.

Introduction A calculation to determine the minimum requirements for SLS crack widths, using Oasys GSA and Adsec

Oasys GSA

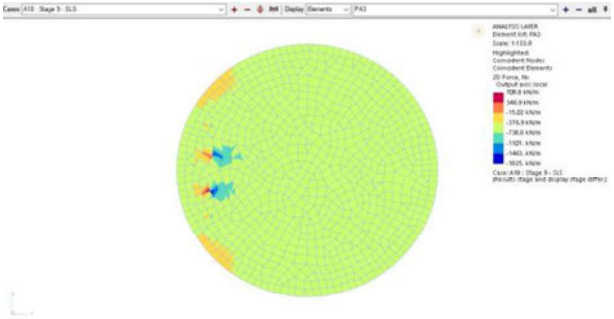
PA3 - RC base slab Cases: A2 A4 A6 A8 A10 A12 A14 A16 A18

				[kNm/m]	[kN/m]	
	Elem	Case	Node	My + sgn(My)|Myx|	Ny	
B : y-direction	42274	A18	55183	384	-492.1	*Max moment with coexisting force*
	42250	A18	54678	-107.8	-411.7	*Min moment with coexisting force*
	42367	A16	37366	-15.22	23.58	*Max axial T(+ve) with coexisting M*
	42834	A16	55247	-20.24	-1116	*Min axial C (-ve) with coexisting M*

				[kNm/m]	[kN/m]	
	Elem	Case	Node	Mx + sgn(Mx)|Mxy|	Nx	
A : x-direction	42274	A18	55183	274.4	-182	*Max moment with coexisting force*
	42188	A18	53606	-216	-605	*Min moment with coexisting force*
	42247	A18	55188	226.9	346.9	*Max axial T(+ve) with coexisting M*
	42582	A18	55188	202.3	-1825	*Min axial C (-ve) with coexisting M*

Nx spike of 708.8kN/m discounted as localised at winching locations

Nx taken as 346.9kN/m



Adsec

SLS crack width check **Minimum reinforcement required for SLS crack width** B25-150 required

Task 2

TABLE CASE DETAILS INTERACTION DIAGRAM MORE CHARTS

Case	Load			Cracked?	Crack width (mm)
	F_x (kN)	M_{yy} (kNm)	M_{zz} (kNm)		
1	-492.1	384	0	UNCRACKED	0.106
2	-411.7	-107.8	0	UNCRACKED	0.001
3	23.6	-15.2	0	UNCRACKED	0.014
4	-1116	-20.2	0	UNCRACKED	
5	-182	274.4	0	UNCRACKED	0.112
6	-605	-216	0	UNCRACKED	0.012
7	346.9	226.9	0	UNCRACKED	0.127
8	-1825	202.3	0	UNCRACKED	

All <0.2mm

Limit 0.2 mm Table 7.1, NEN EN 1992-1-1 OK

B.4. Thrust wall design calculations

ARUP	Job No.	Sheet No.	Rev.
	303154-00	1 of 1	0
	Member/Location		
Job Title: Aramis Launch Shaft	Drg. Ref.		
Calculation: Flexural - RC thrust wall	Made by	Date: 05/06/2024	Chd.

Introduction A calculation to determine the minimum requirements for flexural reinforcement, using Oasys GSA and Adsec

Oasys GSA

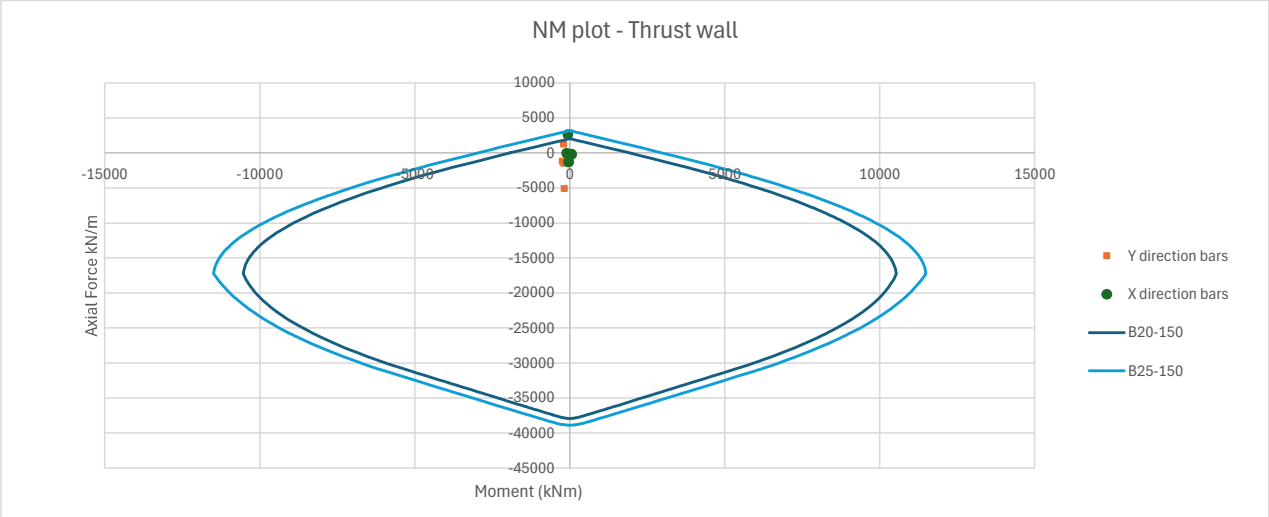
PA4 elements - thrust wall

				[kNm/m]	[kN/m]	
	Elem	**Case**	**Node**	**My + sgn(My)|Myx|**	**Ny**	
B : y-direction	43976	A15	55356	-185.8	-1482	*Max moment with coexisting force*
	44422	A17	55858	-238.1	-1227	*Min moment with coexisting force*
	44192	A15	55653	-197.5	1264	*Max axial T(+ve) with coexisting M*
	44417	A15	55899	-175.9	-5057	*Min axial C (-ve) with coexisting M*

-ve downwards = sagging moment, so rebar at bottom
+ve upwards = hogging moment, so rebar at top

				[kNm/m]	[kN/m]	
	Elem	**Case**	**Node**	**Mx + sgn(Mx)|Mxy|**	**Nx**	
A : x-direction	43976	A17	55356	60.62	-199.2	*Max moment with coexisting force*
	39629	A15	52745	-94.73	-40.16	*Min moment with coexisting force*
	44320	A15	55705	-60.15	2653	*Max axial T(+ve) with coexisting M*
	44293	A15	55753	-32.66	-1320	*Min axial C (-ve) with coexisting M*

-ve downwards = sagging moment, so rebar at bottom
+ve upwards = hogging moment, so rebar at top



Minimum reinforcement required for flexural B25-150

ARUP	Job No.	Sheet No.	Rev.
	303154-00	1 of 1	0
	Member/Location		
Job Title: Aramis Launch Shaft	Drg. Ref.		
Calculation: Shear - RC thrust wall	Made by	Date: 31/05/2024	Chd.

Source/Ref

Checks to undertake:

1. Material properties
2. Minimum reinforcement requirements
3. Shear Capacity

1. Material properties

Tensile strength of steel rebar, fyk (assume B500)	f_{yk} =	500	N/mm^2
Partial factor for steel, γ_s		1.15	
$f_{ywd} = f_{yk}/\gamma_s$	fywd =	435	N/mm^2
Compressive strength of concrete, fck	f_{ck} =	32	N/mm^2
	$f_{ck,cub}$ =	40	N/mm^2
Partial factor for concrete, γ_c		1.5	
a_{cw}		1	NA to Cl 3.1.6
$f_{cd} = a_{cw}f_{ck}/\gamma_c$		21	N/mm^2

2. Minimum reinforcement requirements

Eurocode 2 part 1-1, 9.2.1.1 (1)

Minimum a

C32/40

fctm	3	Mpa
fyk	500	MPa

Element	Depth	$A_{s,min} = 0.26\frac{f_{ctm}b_t d}{f_{yk}}$	*A s,min* = 0.0013btd	*A s,max* = 0.04Ac
	mm	mm2 / m	mm2 / m	mm2/m
Thrust wall	2000	3120	2600	800000

Bar diamet	25	mm
Area	491	mm2
No. bars	6	
Spacing	157	mm

3. Shear Capacity in thrust wall

Check if shear reinforcement is required:

EC1992-1-1 Cl 6.2.2

$$V_{Rd,c} = [C_{Rd,c}k(100\,\rho_1 f_{ck})^{1/3} + k_1\,\sigma_{cp}]\,b_w d \qquad (6.2.a)$$

with a minimum of

$$V_{Rd,c} = (v_{min} + k_1\sigma_{cp})\,b_w d \qquad (6.2.b)$$

where:

f_{ck} is in MPa

$k = 1+\sqrt{\frac{200}{d}} \le 2{,}0$ with d in mm

$\rho_l = \frac{A_{sl}}{b_w d} \le 0{,}02$

A_{sl} is the area of the tensile reinforcement, which extends $\ge (l_{bd} + d)$ beyond the section considered (see Figure 6.3).

b_w is the smallest width of the cross-section in the tensile area [mm]

$\sigma_{cp} = N_{Ed}/A_c < 0{,}2\,f_{cd}$ [MPa]

N_{Ed} is the axial force in the cross-section due to loading or prestressing [in N] (N_{Ed}>0 for compression). The influence of imposed deformations on N_{Ed} may be ignored. (AC1)

A_C is the area of concrete cross section [mm^2]

$V_{Rd,c}$ is [N]

where

f_{ck} =	32	N/mm^2	
d =	2000	mm	
k = 1+(200/d)0.5	1.32		<2.0 ∴ OK
b_w =	1000	mm	per m strip
A_{sl} =	3436	mm^2	Assuming B25 bars at 150mm spacing
ρ_1 =Asl/bwd	0.0017		<0.02 ∴ OK
σ_{cp} =	0.0	N/mm^2	Assumed
$C_{Rd,c}$ = 0.18/γc	0.12		
v_{min} =	0.30	N/mm^2	$v_{min} = 0{,}035\,k^{3/2}\cdot f_{ck}^{1/2}$
k_1 =	0.15		

Design she: $V_{Rd,c}$ =	557534 N		(6.2a)
	558 kN		
$V_{Rd,cmin}$ =	597957 N		(6.2b)
	598 kN		**USE THIS AS BIGGER**
V_{Ed} =	674 kN		From GSA

As VEd > VRd,c shear reinforcement in the slab is required

Check if maximum allowable shear force is exceeded

EC1992-1-1 Cl 6.2.2

The shear force V_{Ed}, calculated without reduction by β, should however always satisfy the condition

$$V_{Ed} \le 0{,}5\, b_w d\, \nu f_{cd} \qquad (6.5)$$

where ν is a strength reduction factor for concrete cracked in shear

Note: The value ν for use in a Country may be found in its National Annex. The recommended value follows from:

$$\nu = 0{,}6\left[1 - \frac{f_{ck}}{250}\right] \quad (f_{ck} \text{ in MPa}) \qquad (6.6N)$$

where

v = 0.523 N/mm²

$V_{Ed,max}$ =	11161600 N	
	11162 kN	

As VEd < VEd,max shear in the slab is acceptable

If shear reinforcement required:

Shear resistance will be calculated in accordance with NEN EN 1992-1-1:2004 clause 6.2.3.

$V_{Rd,s}$ = $[A_{sw} / s] * z * f_{ywd} * \cot\theta$

				outer bar	inner bar
A_{sw} =	402	mm²	Link dia =	16	
Spacing, s =	150	mm	No. legs =	2	
Lever arm, z =	1800	mm (0.9d)			
f_{ywd} =	434.8	N/mm²			
θ =	45	degrees	≡	0.785	radians
α =	45	degrees	≡	0.785	radians

∴ $V_{Rd,s}$ = 2,098 kN

$V_{Rd,max}$ = $\alpha_{cw} * b_w * z * v_1 * f_{cd} / (\cot\theta + \tan\theta)$

α_{cw} =	1.0	(Non-prestressed)			
b_w =	1000.0	mm			
f_{ck} =	32.0	N/mm²	f_{cd} =	21.3	N/mm²
v_1 = v =	0.5232	N/mm²			

∴ $V_{Rd,max}$ = 10,045 kN

$V_{Ed} < V_{Rd,s} < V_{Rd,max}$

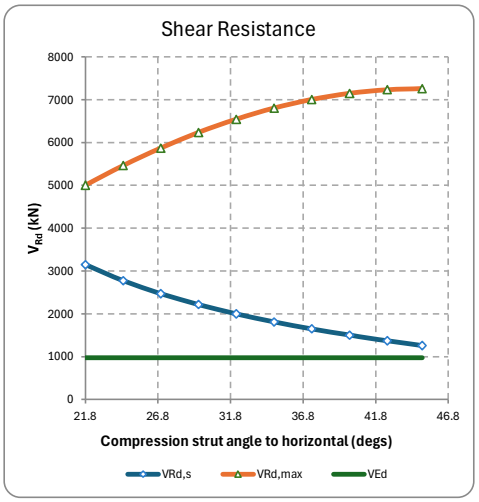
$A_{sw,max}$ = $0.5 * \alpha_{cw} * v_1 * f_{cd} * b_w * s / f_{ywd}$ for θ = 45.0 degrees

∴ $A_{sw,max}$ = 1925.4 mm² ---

θ (degs)	$V_{Rd,s}$ (kN)	$V_{Rd,max}$ (kN)	ΔF_{td} (kN)
21.8	5245	6928	843
24.4	4625	7558	743
27.0	4118	8127	661
29.6	3693	8629	593
32.2	3332	9059	535
34.8	3019	9415	485
37.4	2744	9694	441
40.0	2500	9893	402
42.6	2282	10010	366
45.0	2098	10045	337

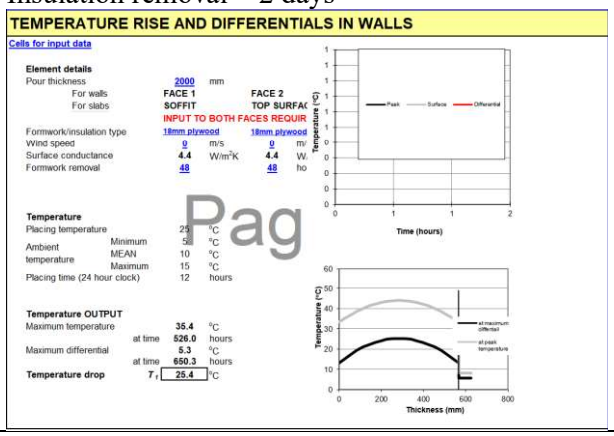
V_{Ed} = 674 kN

$UR_{concrete}$=	0.07
	OK
UR_{links}=	0.32
	OK



21.8	674
45.0	674
2.6	step

Thrust Wall - Early Thermal Checks Summary

	Thrust wall **Assume: internal restraint**
Temperature Risk and Differentials in ground slabs	Pour thickness = 2000mm Insulation type = 18mm plywood Wind speed= 0m/s (since in base of shaft) Insulation removal = 2 days 
Restraint	R=0.42 as per cover sheet of Ciria guide calculation for internal restraint
Crack width	Concrete strength = C32/40 Bar diameter = 20mm Bar spacing = 150mm Cover = 75mm Aggregate type: Ecm kept as default Early age crack width = 0.03mm < 0.3mm (Section 7.3.3, NEN EN 1992-1-1)

C766 Temperature prediction in walls and slabs

This model provides a basis for the comparative assessment of early-age temperature rise and temperature differentials in concrete walls, suspended slabs and ground slabs.

The model is based on the numerical method of Ross and Bray (1949) and is described in detail in Appendix A2

Background data on heat generation were provided by the University of Dundee (Dhir *et al*, 2006) obtained using semi-adiabatic testing to BS EN 196-9:2003

Data for the concrete are input on PAGE 2

Data specific to walls and suspended slabs are input on PAGE 3

Data specific to ground slabs are input on PAGE 4

Each page of this spreadsheet is password protected to prevent accidental deletion of equations. For those wishing to modify or copy the spreadsheet, the password is C766

PAGE 2 ADIABATIC TEMPERATURE			
Concrete mix details	Binder content	kg/m^3	The binder is the total content of cementitious material, including CEM I and CEM I in combination with either fly ash or ground granulated blast-furnace slag, ggbs
	Binder type		Three binder options are available from a drop-down menu CEM I, fly ash, ggbs
	Percent addition	%	The percent of either fly ash or ggbs by weight of binder
	Density	kg/m^3	The wet density of the concrete
	Specific heat	kJ/kg°C	The early-age value when the concrete is only partially hydrated. A value of 1 kJ/kg°C should be used unless more reliable data are available
	Thermal conductivity	W/m°C	Assumed to be 2.0 W/m°C (see Appendix A2 for values for concrete using difference aggregates)
	Multiplier		If results are available for a specific mix, the multiplier may be used to adjust the adiabatic curve to match the data. The default value is 1,
Temperature input	Placing temperature	°C	The temperature of the fresh concrete when poured. As a general rule, the placing temeprature be 5°C above the mean ambinebt temeprature.
	Minimum ambient temperature	°C	These values determine the variable ambient temperature which is assumed to vary on a sinusoidal basis. As a eneral rule the ambient temeprature will vary by ±5°C from the mean value.
	Mean ambient temperature	°C	
	Maximmum ambient temperatur	°C	
	Constant or variable ambient temperature		There is the option to assume either a variable sinusoidal ambient temperature variation or a constant ambient temeprature variation. In the latter case this is assumed to be the MEAN ambient temperature,
	Pacing time	Hours	This assumes a 24 hour clock
Output	Temperature drop T_1		Values are calculated in PAGE 3 and PAGE 4 but are shown on PAGE 2 to enable the effect of changes in the concrete mix and placing temperature to be observed without changing pages. Input on section geometry, formwork etc, is on PAGE 3 and PAGE 4
GRAPHS	Reference curve for the cement		The reference adiabatic curve is estimated from the cement type and content and represents the value at a starting temperature of 20°C
	Adiabatic temperature rise for the concrete		The estimated adiabatic temperature rise is shown based on the reference curve at 20°C and the value obtained when a different placing temperature is used.
PAGE 3 TEMPERATURE CALCULATIONS FOR WALLS AND SUSPENDED SLABS			
Section and formwork details	Section thickness	mm	
	Formwork/insulation type		Formwork/insulation options are available for both faces from drop-down menus. Information for each face must be input. REMEMBER TO INPUT FOR BOTH FACES
	Wind speed	m/s	Assumed to be 4 m/s for typical UK conditions REMEMBER TO INPUT FOR BOTH FACES
	Formwork removal time	hours	The time after casting. Input must be made for both faces. There is the options to remove formwork from opposite faces at different times REMEMBER TO INPUT FOR BOTH FACES
Concrete mix details	Thermal conductivity	W/m°C	Copied from PAGE 2
	Density and specific heat		Copied from PAGE 2
PAGE 4 TEMPERATURE CALCULATIONS FOR GROUND SLABS			
Section and formwork details	Section thickness	mm	This is the minimum dimension through which heat loss is predominant
	Insulation type		Options are available from a drop-down menu for both the soffit formwork and insulation applied to the top surface (if required).
	Wind speed	m/s	Assumed to be 4 m/s for typical UK conditions
	Formwork removal time	hours	The time after casting. Input must be made for both the soffit and the top surface. There is the options to remove opposite faces at different times
Concrete mix details	Thermal conductivity	W/m°C	Copied from PAGE 2
	Density and specific heat		Copied from PAGE 2
Properties of the ground	Specific heat	kJ/kg°C	Default value is 1.2 kJ/kg°C
	Density	kg/m^3	Default value is 2000 kJ/kg°C
	Thermal conductivity	W/m°C	Default value is 1.1 W/m°C
	Temperature	°C	

TEMPERATURE RISE AND DIFFERENTIALS IN WALLS

Cells for input data

Element details

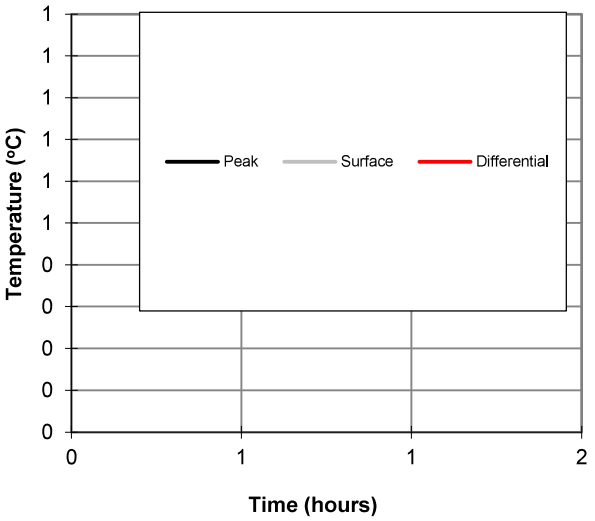
Pour thickness		2000	mm		
For walls		FACE 1		FACE 2	
For slabs		SOFFIT		TOP SURFAC	
		INPUT TO BOTH FACES REQUIRI			
Formwork/insulation type		18mm plywood		18mm plywood	
Wind speed		0	m/s	0	m/
Surface conductance		4.4	W/m^2K	4.4	W/
Formwork removal		48		48	ho

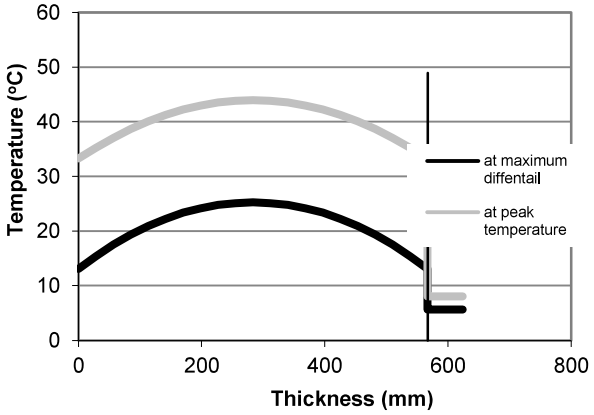
Temperature

Placing temperature		25	°C
Ambient temperature	Minimum	5	°C
	MEAN	10	°C
	Maximum	15	°C
Placing time (24 hour clock)		12	hours

Temperature OUTPUT

Maximum temperature		35.4	°C
	at time	526.0	hours
Maximum differential		5.3	°C
	at time	650.3	hours
Temperature drop	T_1	25.4	°C





C766 Estimation of restraint

This calculator estimates restraint in a wall cast onto a rigid based. The method is based of the ACI approach in which the restraint at the joint is determined by the relative stiffness of the new section and the section against which it is cast. The variation in restraint with height is determined by the length to height ratio.

The method is described in detail in Appendix A5

Each page of this spreadsheet is password protected to prevent accidental deletion of equations. For those wishing to modify or copy the spreadsheet, the password is C766

PAGE 2 RESTRAINT	
Wall geometry	The wall geometry is define by the length L , the height H and the thickness, h. When a new pour that is restrained along one edge abuts an exisiting pour at one end then for the purpose of calculating the restraint profile, the length of the edge restraaining joint should be assumed to be 2 x L, ie it is assumed that there is zero displacement at the joint between the new pour and the adjacent pour
Base geometry	The base geometry is defined by the width W and the thickness, h . The length is assumed to be the same as the length of wall being cast
Relative areas	The ratio of the cross-sectional area (c.s.a.) of the wall A_{new} and the base A_{old} is calculated. The relative areas of influence A_{old} and A_{new} may be difficult to define. For example, when a wall is cast onto a slab, what width of slab should be considered as providing effective restraint? In such circumstances it is recommended that the relative area is assumed to be in proportion to the relative thicknesses, h_{new} and h_{old} , of the wall and slab. The following simple rules may be applied: • For a high wall cast at the edge of a slab assume $A_{new}/A_{old} = h_{new}/h_{old}$ • For a wall cast remote from the edge of a slab assume $A_{new}/A_{old} = h_{new}/2h_{old}$ • For a slab cast against an existing slab assume $A_{new}/A_{old} = h_{new}/h_{old}$
Relative modulus of elasticity	Bamforth and Price (1995) recommends that the ratio E_{new}/E_{old} is assume to be in the range 0.7 to 0.8 at early age. The lower value gives most conservative result and should be used for conditions in which cool-down is most rapid. When estimating restraint to long term deformations, should there be any differential strains, it should be assumed that $E_{new}/E_{old} = 1$
Restraint at the joint R_j	$$R_j = \frac{1}{1 + \frac{A_{new}}{A_{old}} \frac{E_{new}}{E_{old}}}$$
Restraint at the location of the maximum crack width	The maximum restrained strain and crack width occur at a perpendicular distance from the joint of 0.1 times the length of the joint if the new pour is not cast against existing concrete at either end. If the new pour abuts an exisitng pour at one end then the maximum restrained strain and crack width occur at a perpendicular distance from the joint of 0.2 times the length of the joint

C766 Predicting the risk of cracking and controlling crack widths

This calculator provides a basis for estimating the likelihood of cracking and for selecting or checking reinforcement to control crack widths

The approach is based on the method of BS EN 1992-1-1 with adjustments where considered appropriate (see Appendix A8)

Three conditions of restraint are dealt with

PAGE 2 Continuous EDGE restraint.

PAGE 3 END restraint

PAGE 4 INTERNAL restraint from temperature differentials

Each page of this spreadsheet is password protected to prevent accidental deletion of equations. For those wishing to modify or copy the spreadsheet, the password is C766

Input data are as follows (all other parameters are calculated):

PAGE 2 CONTINUOUS EDGE RESTRAINT			
Section details and reinforcement	Section thickness	mm	Section thickness in mm
	Strength class	f_{ck} / $f_{ck,cube}$	Select from the drop-down menu
	Bar diameter	φ	Bar diameter in mm
	Bar spacing	S	Bar spacing in mm
	Cover to reinforcement	c	Minimum cover to reinforcement
	Yield strength of reinforcement	f_{yk}	The characteristic yield strength of the reinforcement in MPa
Early-age restrained strain	Coefficient of expansion	α_c	A value of 12 με / °C is recommended if there is no knowledge of the aggregate type. The recommended value in BS EN 1992-1-1 is 10 με / °C but many aggregates in the UK produce concrete with higher values
	Temperature drop	T_1	This may be obtained from data in C660, the temperature model (Appendix A2) or by independent validated modelling or measurement.
	Creep factor	Kc_1	BS EN 1992-1-1 does not define a creep factor but includes this in the value of restraint R = 0.5 which is the maximum value recommended by BS EN 1992-1-1 and so must cover the worst condition of restraint. When this value of R is used, K_1 should be assumed to be 1. When R is calculated K_1 may be assumed to be 0.65
	Restraint at the location of the maximum crack width	R	BS EN 1992-1-1 permits R to be calculated from the relative stiffness of the element and the member against which it is cast. Where it can be demonstrated that R < 0.8 the calculated value may be used with K_1 = 0.65. The the maximum restrained strain occurs at a distance of 0.1 x length of the wall above the joint with the base. When the wall abuts a previously cast length, the assumed length is calculating the location of the maximum restrained strain is double the actual length.
Risk of early-age cracking	Age at cracking	days	Assumed to be 3 days for early age cracking unless more reliable information is available
	Aggregate type		The tensile strain capacity is the ratio of the tensile strength to the elastic modulus and the latter is affected by the aggregate type. If the aggregate source is known this can be selected from the drop down menu. The default value for unknown aggregate is 1. This is the value dervied from the properties calculated according to BS EN 1992-1-1 for quartz aggregate. Where the elastic modulusat 28 days is known this may be entered and 'measured' should be selected from the drop down menu.
Minimum area of reimforcement $A_{s,min}$	Coefficient for effect edge restraint (Optional)	$k_{R,edge}$	A drop down menu (YES/NO)provides the option to take account of the contributon of the edge restraint in limiting crack opening and the amount of steel required to ensure that yield does not occur.
Crack spacing	Factor for bond	k_1	BS EN 1992-1-1 recommends k_1 = 0.8 but provides a factor of 0.7 where good bond cannot be guaranteed. k_1 = 0.8/0.7 = 1.14 for section >300 mm thick with cover < 50 mm. If cracking is not predicted at early age, then k_1 = 0.8.
Long term cracking	Age at cracking	days	For calculating long term cracking the 28-day properties of the concrete are adopted.
	Long term temperature change	$T2$	Where both the restrained and the restraining element are subject to a change in temperature, T2 is the differential change in temperature between the restrained and the restraining element unless there is external restraint to both elements.
	Drying shrinkage	ε_{cd}	Calculate using the method of BS EN 1992-1-1 unless more reliable information is available. Only apply when causing differential contraction between elements or when the elements acting integrally are subject to external restraint. The drying shrinkage is that occurring at the end of the design life of the structure or part thereof.
	Creep factor	K_{C2}	When restraint is calculated, the long-term creep coefficient is taken as 0.5
	Restraint to T_2	R_2	Long term restraint will be reduced as the ratio of stiffness of the elements reduces and E_n / E_o approaches 1. Consideration must also be given to the element acting integrally.
	Restraint to drying shrinkage	R_3	
Crack spacing	Coefficient for bond characteristics	k_1	If cracking has occurred at early age, the same carack spacing is assumed for long-term cracking. If cracking has not occurred at early age, the crack spacing is recalculated assuming that k_1 =0.8

PAGE 3 END RESTRAINT			
Section details	Section thickness	h	Section thickness in mm
	Strength class	f_{ck} / $f_{ck,cube}$	Select from the drop-down menu.
	Bar diameter	ϕ	Bar diameter in mm
	Bar spacing	s	Bar spacing in mm
	Cover	c	Minimum cover to reinforcement
Properties of reinforcement	Characteristic yield strength	f_{yk}	The characteristic yield strength of the reinforcement = 500 MPa
	Elastic modulus	E_s	200 GPa
Crack sacing	Coefficient for bond characteristics	k_1	See ref to k_1 above
Age at cracking	Early-age	*3 days*	Values of 3 days and 28 days are assumed for the estimation of early age and long-term properties. Values derived on this basis are have been shown to be safe in relation to the design of reinforcement for crack control
	Long term	*28 days*	
Early-age restrained strain	Coefficient of expansion	α_c	A value of 12 με / °C is recommended if there is no knowledge of the aggregate type. The recommended value in BS EN 1992-1-1 is 10 με / °C but many aggregates in the UK produce concrete with higher values
	Temperature drop	T_1	This may be obtained from data in C766, the temperature model (Appendix A2) or by independent validated modelling or measurement.
	Creep factor	Kc_1	BS EN 1992-1-1 does not define a creep factor but includes this in the value of restraint R = 0.5 which is the maximum value recommended by BS EN 1992-1-1 and must cover the worst condition of restraint. When this value of R is used, K_1 should be assumed to be 1. When R is calculated K_1 may be assumed to be 0.65
	Restraint	R	BS EN 1992-1-1 permits R to be calculated from the relative stiffness of the element and the member against which it is cast. Where it can be demonstrated that R < 0.8 the calculated value may be used with K_1 = 0.65. END restraint may be difficult to calculated, particulary when there is a combination of restraints acting on the member, eg a slab on piles with a combination of friction from the base and lateral resistance from piles. Some guidance is provided in Appendix A5, but the user may be required to undertake additional calculations where the arrangement of restraints is complex.
Risk of early-age cracking	Early-age	*Days*	Properties at 3 days are assumed for the estimation of early age cracking. However, when there is confience in the predicted time of cracking there is the option to use other ages.
	Aggregate type		The tensile strain capacity is the ratio of the tensile strength to the elastic modulus and the latter is affected by the aggregate type. If the aggregate source is known this can be selected from the drop down menu. The default value (coefficient = 1) is the value dervied from the properties calculated according to BS EN 1992-1-1 for quartz aggregate. Where the elastic modulus is known this may be entered and 'measured' should be selected from the drop down menu.
Long term cracking	Age at cracking	days	For calculating long-term cracking the 28-day properties of the concrete are adopted.
	Long term temperature change	*T2*	Where both the restrained and the restraining element are subject to a change in temperature, *T2* is the differential change in temperature between the restrained and the restraining element unless there is external restraint to both elements.
	Drying shrinkage	ε_{cd}	Calculate using the method of BS EN 1992-1-1 unless more reliable information is available.
	Creep factor	*Kc2*	
	Restraint to T_2	R_2	Long-term restraint will be reduced as the ratio of stiffness of the elements reduces and E_n / E_o approaches 1 and as elements act integrally.
	Restraint to drying shrinkage	R_3	
	Restraint required to cause cracking	R_{crack}	As restraint may be difficult to calculate, the restraint required to cause cracking is also estimated and the user may judge whether this level of restraint is likely to be achieved in practice.

PAGE 3 INTERNAL RESTRAINT
The input data for dealing with cracking caused by internal restraint is broadly the same as that required for edge restraint with the principal exception that the temperature differential between the centre and the surface ΔT replaces T_1. Other changes inherent in the design calculation are the restraint factor R assumed to be 0.42 and the coefficients k and kc which reflect the difference in the stress distribution within the section compared with the condition of external restraint.

Risk and control of cracking due to internal restraint

Input parameters	Symbol	Unit	Value	Comments/guidance
Section details and reinforcement				
Section thickness	h	mm	2000	
Strength class	f_{ck} / $f_{ck,cube}$	MPa	C32/40	Select from drop down menu
Bar diameter	φ	mm	20	mesh
Bar spacing	s	mm	150	
Cover	c	mm	75	Minimum cover to steel in the direction perpendicular to the crack
Characteristic yield strength of reinforcement	f_{yk}	MPa	500	500 MPa
Area of steel per face per m	A_s	mm^2	**2094**	
Early-age cracking				
Restrained strain				
Coefficient of thermal expansion of concrete	α_c	με/°C	9.5	If aggregate is unknown use 12 με / °C
Temperature differential	DT	°C	25.4	ΔT = Peak temperature - surface temperature
Free differential contraction	$\varepsilon_{free}(ea)$	με	**241**	$\varepsilon_{free(ea)} = \Delta T.\alpha_c$
Creep factor - early age	K_{c1}		0.65	K_{c1} = 0.65
Restraint	R		0.42	taken from cover sheet
Early-age restrained contraction	$\varepsilon_r(ea)$	με	**66**	$\varepsilon_{r(ea)} = R\ K_{c1}\ (\Delta T.\alpha_c)$
Risk of cracking and crack inducing strain				
Aggregate type. Use the 'selected' option and enter the E value if measured or obtained from historical results.	Limestone		0.90	Coefficient for stiffness of aggregate
			30	Enter valueof Ecm(28) if 'measured' or obtained from historical data
Elastic modulus	$E_{cm}(t_c)$	GPa	25.7	Mean value $E_{cm}(t_c)$. BS EN 1992-1-1 values are based on quartz aggregate
Tensile strain capacity	$\varepsilon_{ctu(ea)}$	**με**	**76**	$\varepsilon_{ctu(ea)} = [f_{ctk,0.05}(t_c) / [E_{cm}(t_c) \times K_{c1}]$
Risk of early age cracking	$\varepsilon_{r(ea)}/\varepsilon_{ctu(ea)}$		**0.87**	Low risk of early age cracking if $\varepsilon_{r(ea)}/\varepsilon_{ctu} < 1$.
Early-age crack-inducing strain	$\varepsilon_{cr(ea)}$	με	**28**	$\varepsilon_{cr(ea)} = R_1 K_{c1} (T_1 \alpha_c + \varepsilon_{ca(3)}) - 0.5\ \varepsilon_{ctu(3)} = (\varepsilon_{sm} - \varepsilon_{cm})$
Minimum area of reinforcement $A_{s,min}$				
Age at cracking	t_c	days	3	Assume 3 days unless more reliable information is available
Tensile strength at cracking	$f_{ctk,0.05}(t_c)$	MPa	1.27	Characterisitic tensile strength $f_{ctk,0.05}(t_c)$
Coefficient (BS EN 1992-1-1)	k		0.50	For assumed triangular strain distribution
Coefficient (BS EN 1992-1-1)	k_c		1.00	For pure tension $k_c = 1$
Surface zone representing A_{ct}	$h_{s,min}$	mm	200	$h_{s,min} = k\ k_c\ 0.2\ h$
Minimum area of steel per face per m	$A_{s,min}$	mm^2	**507**	$A_{s,min} = (1\text{-}0.5R)\ k\ k_c (f_{ctk,0.05}(t_c)/ f_{yk})\ A_{ct}$ Highlighted if $A_s < A_{s,min}$
Crack spacing				
Surface zone defining the effective area of concrete in tension, $A_{c,eff}$	$h_{e,ef}$	mm	212.5	$h_{e,ef}$ = lesser of $h/2$ a nd $2.5\ (c + \varphi/2)$
Steel ratio for estimating crack spacing	$\rho_{p,eff}$		0.00986	$\rho_{p,eff} = A_s / A_{c,eff} = A_s / (h_{e,ef} \times 1000)$
Coefficient for bond characteristics.	k_1		0.8	BS EN 1992-1-1 recommends k_1 = 0.8, but provides a factor of 0.7 where good bond cannot be guaranteed. Recommneded k_1 = 0.8/0.7 = 1.14 at early age for sections >300 mm thick with cover <50mm.
Crack spacing	$S_{r,max}$	mm	**945**	$S_{r,max} = 3.4c + 0.425\ k_1\ \varphi/\rho_{p,eff}$
Early age crack width	w_k	mm	**0.03**	$w_k = \varepsilon_{c(ea)}\ S_{r,max}$
Maximum stress in the reinforcement	σ_s	N/mm^2	**26**	$\sigma_s = 2(\varepsilon_{sm} - \varepsilon_{cm})E_s$

ARUP	Job No.	Sheet No.	Rev.
	303154-00	1 of 1	0
	Member/Location		
Job Title: Aramis Launch Shaft	Drg. Ref.		
Calculation: SLS crack width - RC thrust wall	Made by [redacted]	Date: 10/06/2024	Chd.

Introduction A calculation to determine the minimum requirements for SLS crack widths, using Oasys GSA and Adsec

Oasys GSA

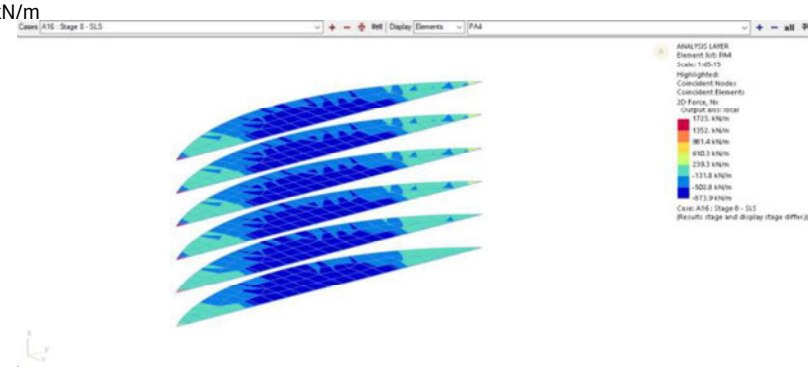
PA4 - RC thrust wall Cases: A2 A4 A6 A8 A10 A12 A14 A16 A18

				[kNm/m] My + sgn(My)|Myx|	[kN/m] Ny	
	Elem	**Case**	**Node**			
B : y-direction	44126	A14	55526	-94.71	-1661	*Max moment with coexisting force*
	44422	A18	55858	-176.2	-904.5	*Min moment with coexisting force*
	44192	A16	55653	-145.3	724.6	*Max axial T(+ve) with coexisting M*
	44417	A16	55899	-126.6	-3628	*Min axial C (-ve) with coexisting M*

				[kNm/m] Mx + sgn(Mx)|Mxy|	[kN/m] Nx	
	Elem	**Case**	**Node**			
A : x-direction	43976	A18	55356	44.86	-145.8	*Max moment with coexisting force*
	39629	A16	52745	-69.07	-37.34	*Min moment with coexisting force*
	44320	A16	55705	-42.71	610.3	*Max axial T(+ve) with coexisting M*
	44293	A16	55753	-24.15	-873.9	*Min axial C (-ve) with coexisting M*

Nx spike of 1723kN/m discounted as localised at winching locations

Nx taken as 610.3kN/m



Adsec

SLS crack width check **Minimum reinforcement required for SLS crack width** B32-150 required

Case	Load			Cracked?	Crack width (mm)
	F_x (kN)	M_{yy} (kNm)	M_{zz} (kNm)		
1	-1661	-94.7	0	UNCRACKED	
2	-904.5	-176.2	0	UNCRACKED	
3	724.6	-145.3	0	UNCRACKED	0.151
4	-3628	-126.6	0	UNCRACKED	
5	-145.8	44.9	0	UNCRACKED	
6	-37.3	-69.1	0	UNCRACKED	0.006
7	610.3	-42.7	0	UNCRACKED	0.122
8	-873.9	-24.2	0	UNCRACKED	

All <0.2mm

Limit 0.2 mm Table 7.1, NEN EN 1992-1-1 OK

B.5. Tunnel eye design calculations

ARUP	Job No.	Sheet No.	Rev.
	303154-00	1 of 1	0
	Member/Location		
Job Title: Aramis Launch Shaft	Drg. Ref.		
Calculation: Flexural - RC tunnel eye	Made by	Date: 05/06/2024	Chd.

Introduction A calculation to determine the minimum requirements for flexural reinforcement, using Oasys GSA and Adsec

Oasys GSA

PA5 elements

				[kNm/m]	[kN/m]
	Elem	Case	Node	My + sgn(My)|Myx|	Ny
B : y-direction	41304	A15	53088	52.97	-789.2
	41297	A15	53203	-191.4	766.7
	41327	A15	53122	-15.78	1500
	42061	A15	54269	-58.86	-6424

Max moment with coexisting force
Min moment with coexisting force
Max axial T(+ve) with coexisting M
Min axial C (-ve) with coexisting M

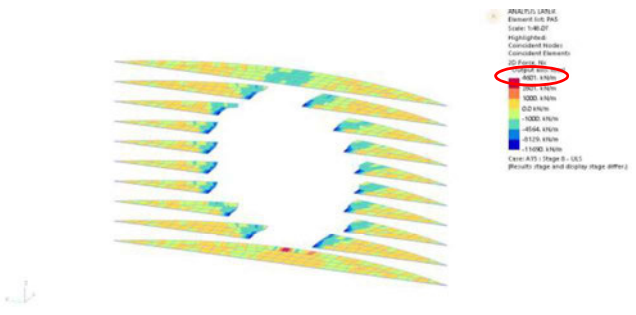
-ve downwards = sagging moment, so rebar at bottom
+ve upwards = hogging moment, so rebar at top

				[kNm/m]	[kN/m]
	Elem	Case	Node	Mx + sgn(Mx)|Mxy|	Nx
A : x-direction	41302	A15	53185	40.39	12.29
	41338	A15	53088	-83.38	312.9
	39681	A15	52786	-4.251	1000
	41827	A15	54041	-8.199	-11690

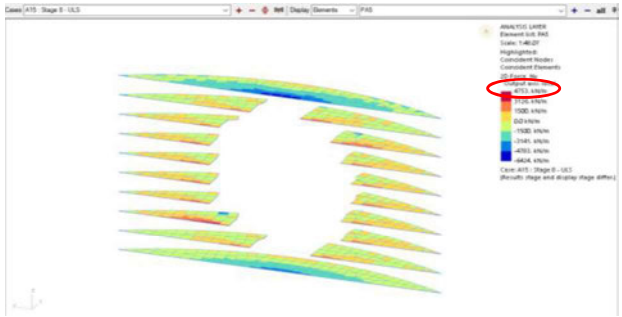
Max moment with coexisting force
Min moment with coexisting force
Max axial T(+ve) with coexisting M
Min axial C (-ve) with coexisting M

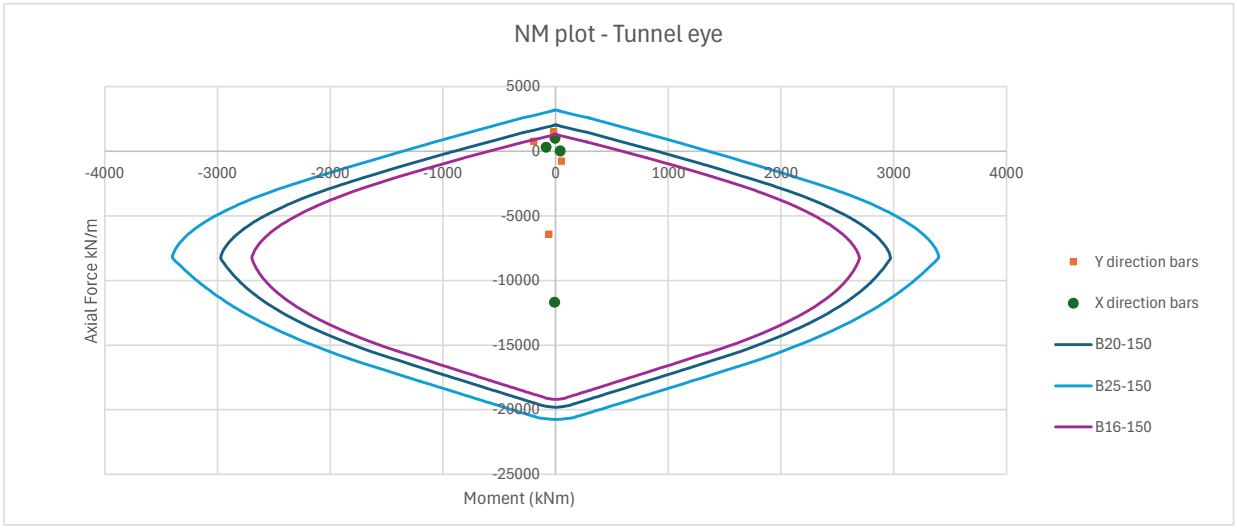
-ve downwards = sagging moment, so rebar at bottom
+ve upwards = hogging moment, so rebar at top

Nx spike of 4601kN/m discounted as localised around tunnel adit
Nx taken as 1000kN/m



Ny spike of 4753kN/m discounted as localised around tunnel adit
Ny taken as 1500kN/m





Minimum reinforcement required for flexural B20-150

ARUP	Job No.	Sheet No.	Rev.
	303154-00	1 of 1	0
	Member/Location		
Job Title: Aramis Launch Shaft	Drg. Ref.		
Calculation: Shear - RC tunnel eye	Made by	Date: 31/05/2024	Chd.

Source/Ref

Checks to undertake:

1. Material properties
2. Minimum reinforcement requirements
3. Shear Capacity

1. Material properties

Tensile strength of steel rebar, fyk (assume B500)	f_{yk} =	500	N/mm²
Partial factor for steel, γ_s		1.15	
$f_{ywd} = f_{yk}/\gamma_s$	fywd =	435	N/mm²
Compressive strength of concrete, fck	f_{ck} =	32	N/mm²
	$f_{ck,cub}$ =	40	N/mm²
Partial factor for concrete, γ_c		1.5	
a_{cw}		1	NA to Cl 3.1.6
$f_{cd} = a_{cw} f_{ck}/\gamma_c$		21	N/mm²

2. Minimum reinforcement requirements

Eurocode 2 part 1-1, 9.2.1.1 (1)

Minimum a

C32/40

fctm	3	Mpa
fyk	500	MPa

Element	Depth	$A_{s,min} = 0.26 \frac{f_{ctm} b_t d}{f_{yk}}$	$A_{s,min}$ = 0.0013btd	$A_{s,max}$ = 0.04Ac
	mm	mm2 / m	mm2 / m	mm2/m
Tunnel eye	1000	1560	1300	400000

Bar diamet	20	mm
Area	314	mm2
No. bars	5	
Spacing	201	mm

3. Shear Capacity in tunnel eye

Check if shear reinforcement is required:

EC1992-1-1 Cl 6.2.2

$$V_{Rd,c} = [C_{Rd,c} k (100 \rho_l f_{ck})^{1/3} + k_1 \sigma_{cp}] b_w d \quad (6.2.a)$$

with a minimum of

$$V_{Rd,c} = (v_{min} + k_1 \sigma_{cp}) b_w d \quad (6.2.b)$$

where:

f_{ck} is in MPa

$k = 1 + \sqrt{\frac{200}{d}} \le 2,0$ with d in mm

$\rho_l = \frac{A_{sl}}{b_w d} \le 0,02$

A_{sl} is the area of the tensile reinforcement, which extends $\ge (l_{bd} + d)$ beyond the section considered (see Figure 6.3).

b_w is the smallest width of the cross-section in the tensile area [mm]

$\sigma_{cp} = N_{Ed}/A_c < 0,2\, f_{cd}$ [MPa]

N_{Ed} is the axial force in the cross-section due to loading or prestressing [in N] ($N_{Ed}>0$ for compression). The influence of imposed deformations on N_{Ed} may be ignored.

A_C is the area of concrete cross section [mm²]

$V_{Rd,c}$ is [N]

where

f_{ck} =	32	N/mm²		
d =	1000	mm		
k = 1+(200/d)0.5	1.45		<2.0 ∴ OK	
b_w =	1000	mm	per m strip	
A_{sl} =	2199	mm²	Assuming B20 bars at 150mm spacing	
ρ_l =Asl/bwd	0.0022		<0.02 ∴ OK	
σ_{cp} =	0.0	N/mm²	Assumed	
$C_{Rd,c}$ = 0.18/γc	0.12			
v_{min} =	0.34	N/mm²	$v_{min} = 0,035 k^{3/2} \cdot f_{ck}^{1/2}$	
k_1 =	0.15			

Design she: $V_{Rd,c}$ =	332797	N	(6.2a)	
	333	kN		
$V_{Rd,cmin}$ =	344701	N	(6.2b)	
	345	kN	**USE THIS AS BIGGER**	
V_{Ed} =	340	kN	From GSA	

As VEd < VRd,c shear reinforcement in the slab is NOT required

Check if maximum allowable shear force is exceeded

EC1992-1-1 Cl 6.2.2

The shear force V_{Ed}, calculated without reduction by β, should however always satisfy the condition

$$V_{Ed} \le 0{,}5\, b_{wd}\, \nu\, f_{cd} \qquad (6.5)$$

where ν is a strength reduction factor for concrete cracked in shear

Note: The value ν for use in a Country may be found in its National Annex. The recommended value follows from:

$$\nu = 0{,}6\left[1 - \frac{f_{ck}}{250}\right] \quad (f_{ck} \text{ in MPa}) \qquad (6.6N)$$

where

v = 0.523 N/mm²

$V_{Ed,max}$ = 5580800 N

5581 kN

As VEd < VEd,max shear in the slab is acceptable

If shear reinforcement required:

Shear resistance will be calculated in accordance with BS EN 1992-1-1:2004 clause 6.2.3.

$V_{Rd,s}$ = $[A_{sw} / s] * z * f_{ywd} * \cot\theta$

				outer bar	inner bar
A_{sw} =	402	mm²	Link dia =	16	
Spacing, s =	150	mm	No. legs =	2	
Lever arm, z =	900	mm (0.9d)			
f_{ywd} =	434.8	N/mm²			
θ =	45	degrees	≡	0.785	radians
α =	45	degrees	≡	0.785	radians

$\therefore V_{Rd,s}$ = 1,049 kN

$V_{Rd,max}$ = $\alpha_{cw} * b_w * z * v_1 * f_{cd} / (\cot\theta + \tan\theta)$

α_{cw} =	1.0	(Non-prestressed)			
b_w =	1000.0	mm			
f_{ck} =	32.0	N/mm²	f_{cd} =	21.3	N/mm²
$v_1 = v$ =		0.5232	N/mm²		

$\therefore V_{Rd,max}$ = 5,023 kN

$V_{Ed} < V_{Rd,s} < V_{Rd,ma}$

$A_{sw,max}$ = $0.5 * \alpha_{cw} * v_1 * f_{cd} * b_w * s / f_{ywd}$ for θ = 45.0 degrees

$\therefore A_{sw,max}$ = 1925.4 mm² ---

θ (degs)	$V_{Rd,s}$ (kN)	$V_{Rd,max}$ (kN)	ΔF_{td} (kN)
21.8	2623	3464	425
24.4	2313	3779	375
27.0	2059	4063	334
29.6	1847	4314	299
32.2	1666	4530	270
34.8	1509	4708	245
37.4	1372	4847	222
40.0	1250	4946	203
42.6	1141	5005	185
45.0	1049	5023	170

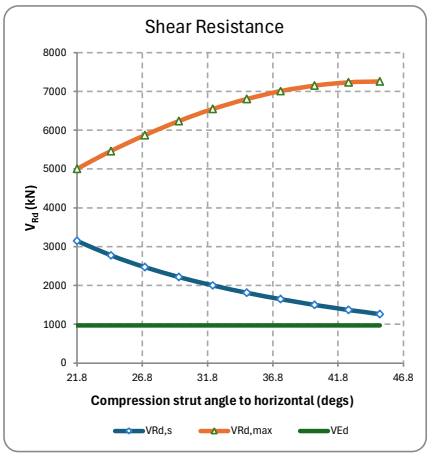
V_{Ed} = 340 kN

$UR_{concrete}$= 0.07

OK

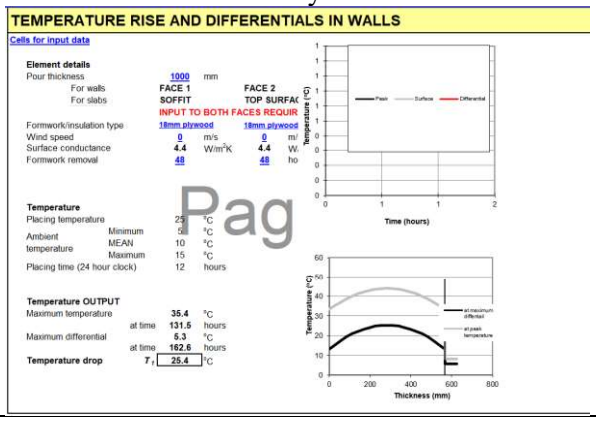
UR_{links}= 0.32

OK



21.8	340
45.0	340
2.6	step

Tunnel Eye - Early Thermal Checks Summary

	Tunnel eye **Assume: internal restraint**
Temperature Risk and Differentials in ground slabs	Pour thickness = 1000mm Insulation type = 18mm plywood Wind speed= 0m/s (since in base of shaft) Insulation removal = 2 days 
Restraint	R=0.42 as per cover sheet of Ciria guide calculation for internal restraint
Crack width	Concrete strength = C32/40 Bar diameter = 16mm Bar spacing = 150mm Cover = 75mm Aggregate type: Ecm kept as default Early age crack width = 0.03mm < 0.3mm (Section 7.3.3, NEN EN 1992-1-1)

C766 Temperature prediction in walls and slabs

This model provides a basis for the comparative assessment of early-age temperature rise and temperature differentials in concrete walls, suspended slabs and ground slabs.

The model is based on the numerical method of Ross and Bray (1949) and is described in detail in Appendix A2

Background data on heat generation were provided by the University of Dundee (Dhir *et al*, 2006) obtained using semi-adiabatic testing to BS EN 196-9:2003

Data for the concrete are input on PAGE 2

Data specific to walls and suspended slabs are input on PAGE 3

Data specific to ground slabs are input on PAGE 4

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PAGE 2 ADIABATIC TEMPERATURE			
Concrete mix details	Binder content	kg/m^3	The binder is the total content of cementitious material, including CEM I and CEM I in combination with either fly ash or ground granulated blast-furnace slag, ggbs
	Binder type		Three binder options are available from a drop-down menu CEM I, fly ash, ggbs
	Percent addition	%	The percent of either fly ash or ggbs by weight of binder
	Density	kg/m^3	The wet density of the concrete
	Specific heat	kJ/kg°C	The early-age value when the concrete is only partially hydrated. A value of 1 kJ/kg°C should be used unless more reliable data are available
	Thermal conductivity	W/m°C	Assumed to be 2.0 W/m°C (see Appendix A2 for values for concrete using difference aggregates)
	Multiplier		If results are available for a specific mix, the multiplier may be used to adjust the adiabatic curve to match the data. The default value is 1,
Temperature input	Placing temperature	°C	The temperature of the fresh concrete when poured. As a general rule, the placing temeprature be 5°C above the mean ambinebt temeprature.
	Minimum ambient temperature	°C	These values determine the variable ambient temperature which is assumed to vary on a sinusoidal basis. As a eneral rule the ambient temeprature will vary by ±5°C from the mean value.
	Mean ambient temperature	°C	
	Maximmum ambient temperatur	°C	
	Constant or variable ambient temperature		There is the option to assume either a variable sinusoidal ambient temperature variation or a constant ambient temeprature variation. In the latter case this is assumed to be the MEAN ambient temperature,
	Pacing time	Hours	This assumes a 24 hour clock
Output	Temperature drop T_1		Values are calculated in PAGE 3 and PAGE 4 but are shown on PAGE 2 to enable the effect of changes in the concrete mix and placing temperature to be observed without changing pages. Input on section geometry, formwork etc, is on PAGE 3 and PAGE 4
GRAPHS	Reference curve for the cement		The reference adiabatic curve is estimated from the cement type and content and represents the value at a starting temperature of 20°C
	Adiabatic temperature rise for the concrete		The estimated adiabatic temperature rise is shown based on the reference curve at 20°C and the value obtained when a different placing temperature is used.
PAGE 3 TEMPERATURE CALCULATIONS FOR WALLS AND SUSPENDED SLABS			
Section and formwork details	Section thickness	mm	
	Formwork/insulation type		Formwork/insulation options are available for both faces from drop-down menus. Information for each face must be input. REMEMBER TO INPUT FOR BOTH FACES
	Wind speed	m/s	Assumed to be 4 m/s for typical UK conditions REMEMBER TO INPUT FOR BOTH FACES
	Formwork removal time	hours	The time after casting. Input must be made for both faces. There is the options to remove formwork from opposite faces at different times REMEMBER TO INPUT FOR BOTH FACES
Concrete mix details	Thermal conductivity	W/m°C	Copied from PAGE 2
	Density and specific heat		Copied from PAGE 2
PAGE 4 TEMPERATURE CALCULATIONS FOR GROUND SLABS			
Section and formwork details	Section thickness	mm	This is the minimum dimension through which heat loss is predominant
	Insulation type		Options are available from a drop-down menu for both the soffit formwork and insulation applied to the top surface (if required).
	Wind speed	m/s	Assumed to be 4 m/s for typical UK conditions
	Formwork removal time	hours	The time after casting. Input must be made for both the soffit and the top surface. There is the options to remove opposite faces at different times
Concrete mix details	Thermal conductivity	W/m°C	Copied from PAGE 2
	Density and specific heat		Copied from PAGE 2
Properties of the ground	Specific heat	kJ/kg°C	Default value is 1.2 kJ/kg°C
	Density	kg/m^3	Default value is 2000 kJ/kg°C
	Thermal conductivity	W/m°C	Default value is 1.1 W/m°C
	Temperature	°C	

TEMPERATURE RISE AND DIFFERENTIALS IN WALLS

Cells for input data

Element details

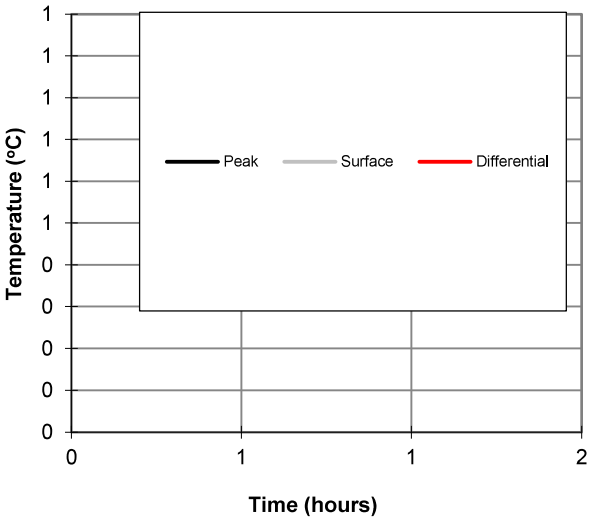
Pour thickness		1000	mm		
For walls		FACE 1		FACE 2	
For slabs		SOFFIT		TOP SURFAC	
		INPUT TO BOTH FACES REQUIRI			
Formwork/insulation type		18mm plywood		18mm plywood	
Wind speed		0	m/s	0	m/
Surface conductance		4.4	W/m^2K	4.4	W/
Formwork removal		48		48	ho

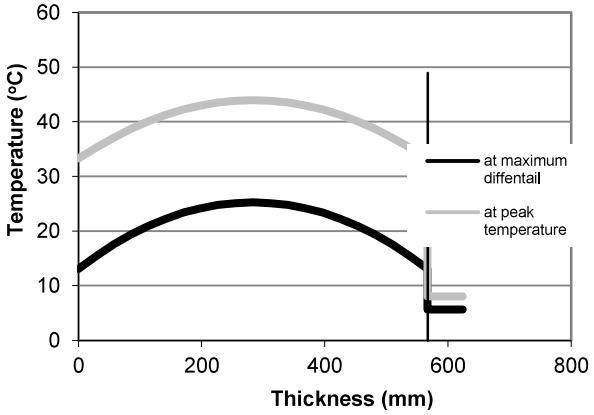
Temperature

Placing temperature		25	°C
Ambient temperature	Minimum	5	°C
	MEAN	10	°C
	Maximum	15	°C
Placing time (24 hour clock)		12	hours

Temperature OUTPUT

Maximum temperature		**35.4**	°C
	at time	**131.5**	hours
Maximum differential		**5.3**	°C
	at time	**162.6**	hours
Temperature drop	T_1	**25.4**	°C





C766 Estimation of restraint

This calculator estimates restraint in a wall cast onto a rigid based. The method is based of the ACI approach in which the restraint at the joint is determined by the relative stiffness of the new section and the section against which it is cast. The variation in restraint with height is determined by the length to height ratio.

The method is described in detail in Appendix A5

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PAGE 2 RESTRAINT	
Wall geometry	The wall geometry is define by the length L , the height H and the thickness, h. When a new pour that is restrained along one edge abuts an exisiting pour at one end then for the purpose of calculating the restraint profile, the length of the edge restraaining joint should be assumed to be 2 x L, ie it is assumed that there is zero displacement at the joint between the new pour and the adjacent pour
Base geometry	The base geometry is defined by the width W and the thickness, h . The length is assumed to be the same as the length of wall being cast
Relative areas	The ratio of the cross-sectional area (c.s.a.) of the wall A_{new} and the base A_{old} is calculated. The relative areas of influence A_{old} and A_{new} may be difficult to define. For example, when a wall is cast onto a slab, what width of slab should be considered as providing effective restraint? In such circumstances it is recommended that the relative area is assumed to be in proportion to the relative thicknesses, h_{new} and h_{old} , of the wall and slab. The following simple rules may be applied: • For a high wall cast at the edge of a slab assume $A_{new}/A_{old} = h_{new}/h_{old}$ • For a wall cast remote from the edge of a slab assume $A_{new}/A_{old} = h_{new}/2h_{old}$ • For a slab cast against an existing slab assume $A_{new}/A_{old} = h_{new}/h_{old}$
Relative modulus of elasticity	Bamforth and Price (1995) recommends that the ratio E_{new}/E_{old} is assume to be in the range 0.7 to 0.8 at early age. The lower value gives most conservative result and should be used for conditions in which cool-down is most rapid. When estimating restraint to long term deformations, should there be any differential strains, it should be assumed that $E_{new}/E_{old} = 1$
Restraint at the joint R_j	$$R_j = \frac{1}{1 + \frac{A_{new}}{A_{old}} \frac{E_{new}}{E_{old}}}$$
Restraint at the location of the maximum crack width	The maximum restrained strain and crack width occur at a perpendicular distance from the joint of 0.1 times the length of the joint if the new pour is not cast against existing concrete at either end. If the new pour abuts an exisitng pour at one end then the maximum restrained strain and crack width occur at a perpendicular distance from the joint of 0.2 times the length of the joint

C766 Predicting the risk of cracking and controlling crack widths

This calculator provides a basis for estimating the likelihood of cracking and for selecting or checking reinforcement to control crack widths

The approach is based on the method of BS EN 1992-1-1 with adjustments where considered appropriate (see Appendix A8)

Three conditions of restraint are dealt with

PAGE 2 Continuous EDGE restraint.

PAGE 3 END restraint

PAGE 4 INTERNAL restraint from temperature differentials

Each page of this spreadsheet is password protected to prevent accidental deletion of equations. For those wishing to modify or copy the spreadsheet, the password is C766

Input data are as follows (all other parameters are calculated):

PAGE 2 CONTINUOUS EDGE RESTRAINT			
Section details and reinforcement	Section thickness	mm	Section thickness in mm
	Strength class	f_{ck} / $f_{ck,cube}$	Select from the drop-down menu
	Bar diameter	φ	Bar diameter in mm
	Bar spacing	S	Bar spacing in mm
	Cover to reinforcement	c	Minimum cover to reinforcement
	Yield strength of reinforcement	f_{yk}	The characteristic yield strength of the reinforcement in MPa
Early-age restrained strain	Coefficient of expansion	α_c	A value of 12 με / °C is recommended if there is no knowledge of the aggregate type. The recommended value in BS EN 1992-1-1 is 10 με / °C but many aggregates in the UK produce concrete with higher values
	Temperature drop	T_1	This may be obtained from data in C660, the temperature model (Appendix A2) or by independent validated modelling or measurement.
	Creep factor	Kc_1	BS EN 1992-1-1 does not define a creep factor but includes this in the value of restraint R = 0.5 which is the maximum value recommended by BS EN 1992-1-1 and so must cover the worst condition of restraint. When this value of R is used, K_1 should be assumed to be 1. When R is calculated K_1 may be assumed to be 0.65
	Restraint at the location of the maximum crack width	R	BS EN 1992-1-1 permits R to be calculated from the relative stiffness of the element and the member against which it is cast. Where it can be demonstrated that R < 0.8 the calculated value may be used with K_1 = 0.65. The the maximum restrained strain occurs at a distance of 0.1 x length of the wall above the joint with the base. When the wall abuts a previously cast length, the assumed length is calculating the location of the maximum restrained strain is double the actual length.
Risk of early-age cracking	Age at cracking	days	Assumed to be 3 days for early age cracking unless more reliable information is available
	Aggregate type		The tensile strain capacity is the ratio of the tensile strength to the elastic modulus and the latter is affected by the aggregate type. If the aggregate source is known this can be selected from the drop down menu. The default value for unknown aggregate is 1. This is the value dervied from the properties calculated according to BS EN 1992-1-1 for quartz aggregate. Where the elastic modulusat 28 days is known this may be entered and 'measured' should be selected from the drop down menu.
Minimum area of reimforcement $A_{s,min}$	Coefficient for effect edge restraint (Optional)	$k_{R,edge}$	A drop down menu (YES/NO)provides the option to take account of the contributon of the edge restraint in limiting crack opening and the amount of steel required to ensure that yield does not occur.
Crack spacing	Factor for bond	k_1	BS EN 1992-1-1 recommends k_1 = 0.8 but provides a factor of 0.7 where good bond cannot be guaranteed. k_1 = 0.8/0.7 = 1.14 for section >300 mm thick with cover < 50 mm. If cracking is not predicted at early age, then k_1 = 0.8.
Long term cracking	Age at cracking	days	For calculating long term cracking the 28-day properties of the concrete are adopted.
	Long term temperature change	$T2$	Where both the restrained and the restraining element are subject to a change in temperature, T2 is the differential change in temperature between the restrained and the restraining element unless there is external restraint to both elements.
	Drying shrinkage	ε_{cd}	Calculate using the method of BS EN 1992-1-1 unless more reliable information is available. Only apply when causing differential contraction between elements or when the elements acting integrally are subject to external restraint. The drying shrinkage is that occurring at the end of the design life of the structure or part thereof.
	Creep factor	K_{C2}	When restraint is calculated, the long-term creep coefficient is taken as 0.5
	Restraint to T_2	R_2	Long term restraint will be reduced as the ratio of stiffness of the elements reduces and E_n / E_o approaches 1. Consideration must also be given to the element acting integrally.
	Restraint to drying shrinkage	R_3	
Crack spacing	Coefficient for bond characteristics	k_1	If cracking has occurred at early age, the same carack spacing is assumed for long-term cracking. If cracking has not occurred at early age, the crack spacing is recalculated assuming that k_1 =0.8

PAGE 3 END RESTRAINT			
Section details	Section thickness	h	Section thickness in mm
	Strength class	f_{ck} / $f_{ck,cube}$	Select from the drop-down menu.
	Bar diameter	ϕ	Bar diameter in mm
	Bar spacing	s	Bar spacing in mm
	Cover	c	Minimum cover to reinforcement
Properties of reinforcement	Characteristic yield strength	f_{yk}	The characteristic yield strength of the reinforcement = 500 MPa
	Elastic modulus	E_s	200 GPa
Crack sacing	Coefficient for bond characteristics	k_1	See ref to k_1 above
Age at cracking	Early-age	*3 days*	Values of 3 days and 28 days are assumed for the estimation of early age and long-term properties. Values derived on this basis are have been shown to be safe in relation to the design of reinforcement for crack control
	Long term	*28 days*	
Early-age restrained strain	Coefficient of expansion	α_c	A value of 12 με / °C is recommended if there is no knowledge of the aggregate type. The recommended value in BS EN 1992-1-1 is 10 με / °C but many aggregates in the UK produce concrete with higher values
	Temperature drop	T_1	This may be obtained from data in C766, the temperature model (Appendix A2) or by independent validated modelling or measurement.
	Creep factor	Kc_1	BS EN 1992-1-1 does not define a creep factor but includes this in the value of restraint R = 0.5 which is the maximum value recommended by BS EN 1992-1-1 and must cover the worst condition of restraint. When this value of R is used, K_1 should be assumed to be 1. When R is calculated K_1 may be assumed to be 0.65
	Restraint	R	BS EN 1992-1-1 permits R to be calculated from the relative stiffness of the element and the member against which it is cast. Where it can be demonstrated that R < 0.8 the calculated value may be used with K_1 = 0.65. END restraint may be difficult to calculated, particulary when there is a combination of restraints acting on the member, eg a slab on piles with a combination of friction from the base and lateral resistance from piles. Some guidance is provided in Appendix A5, but the user may be required to undertake additional calculations where the arrangement of restraints is complex.
Risk of early-age cracking	Early-age	*Days*	Properties at 3 days are assumed for the estimation of early age cracking. However, when there is confience in the predicted time of cracking there is the option to use other ages.
	Aggregate type		The tensile strain capacity is the ratio of the tensile strength to the elastic modulus and the latter is affected by the aggregate type. If the aggregate source is known this can be selected from the drop down menu. The default value (coefficient = 1) is the value dervied from the properties calculated according to BS EN 1992-1-1 for quartz aggregate. Where the elastic modulus is known this may be entered and 'measured' should be selected from the drop down menu.
Long term cracking	Age at cracking	days	For calculating long-term cracking the 28-day properties of the concrete are adopted.
	Long term temperature change	*T2*	Where both the restrained and the restraining element are subject to a change in temperature, *T2* is the differential change in temperature between the restrained and the restraining element unless there is external restraint to both elements.
	Drying shrinkage	ε_{cd}	Calculate using the method of BS EN 1992-1-1 unless more reliable information is available.
	Creep factor	*Kc2*	
	Restraint to T_2	R_2	Long-term restraint will be reduced as the ratio of stiffness of the elements reduces and E_n / E_o approaches 1 and as elements act integrally.
	Restraint to drying shrinkage	R_3	
	Restraint required to cause cracking	R_{crack}	As restraint may be difficult to calculate, the restraint required to cause cracking is also estimated and the user may judge whether this level of restraint is likely to be achieved in practice.

PAGE 3 INTERNAL RESTRAINT
The input data for dealing with cracking caused by internal restraint is broadly the same as that required for edge restraint with the principal exception that the temperature differential between the centre and the surface ΔT replaces T_1. Other changes inherent in the design calculation are the restraint factor R assumed to be 0.42 and the coefficients k and kc which reflect the difference in the stress distribution within the section compared with the condition of external restraint.

Risk and control of cracking due to internal restraint

Input parameters	Symbol	Unit	Value	Comments/guidance
Section details and reinforcement				
Section thickness	h	mm	1000	
Strength class	f_{ck} / $f_{ck,cube}$	MPa	C32/40	Select from drop down menu
Bar diameter	φ	mm	16	mesh
Bar spacing	s	mm	150	
Cover	c	mm	75	Minimum cover to steel in the direction perpendicular to the crack
Characteristic yield strength of reinforcement	f_{yk}	MPa	500	500 MPa
Area of steel per face per m	A_s	mm^2	**1340**	
Early-age cracking				
Restrained strain				
Coefficient of thermal expansion of concrete	α_c	με/°C	9.5	If aggregate is unknown use 12 με / °C
Temperature differential	DT	°C	25.4	ΔT = Peak temperature - surface temperature
Free differential contraction	$\varepsilon_{free}(ea)$	με	**241**	$\varepsilon_{free(ea)} = \Delta T.\alpha_c$
Creep factor - early age	K_{c1}		0.65	K_{c1} = 0.65
Restraint	R		0.42	taken from cover sheet
Early-age restrained contraction	$\varepsilon_r(ea)$	με	**66**	$\varepsilon_{r(ea)} = R\ K_{c1}\ (\Delta T.\alpha_c)$
Risk of cracking and crack inducing strain				
Aggregate type. Use the 'selected' option and enter the E value if measured or obtained from historical results.	Limestone		0.90	Coefficient for stiffness of aggregate
			30	Enter valueof Ecm(28) if 'measured' or obtained from historical data
Elastic modulus	$E_{cm}(t_c)$	GPa	25.7	Mean value $E_{cm}(t_c)$. BS EN 1992-1-1 values are based on quartz aggregate
Tensile strain capacity	$\varepsilon_{ctu(ea)}$	**με**	**76**	$\varepsilon_{ctu(ea)} = [f_{ctk,0.05}(t_c) / [E_{cm}(t_c) \times K_{c1}]$
Risk of early age cracking	$\varepsilon_{r(ea)}/\varepsilon_{ctu(ea)}$		**0.87**	Low risk of early age cracking if $\varepsilon_{r(ea)}/\varepsilon_{ctu} < 1$.
Early-age crack-inducing strain	$\varepsilon_{cr(ea)}$	με	**28**	$\varepsilon_{cr(ea)} = R_1 K_{c1} (T_1 \alpha_c + \varepsilon_{ca(3)}) - 0.5\ \varepsilon_{ctu(3)} = (\varepsilon_{sm} - \varepsilon_{cm})$
Minimum area of reinforcement $A_{s,min}$				
Age at cracking	t_c	days	3	Assume 3 days unless more reliable information is available
Tensile strength at cracking	$f_{ctk,0.05}(t_c)$	MPa	1.27	Characterisitic tensile strength $f_{ctk,0.05}(t_c)$
Coefficient (BS EN 1992-1-1)	k		0.50	For assumed triangular strain distribution
Coefficient (BS EN 1992-1-1)	k_c		1.00	For pure tension $k_c = 1$
Surface zone representing A_{ct}	$h_{s,min}$	mm	100	$h_{s,min} = k\ k_c\ 0.2\ h$
Minimum area of steel per face per m	$A_{s,min}$	mm^2	**253**	$A_{s,min} = (1\text{-}0.5R)\ k\ k_c (f_{ctk,0.05}(t_c)/ f_{yk})\ A_{ct}$ Highlighted if $A_s < A_{s,min}$
Crack spacing				
Surface zone defining the effective area of concrete in tension, $A_{c,eff}$	$h_{e,ef}$	mm	207.5	$h_{e,ef}$ = lesser of $h/2$ a nd $2.5\ (c + \varphi/2)$
Steel ratio for estimating crack spacing	$\rho_{p,eff}$		0.00646	$\rho_{p,eff} = A_s / A_{c,eff} = A_s / (h_{e,ef} \times 1000)$
Coefficient for bond characteristics.	k_1		0.8	BS EN 1992-1-1 recommends k_1 = 0.8, but provides a factor of 0.7 where good bond cannot be guaranteed. Recommneded k_1 = 0.8/0.7 = 1.14 at early age for sections >300 mm thick with cover <50mm.
Crack spacing	$S_{r,max}$	mm	**1097**	$S_{r,max} = 3.4c + 0.425\ k_1\ \varphi/\rho_{p,eff}$
Early age crack width	w_k	mm	**0.03**	$w_k = \varepsilon_{c(ea)}\ S_{r,max}$
Maximum stress in the reinforcement	σ_s	N/mm^2	**26**	$\sigma_s = 2(\varepsilon_{sm} - \varepsilon_{cm})E_s$

ARUP	Job No.	Sheet No.	Rev.
	303154-00	1 of 1	02
	Member/Location		
Job Title: Aramis Launch Shaft	Drg. Ref.		
Calculation: SLS crack width - RC tunnel eye	Made by	Date: 17/10/2024	Chd.

Introduction A calculation to determine the minimum requirements for SLS crack widths, using Oasys GSA and Adsec

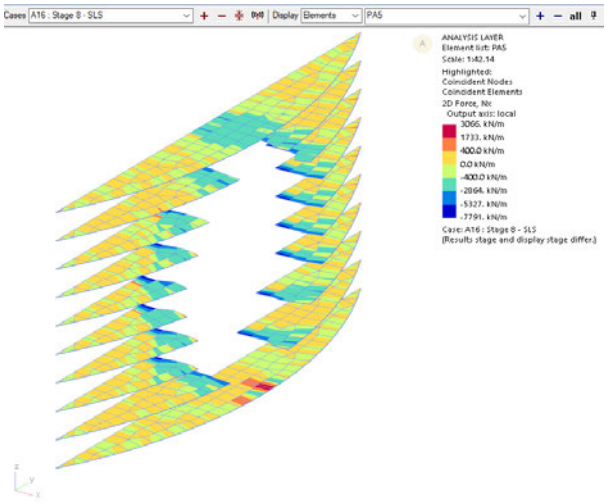
Oasys GSA

PA5 - tunnel eye Cases: A2 A4 A6 A8 A10 A12 A14 A16 A18

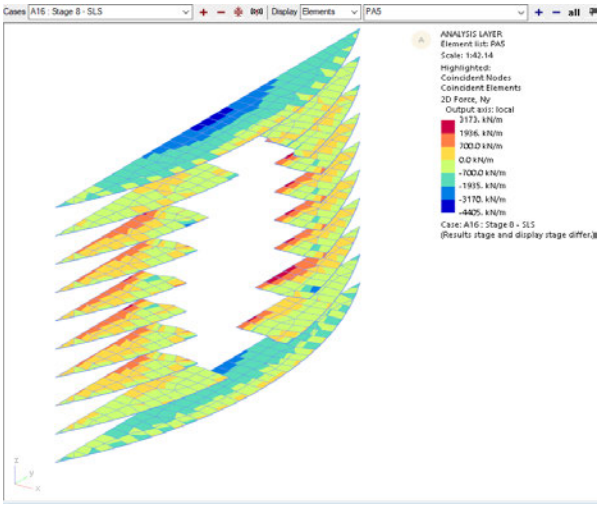
	Elem	Case	Node	[kNm/m] My + sgn(My)|Myx|	[kN/m] Ny	
B : y-direction	41304	A16	53088	31.24	-582.1	*Max moment with coexisting force*
	41297	A16	53203	-114.6	400	*Min moment with coexisting force*
	41327	A16	53122	-9.564	700	*Max axial T(+ve) with coexisting M*
	42061	A16	54269	-41.06	-4405	*Min axial C (-ve) with coexisting M*

	Elem	Case	Node	[kNm/m] Mx + sgn(Mx)|Mxy|	[kN/m] Nx	
A : x-direction	41850	A16	46270	24.29	10.23	*Max moment with coexisting force*
	41338	A16	53088	-49.43	201.1	*Min moment with coexisting force*
	39681	A16	52786	-3.949	400	*Max axial T(+ve) with coexisting M*
	41827	A16	54041	-5.649	-7791	*Min axial C (-ve) with coexisting M*

Nx spike of 3066kN/m discounted as localised peak
Nx taken as 400kN/m



Ny spike of 3173kN/m discounted as localised peak
Ny taken as 700kN/m



Adsec

SLS crack width check **Minimum reinforcement required for SLS crack width** B16-150 required

Case	Load F_x (kN)	Load M_{yy} (kNm)	Load M_{zz} (kNm)	Cracked?
1	-582.1	31.2	0	UNCRACKED
2	400	-114.6	0	UNCRACKED
3	700	-9.6	0	UNCRACKED
4	-4405	-41.1	0	UNCRACKED
5	10.2	24.3	0	UNCRACKED
6	201.1	-49.4	0	UNCRACKED
7	400	-3.9	0	UNCRACKED
8	-7791	-5.6	0	UNCRACKED

All Uncracked

Limit 0.2 mm Table 7.1, NEN EN 1992-1-1 OK

B.6. Capping beam design calculations

ARUP	Job No.	Sheet No.	Rev.
	303154-00	1 of 1	0
	Member/Location		
Job Title: Aramis Launch Shaft	Drg. Ref.		
Calculation: Flexural - RC capping beam	Made by	Date: 05/06/2024	Chd.

Introduction A calculation to determine the minimum requirements for flexural reinforcement, using Oasys GSA and Adsec

Oasys GSA

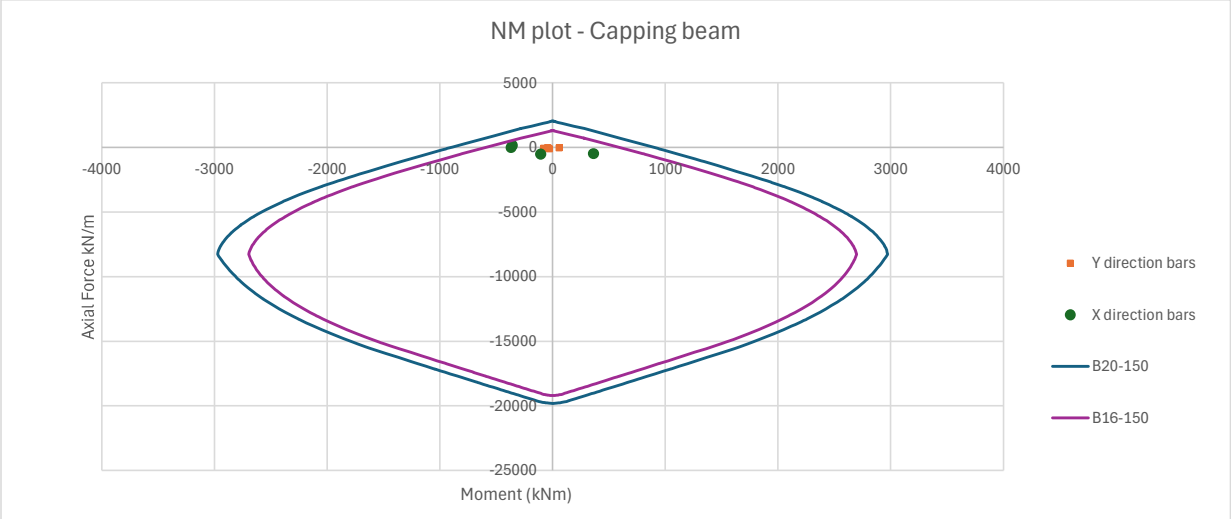
Top 1m capping slab

				[kNm/m]	[kN/m]	
	Elem	Case	Node	My + sgn(My)|Myx|	Ny	
B : y-direction	60	A17	119	59.88	-51.8	*Max moment with coexisting force*
	173	A17	286	-79.16	-65.21	*Min moment with coexisting force*
	176	A17	288	-42.51	-43.48	*Max axial T(+ve) with coexisting M*
	160	A17	95	-28.91	-125.4	*Min axial C (-ve) with coexisting M*

-ve downwards = sagging moment, so rebar at bottom
+ve upwards = hogging moment, so rebar at top

				[kNm/m]	[kN/m]	
	Elem	Case	Node	Mx + sgn(Mx)|Mxy|	Nx	
A : x-direction	113	A15	225	363.3	-482.1	*Max moment with coexisting force*
	140	A15	55	-368.2	-18.4	*Min moment with coexisting force*
	83	A15	168	-356	146.3	*Max axial T(+ve) with coexisting M*
	171	A17	284	-105.7	-514.3	*Min axial C (-ve) with coexisting M*

-ve downwards = sagging moment, so rebar at bottom
+ve upwards = hogging moment, so rebar at top



Minimum reinforcement required for flexural B16-150

ARUP	Job No.	Sheet No.	Rev.
	303154-00	1 of 1	0
	Member/Location		
Job Title: Aramis Launch Shaft	Drg. Ref.		
Calculation: Shear - RC capping beam	Made by	Date: 11/06/2024	Chd.

Source/Ref

Checks to undertake:

1. Material properties
2. Minimum reinforcement requirements
3. Shear Capacity

1. Material properties

Tensile strength of steel rebar, fyk (assume B500)	f_{yk} =	500	N/mm²
Partial factor for steel, γ_s		1.15	
$f_{ywd} = f_{yk}/\gamma_s$	fywd =	435	N/mm²
Compressive strength of concrete, fck	f_{ck} =	32	N/mm²
	$f_{ck,cub}$ =	40	N/mm²
Partial factor for concrete, γ_c		1.5	
a_{cw}		1	NA to Cl 3.1.6
$f_{cd} = a_{cw}f_{ck}/\gamma_c$		21	N/mm²

2. Minimum reinforcement requirements

Eurocode 2 part 1-1, 9.2.1.1 (1)

Minimum a

C32/40

fctm	3	Mpa
fyk	500	MPa

Element	Depth	$A_{s,min} = 0.26\frac{f_{ctm}b_t d}{f_{yk}}$	As,min = 0.0013btd	As,max = 0.04Ac
	mm	mm2 / m	mm2 / m	mm2/m
Capping sl	1000	1560	1300	400000

Bar diamet	20	mm
Area	314	mm2
No. bars	5	
Spacing	201	mm

3. Shear Capacity in capping beam

Check if shear reinforcement is required:

EC1992-1-1 Cl 6.2.2

$$V_{Rd,c} = [C_{Rd,c}k(100\,\rho_1 f_{ck})^{1/3} + k_1\,\sigma_{cp}]\,b_w d \qquad (6.2.a)$$

with a minimum of

$$V_{Rd,c} = (v_{min} + k_1\sigma_{cp})\,b_w d \qquad (6.2.b)$$

where:

f_{ck} is in MPa

$k = 1+\sqrt{\frac{200}{d}} \le 2{,}0$ with d in mm

$\rho_l = \frac{A_{sl}}{b_w d} \le 0{,}02$

A_{sl} is the area of the tensile reinforcement, which extends $\ge (l_{bd} + d)$ beyond the section considered (see Figure 6.3).

b_w is the smallest width of the cross-section in the tensile area [mm]

$\sigma_{cp} = N_{Ed}/A_c < 0{,}2\,f_{cd}$ [MPa]

N_{Ed} is the axial force in the cross-section due to loading or prestressing [in N] (N_{Ed}>0 for compression). The influence of imposed deformations on N_{Ed} may be ignored.

A_C is the area of concrete cross section [mm²]

$V_{Rd,c}$ is [N]

where

f_{ck} =	32	N/mm²	
d =	1000	mm	
k = 1+(200/d)0.5	1.45		<2.0 ∴ OK
b_w =	1000	mm	per m strip
A_{si} =	2199	mm²	Assuming B20 bars at 150mm spacing
ρ_1 =Asl/bwd	0.0022		<0.02 ∴ OK
σ_{cp} =	0.0	N/mm²	Assumed
$C_{Rd,c}$ = 0.18/γc	0.12		
v_{min} =	0.34	N/mm²	$v_{min} = 0{,}035\,k^{3/2}\,f_{ck}^{1/2}$
k_1 =	0.15		

Design she: $V_{Rd,c}$ =	332797	N	(6.2a)	
	333	kN		
$V_{Rd,cmin}$ =	344701	N	(6.2b)	
	345	kN	**USE THIS AS BIGGER**	
V_{Ed} =	122	kN	From GSA	

As VEd < VRd,c shear reinforcement in the slab is NOT required

Check if maximum allowable shear force is exceeded

EC1992-1-1 Cl 6.2.2

The shear force V_{Ed}, calculated without reduction by β, should however always satisfy the condition

$V_{Ed} \le 0{,}5\, b_w d\, \nu f_{cd}$ (6.5)

where ν is a strength reduction factor for concrete cracked in shear

Note: The value ν for use in a Country may be found in its National Annex. The recommended value follows from:

$\nu = 0{,}6\left[1 - \frac{f_{ck}}{250}\right]$ (f_{ck} in MPa) (6.6N)

where

v = 0.523 N/mm²

$V_{Ed,max}$ = 5580800 N

5581 kN

As VEd < VEd,max shear in the slab is acceptable

If shear reinforcement required:

Shear resistance will be calculated in accordance with BS EN 1992-1-1:2004 clause 6.2.3.

$V_{Rd,s}$ = $[A_{sw} / s] * z * f_{ywd} * \cot\theta$

				outer bar	inner bar
A_{sw} =	402	mm²	Link dia =	16	
Spacing, s =	150	mm	No. legs =	2	
Lever arm, z =	900	mm (0.9d)			
f_{ywd} =	434.8	N/mm²			
θ =	45	degrees	=	0.785	radians
α =	45	degrees	=	0.785	radians

∴ $V_{Rd,s}$ = 1,049 kN

$V_{Rd,max}$ = $\alpha_{cw} * b_w * z * \nu_1 * f_{cd} / (\cot\theta + \tan\theta)$

α_{cw} =	1.0	(Non-prestressed)			
b_w =	1000.0	mm			
f_{ck} =	32.0	N/mm²	f_{cd} =	21.3	N/mm²
$\nu_1 = \nu$ =		0.5232	N/mm²		

∴ $V_{Rd,max}$ = 5,023 kN

$V_{Ed} < V_{Rd,s} < V_{Rd,max}$

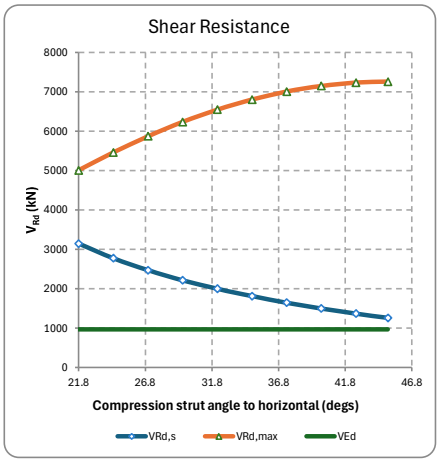
[$A_{sw,max}$ = $0.5 * \alpha_{cw} * \nu_1 * f_{cd} * b_w * s / f_{ywd}$ for θ = 45.0 degrees

∴ $A_{sw,max}$ = 1925.4 mm² ---]

θ (degs)	$V_{Rd,s}$ (kN)	$V_{Rd,max}$ (kN)	ΔF_{td} (kN)
21.8	2623	3464	153
24.4	2313	3779	134
27.0	2059	4063	120
29.6	1847	4314	107
32.2	1666	4530	97
34.8	1509	4708	88
37.4	1372	4847	80
40.0	1250	4946	73
42.6	1141	5005	66
45.0	1049	5023	61

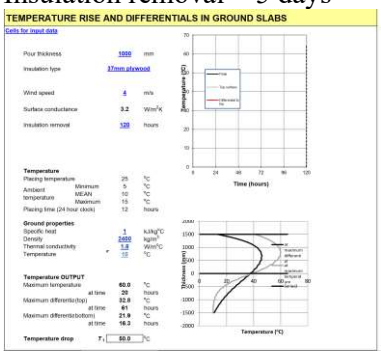
V_{Ed} = 122 kN

$UR_{concrete}$=	0.02
	OK
UR_{links}=	0.12
	OK



21.8	122
45.0	122
2.6	step

RC capping beam - Early Thermal Checks Summary

	RC capping beam **Assume: internal restraint**
Temperature Risk and Differentials in ground slabs	Pour thickness = 1000mm Insulation type (top surface) = none Insulation type (bottom surface) = none Wind speed= 4m/s Insulation removal = 5 days 
Restraint	R=0.42 as per cover sheet of Ciria guide calculation for internal restraint
Crack width	Concrete strength = C32/40 Bar diameter = 20mm Bar spacing = 150mm Cover = 75mm top / 75mm bottom Aggregate type: Ecm kept as default Early age crack width = 0.09mm < 0.3mm (Section 7.3.3, NEN EN 1992-1-1)

C766 Temperature prediction in walls and slabs

This model provides a basis for the comparative assessment of early-age temperature rise and temperature differentials in concrete walls, suspended slabs and ground slabs.

The model is based on the numerical method of Ross and Bray (1949) and is described in detail in Appendix A2

Background data on heat generation were provided by the University of Dundee (Dhir *et al*, 2006) obtained using semi-adiabatic testing to BS EN 196-9:2003

Data for the concrete are input on PAGE 2

Data specific to walls and suspended slabs are input on PAGE 3

Data specific to ground slabs are input on PAGE 4

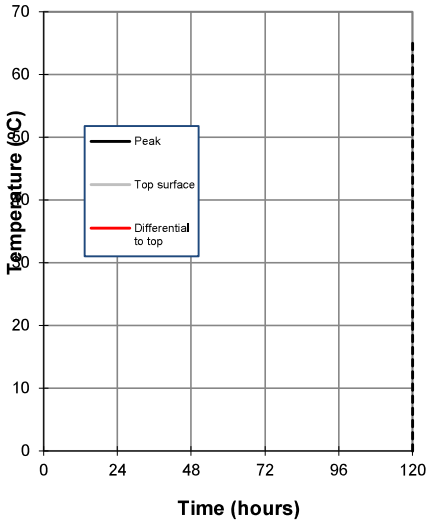
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PAGE 2 ADIABATIC TEMPERATURE			
Concrete mix details	Binder content	kg/m^3	The binder is the total content of cementitious material, including CEM I and CEM I in combination with either fly ash or ground granulated blast-furnace slag, ggbs
	Binder type		Three binder options are available from a drop-down menu CEM I, fly ash, ggbs
	Percent addition	%	The percent of either fly ash or ggbs by weight of binder
	Density	kg/m^3	The wet density of the concrete
	Specific heat	kJ/kg°C	The early-age value when the concrete is only partially hydrated. A value of 1 kJ/kg°C should be used unless more reliable data are available
	Thermal conductivity	W/m°C	Assumed to be 2.0 W/m°C (see Appendix A2 for values for concrete using difference aggregates)
	Multiplier		If results are available for a specific mix, the multiplier may be used to adjust the adiabatic curve to match the data. The default value is 1,
Temperature input	Placing temperature	°C	The temperature of the fresh concrete when poured. As a general rule, the placing temeprature be 5°C above the mean ambinebt temeprature.
	Minimum ambient temperature	°C	These values determine the variable ambient temperature which is assumed to vary on a sinusoidal basis. As a eneral rule the ambient temeprature will vary by ±5°C from the mean value.
	Mean ambient temperature	°C	
	Maximmum ambient temperatur	°C	
	Constant or variable ambient temperature		There is the option to assume either a variable sinusoidal ambient temperature variation or a constant ambient temeprature variation. In the latter case this is assumed to be the MEAN ambient temperature,
	Pacing time	Hours	This assumes a 24 hour clock
Output	Temperature drop T_1		Values are calculated in PAGE 3 and PAGE 4 but are shown on PAGE 2 to enable the effect of changes in the concrete mix and placing temperature to be observed without changing pages. Input on section geometry, formwork etc, is on PAGE 3 and PAGE 4
GRAPHS	Reference curve for the cement		The reference adiabatic curve is estimated from the cement type and content and represents the value at a starting temperature of 20°C
	Adiabatic temperature rise for the concrete		The estimated adiabatic temperature rise is shown based on the reference curve at 20°C and the value obtained when a different placing temperature is used.
PAGE 3 TEMPERATURE CALCULATIONS FOR WALLS AND SUSPENDED SLABS			
Section and formwork details	Section thickness	mm	
	Formwork/insulation type		Formwork/insulation options are available for both faces from drop-down menus. Information for each face must be input. REMEMBER TO INPUT FOR BOTH FACES
	Wind speed	m/s	Assumed to be 4 m/s for typical UK conditions REMEMBER TO INPUT FOR BOTH FACES
	Formwork removal time	hours	The time after casting. Input must be made for both faces. There is the options to remove formwork from opposite faces at different times REMEMBER TO INPUT FOR BOTH FACES
Concrete mix details	Thermal conductivity	W/m°C	Copied from PAGE 2
	Density and specific heat		Copied from PAGE 2
PAGE 4 TEMPERATURE CALCULATIONS FOR GROUND SLABS			
Section and formwork details	Section thickness	mm	This is the minimum dimension through which heat loss is predominant
	Insulation type		Options are available from a drop-down menu for both the soffit formwork and insulation applied to the top surface (if required).
	Wind speed	m/s	Assumed to be 4 m/s for typical UK conditions
	Formwork removal time	hours	The time after casting. Input must be made for both the soffit and the top surface. There is the options to remove opposite faces at different times
Concrete mix details	Thermal conductivity	W/m°C	Copied from PAGE 2
	Density and specific heat		Copied from PAGE 2
Properties of the ground	Specific heat	kJ/kg°C	Default value is 1.2 kJ/kg°C
	Density	kg/m^3	Default value is 2000 kJ/kg°C
	Thermal conductivity	W/m°C	Default value is 1.1 W/m°C
	Temperature	°C	

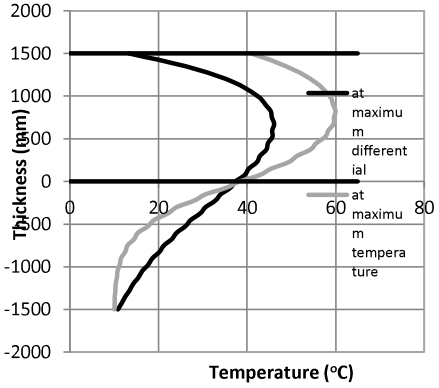
TEMPERATURE RISE AND DIFFERENTIALS IN GROUND SLABS

Cells for input data

Pour thickness		1000	mm
Insulation type		37mm plywood	
Wind speed		4	m/s
Surface conductance		3.2	W/m^{2}K
Insulation removal		120	hours



Temperature			
Placing temperature		25	°C
Ambient temperature	Minimum	5	°C
	MEAN	10	°C
	Maximum	15	°C
Placing time (24 hour clock)		12	hours
Ground properties			
Specific heat		1	kJ/kg°C
Density		2400	kg/m^3
Thermal conductivity		1.8	W/m°C
Temperature		10	°C
Temperature OUTPUT			
Maximum temperature		**60.0**	°C
	at time	**20**	hours
Maximum differential (top)		**32.8**	°C
	at time	**61**	hours
Maximum differential bottom)		**21.9**	°C
	at time	**16.3**	hours
Temperature drop	T_1	**50.0**	°C



C766 Estimation of restraint

This calculator estimates restraint in a wall cast onto a rigid based. The method is based of the ACI approach in which the restraint at the joint is determined by the relative stiffness of the new section and the section against which it is cast. The variation in restraint with height is determined by the length to height ratio.

The method is described in detail in Appendix A5

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PAGE 2 RESTRAINT	
Wall geometry	The wall geometry is define by the length L , the height H and the thickness, h. When a new pour that is restrained along one edge abuts an exisiting pour at one end then for the purpose of calculating the restraint profile, the length of the edge restraaining joint should be assumed to be 2 x L, ie it is assumed that there is zero displacement at the joint between the new pour and the adjacent pour
Base geometry	The base geometry is defined by the width W and the thickness, h . The length is assumed to be the same as the length of wall being cast
Relative areas	The ratio of the cross-sectional area (c.s.a.) of the wall A_{new} and the base A_{old} is calculated. The relative areas of influence A_{old} and A_{new} may be difficult to define. For example, when a wall is cast onto a slab, what width of slab should be considered as providing effective restraint? In such circumstances it is recommended that the relative area is assumed to be in proportion to the relative thicknesses, h_{new} and h_{old} , of the wall and slab. The following simple rules may be applied: • For a high wall cast at the edge of a slab assume $A_{new}/A_{old} = h_{new}/h_{old}$ • For a wall cast remote from the edge of a slab assume $A_{new}/A_{old} = h_{new}/2h_{old}$ • For a slab cast against an existing slab assume $A_{new}/A_{old} = h_{new}/h_{old}$
Relative modulus of elasticity	Bamforth and Price (1995) recommends that the ratio E_{new}/E_{old} is assume to be in the range 0.7 to 0.8 at early age. The lower value gives most conservative result and should be used for conditions in which cool-down is most rapid. When estimating restraint to long term deformations, should there be any differential strains, it should be assumed that $E_{new}/E_{old} = 1$
Restraint at the joint R_j	$$R_j = \frac{1}{1 + \frac{A_{new}}{A_{old}} \frac{E_{new}}{E_{old}}}$$
Restraint at the location of the maximum crack width	The maximum restrained strain and crack width occur at a perpendicular distance from the joint of 0.1 times the length of the joint if the new pour is not cast against existing concrete at either end. If the new pour abuts an exisitng pour at one end then the maximum restrained strain and crack width occur at a perpendicular distance from the joint of 0.2 times the length of the joint

C766 Predicting the risk of cracking and controlling crack widths

This calculator provides a basis for estimating the likelihood of cracking and for selecting or checking reinforcement to control crack widths

The approach is based on the method of BS EN 1992-1-1 with adjustments where considered appropriate (see Appendix A8)

Three conditions of restraint are dealt with

PAGE 2 Continuous EDGE restraint.

PAGE 3 END restraint

PAGE 4 INTERNAL restraint from temperature differentials

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Input data are as follows (all other parameters are calculated):

PAGE 2 CONTINUOUS EDGE RESTRAINT			
Section details and reinforcement	Section thickness	mm	Section thickness in mm
	Strength class	f_{ck} / $f_{ck,cube}$	Select from the drop-down menu
	Bar diameter	φ	Bar diameter in mm
	Bar spacing	S	Bar spacing in mm
	Cover to reinforcement	c	Minimum cover to reinforcement
	Yield strength of reinforcement	f_{yk}	The characteristic yield strength of the reinforcement in MPa
Early-age restrained strain	Coefficient of expansion	α_c	A value of 12 με / °C is recommended if there is no knowledge of the aggregate type. The recommended value in BS EN 1992-1-1 is 10 με / °C but many aggregates in the UK produce concrete with higher values
	Temperature drop	T_1	This may be obtained from data in C660, the temperature model (Appendix A2) or by independent validated modelling or measurement.
	Creep factor	Kc_1	BS EN 1992-1-1 does not define a creep factor but includes this in the value of restraint R = 0.5 which is the maximum value recommended by BS EN 1992-1-1 and so must cover the worst condition of restraint. When this value of R is used, K_1 should be assumed to be 1. When R is calculated K_1 may be assumed to be 0.65
	Restraint at the location of the maximum crack width	R	BS EN 1992-1-1 permits R to be calculated from the relative stiffness of the element and the member against which it is cast. Where it can be demonstrated that R < 0.8 the calculated value may be used with K_1 = 0.65. The the maximum restrained strain occurs at a distance of 0.1 x length of the wall above the joint with the base. When the wall abuts a previously cast length, the assumed length is calculating the location of the maximum restrained strain is double the actual length.
Risk of early-age cracking	Age at cracking	days	Assumed to be 3 days for early age cracking unless more reliable information is available
	Aggregate type		The tensile strain capacity is the ratio of the tensile strength to the elastic modulus and the latter is affected by the aggregate type. If the aggregate source is known this can be selected from the drop down menu. The default value for unknown aggregate is 1. This is the value dervied from the properties calculated according to BS EN 1992-1-1 for quartz aggregate. Where the elastic modulusat 28 days is known this may be entered and 'measured' should be selected from the drop down menu.
Minimum area of reimforcement $A_{s,min}$	Coefficient for effect edge restraint (Optional)	$k_{R,edge}$	A drop down menu (YES/NO)provides the option to take account of the contributon of the edge restraint in limiting crack opening and the amount of steel required to ensure that yield does not occur.
Crack spacing	Factor for bond	k_1	BS EN 1992-1-1 recommends k_1 = 0.8 but provides a factor of 0.7 where good bond cannot be guaranteed. k_1 = 0.8/0.7 = 1.14 for section >300 mm thick with cover < 50 mm. If cracking is not predicted at early age, then k_1 = 0.8.
Long term cracking	Age at cracking	days	For calculating long term cracking the 28-day properties of the concrete are adopted.
	Long term temperature change	$T2$	Where both the restrained and the restraining element are subject to a change in temperature, T2 is the differential change in temperature between the restrained and the restraining element unless there is external restraint to both elements.
	Drying shrinkage	ε_{cd}	Calculate using the method of BS EN 1992-1-1 unless more reliable information is available. Only apply when causing differential contraction between elements or when the elements acting integrally are subject to external restraint. The drying shrinkage is that occurring at the end of the design life of the structure or part thereof.
	Creep factor	K_{C2}	When restraint is calculated, the long-term creep coefficient is taken as 0.5
	Restraint to T_2	R_2	Long term restraint will be reduced as the ratio of stiffness of the elements reduces and E_n / E_o approaches 1. Consideration must also be given to the element acting integrally.
	Restraint to drying shrinkage	R_3	
Crack spacing	Coefficient for bond characteristics	k_1	If cracking has occurred at early age, the same carack spacing is assumed for long-term cracking. If cracking has not occurred at early age, the crack spacing is recalculated assuming that k_1 =0.8

PAGE 3 END RESTRAINT			
Section details	Section thickness	h	Section thickness in mm
	Strength class	f_{ck} / $f_{ck,cube}$	Select from the drop-down menu.
	Bar diameter	ϕ	Bar diameter in mm
	Bar spacing	s	Bar spacing in mm
	Cover	c	Minimum cover to reinforcement
Properties of reinforcement	Characteristic yield strength	f_{yk}	The characteristic yield strength of the reinforcement = 500 MPa
	Elastic modulus	E_s	200 GPa
Crack sacing	Coefficient for bond characteristics	k_1	See ref to k_1 above
Age at cracking	Early-age	*3 days*	Values of 3 days and 28 days are assumed for the estimation of early age and long-term properties. Values derived on this basis are have been shown to be safe in relation to the design of reinforcement for crack control
	Long term	*28 days*	
Early-age restrained strain	Coefficient of expansion	α_c	A value of 12 με / °C is recommended if there is no knowledge of the aggregate type. The recommended value in BS EN 1992-1-1 is 10 με / °C but many aggregates in the UK produce concrete with higher values
	Temperature drop	T_1	This may be obtained from data in C766, the temperature model (Appendix A2) or by independent validated modelling or measurement.
	Creep factor	Kc_1	BS EN 1992-1-1 does not define a creep factor but includes this in the value of restraint R = 0.5 which is the maximum value recommended by BS EN 1992-1-1 and must cover the worst condition of restraint. When this value of R is used, K_1 should be assumed to be 1. When R is calculated K_1 may be assumed to be 0.65
	Restraint	R	BS EN 1992-1-1 permits R to be calculated from the relative stiffness of the element and the member against which it is cast. Where it can be demonstrated that R < 0.8 the calculated value may be used with K_1 = 0.65. END restraint may be difficult to calculated, particulary when there is a combination of restraints acting on the member, eg a slab on piles with a combination of friction from the base and lateral resistance from piles. Some guidance is provided in Appendix A5, but the user may be required to undertake additional calculations where the arrangement of restraints is complex.
Risk of early-age cracking	Early-age	*Days*	Properties at 3 days are assumed for the estimation of early age cracking. However, when there is confience in the predicted time of cracking there is the option to use other ages.
	Aggregate type		The tensile strain capacity is the ratio of the tensile strength to the elastic modulus and the latter is affected by the aggregate type. If the aggregate source is known this can be selected from the drop down menu. The default value (coefficient = 1) is the value dervied from the properties calculated according to BS EN 1992-1-1 for quartz aggregate. Where the elastic modulus is known this may be entered and 'measured' should be selected from the drop down menu.
Long term cracking	Age at cracking	days	For calculating long-term cracking the 28-day properties of the concrete are adopted.
	Long term temperature change	*T2*	Where both the restrained and the restraining element are subject to a change in temperature, *T2* is the differential change in temperature between the restrained and the restraining element unless there is external restraint to both elements.
	Drying shrinkage	ε_{cd}	Calculate using the method of BS EN 1992-1-1 unless more reliable information is available.
	Creep factor	*Kc2*	
	Restraint to T_2	R_2	Long-term restraint will be reduced as the ratio of stiffness of the elements reduces and E_n / E_o approaches 1 and as elements act integrally.
	Restraint to drying shrinkage	R_3	
	Restraint required to cause cracking	R_{crack}	As restraint may be difficult to calculate, the restraint required to cause cracking is also estimated and the user may judge whether this level of restraint is likely to be achieved in practice.

PAGE 3 INTERNAL RESTRAINT
The input data for dealing with cracking caused by internal restraint is broadly the same as that required for edge restraint with the principal exception that the temperature differential between the centre and the surface ΔT replaces T_1. Other changes inherent in the design calculation are the restraint factor R assumed to be 0.42 and the coefficients k and kc which reflect the difference in the stress distribution within the section compared with the condition of external restraint.

Risk and control of cracking due to internal restraint

Input parameters	Symbol	Unit	Value	Comments/guidance
Section details and reinforcement				
Section thickness	h	mm	1000	
Strength class	f_{ck} / $f_{ck,cube}$	MPa	C32/40	Select from drop down menu
Bar diameter	φ	mm	20	mesh
Bar spacing	s	mm	150	
Cover	c	mm	75	Minimum cover to steel in the direction perpendicular to the crack
Characteristic yield strength of reinforcement	f_{yk}	MPa	500	500 MPa
Area of steel per face per m	A_s	mm^2	**2094**	
Early-age cracking				
Restrained strain				
Coefficient of thermal expansion of concrete	α_c	με/°C	9.5	If aggregate is unknown use 12 με / °C
Temperature differential	DT	°C	50	ΔT = Peak temperature - surface temperature
Free differential contraction	$\varepsilon_{free}(ea)$	με	**475**	$\varepsilon_{free(ea)} = \Delta T.\alpha_c$
Creep factor - early age	K_{c1}		0.65	K_{c1} = 0.65
Restraint	R		0.42	taken from cover sheet
Early-age restrained contraction	$\varepsilon_r(ea)$	με	**130**	$\varepsilon_{r(ea)} = R\ K_{c1}\ (\Delta T.\alpha_c)$
Risk of cracking and crack inducing strain				
Aggregate type. Use the 'selected' option and enter the E value if measured or obtained from historical results.	Limestone		0.90	Coefficient for stiffness of aggregate
			30	Enter valueof Ecm(28) if 'measured' or obtained from historical data
Elastic modulus	$E_{cm}(t_c)$	GPa	25.7	Mean value $E_{cm}(t_c)$. BS EN 1992-1-1 values are based on quartz aggregate
Tensile strain capacity	$\varepsilon_{ctu(ea)}$	**με**	**76**	$\varepsilon_{ctu(ea)} = [f_{ctk,0.05}(t_c) / [E_{cm}(t_c) \times K_{c1}]$
Risk of early age cracking	$\varepsilon_{r(ea)}/\varepsilon_{ctu(ea)}$		**1.71**	Low risk of early age cracking if $\varepsilon_{r(ea)}/\varepsilon_{ctu} < 1$.
Early-age crack-inducing strain	$\varepsilon_{cr(ea)}$	με	**92**	$\varepsilon_{cr(ea)} = R_1 K_{c1} (T_1 \alpha_c + \varepsilon_{ca(3)}) - 0.5\ \varepsilon_{ctu(3)} = (\varepsilon_{sm} - \varepsilon_{cm})$
Minimum area of reinforcement $A_{s,min}$				
Age at cracking	t_c	days	3	Assume 3 days unless more reliable information is available
Tensile strength at cracking	$f_{ctk,0.05}(t_c)$	MPa	1.27	Characterisitic tensile strength $f_{ctk,0.05}(t_c)$
Coefficient (BS EN 1992-1-1)	k		0.50	For assumed triangular strain distribution
Coefficient (BS EN 1992-1-1)	k_c		1.00	For pure tension $k_c = 1$
Surface zone representing A_{ct}	$h_{s,min}$	mm	100	$h_{s,min} = k\ k_c\ 0.2\ h$
Minimum area of steel per face per m	$A_{s,min}$	mm^2	**253**	$A_{s,min} = (1\text{-}0.5R)\ k\ k_c (f_{ctk,0.05}(t_c)/ f_{yk})\ A_{ct}$ Highlighted if $A_s < A_{s,min}$
Crack spacing				
Surface zone defining the effective area of concrete in tension, $A_{c,eff}$	$h_{e,ef}$	mm	212.5	$h_{e,ef}$ = lesser of $h/2$ a nd $2.5\ (c + \varphi/2)$
Steel ratio for estimating crack spacing	$\rho_{p,eff}$		0.00986	$\rho_{p,eff} = A_s / A_{c,eff} = A_s / (h_{e,ef} \times 1000)$
Coefficient for bond characteristics.	k_1		0.8	BS EN 1992-1-1 recommends k_1 = 0.8, but provides a factor of 0.7 where good bond cannot be guaranteed. Recommneded k_1 = 0.8/0.7 = 1.14 at early age for sections >300 mm thick with cover <50mm.
Crack spacing	$S_{r,max}$	mm	**945**	$S_{r,max} = 3.4c + 0.425\ k_1\ \varphi/\rho_{p,eff}$
Early age crack width	w_k	mm	**0.09**	$w_k = \varepsilon_{c(ea)}\ S_{r,max}$
Maximum stress in the reinforcement	σ_s	N/mm^2	**52**	$\sigma_s = 2(\varepsilon_{sm} - \varepsilon_{cm})E_s$

ARUP	Job No.	Sheet No.	Rev.
	303154-00	1 of 1	0
	Member/Location		
Job Title: Aramis Launch Shaft	Drg. Ref.		
Calculation: SLS crack width - RC capping slab	Made by	Date: 10/06/2024	Chd.

Introduction A calculation to determine the minimum requirements for SLS crack widths, using Oasys GSA and Adsec

Oasys GSA

Top 1m capping slab Cases: A2 A4 A6 A8 A10 A12 A14 A16 A18

				[kNm/m] My + sgn(My)|Myx|	[kN/m] Ny	
	Elem	**Case**	**Node**			
B : y-direction	60	A18	119	40.56	-38.73	*Max moment with coexisting force*
	173	A18	286	-53.05	-49.01	*Min moment with coexisting force*
	31	A2	63	0.7178	-7.117	*Max axial T(+ve) with coexisting M*
	160	A18	95	-19.88	-89.25	*Min axial C (-ve) with coexisting M*

				[kNm/m] Mx + sgn(Mx)|Mxy|	[kN/m] Nx	
	Elem	**Case**	**Node**			
A : x-direction	113	A16	225	242.1	-336.7	*Max moment with coexisting force*
	140	A16	55	-245.4	-21.45	*Min moment with coexisting force*
	83	A16	168	-237.2	95.01	*Max axial T(+ve) with coexisting M*
	171	A18	284	-70.65	-359.1	*Min axial C (-ve) with coexisting M*

Adsec

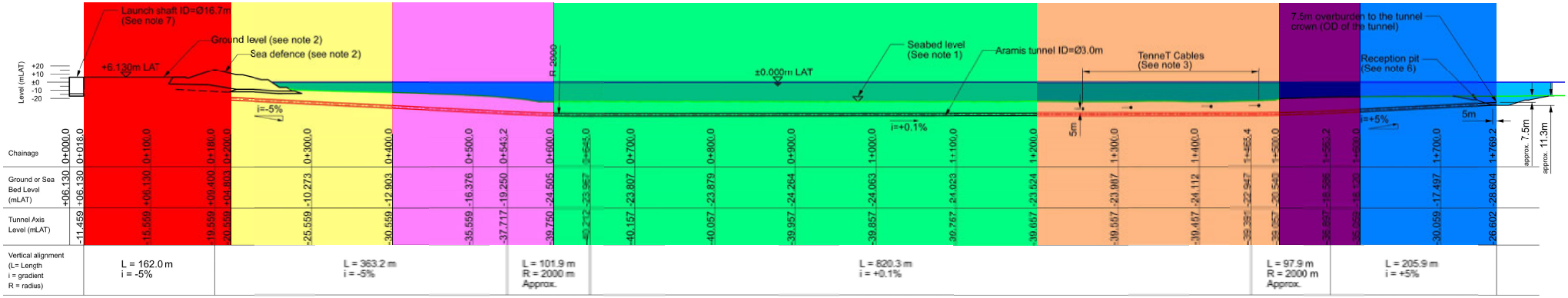
SLS crack width check **Minimum reinforcement required for SLS crack width** B25-150 required

Case	Load F_x (kN)	Load M_{yy} (kNm)	Load M_{zz} (kNm)	Cracked?	Crack width (mm)
1	-38.7	40.6	0	UNCRACKED	0.014
2	-49	-53.1	0	UNCRACKED	0.019
3	-7.1	0.7	0	UNCRACKED	
4	-89.3	-19.9	0	UNCRACKED	71e-6
5	-336.7	242.1	0	UNCRACKED	0.062
6	-21.5	-245.4	0	UNCRACKED	0.132
7	95	-237.2	0	UNCRACKED	0.155
8	-359.1	-70.7	0	UNCRACKED	

All <0.2mm

Limit 0.2 mm Table 7.1, NEN EN 1992-1-1 OK

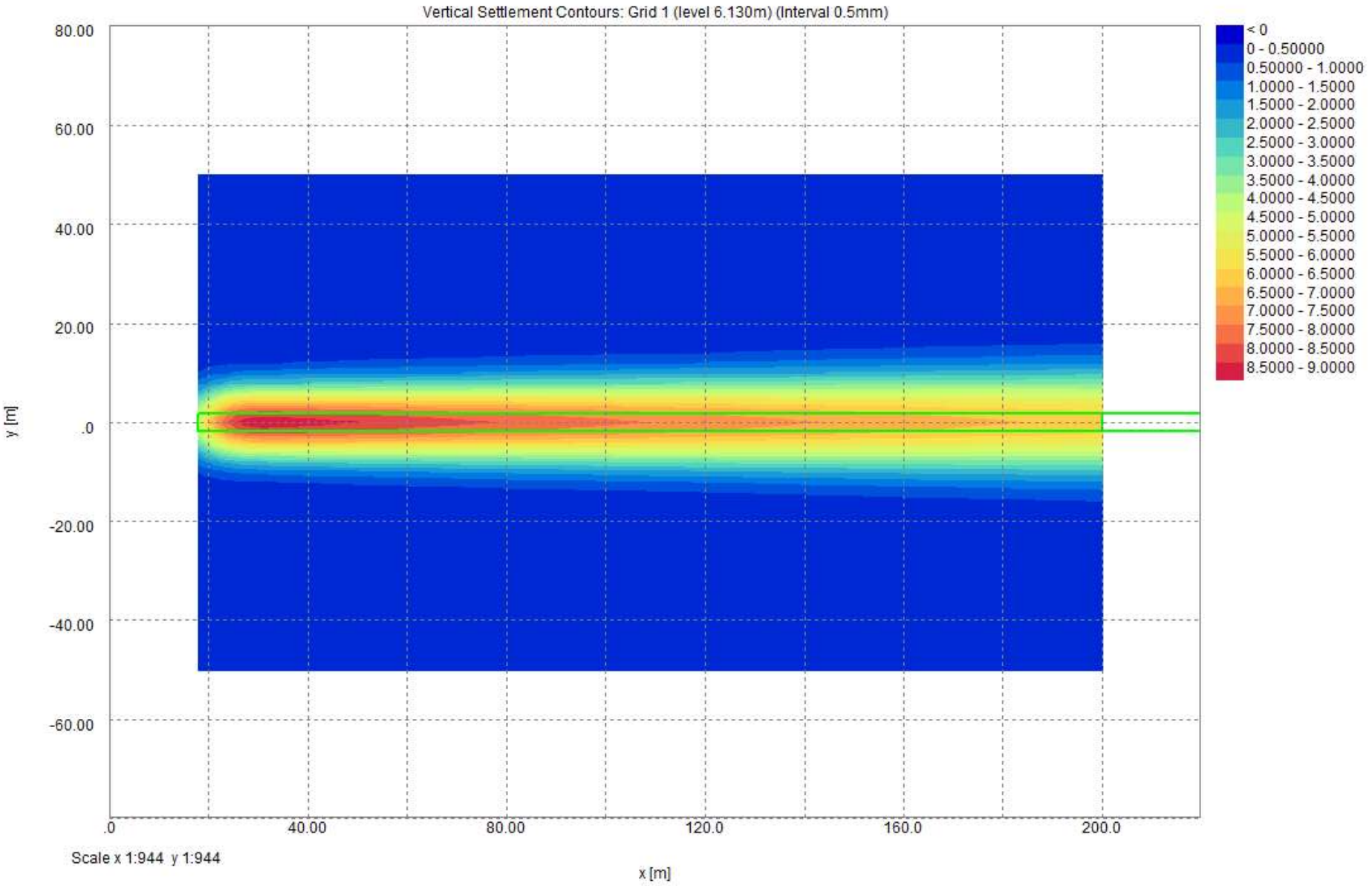
APPENDIX C XDISP SETTLEMENT CONTOURS



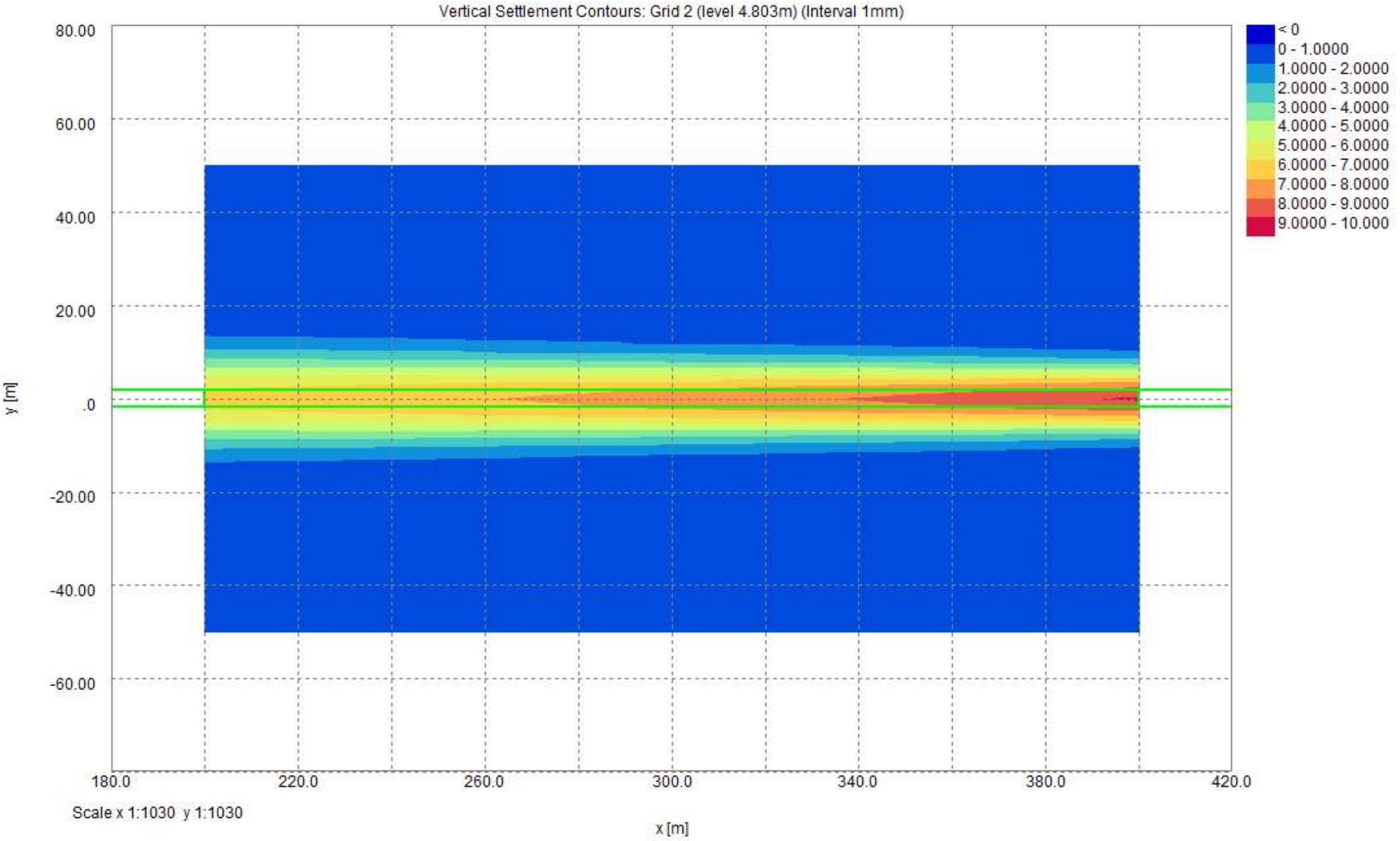
Longitudinal Profile View

The following contours are based on the above 7 coloured sections. Each section follows the Ground or seabed level (mLat) as illustrated in the figure for longitudinal profile view above.

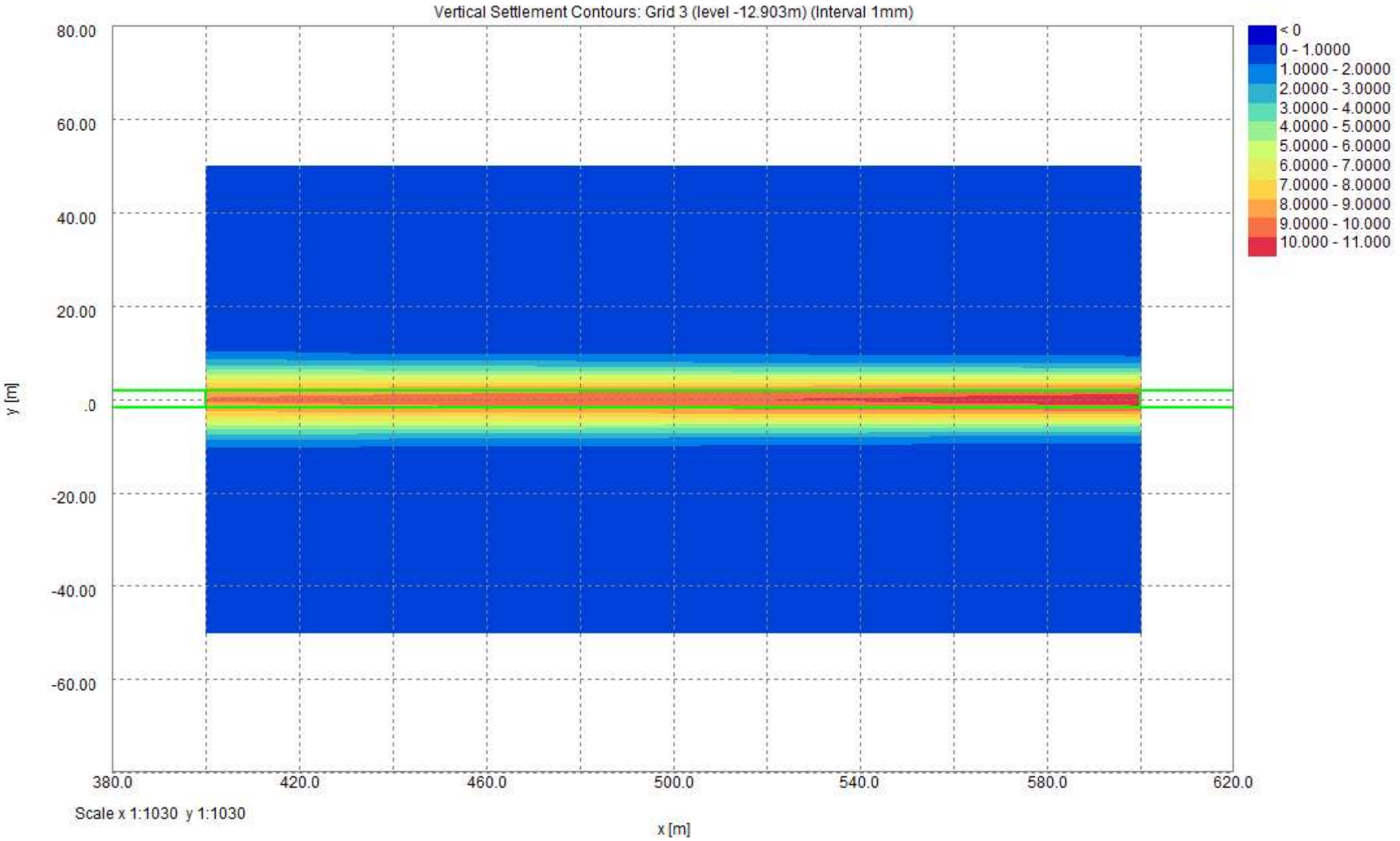
Plan view contour of seabed at ch 0+018 to ch0+200 for 1% volume loss



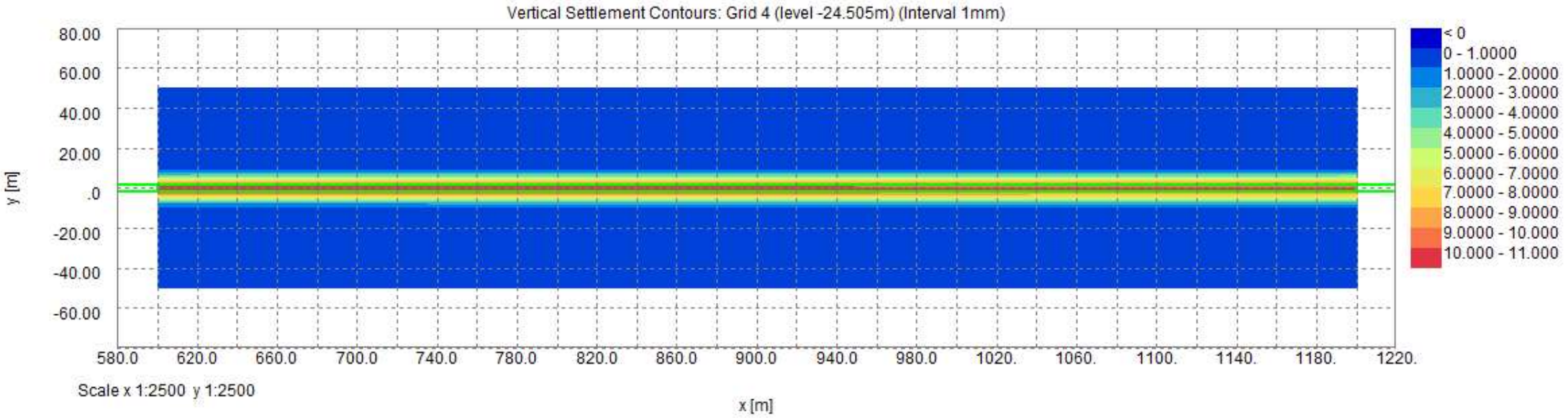
Plan view contour of seabed at ch 0+200 to ch 0+400 for 1% volume loss



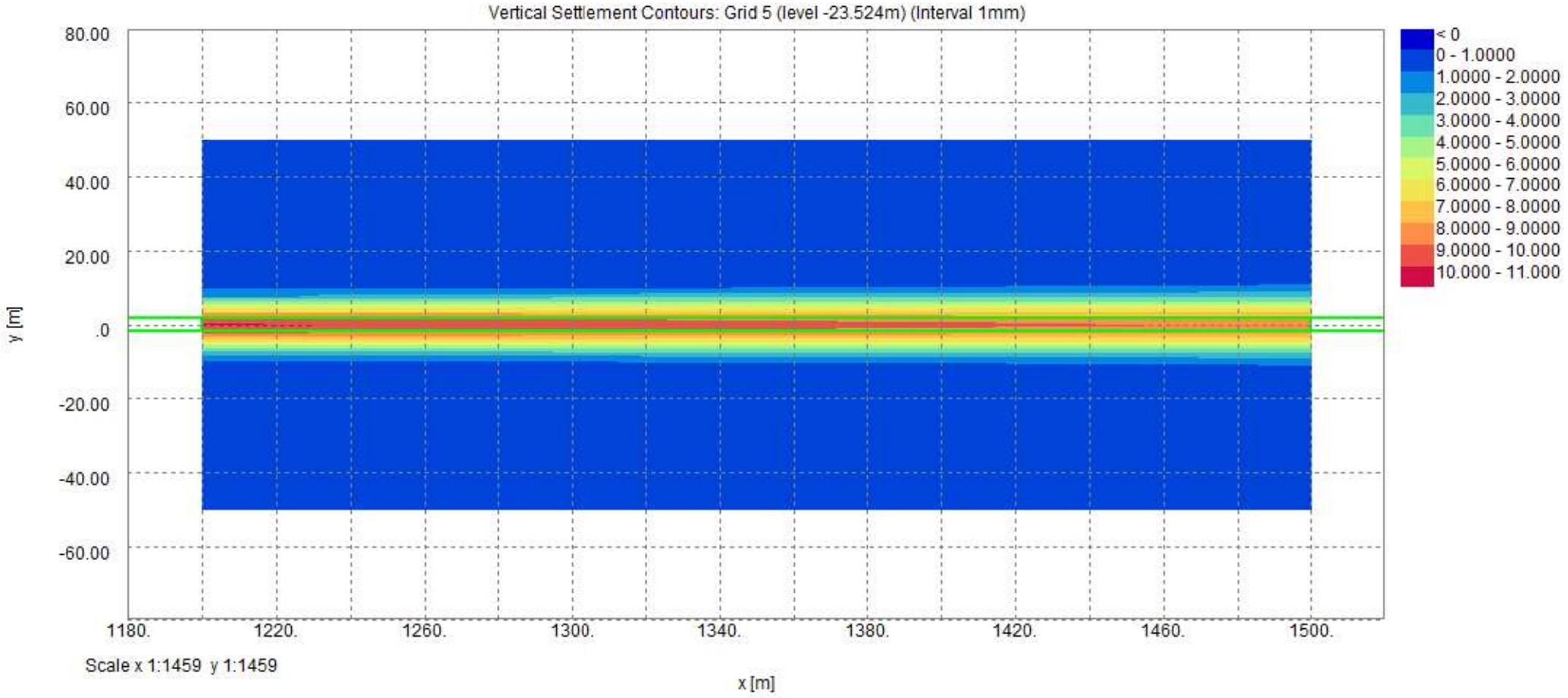
Plan view contour of seabed at ch 0+400 to ch 0+600 for 1% volume loss



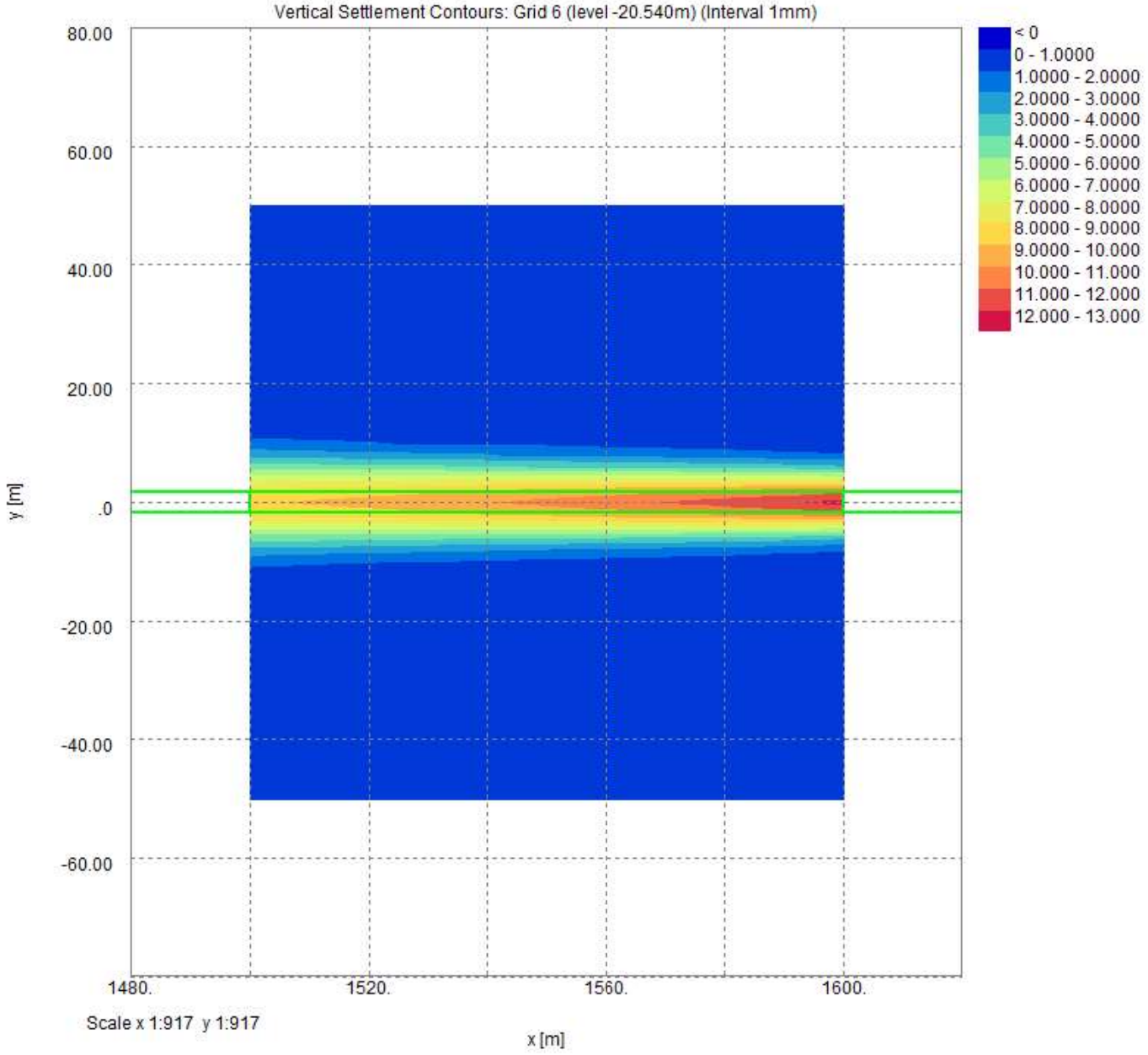
Plan view contour of seabed at ch 0+600 to ch 1+200 for 1% volume loss



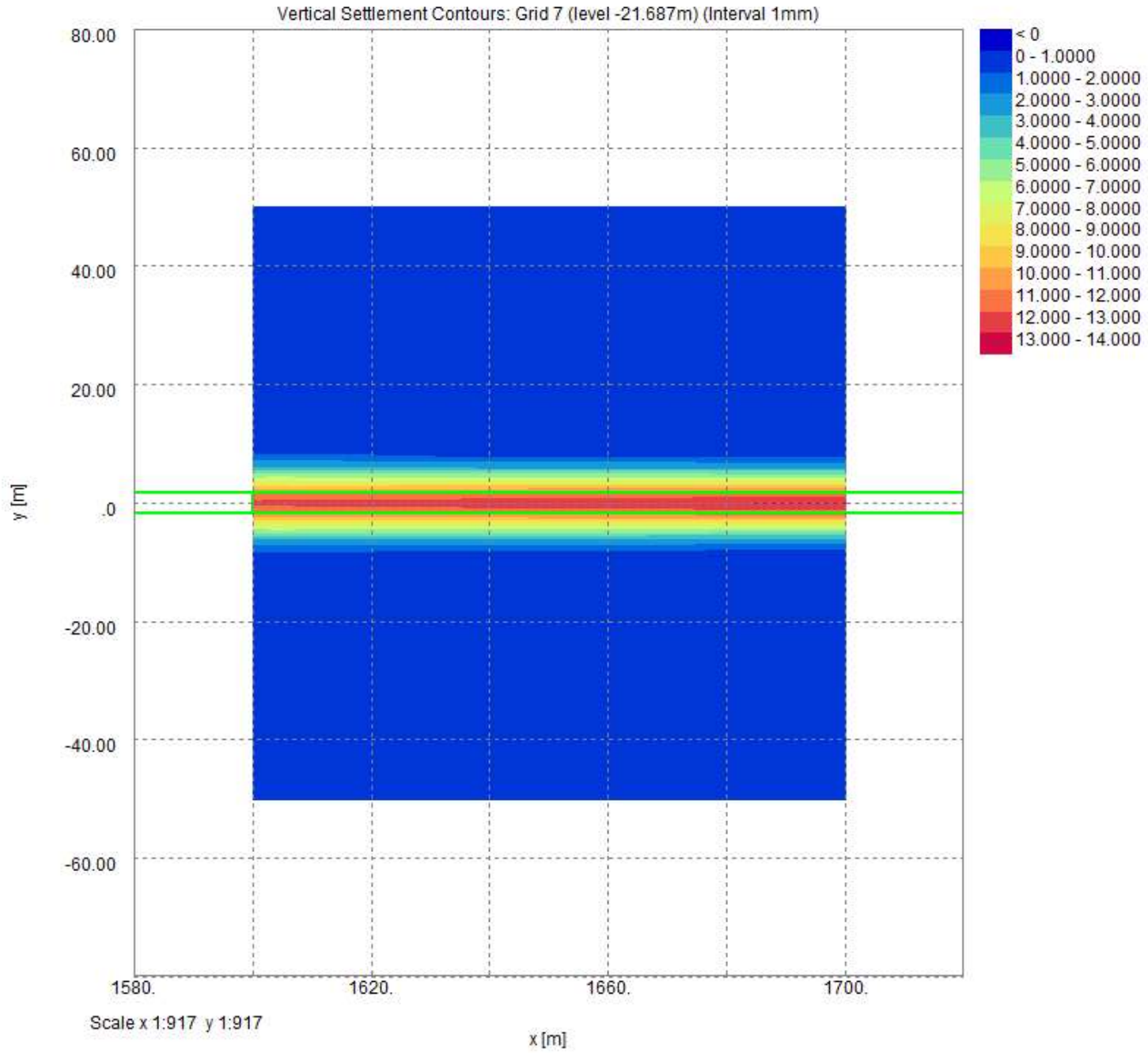
Plan view contour of seabed at ch 1+200 to ch 1+500 for 1% volume loss



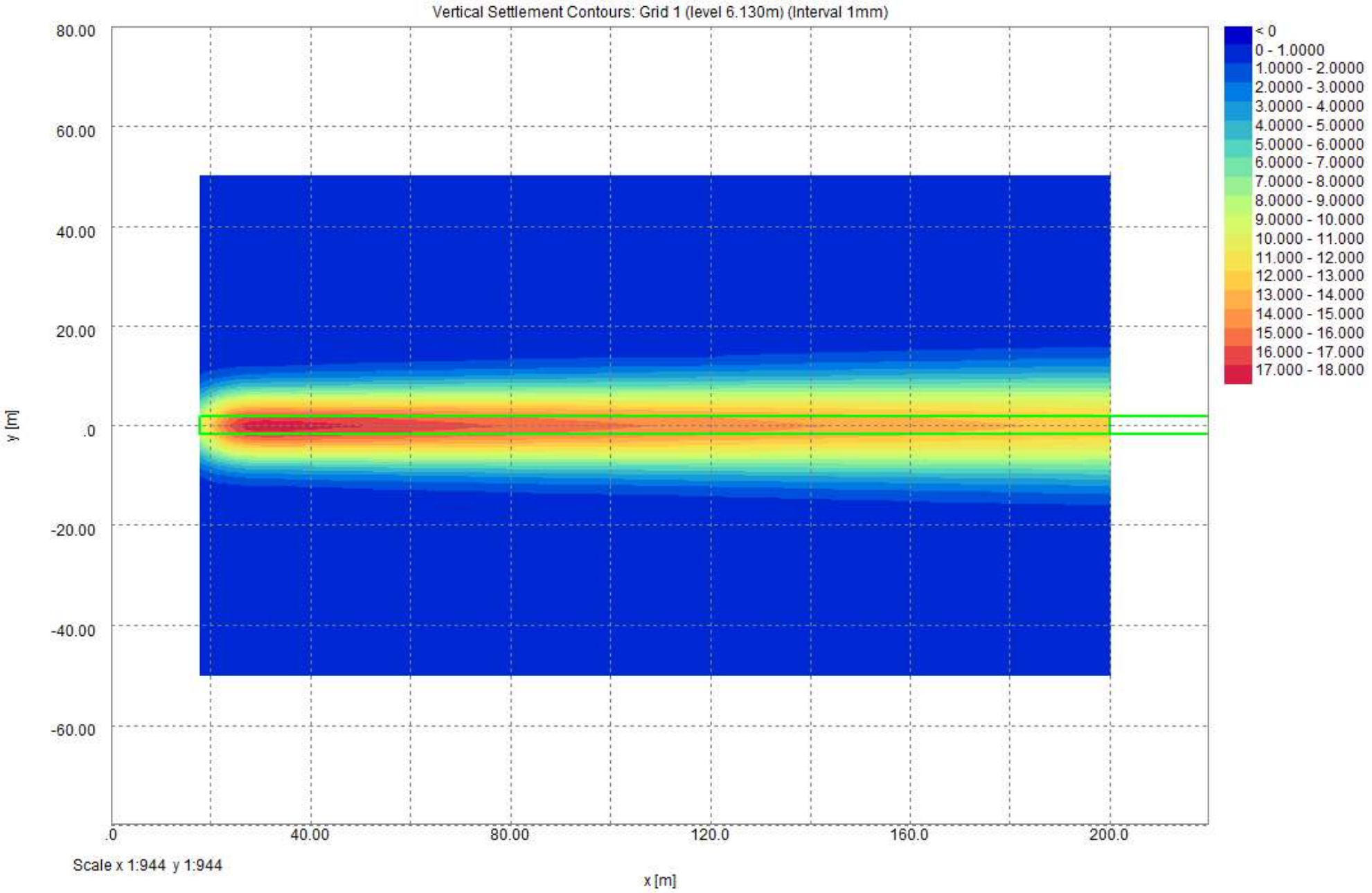
Plan view contour of seabed at ch 1+500 to ch 1+600 for 1% volume loss



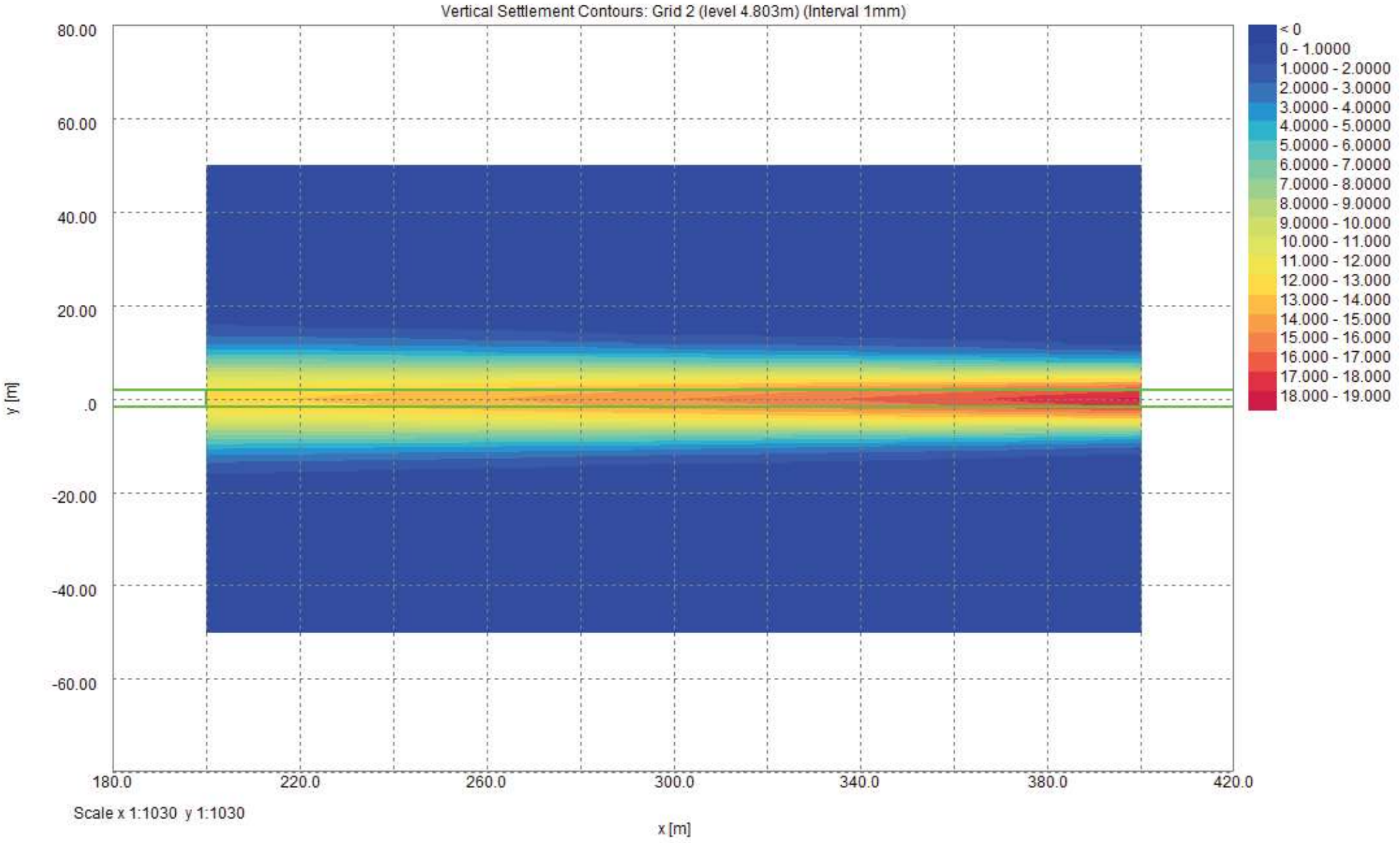
Plan view contour of seabed at ch 1+600 to ch 1+700 for 1% volume loss



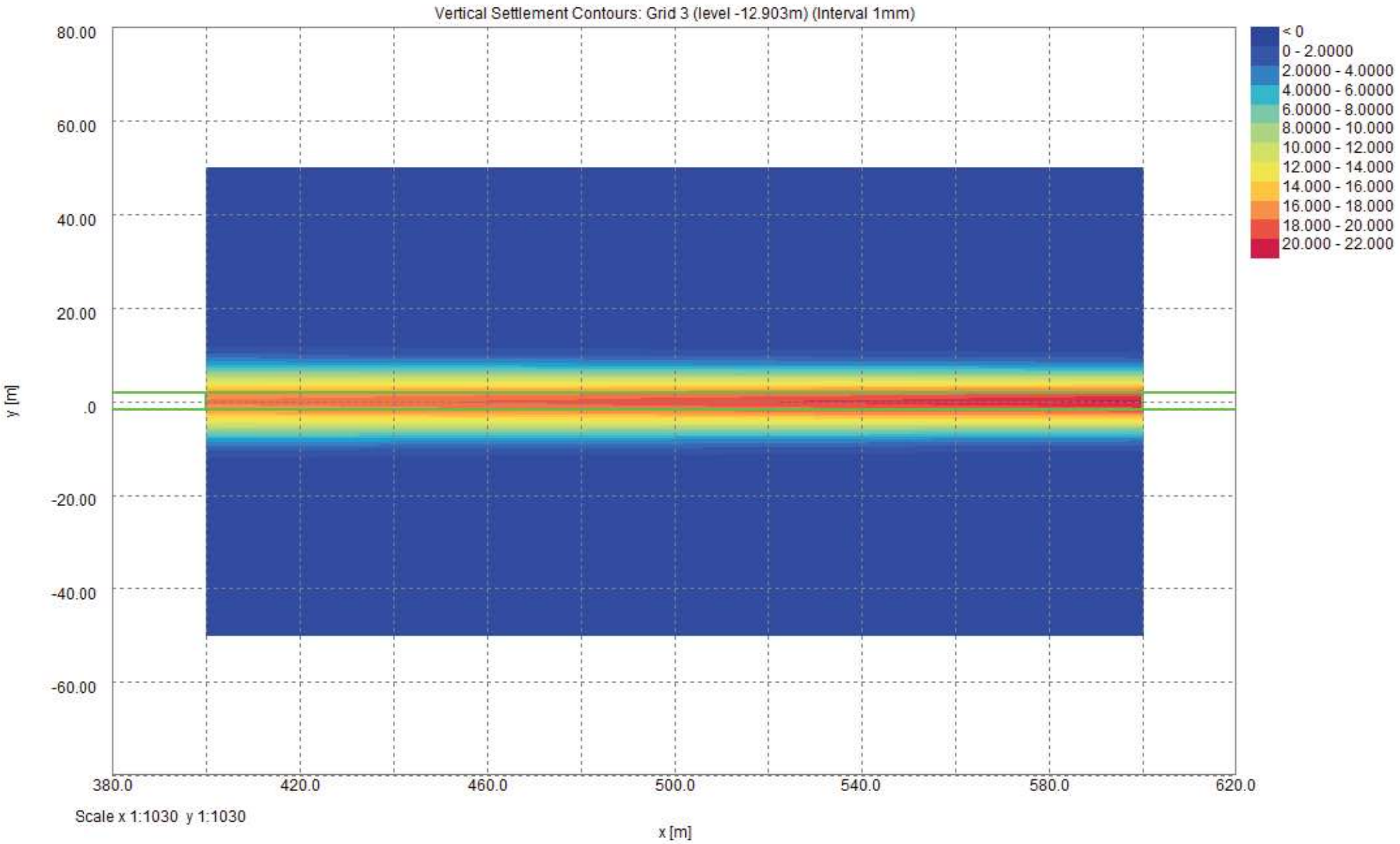
Plan view contour of seabed at ch 0+018 to ch0+200 for 2% volume loss



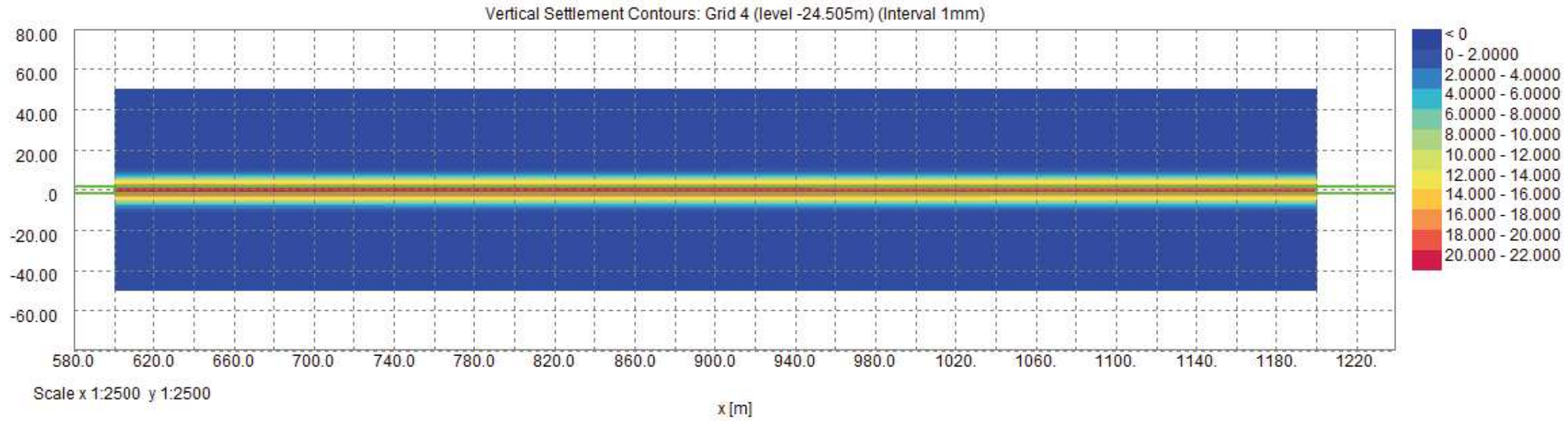
Plan view contour of seabed at ch 0+200 to ch 0+400 for 2% volume loss



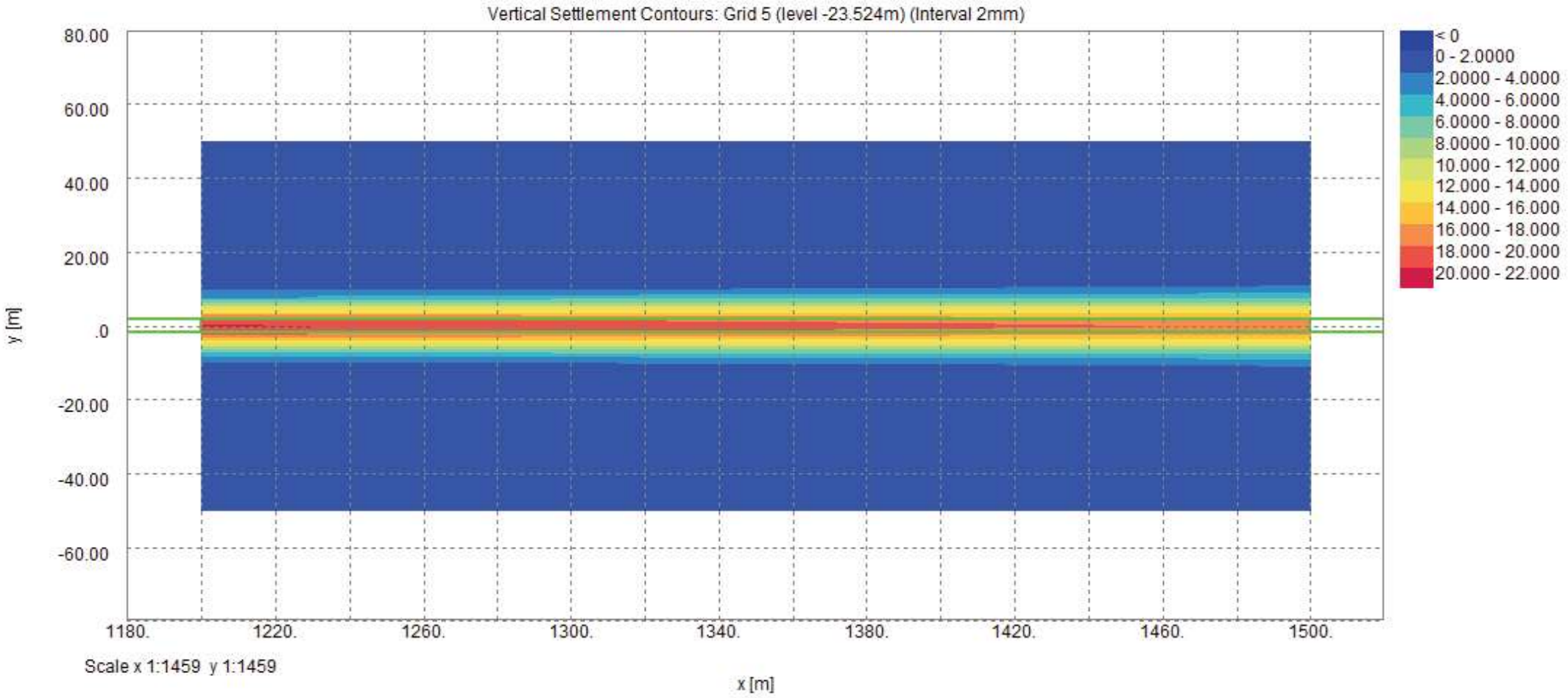
Plan view contour of seabed at ch 0+400 to ch 0+600 for 2% volume loss



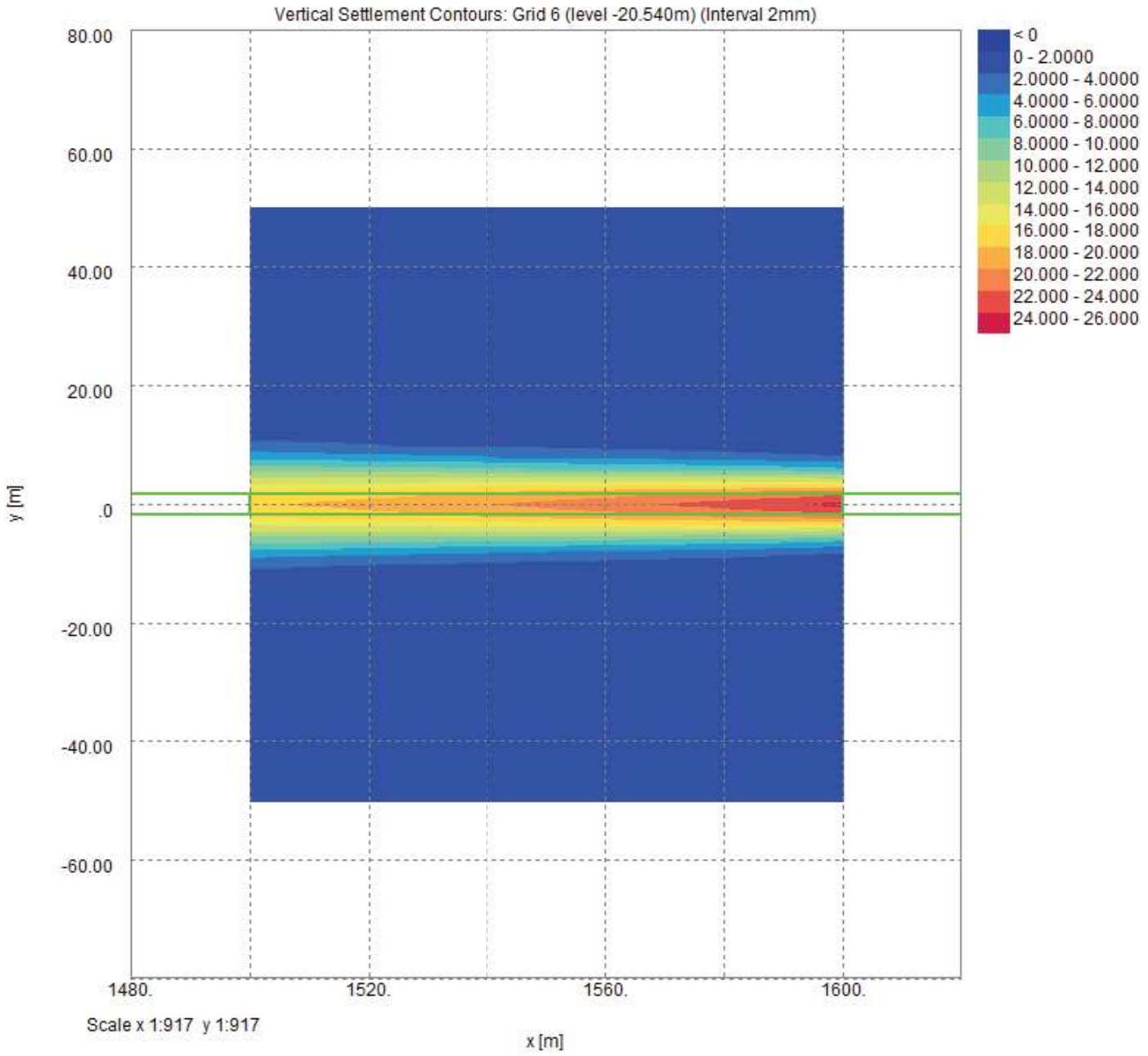
Plan view contour of seabed at ch 0+600 to ch 1+200 for 2% volume loss



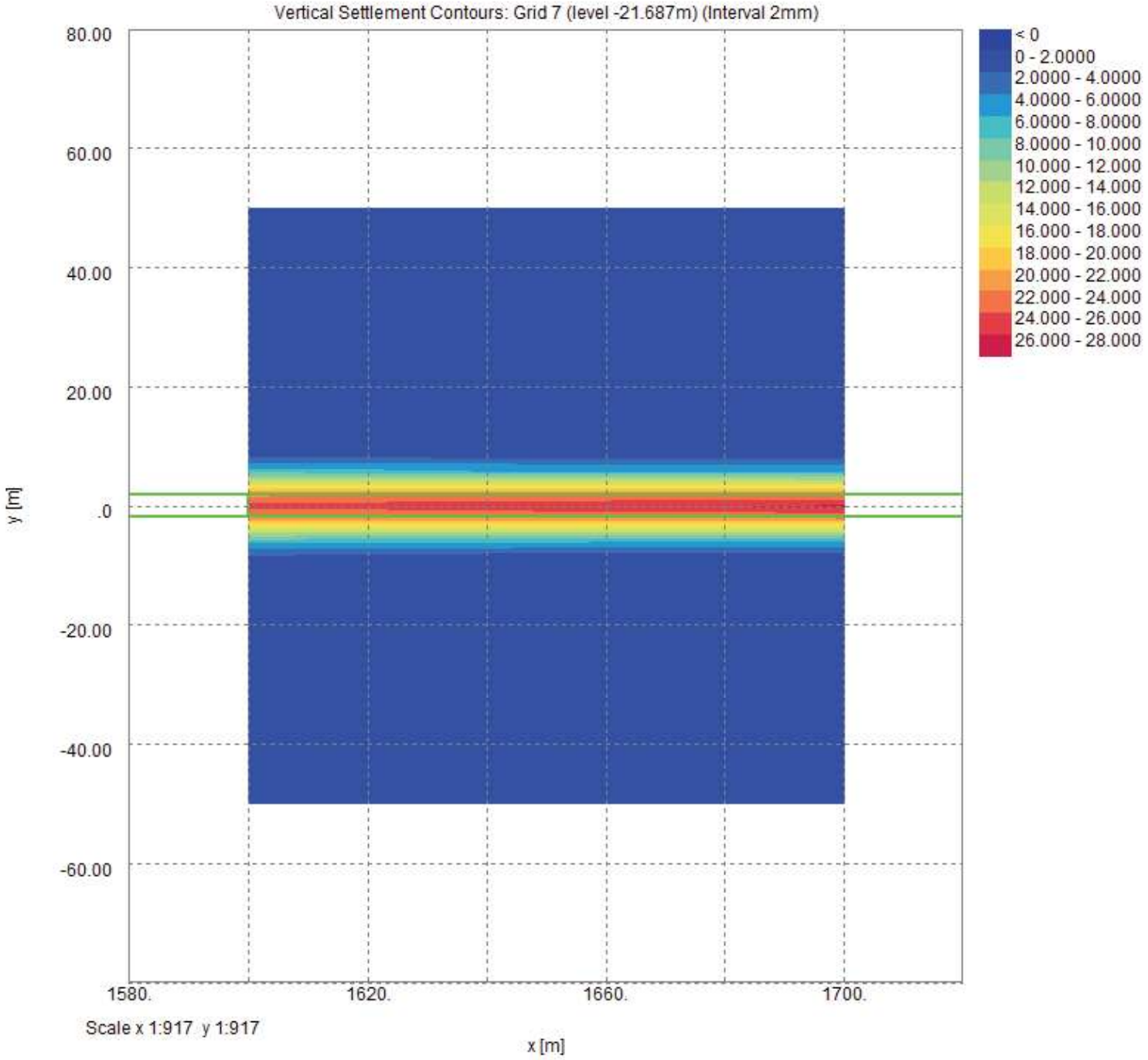
Plan view contour of seabed at ch 1+200 to ch 1+500 for 2% volume loss



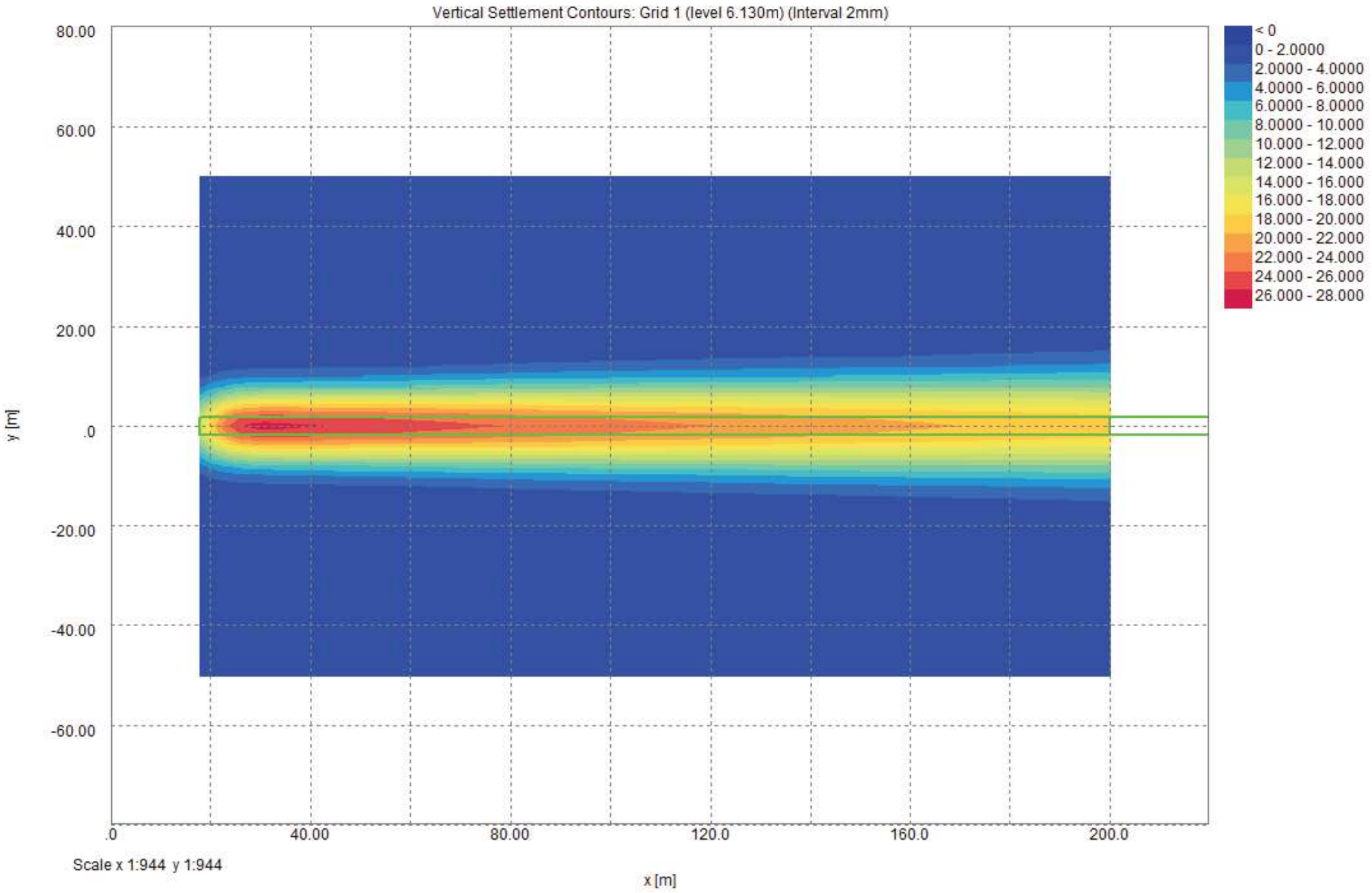
Plan view contour of seabed at ch 1+500 to ch 1+600 for 2% volume loss



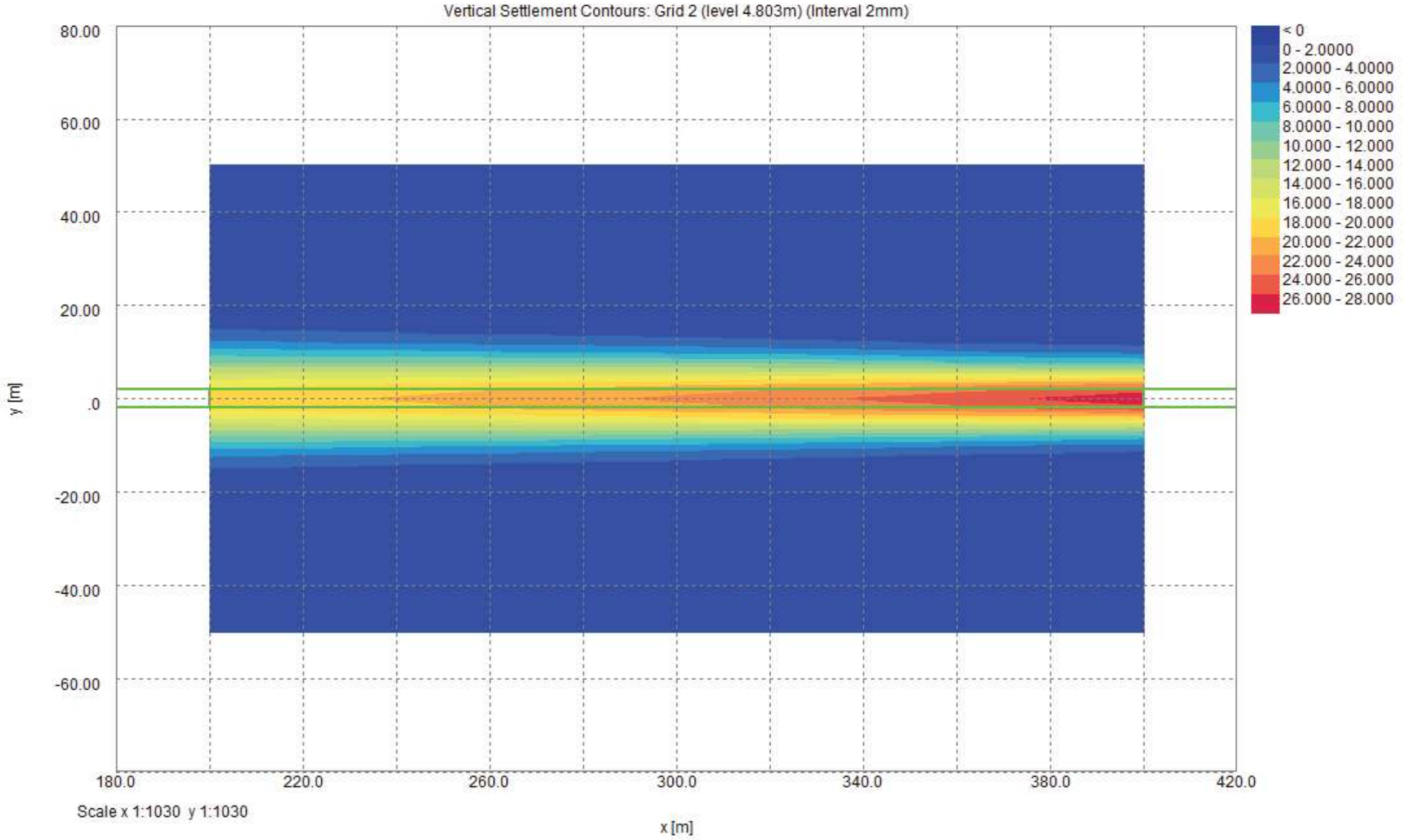
Plan view contour of seabed at ch 1+600 to ch 1+700 for 2% volume loss



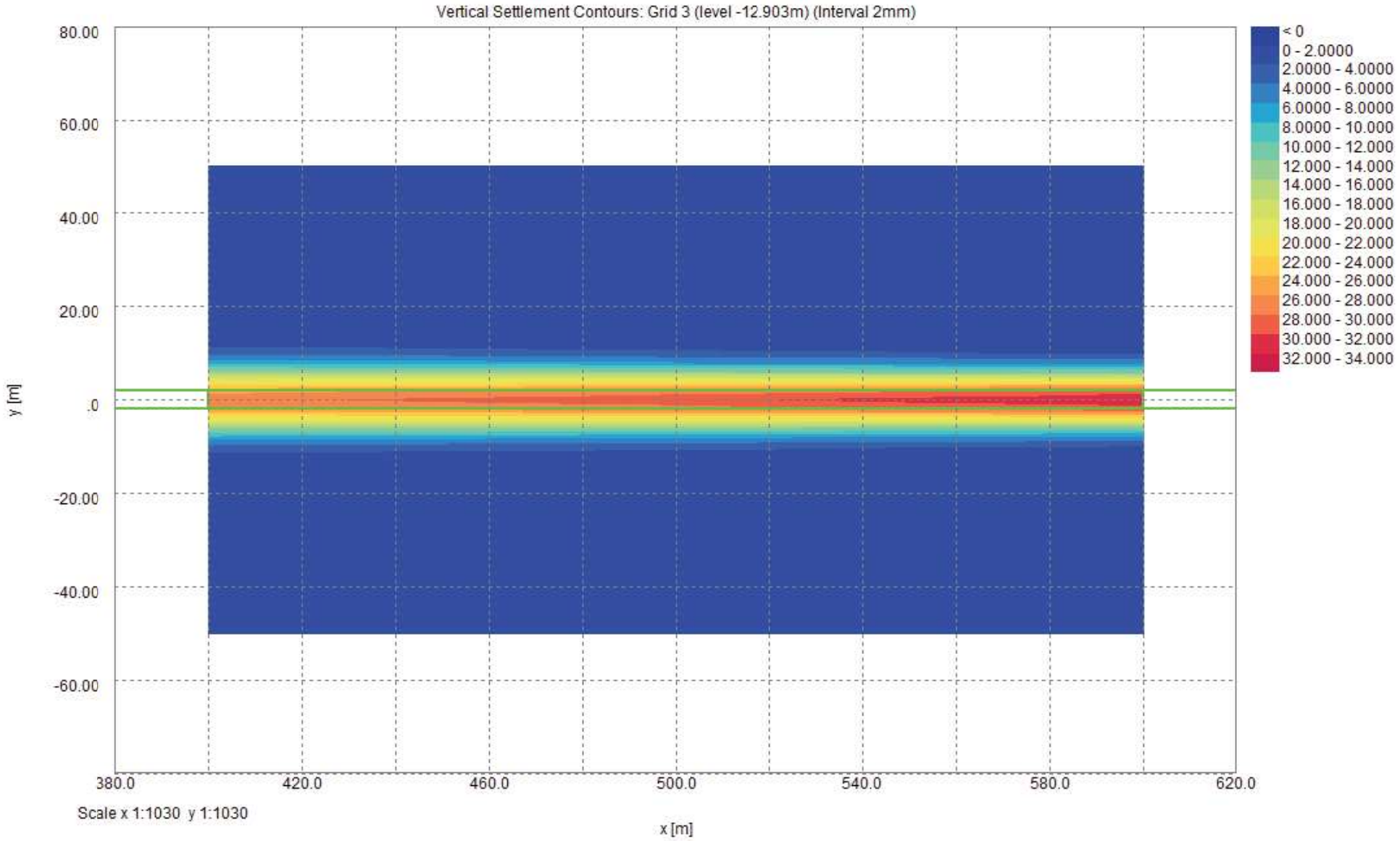
Plan view contour of seabed at ch 0+018 to ch0+200 for 3% volume loss



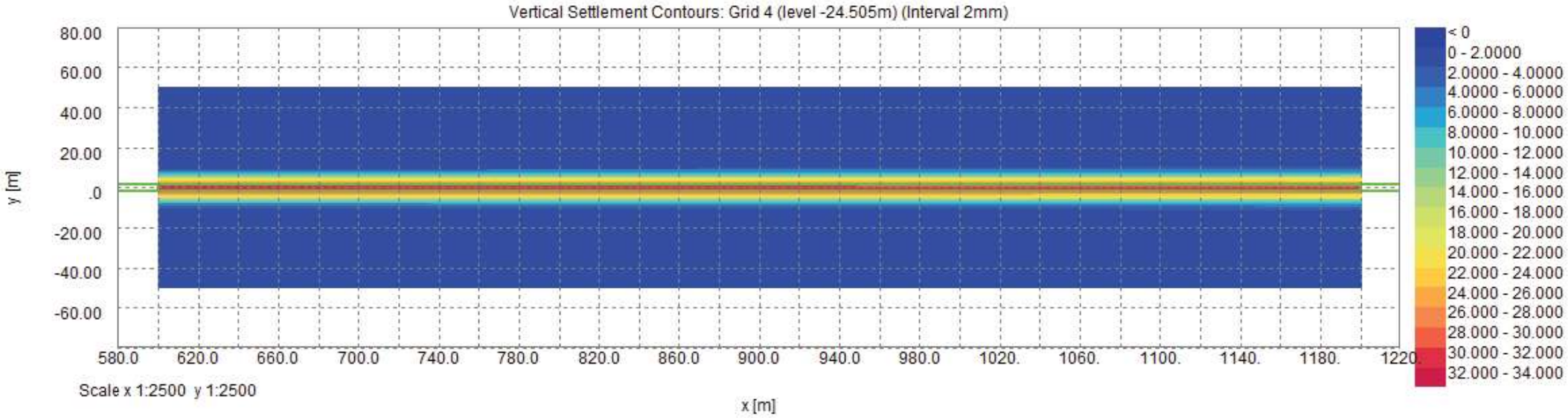
Plan view contour of seabed at ch 0+200 to ch 0+400 for 3% volume loss



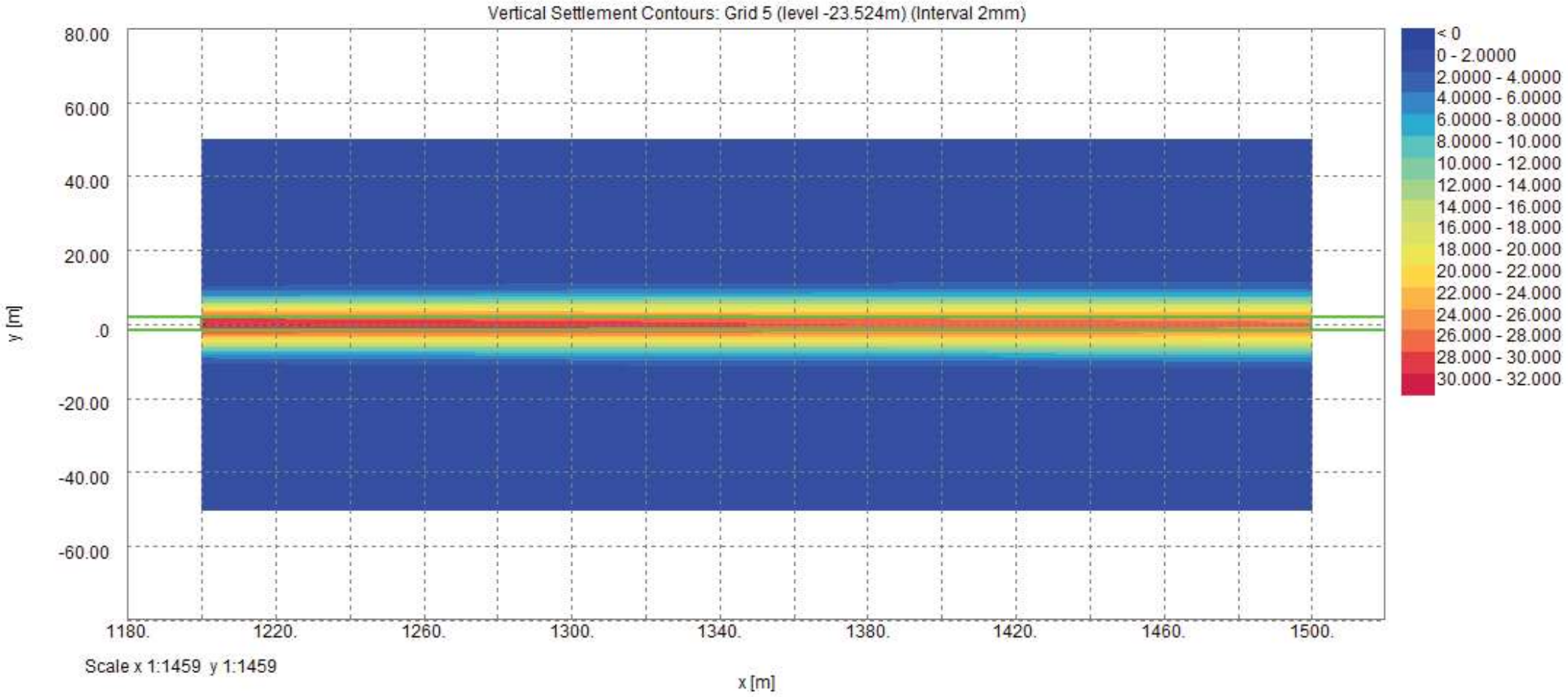
Plan view contour of seabed at ch 0+400 to ch 0+600 for 3% volume loss



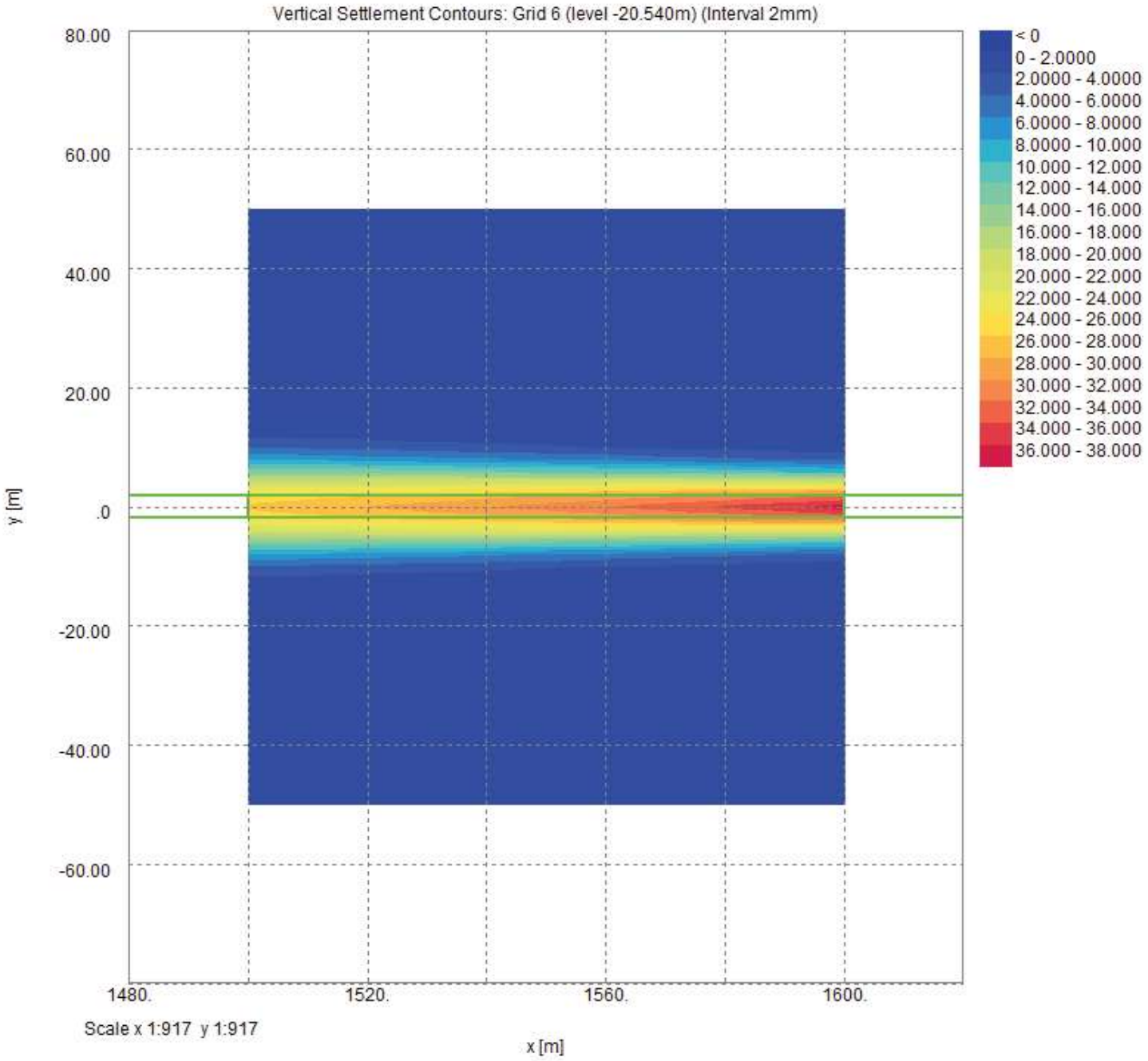
Plan view contour of seabed at ch 0+600 to ch 1+200 for 3% volume loss



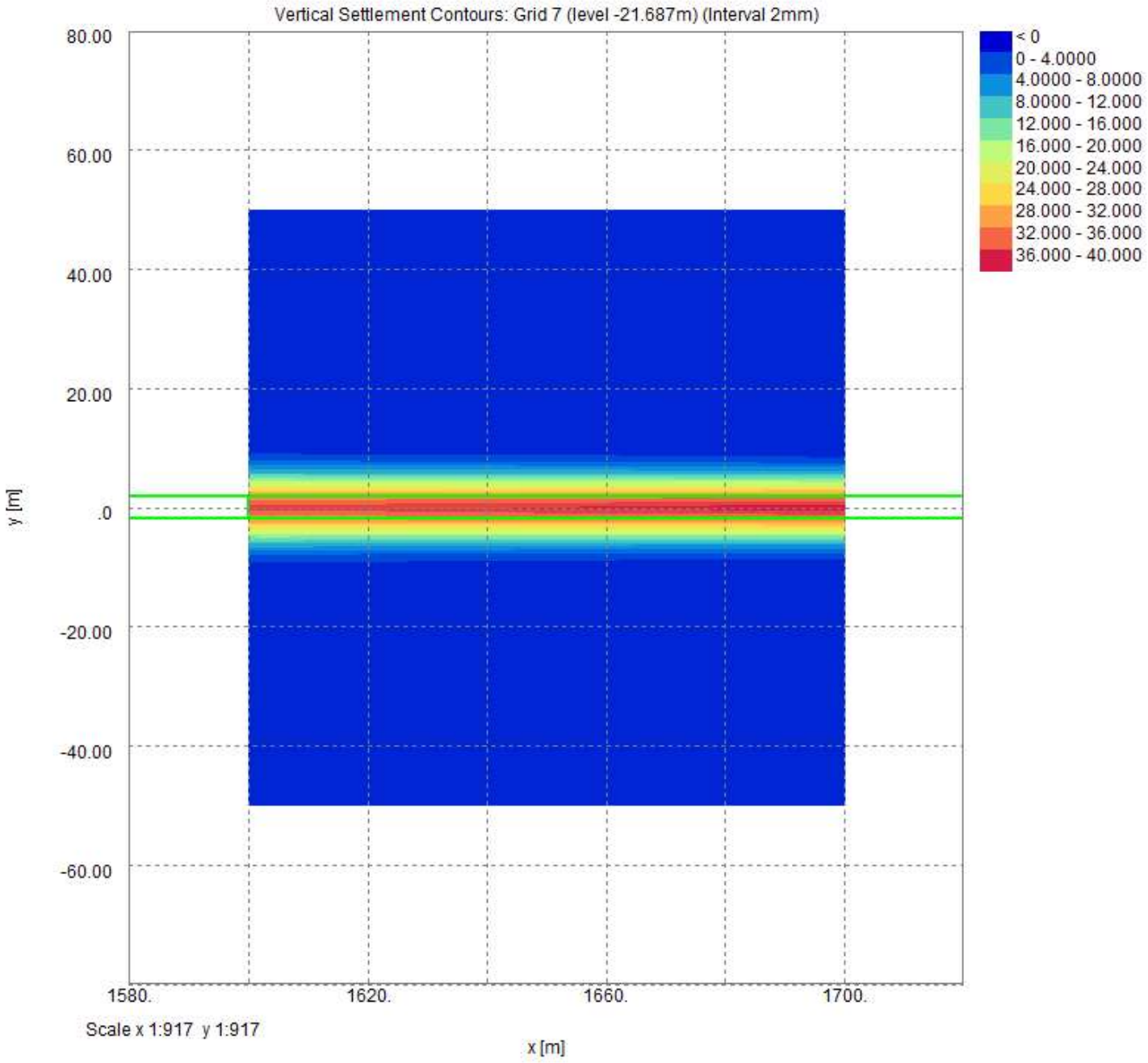
Plan view contour of seabed at ch 1+200 to ch 1+500 for 3% volume loss



Plan view contour of seabed at ch 1+500 to ch 1+600 for 3% volume loss



Plan view contour of seabed at ch 1+600 to ch 1+700 for 3% volume loss



APPENDIX D LAKE MEAD MONOPILE PROJECT EXPERIENCE

ARUP

Aramis : TBM retrieval – use of steel tube

Lake Mead project example

07 May 2024

ARUP

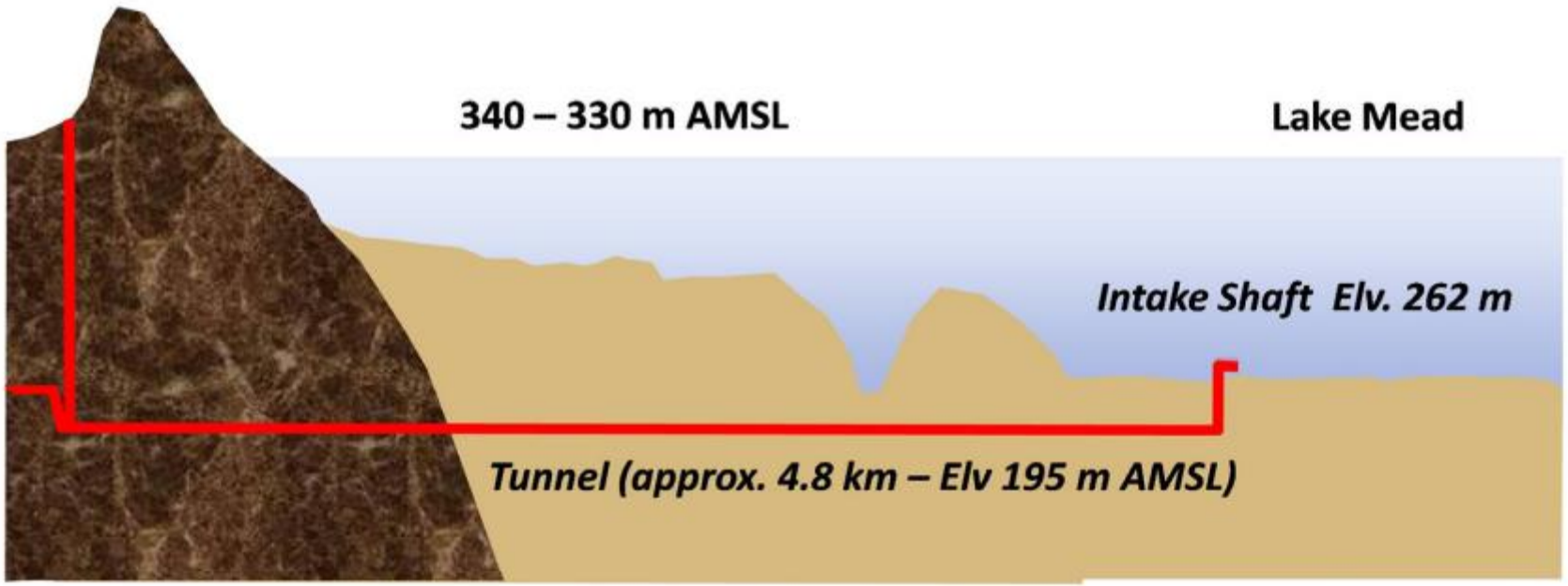
Lake Mead experience

General Scheme

International Forum on Subsea Tunnels

ARUP

Access Shafts



salini impregilo

Intake

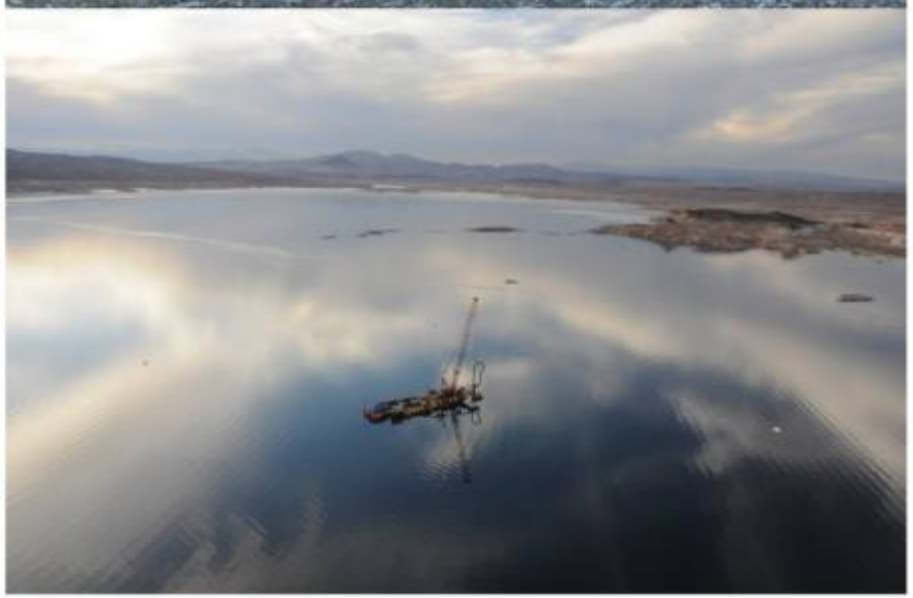
International Forum on Subsea Tunnels

ARUP

- Excavation and Blasting
- Prefab structure
- Placement and Survey
- Underwater Tremie concrete







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Intake - Excavation

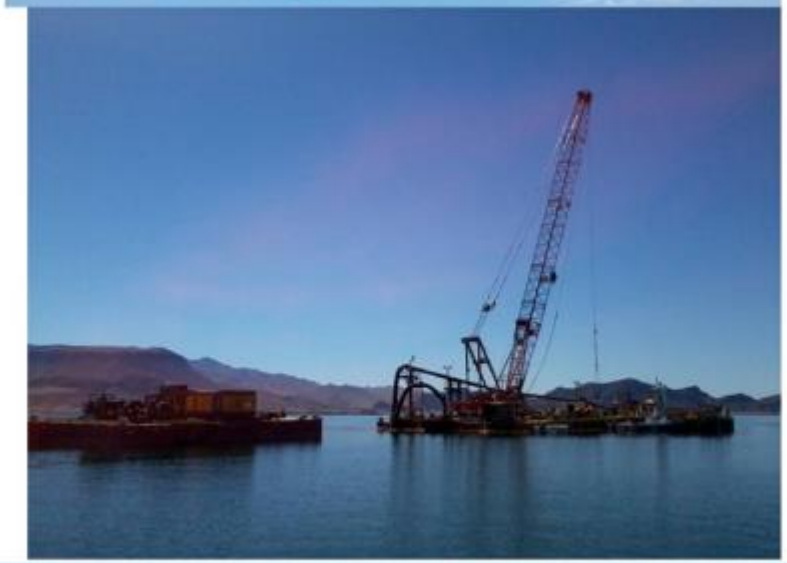
International Forum on Subsea Tunnels

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- Excavation and Blasting done from crane barge
- Blasting with shaped charges
- Clamshell, airlift for muck removal.
- 19,000 m^3 rock removed
- Elevation 250 to 230 m



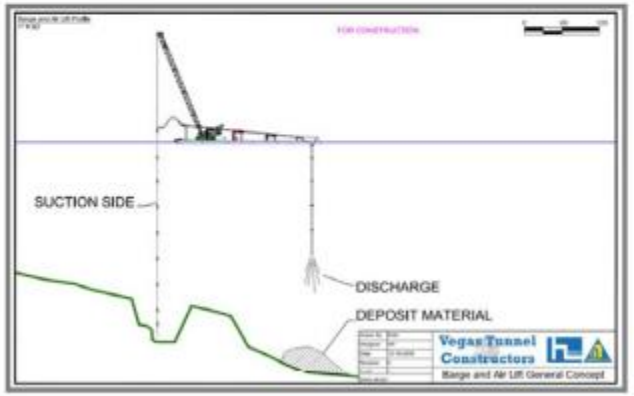
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Intake - Excavation

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ARUP





- Clamming used for removal of most material
- Airlift "cleaned" rock surface prior to blasting



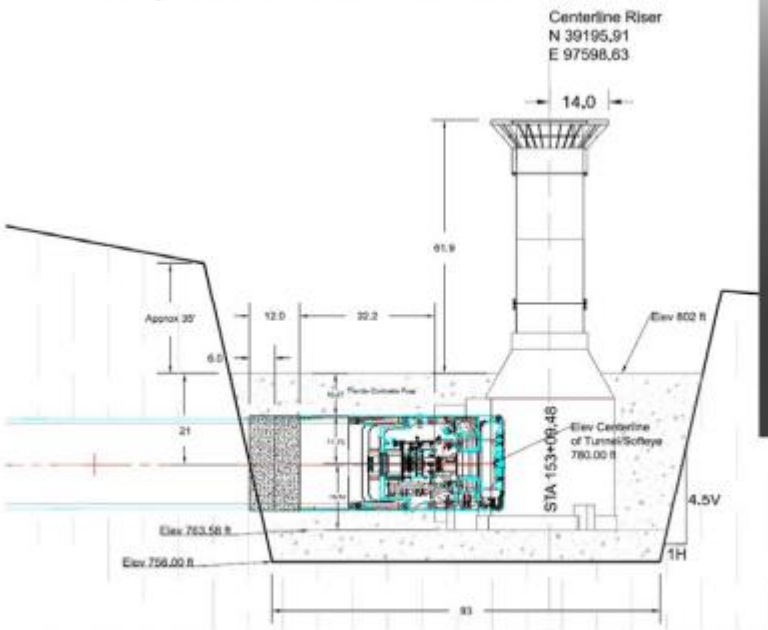
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Intake - Construction

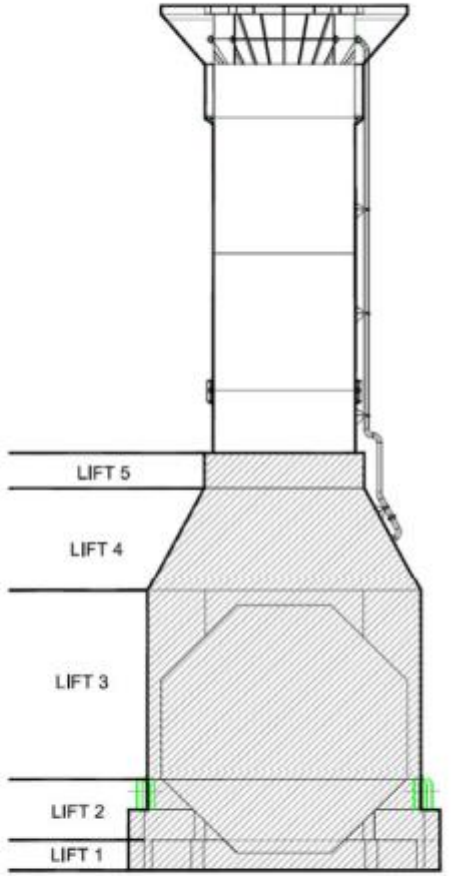
International Forum on Subsea Tunnels

ARUP

- Prefabricated structure
- Concrete and Stainless steel
- Internal bulkhead
- No Divers
- Lowered into excavation
- Surveyed
- Encased in tremie concrete
- Reception chamber for TBM







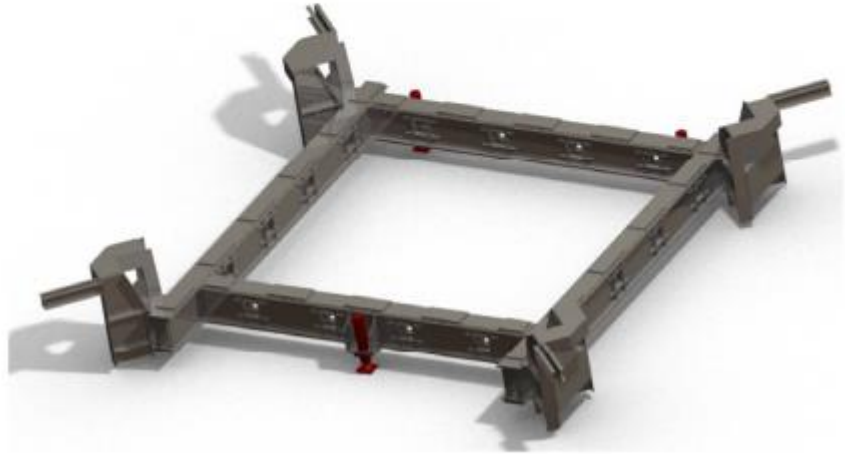
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Intake - Launching

International Forum on Subsea Tunnels

ARUP

1. Placed frame on bottom of excavation.
2. Leveled guiding frame with hydraulic jacks. Placed tremie concrete around fr ame to support it.
3. Lower structure with strand jacks to guiding frame.
4. Filled excavation to elevation 244 m with tremie concrete and permanently secure structure. Appro x 8,400 cum.
5. All operations were monitored with an ROV





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Intake - Launching

International Forum on Subsea Tunnels

ARUP

- Was done with “cigar” buoys attached to guiding frame and structure
- Continuously monitored with total station and GPS on calm day for several hours on multiple occasions
- Good results, actual error of +/- 5 cm
- TBM has margin of error of +/- 30 cm

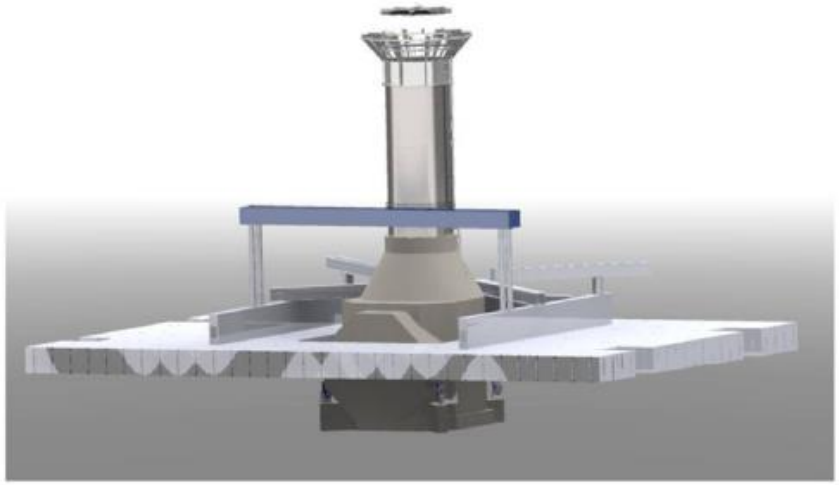




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Intake - Construction

International Forum on Subsea Tunnels



- Built in lifts on barge
- 1200 Tons
- Supported by 4 strand jacks





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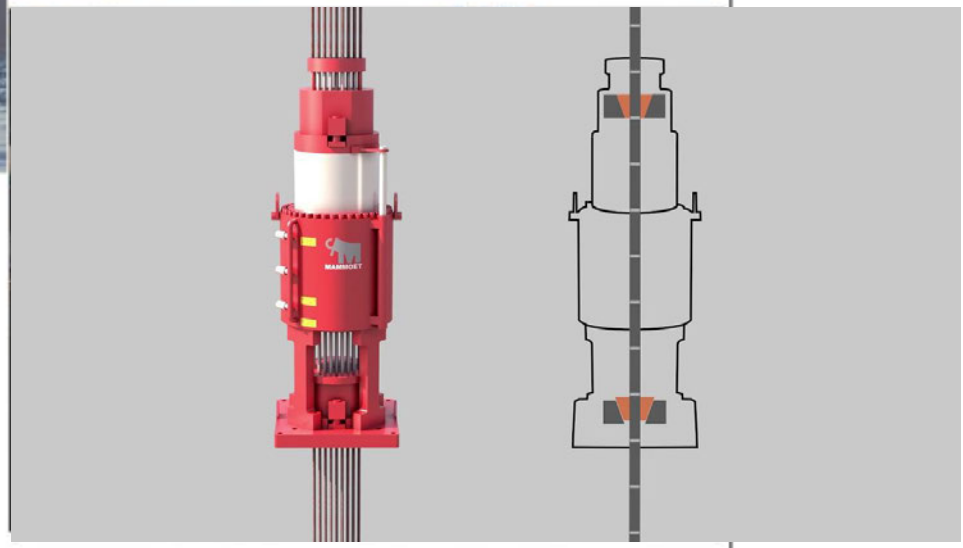
Intake - Launching

International Forum on Subsea Tunnels

ARUP







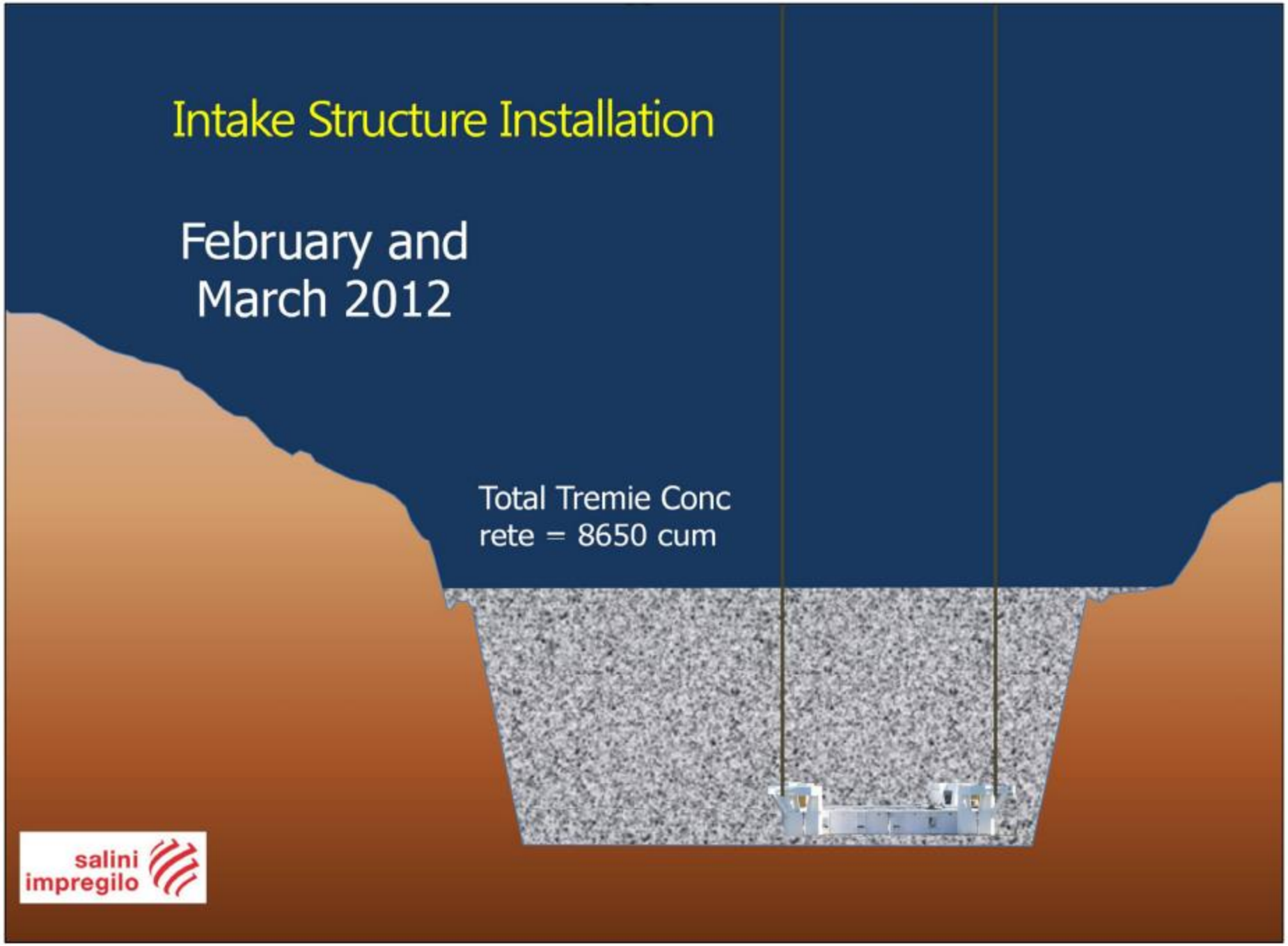
- 3 Hour transport from dock to location.
- Lowering done over course of 3 days

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ARUP

Intake Structure Installation

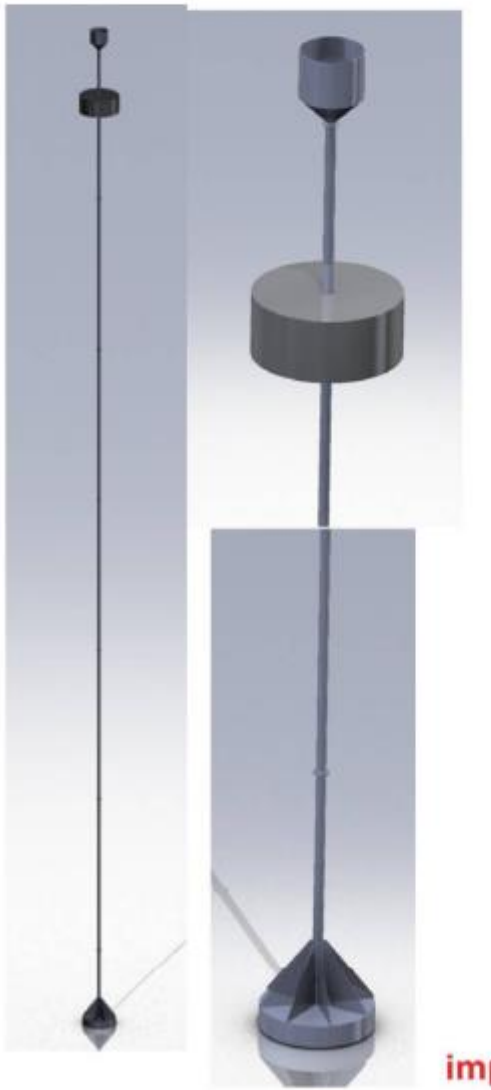
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March 2012



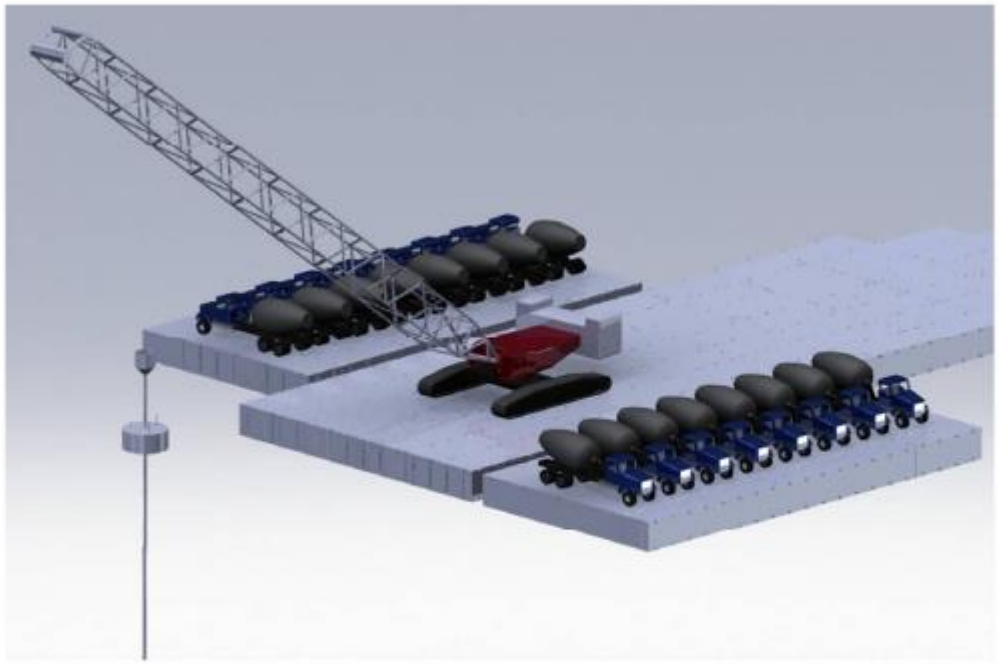
Intake – Concrete Pouring

International Forum on Subsea Tunnels

ARUP



- Self Consolidating mix
- Batch plant on site
- 4 barges with 8 trucks each
- 12 days of pouring
- Trucks supply hopper conveyed to pump on barge
- Tremie pipe moved around excavation in circular manner as hole filled
- Pipe shortened as concrete level increases



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Intake – Concrete Pouring

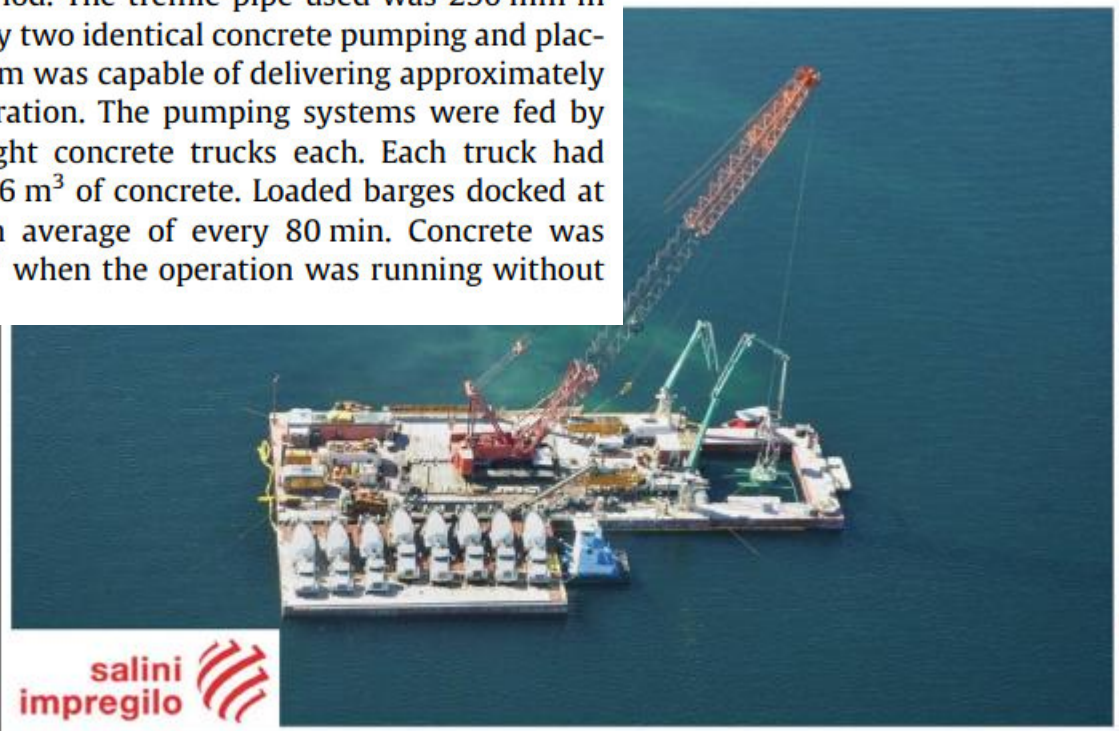
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ARUP



6.4. Tremie concrete placement

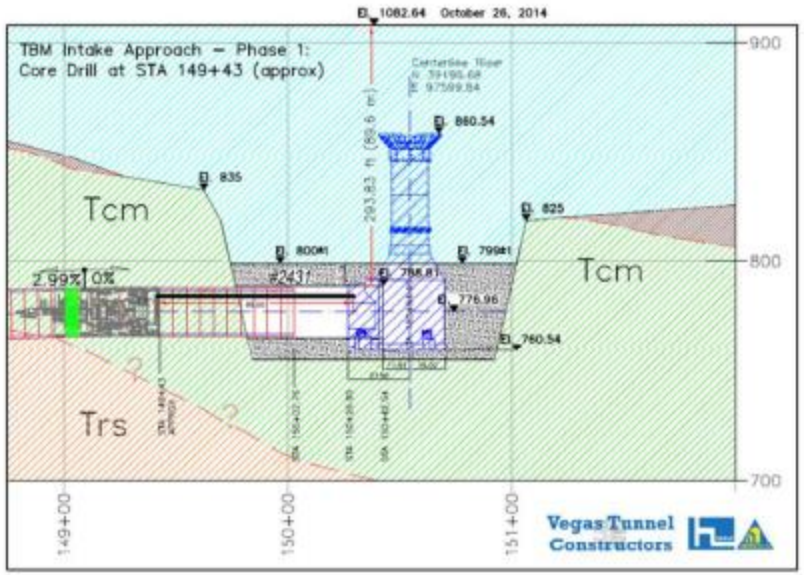
Approximately 9200 m^3 of concrete was placed in a continuous pour over a 12-day period. The tremie pipe used was 250 mm in diameter and was fed by two identical concrete pumping and placing systems. Each system was capable of delivering approximately 92 $m^3 \cdot h^{-1}$ at peak operation. The pumping systems were fed by barges loaded with eight concrete trucks each. Each truck had the capacity to carry 7.6 m^3 of concrete. Loaded barges docked at the crane barge on an average of every 80 min. Concrete was placed at 42–46 $m^3 \cdot h^{-1}$ when the operation was running without interruption.

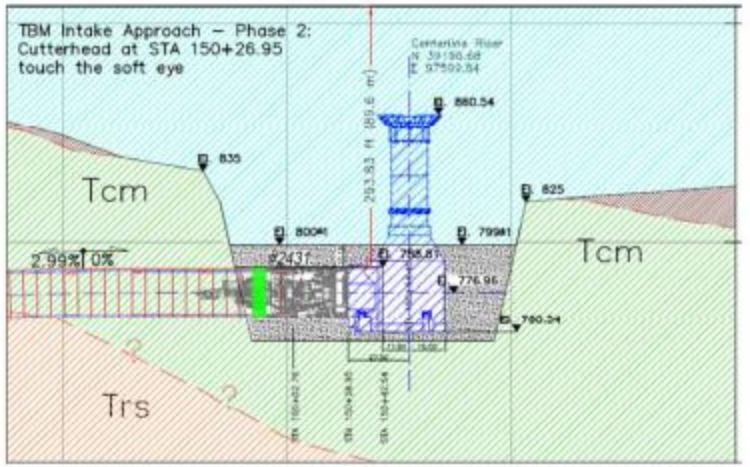


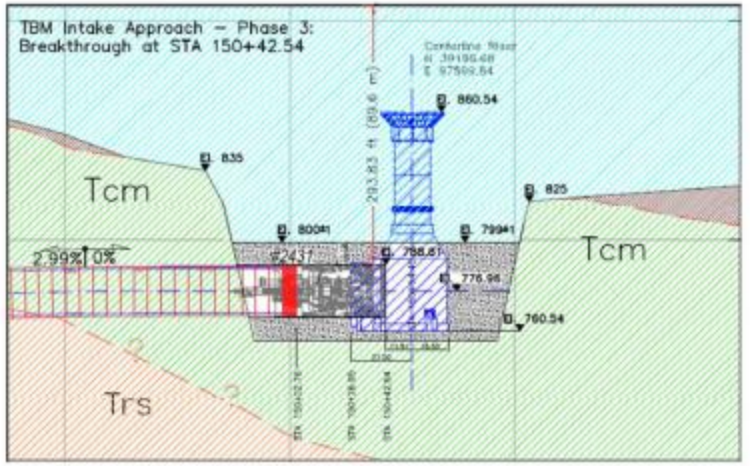


TBM Break in

ARUP





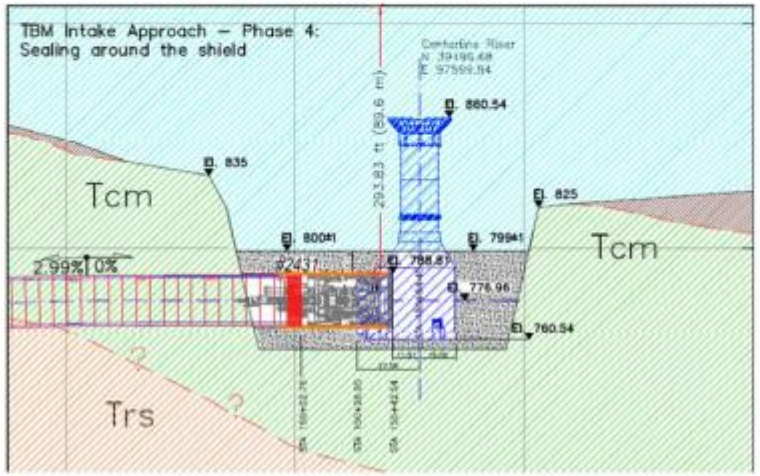


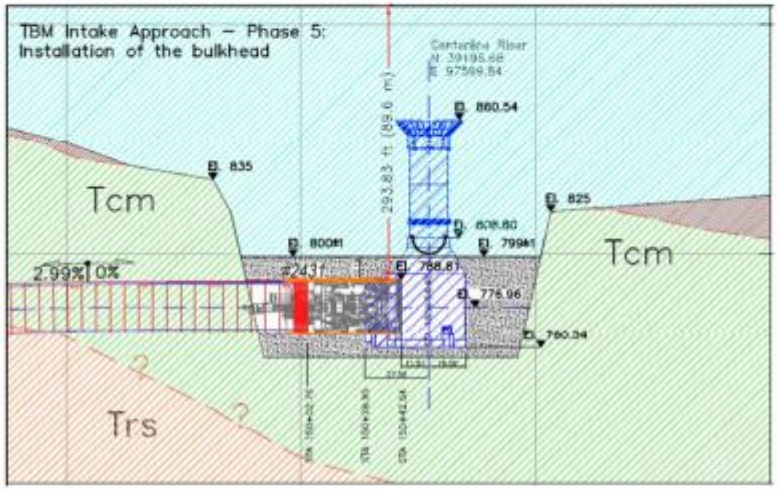
Phases of Break in:

1. Probe hole to check permeability of tremie concrete
2. TBM advance to touch soft eye
3. Break in at ring 2,427

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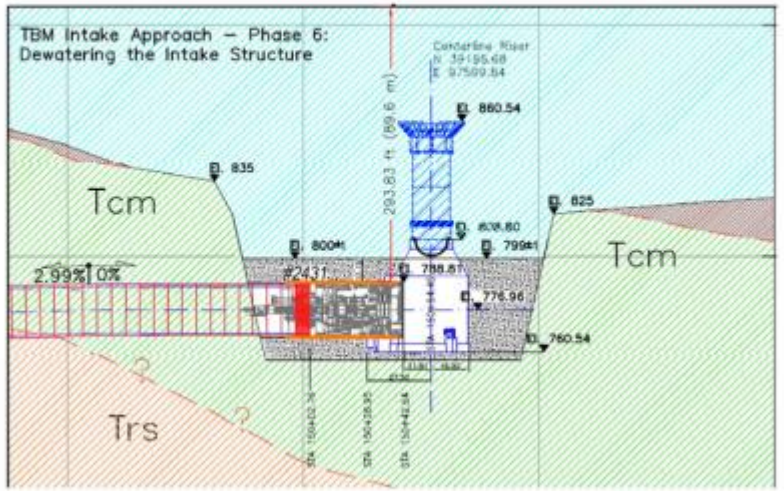
TBM Break in





Phases of Break in:

4. Sealing around the shield
5. Installation of the bulkhead
6. Dewatering of the Intake



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ARUP

Bijlage

7. Settlement Impact Analysis and Monitoring Plan - Sea Defence

NL-ARM-040-ARU1-110381_02_IFD_20240824_01

Royal HaskoningDHV

Tunnel FEED - Settlement Impact Analysis and Monitoring Plan - Sea Defence				**Document Reference** NL-ARM-040-ARU1-110381	
				Revision: 02	**Step**: IFD
				Date: 24/10/2024	
Document Type: REP	**Family Equip:** n/a	**Discipline:** CIV	**Class:** 2	**Page** 1 of 63	



ARAMIS PROJECT



Rev	Step	Date	Revision memo	Issued by	Checked by	Approved by
00	[redacted]	11/06/2024	Issued for Review	[redacted]	[redacted]	[redacted]
01	[redacted]	09/10/2024	Issued for Design	[redacted]	[redacted]	[redacted]
02	[redacted]	24/10/2024	Re-Issued for Design	[redacted]	[redacted]	[redacted]

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ABBREVIATIONS

Abbreviation	Description
GDR	Geotechnical Design Report
VL	Volume Loss
FE	Finite Element
C/D	Cover to diameter ratio
TBM	Tunnel Boring Machine
SLS	Serviceability Limit State

DEFINITIONS

Terminology	Definition
Company	Aramis, a joint development by TotalEnergies, Shell Netherlands, Energie Beheer Nederland (EBN) and NV Nederlandse Gasunie.
Aramis	Refers to assets owned and operated by Aramis
Project	Aramis Project consisting large-scale infrastructure to transport CO2 captured by other industrial enterprises and to deliver this offshore to depleted gas fields in which the CO2 will be stored permanently.
FEED Contractor	Petrofac Facilities Management Ltd.
EPC Contractor	Party or organisation selected by the Company to execute the detailed engineering, procurement (supply) and construction (and commissioning) phase of the project.
Supplier	Party or organisation that manufactures, supplies equipment packages and/or services to perform duties specified by the Company, FEED Contractor or EPC Contractor.
Shall	Is used to indicate a mandatory requirement.
Should	Is used to indicate that the provision is preferred.
May	Is used to indicate alternatives, which are equally acceptable, at Contractor discretion.
Must	Is used to indicate compulsory, legally binding requirement.

HOLDS

No.	Section	Hold

1 INTRODUCTION

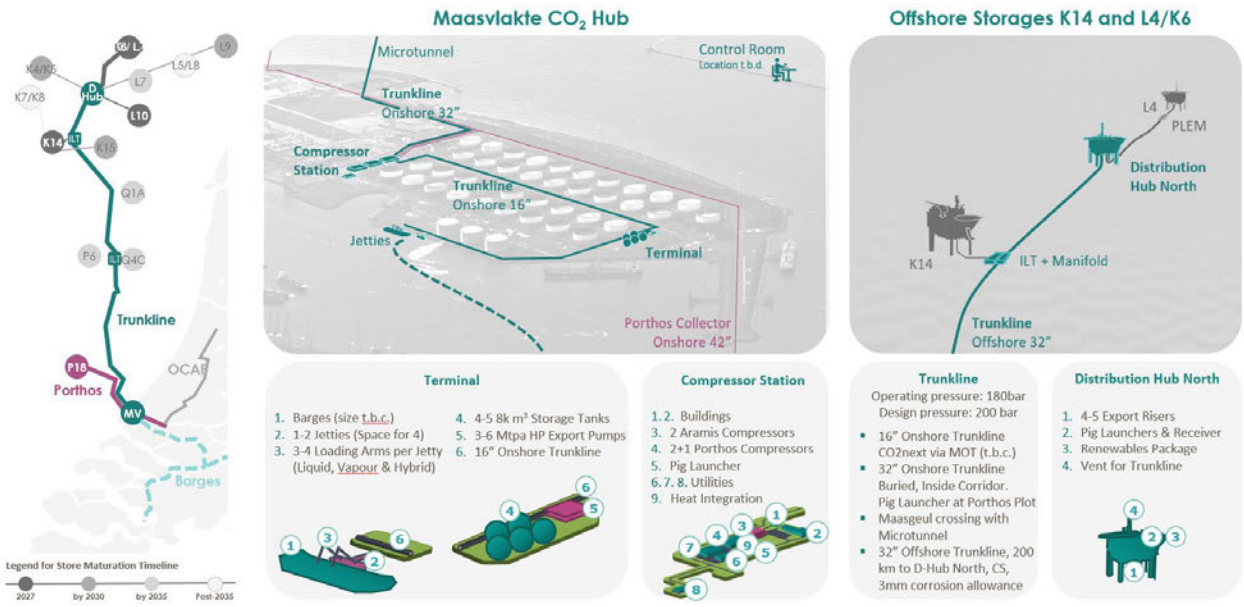
1.1 Background

The Aramis project is a cooperation between Shell, TotalEnergies, Gasunie and EBN to utilise depleted gas fields and potentially aquifers in the Dutch North Sea to establish an open-access CCS infrastructure for transport and storage, targeting hard-to abate industrial emissions in the Netherlands and Europe.

This part of the project provides the transport of CO_2 from the onshore terminal to an offshore platform, which is used as distribution hub (D-Hubn). Other parts of the project are provided by CO2next and Porthos, while the actual storage in the depleted offshore gas fields under the North Sea is performed by Shell and TotalEnergies as part of their offshore operations.

In Figure 1-1 below an overview of the envisaged CCS infrastructure is given. The left panel illustrates the trunkline that transports the CO_2 from the Maasvlakte to the offshore storage. The middle panels illustrate the Maasvlakte Hub with the Porthos compressor station and the CO2next Terminal. The right panel shows the platforms of TTE and Shell.

Figure 1-1 - Overview of CCS Infrastructure



1.2 Purpose of Document

This technical note assesses the influence of the tunnel excavation below the sea defence through settlement analysis as part of the FEED design. The construction of the tunnel will inevitably cause movement of the ground around the sea defence. The aim of this technical note is to evaluate the vertical settlement, horizontal displacement and tensile strain across the sea defence and allow for better understanding of the implications of ground disturbance associated with the proposed ground excavation strategy for tunnel construction.

The impact of the associated shaft construction on the sea defence has also been addressed within this technical note.

This document also provides recommendations for instrumentation and monitoring of the sea defence to provide confidence in design and verify predicted behaviours, and also includes comment on potential extreme tunnel failure scenarios (e.g. very high-volume losses which could be associated with tunnel collapse) and the potential impact on the sea defence levels.

Reference is made to the following documents:

- Tunnel Basis of Design: NL-ARM-040-ARU1-110301 [1]
- Tunnel Alignment: NL-ARM-040-ARU1-110323 [2]
- Geotechnical Design Report: NL-ARM-040-ARU1-110305 [3]

2 PROJECT INFORMATION

2.1 Tunnel

The alignment for the CO_2 pipeline is presented in drawing NL-ARM-040-ARU1-110323 [2] and the location of the tunnel relative to the sea defence is presented in Figure 2-2. This alignment will be considered as the basis for the analysis covered within this technical note.

The proposed tunnel has 3m internal diameter (3.5m external diameter) constructed using Slurry TBM – hybrid pipejacking to segmental lining method. The tunnel alignment length is approximately 1.7km.

For the impact of the tunnel itself, the site considered at the location of the sea defence is between chainage 0+100.0 to 0+300.0 (Figure 2-1). The location of the tunnel relative to the sea defence is presented in Figure 2-1 and Figure 2-2. Comment on the impact of shaft construction is included in section 2.4.

Figure 2-1 – Tunnel vertical alignment [2]

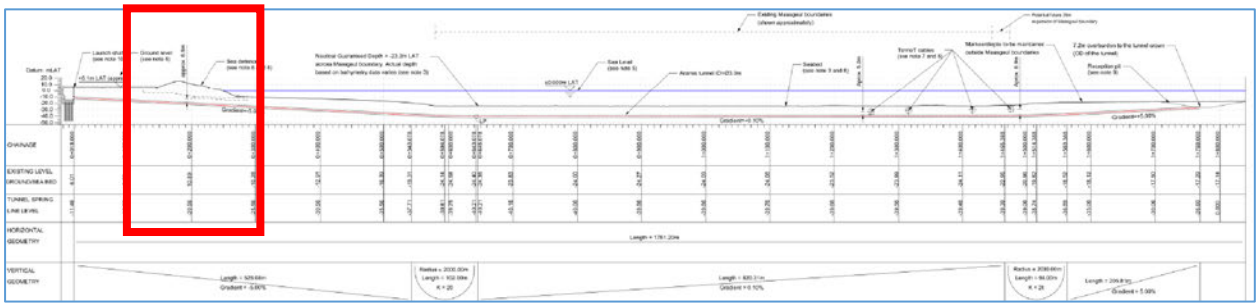
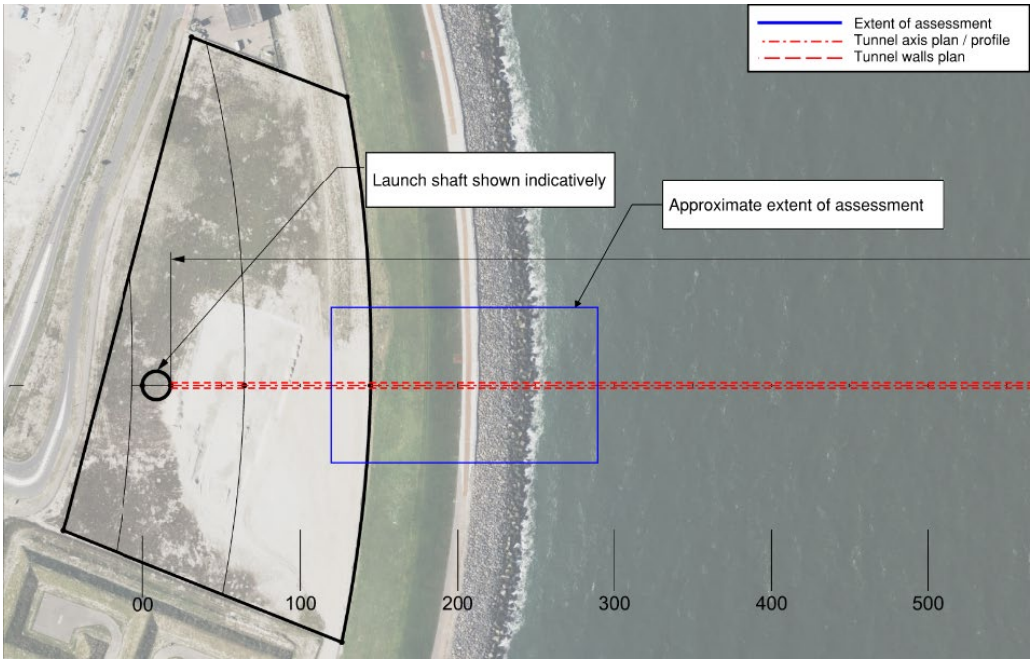


Figure 2-2 – Plan view of tunnel alignment relative to sea defence



2.2 Sea Defence description

The general composition of the structure consists of a sand core with concrete components in addition to large stones and concrete blocks as the structure approaches the shore.

An annotated cross-section of the sea defence is shown in Figure 2-3, based on “as-built” information originating from RWS (“As built tekeningen Haaienvin" i.e. “as-built drawings for sharkfin”) and provided by Company in response to Technical Query NL-ARM-040-PTC1-TQ0002 [4]. The figure shows drawing *GZ079.419* which is a modernised version of Section A-A shown on *A7H_9924_2_van_14* (see Figure 2-4) with the section location identified on *A7H_9924_1_van_14* (see Figure 2-5) as contained with the Technical Query attachments. While the location of the section A-A does not exactly align to the location of the Aramis shaft and tunnel alignment, it is assumed from the as-built drawings that the cross section represents a continuous section along the outer perimeter of the sharkfin at the location of the tunnel crossing.

The proposed tunnel excavation occurs directly beneath the sea defence which spans from approx. Ch120m to Ch290m along the tunnel. The distance between the base of the sea defence and the tunnel crown ranges from approx. 6m to 16m (estimated with reference to the as-built drawings). For information, the top of the concrete block portion of the sea defence is around c.8m NAP (9m LAT) and the top of the sea defence bund is c.15mNAP (16mLAT). For comparison, the highest 40-year tide level referenced in the Tunnel FEED Basis of Design [1] and Geotechnical Design Report [3] is around 4.5mLAT.

Figure 2-3 – annotated cross section of shark-fin from RWS as-built drawings. Source: [4]

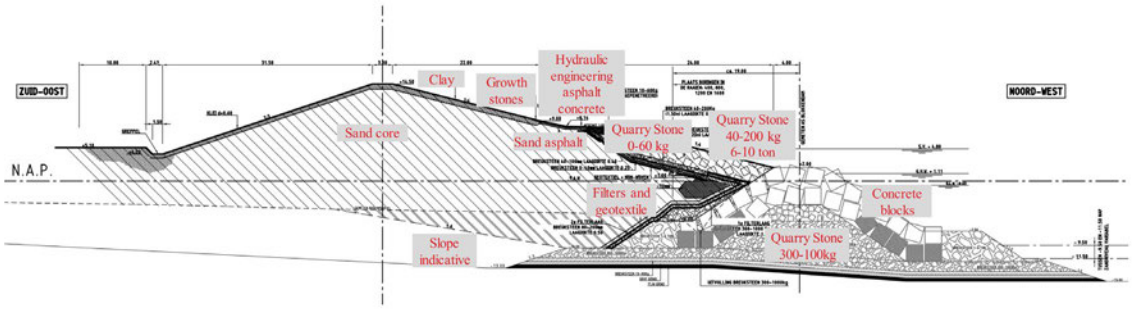


Figure 2-4 – Section A-A from RWS as-built drawings. Source: [4]

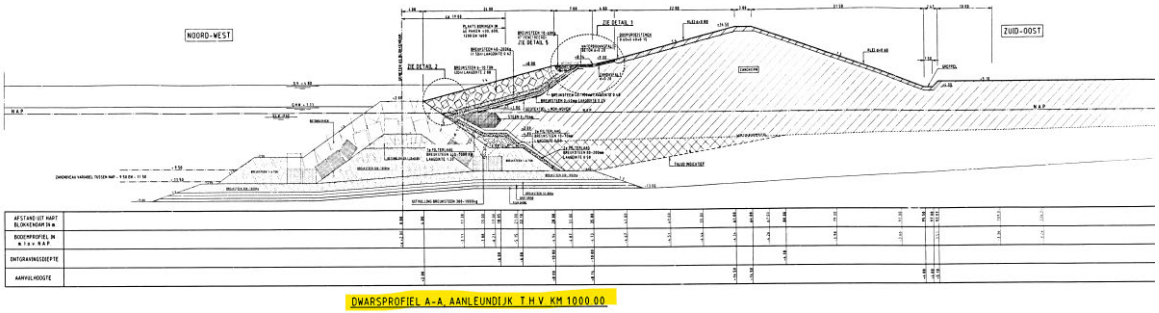
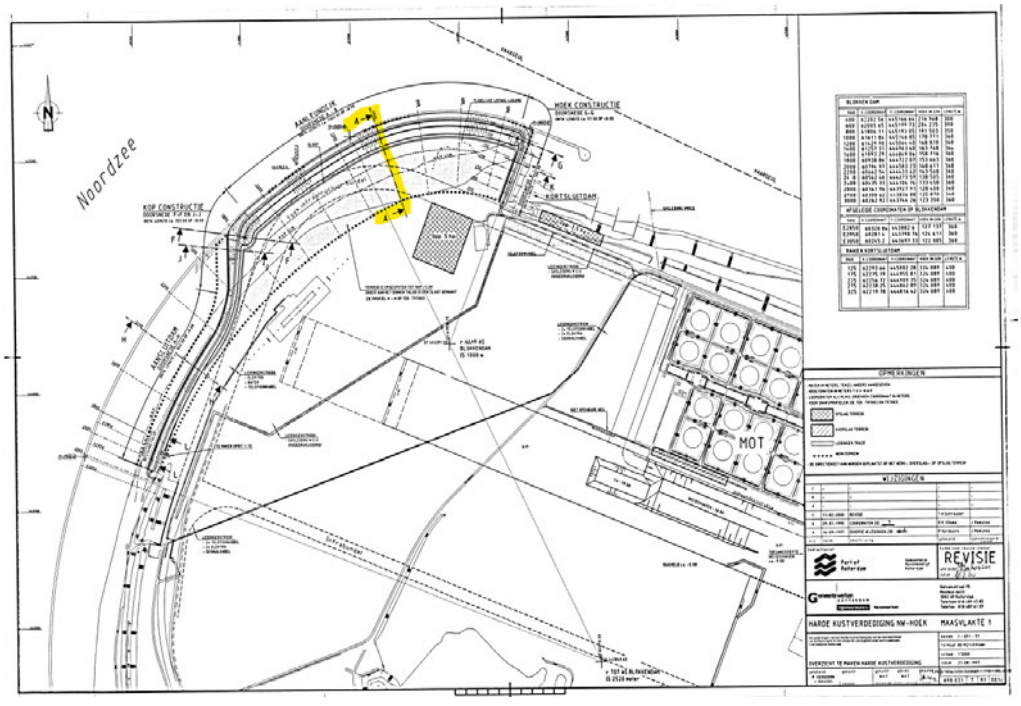


Figure 2-5 – Plan location of Section A-A from RWS as-built drawings. Source: [4]



Due to limited third-party information on geological or mechanical materials to define the properties of the sea defence, Table 2-1 shows the mechanical parameters of the sea defence that have been assumed based on the properties of the made ground material taken from a nearby borehole.

Table 2-1 - Assumed Mechanical Properties of the Sea Defence

Property	Symbol	Value
Elastic Modulus	E	30 MPa
Poisson Ratio	υ	0.25
Shear Modulus	G	18.75 MPa

The sea defence spans approximately 170m from end to end. The construction materials and thickness of the structure is variable across the span. The thickest section is located below the summit of the sand core measuring at approx. 19.7m from the bottom to the surface. The sea defence follows a general taper as it approaches the shore with a rubble stone layer being introduced below the blocks and quarry stone.

2.3 Geotechnical Conditions at the Sea Defence

In the GDR [3], several typical sections of the simplified soil profiles were developed. The simplified soil profile for the site considered is Section B. The corresponding sets of Lower and Upper bound soil parameters are given in Table 2-2 and Table 2-3.

Settlement of the sea defence could induce stress induced tension cracks to manifest within the upper clay capping as the tunnelling progresses beneath, these tension cracks could allow the ingress rather than run off of surface water into the sand core. It is assumed based on the drawings that the slopes on the ‘dry’ side is 1 (v) in 3 (h) and 1 (v) in 4 (h) for the ‘seaward’ side. The introduction of water in the defence embankment could result in a circular slip failure within the material. This is less of a risk on the seaward side due to the shallow gradient and significant toe weight imposed by the quarried and concrete blocks, any failures are therefore likely to be shallow and restricted to the upper slope on the landward side. It is noted that there is some evidence of a local slip failure at the toe of the sea defence embankment around 2004 as discussed in the Ground Investigation Report Section 2.4 [5] in which it is also noted that remedial works appear to have been undertaken.

Table 2-2 – Table of Lower Bound soil parameters

Lower model (LM)	Layer thickness	Unit wt	Poisson's Ratio	Friction angle	Cohesion	Undrained shear str	Avg. Stiffness	EP coeff.	Permeability
SYMBOL		**γ**	**v**	**ϕp**	**c'**	**Su**	**E'**	**K0**	**k**
UNIT	**m**	**kN/m3**	**-**	**degrees**	**kPa**	**kPa**	**kN/m2**	**-**	**m/s**
MG Silty sand (LM)	21.5	18.5	0.25	29	0	-	40000	0.52	0
NLD Silty sand LB	10	18.5	0.25	31	0	-	27000	0.52	1E-6
NLD clay	2	17	0.35	-	0	72	12960	0.67	1E-9
NLD Silty sand LM	4.5	17.5	0.25	30	0	-	30000	0.52	1E-5
Peat	2	15	0.45	15	0	-	1500	0.67	1E-7
NLD Silty sand LM	7.5	17.5	0.25	29	0	-	30000	0.52	1E-5

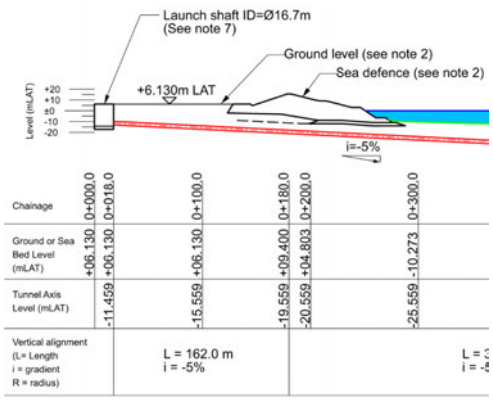
Table 2-3 – Table of Upper Bound soil parameters

Upper model (UB)	Layer thickness	Unit wt	Poisson's Ratio	Friction angle	Cohesion	Undrained shear str	Avg. Stiffness	EP coeff.	Permeability
SYMBOL		γ	v	ϕp	c'	Su	E'	K0	k
UNIT	m	kN/m3	-	degrees	kPa	kPa	kN/m2	-	m/s
MG Sand (UM)	21.5	20	0.2	38	0	-	62000	0.67	0
NLD Sand (UM)	31.6	20	0.2	38	0	-	60000	0.67	0
NLD Sand (UB)	4	20	0.2	42	0	-	76000	0.65	0
NLD Sand (UM)	10	20	0.2	38	0	-	60000	0.67	0

2.4 Impact of shaft construction on the Sea Defence

The shaft will facilitate the tunnel construction and pipeline installation. It has an internal diameter of 16.7m and its primary and permanent support comprises of a 1.2m thick diaphragm wall as described in the Tunnel FEED Design Report [6]. The shaft is located more than 100m away from the sea defence see Figure 2-6 below.

Figure 2-6 - Shaft location relative to sea defence (from NL-ARM-040-ARU1-110321)



The effect of the shaft construction on the sea defence was assessed in accordance with *"CIRIA 760* [7]. The ground movements due to the shaft were estimated based on empirical methods as per §6.2.1 for coarse-grained soils, for two construction stages: installation of the diaphragm wall and subsequent excavation of the shaft:

<u>During diaphragm wall installation</u>

Data for this case are very limited. Two examples are provided by CIRIA 760 (Symons et al (1987) and Puller (2003)). The sheet piles example (Symons et al) is not considered to be relevant, as the main cause of the large settlements is the vibration from driving the piles from the ground through sand and gravel layers.

The Puller (2003) report is more relevant to our case. This case study reports 0.03% horizontal ground movements/depth at ground level at a distance corresponding to half of the wall length (L/2), which reduces to zero at a distance corresponding to the wall depth. Subsequently, this can be extrapolated to about 0.06% horizontal ground movement/depth just behind the wall.

Assuming a diaphragm wall of an approximate length of 26.3m, the following horizontal ground movements are estimated:

0.06% x 26300= 16mm behind the shaft wall

0.03% x 26300= 8mm at a distance of x=26.3/2= 13.15m from the shaft wall

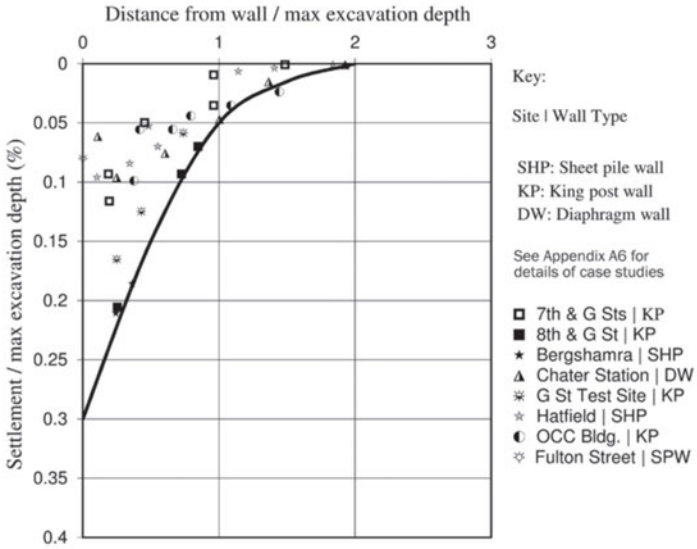
0 at a distance of x=26.3m from the shaft wall

It is noted that settlement due to other construction methods may differ; and the settlements should be verified for the selected shaft detailed design during the EPC Contractor detailed design phase,

<u>During the shaft excavation</u>

Figure 2-7, included below as well, shows the ground surface settlement due to excavation in front of walls embedded in coarse-grained soils.

Figure 2-7 – Ground surface settlement due to excavation in front of wall in sand (ref: CIRIA 760 – Figure 6.16)



Based on the data presented on the figure above and an approximate shaft depth of 23.3m (note that the diaphragm wall extends 3m beyond the shaft base slab soffit level), the ground surface settlements are estimated:

0.3% x 23300= 70mm behind the shaft wall

0.15% x 23300= 35mm at a distance of x=23.3/2= 11.65m from the shaft wall

0.05% x 23300= 12mm at a distance of x=23.3m from the shaft wall

0 at a distance of x= 2x23.3= 46.6m from the shaft wall

In conclusion, the installation of the diaphragm wall and excavation of the shaft are not envisaged to affect the sea defence works.

3 METHODOLOGY

3.1 Introduction

The settlement assessment has considered two approaches using XDisp as an analytical method and Plaxis 2D as a numerical method. These software analyse the greenfield ground movement in terms of settlement and horizontal displacement. The initial analytical assessment considers elastic behaviour and leaves gaps to address the variability of ground material along the alignment, as such, Plaxis 2D has been used to consider stress redistribution during tunnelling construction and provide an additional perspective on the assessment.

The ground movement caused by the excavation of a bored tunnel is obtained from the tunnel displacement corresponding to a percentage volume loss (VL). The following assessment considers 1%, 2% and 3% VL to address typical operational range of volume loss for a slurry TBM. A volume loss of 2% has been considered as the baseline for tunnel excavation – this is based on industry experience of typically achievable values for TBMs and is further supported by Vu (2016) [8] stating that a maximum volume loss of less than 1.5% can be expected when the cover-to-diameter C/D ratio is greater than 1, as is the case for the Aramis tunnel. Thus 2% is considered a reasonable upper bound for purpose of assessment, and 3% has been considered in order to test the sensitivity of the system to higher volume losses. The parameters adopted are considered appropriate for tunnelling in soft ground and are relatively conservative when compared with recorded parameters for similar tunnel projects. As an additional check, for the readers information only, further extreme scenarios are presented briefly in Section 4.4, to provide indicative settlements corresponding to various tunnel failure scenarios (e.g. 100% VL).

3.2 XDisp

Oasys XDisp is an asset damage assessment analysis software. It is used to estimate three-dimensional displacements caused by different geotechnical engineering activities and their potential impact on nearby structures. The software uses well established empirical methods to provide analytical solutions. These methods are based on the understanding whereby ground movements associated with tunnel excavation commonly resemble an inverted normal Gaussian distribution curve.

The analysis methodology proposed by O'Reilly & New (1982) [9] was adopted to determine vertical settlement and horizontal displacements at the sea defence, which has been considered as a surface structure based on Figure 2-3.

The methodology considers a greenfield ground movement profile and determines horizontal and vertical ground settlements due to the proposed tunnel excavation. Further data processes allow for generation of maximum tensile strain and angular distortion values.

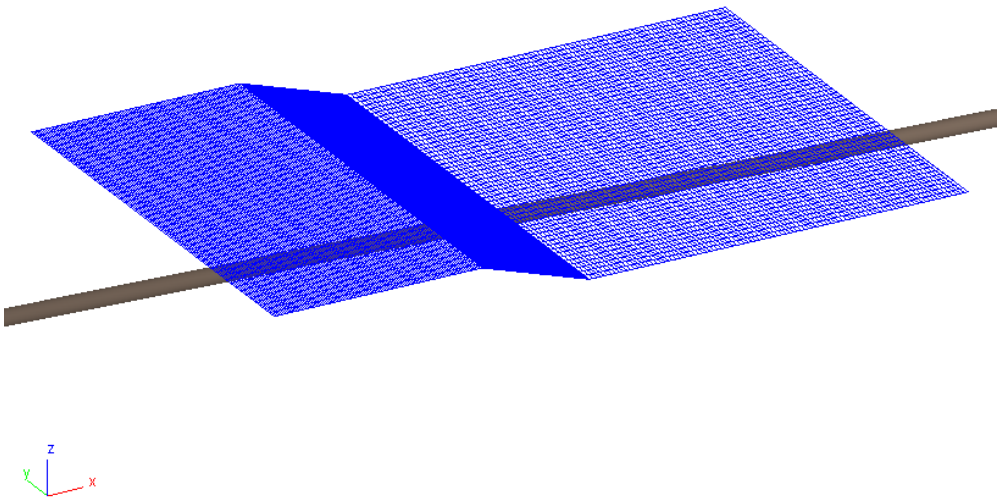
3.2.1 Model assumptions and properties

The XDisp model utilises a “wished into place” construction resulting in maximum values to be calculated and does not consider phases of construction. Furthermore, a limitation of the analysis is the basis of greenfield calculations whereby displacements are calculated solely on the modelled excavation, independent of influence by other existing structures.

The trough width parameter, K, has been calculated within the software using relative distance between the tunnel axis and the location of the analysis. This K parameter is an input dependent on ground conditions, and in this instance is based on the equations presented by O’Reilly and New [9] under the assumed condition of granular soil.

3.2.2 Critical Design Sections

Figure 3-1- XDisp model of the sea defence



To provide an overview of the predicted settlement across the structure, the XDisp model has assessed nine sections across the sea defence profile, spanning 50m either side of the tunnel line, in addition to two longitudinal assessments taken along the base and surface of the sea defence. Noted the geometry of the sea defence has been simplified to consider key locations with the assumption the section remains consistent along the coast 50m either side of the tunnel as suggested by satellite imagery. These simplifications suffice to cover the zone of influence of the tunnel excavation. The mesh shown in Figure 3-1 illustrates the region of assessment in 2D space. Further details of the analysis locations outlined in Figure 3-2 and Figure 3-3. The relative depths of locations are presented in Table 3-1.

Figure 3-2 - Plan view of assessed sea defence locations

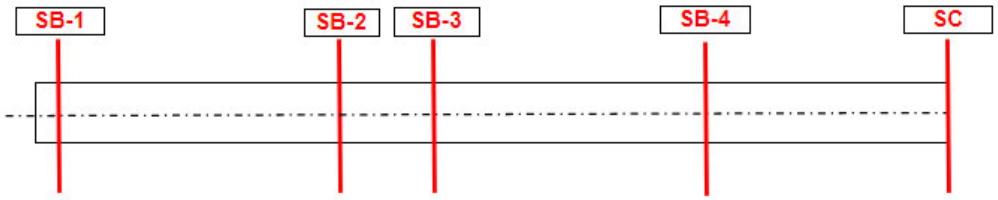


Figure 3-3 - Cross section view of assessed sea defence locations

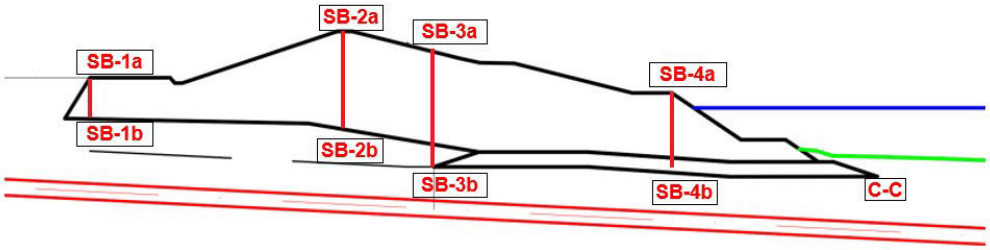


Table 3-1 - Table of relative depths of assessed points

Location	Location Elevation (mLat)	Tunnel Axis (mLat)	Distance from Tunnel Axis [m]
SB-1a	6.1	-17.0	23.1
SB-1b	-2.3	-17.0	14.7
SB-2a	15.5	-19.5	35.0
SB-2b	-4.1	-19.5	15.4
SB-3a	11.4	-20.6	32.0
SB-3b	-12.0	-20.6	8.6
SB-4a	3.0	-22.9	25.9
SB-4b	-13.2	-22.9	9.7
S C-C	-13.9	-25.0	11.1

3.3 Plaxis 2D

Plaxis 2D program is a finite element (FE) software that enables numerical analysis to examine the nonlinear interaction between the tunnel, surrounding soils and the related elasto-plastic behaviour. The methodology provides an approximate solution using a range of different soil parameters to analyse their individual effects on the settlement. Compared to analytical method, this methodology differs from the empirically derived approximations of Xdisp. Comparing the output of Plaxis 2D with those obtained from XDisp enables a better understanding of the potential settlement behaviour due to the ground conditions at the location of the Sea defence.

3.3.1 Model assumptions and properties

A 2D Plaxis model comprising the tunnel and the sea defence at Section B (Figure 3-4), which corresponds to Section SB-2a-2b (Figure 3-3) was set up to estimate the settlements due to the construction of the tunnel (Figure 3-5). In this plane strain model, the strains can take place only in the x-y plane.

The soil models, Lower and Upper bounds, produced using the soil parameters in Section 2.3 are shown in Figure 3-6 and Figure 3-7. NLD clay and Peat has been modelled using Mohr-Coulomb model, while the rest of the soil strata has been modelled using HS-Small model that incorporates strain-hardening and small-strain stiffness effects. As the NLD clay lies less than 1.0x diameter away from the tunnel, it is considered necessary to consider short term vs long term behaviour and is modelled as Undrained B material. For soil-structure interaction the interface value of R_{inter} is assumed to be 0.67.

The sea defence is modelled as a Made Ground (MG) soil layer with properties given in Table 2-2 and Table 2-3 for LM and UB model respectively. In the absence of the information about the sand core of the sea defence, the properties of the made ground material is taken from a nearby borehole.

The possible construction staging has been simulated in Plaxis 2D with 1 to 3 % volume loss for tunnel construction, this FE model also take into consideration the possible construction staging for sea defence, in order to stimulate a more accurate stress regime of the site. In addition, 250mm segmental lining is assumed for the tunnel with the properties given in Table 3-2.

2D analysis was considered appropriate at FEED stage based on the linear nature of the tunnel and associated settlements along its length.

Figure 3-4 - Section B for Plaxis 2D model.

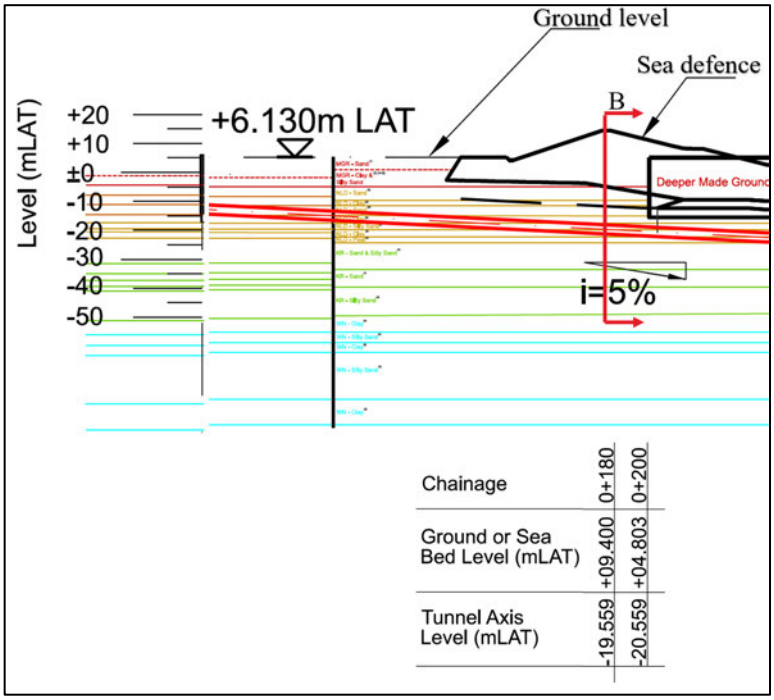


Table 3-2 - Plaxis 2D plate properties of segmental lining (assumed concrete Class is C50/60 and unit weight of the concrete is 24kN/m^3).

Equivalent Unit weight	Section area	Second moment of area	Equivalent depth of lining	Elastic modulus	Stiffness	
w	A1	I	d		EA1	EI
kN/m/m	m^2/m	m^4/m	m	MPa	kN/m	kNm^2/m
4.8	0.1983347	0.00065015	0.198	37278	7393519.7	24236.35

Figure 3-5 - Plaxis 2D model with generated mesh

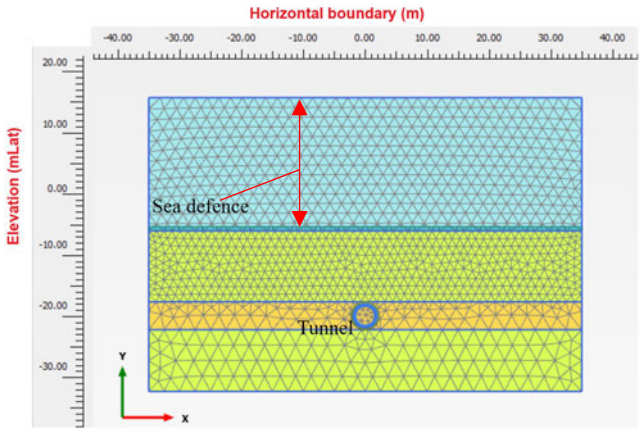


Figure 3-6 - Plaxis 2D soil model at Section B (LM)

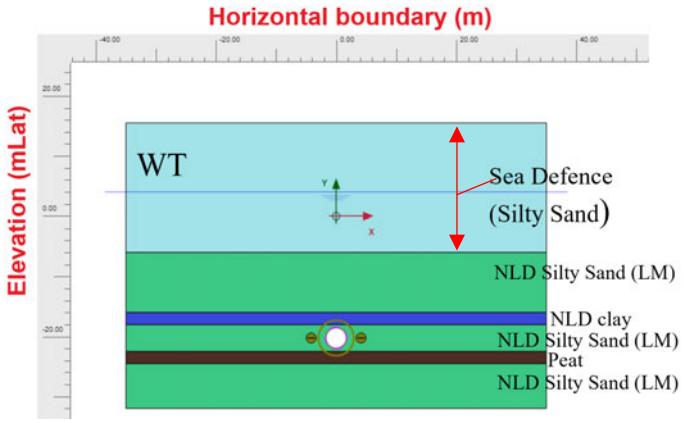
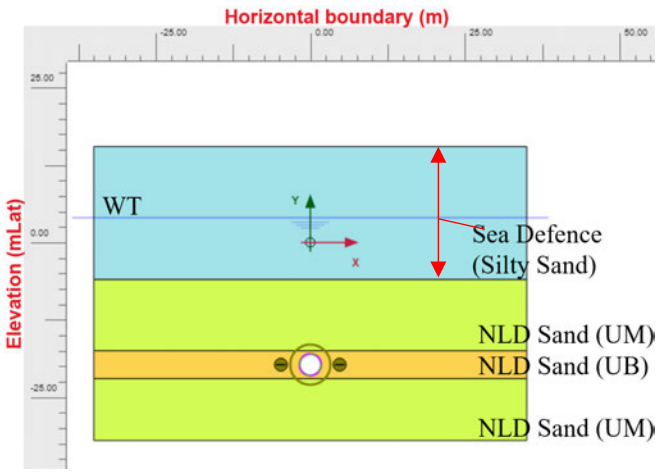


Figure 3-7 – Plaxis 2D soil model at Section B (UB)



4 SETTLEMENT ANALYSIS RESULTS

4.1 XDisp

4.1.1 Summary of analysed sections

This section summarises the results of the analysis in tabulated form. These locations have been assessed to provide insight into the behaviour of the structure as a single entity considering points along the top and bottom of the structure as a form of upper and lower bound in the absence of confirmed material parameters.

The analysis results for the considered sections illustrated in Figure 3-2 and Figure 3-3 cover the following criteria:

- Vertical displacement
- Horizontal displacement (perpendicular to tunnel axis)
- Angular distortion
- Maximum tensile strain

The results for volume loss 1%, 2% and 3% are summarised in Table 4-1 to Table 4-3, respectively.

Table 4-1 - Table of XDisp results for 1% Volume Loss

Section	Maximum Vertical displacement [mm]	Maximum Horizontal displacement [mm]	Angular distortion	Maximum tensile strain [%]
SB-1a	7	1	1/3286	0.03
SB-1b	11	2	1/2000	0.05
SB-2a	5	1	1/6800	0.01
SB-2b	11	2	1/1455	0.04
SB-3a	5	1	1/6300	0.02
SB-3b	19	3	1/1208	0.14
SB-4a	6	1	1/4333	0.02
SB-4b	17	3	1/618	0.10
S C-C	15	2	1/767	N/A
Longitudinal Section				
Base	20	-	1/5200	0.19
Surface	15	-	1/2500	0.07

Table 4-2 - Table of XDisp results for 2% Volume Loss

Section	Maximum Vertical displacement [mm]	Maximum Horizontal displacement [mm]	Angular distortion	Maximum tensile strain [%]
SB-1a	14	2	1/799	0.05
SB-1b	22	4	1/727	0.10
SB-2a	9	2	1/3846	0.02
SB-2b	21	3	1/782	0.07
SB-3a	10	2	1/3300	0.03
SB-3b	39	6	1/244	0.27
SB-4a	12	2	1/2177	0.05
SB-4b	34	5	1/325	0.20
S C-C	30	5	1/400	N/A
Longitudinal Section				
Base	39	-	1/1316	0.38
Surface	30	-	1/2471	0.15

Table 4-3 - Table of XDisp results for 3% Volume Loss

Section	Maximum Vertical displacement [mm]	Maximum Horizontal displacement [mm]	Angular distortion	Maximum tensile strain [%]
SB-1a	21	3	1/1190	0.07
SB-1b	33	5	1/500	0.15
SB-2a	14	2	1/2643	0.03
SB-2b	32	5	1/516	0.11
SB-3a	15	3	1/2267	0.05
SB-3b	58	9	1/164	0.41
SB-4a	19	3	1/1556	0.07
SB-4b	51	8	1/216	0.30
S C-C	44	7	1/284	N/A
Longitudinal Section				
Base	58	-	1/926	0.57
Surface	44	-	1/1680	0.23

4.1.2 Critical Design Cases

Due to having the closest absolute distance from the tunnel, section SB-3a has been identified as the critical assessment location. This location yields the greatest theoretical values compared to other considered sections due to being the closest location to the tunnel excavation. Additionally, Section SB-3b has been included to provide comparison and insight between the top and bottom of the structure.

The calculated values of vertical displacement at section SB-3a and SB-3b for VL = 1%, 2% and 3% are presented in Figure 4-1 and Figure 4-2. The graphs of vertical displacement for the remaining sections are available in Appendix A.

Figure 4-1 - Vertical Displacement at Section SB-3a

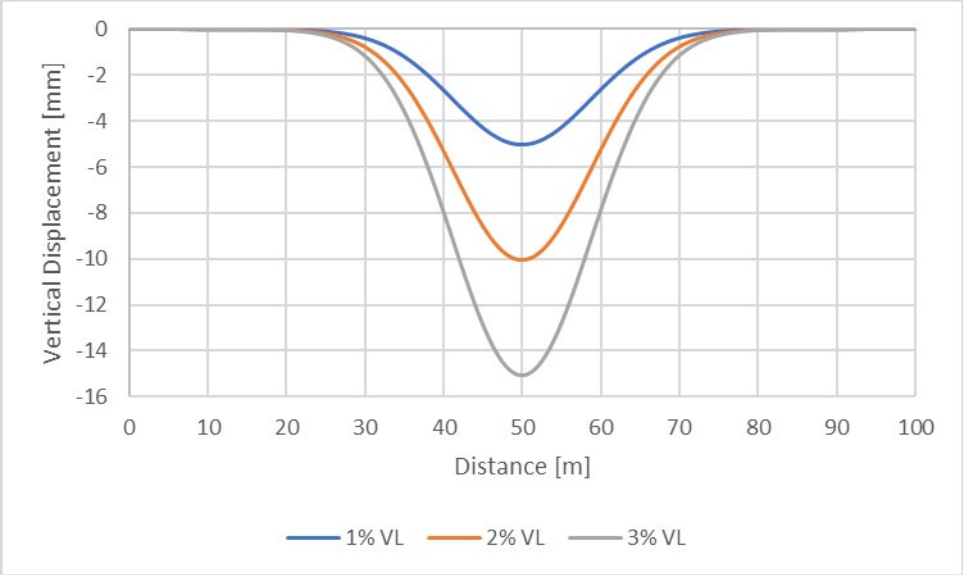


Figure 4-1 shows the maximum vertical displacement considering volume loss range from 1% to 3%. The results indicate vertical displacement increases proportionally with increase in volume loss. Maximum displacement values are 5mm for 1% VL; 10mm for 2% VL; 15mm for 3% VL.

Figure 4-2 - Vertical Displacement at Section SB-3b

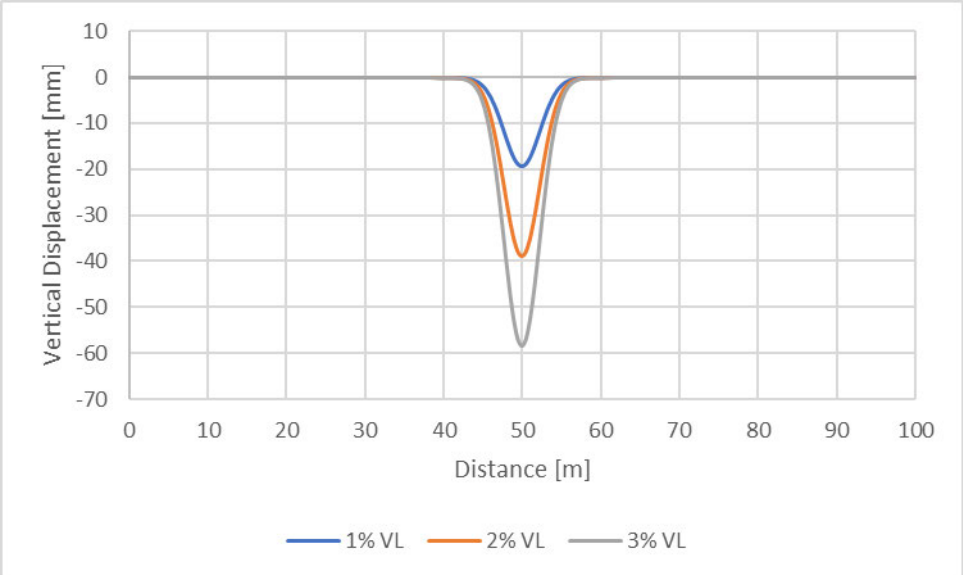


Figure 4-2 shows the maximum vertical displacement considering volume loss range from 1% to 3%. The results indicate vertical displacement increases proportionally with increase in volume loss. Maximum displacement values are 19mm for 1% VL; 39mm for 2% VL; 58mm for 3% VL.

4.1.3 Longitudinal sections

The vertical displacements along the surface and base of the sea defence have been modelled to provide a higher resolution depiction of displacements from a longitudinal perspective across the structure. The lines of assessment are depicted in Figure 4-3 and Figure 4-5, with results shown in Figure 4-4 and Figure 4-6, respectively.

The vertical settlement across 1%,2% and 3% volume loss has been captured and illustrated in Figure 4-4 and Figure 4-6 below. The charts demonstrate an increase in volume loss leads to increase in displacement.

Surface of Sea Defence

Along the surface of the sea defence for 2% volume loss, the maximum vertical displacement is 30mm occurring at the toe of the sea defence at ch160 and the minimum vertical displacement is 9mm at the surface of the sea defence where the distance from the tunnel is greatest at ch50.

Figure 4-3 - Polyline of Surface of Sea Defence

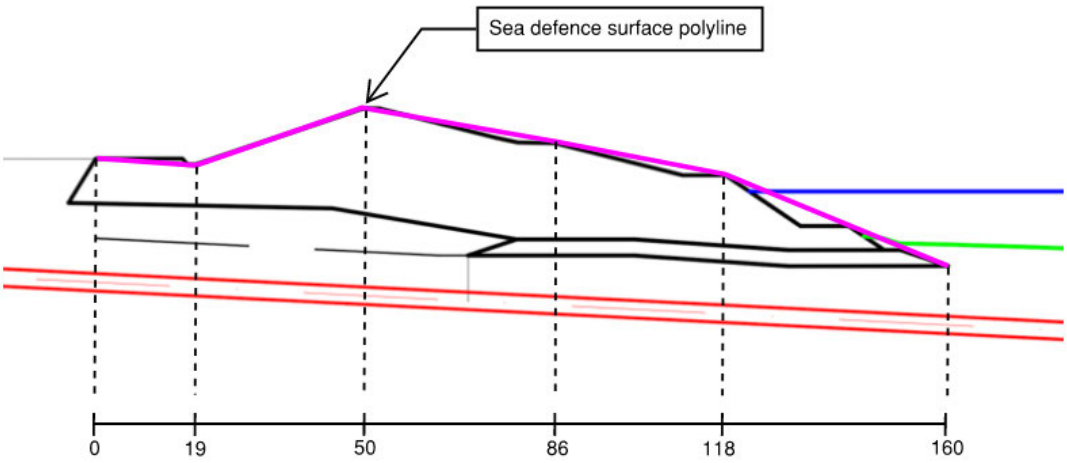
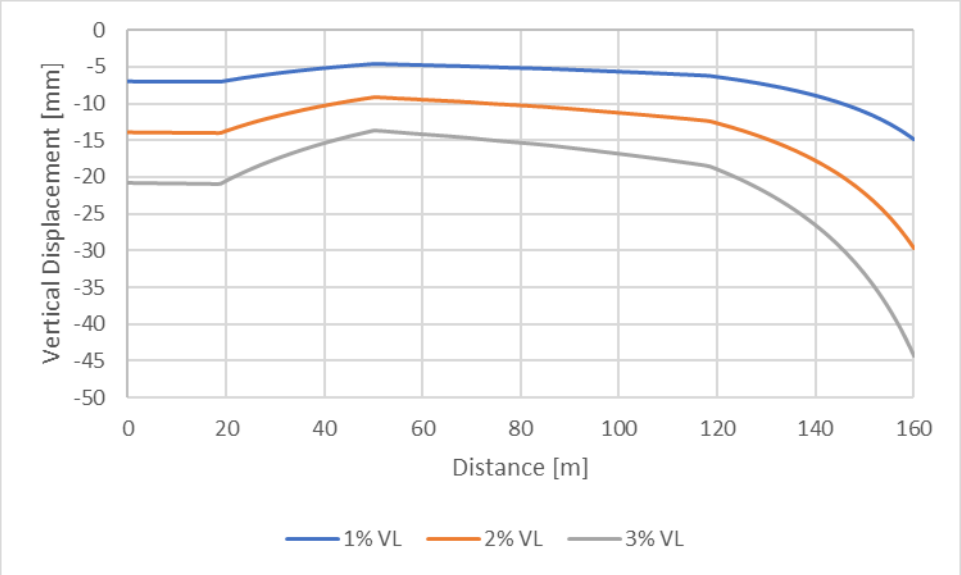


Figure 4-3 illustrates the polyline used to model the surface of the sea defence structure. The local chainage of 50m indicates the highest point with greatest distance from the tunnel. The location with the shallowest distance from the tunnel is at local chainage 160m.

Figure 4-4 - Vertical Displacement along the surface of Sea Defence



In relation to Figure 4-3, the vertical displacements shown in Figure 4-4 display consistency with the distance between the analysis location and the tunnel. Across the first 19m, the vertical displacement remains consistent. Between 19m to 50m, the vertical displacement reduces as the distance from the tunnel increases to the peak (of the sea defence). At 50m, the vertical displacements are 5mm for 1%VL, 9mm for 2%VL, and 14mm for 3%VL. As the distance between the surface and tunnel decreases, the vertical displacements can be seen to increase to the maximum at 160m, at the toe. At 160m, the vertical displacements are 15mm for 1%VL, 30mm for 2%VL, and 44mm for 3%VL.

Bottom of Sea Defence

Along the bottom of the sea defence for 2% volume loss, the maximum vertical displacement is 39mm where the distance to the tunnel is the least. The minimum vertical displacement is 20mm occurring where there is the greatest distance from the tunnel.

Figure 4-5 - Polyline of bottom of Sea Defence

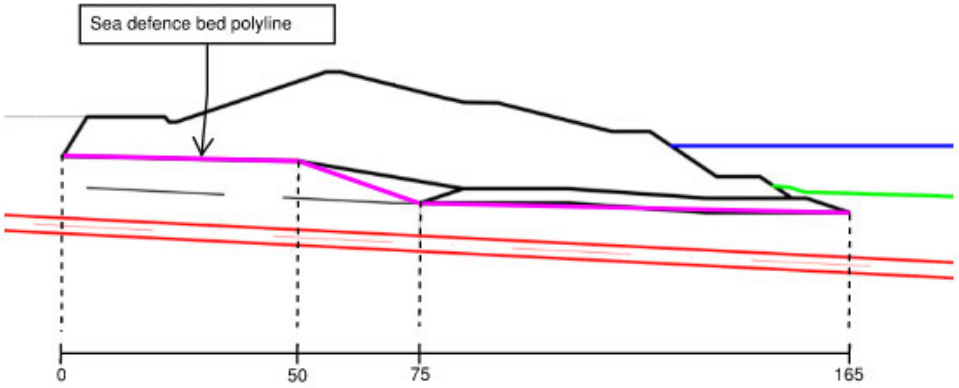


Figure 4-5 illustrates the polyline used to model the bottom of the sea defence structure. Between 0m to 50m, the distance from the tunnel gradually increases. From 50m to 75m, the distance from the tunnel decreases significantly to the closest point.

Figure 4-6 - Vertical Displacement along the bottom of Sea Defence

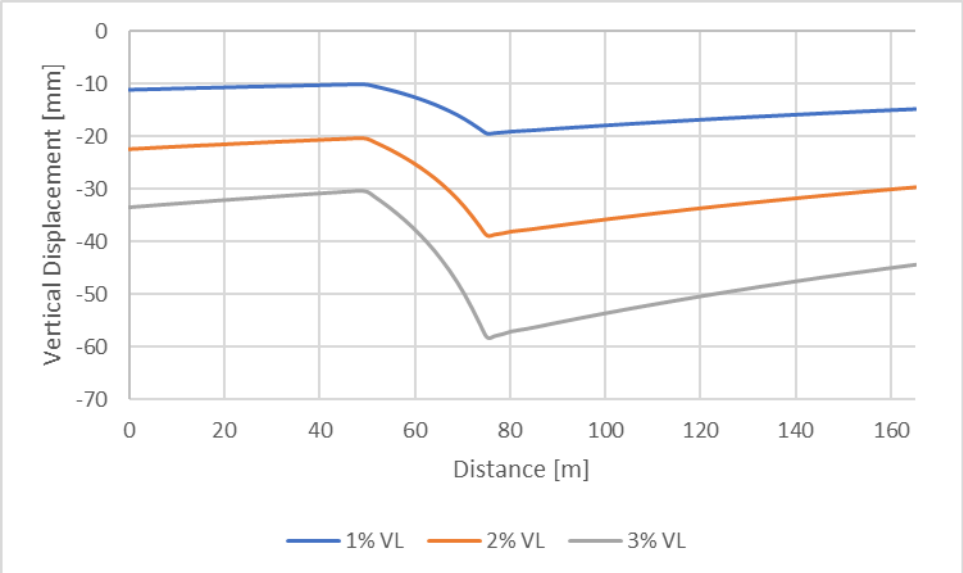
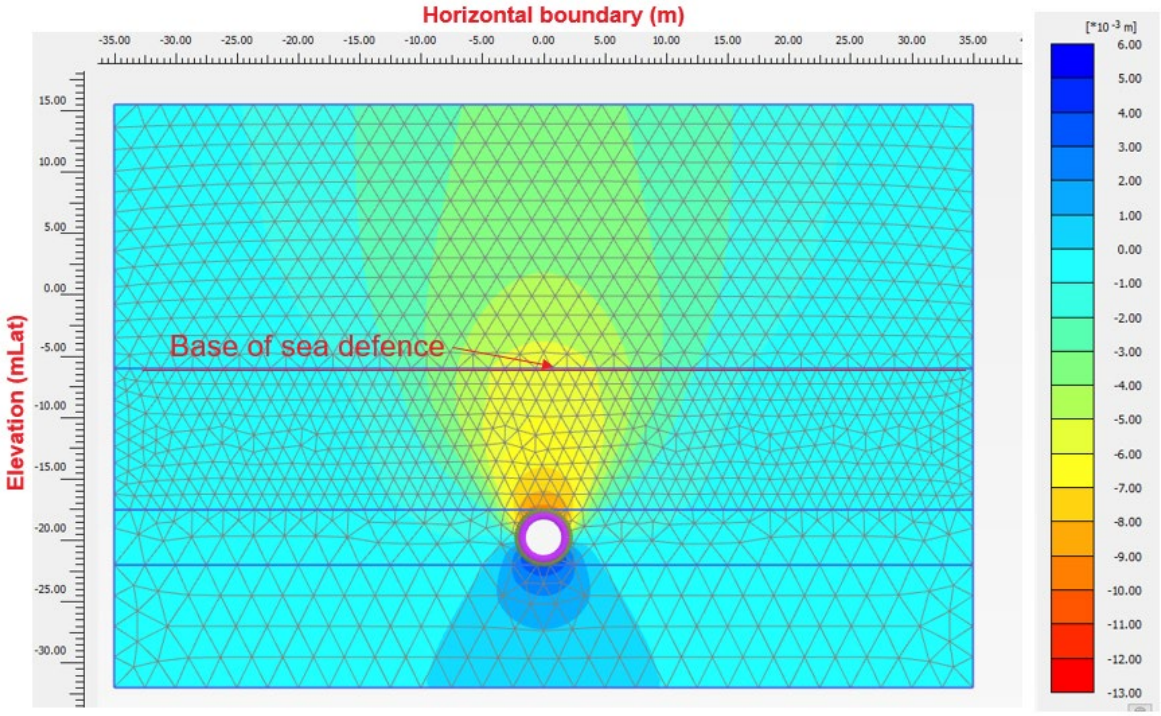


Figure 4-5 illustrates the polyline used to model the bottom of the sea defence structure. The vertical displacements shown in Figure 4-6 displays consistency with the distance between the analysis location and the tunnel. Across the first 50m, the vertical displacement remains consistent. Between 0m to 50m, the vertical displacement reduces as the distance from the tunnel increases. The location with the greatest distance from the tunnel is at local chainage 50m. At 50m, the vertical displacements are 10mm for 1%VL, 20mm for 2%VL, and 30mm for 3%VL. The local chainage of 75m indicates the lowest point with shallowest distance from the tunnel where the vertical displacement is at its maximum. At 75m, the vertical displacements are 19mm for 1%VL, 39mm for 2%VL, and 58mm for 3%VL.

4.2 Plaxis 2D

The settlement contours at the section resulting from the tunnel excavation for volume loss 1% is shown in Figure 4-7. The zone of influence of tunnel excavation is 20m either side of the tunnel and the settlement at the base of the sea defence is 6mm.

Figure 4-7 - Vertical settlement from tunnel construction for volume loss 1% (UB model).



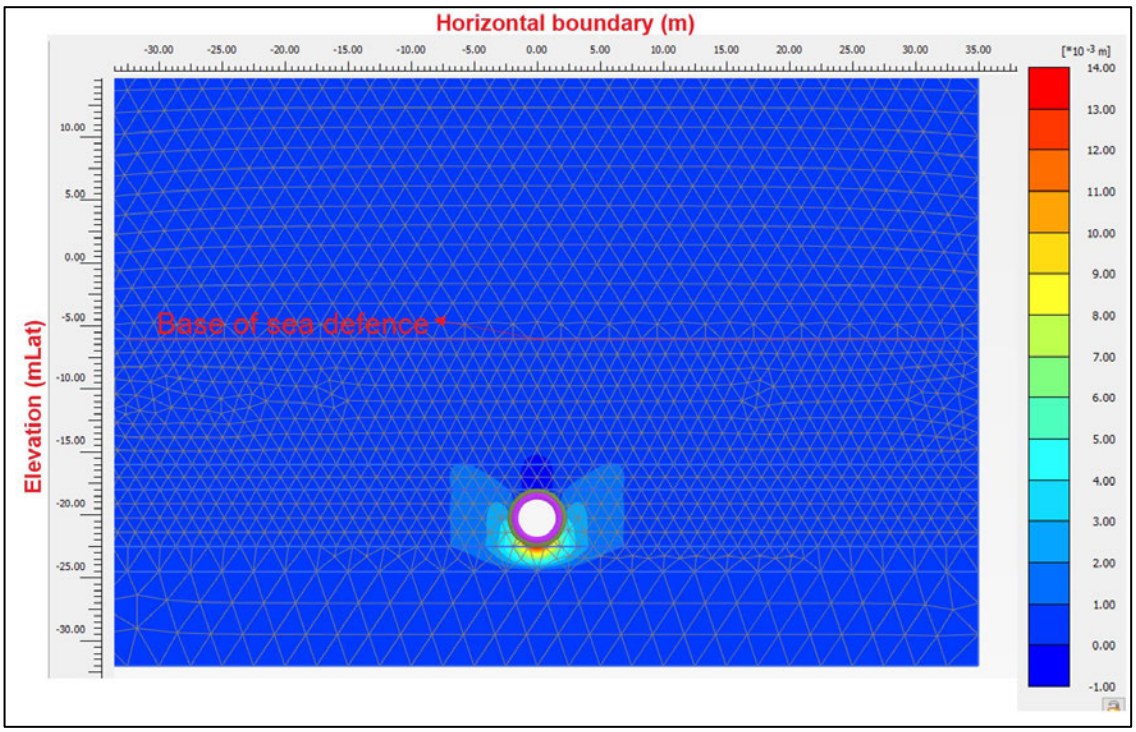
A further sensitivity analysis with volume loss 2% and 3% was performed to analyse worst case scenario, which is presented in Table 4-4 with the corresponding displacements for Volume loss 1%, 2% and 3%.

Table 4-4 - Vertical settlement at the base of the sea defence from Plaxis 2D analysis at Section B-2b

Volume Loss (%)	Maximum Vertical displacement [mm]
1	6
2	10
3	14

In addition, the lower bound soil profile was analysed to observe the sensitivity of the settlement (Figure 4-8). In the lower bound model, there is a positive (upward) vertical movement of 14mm at the tunnel invert located in the silty sand layer, which is partially due to numerical modelling and partially due to the high-water pressure induced uplift of the tunnel. The uplift check will be undertaken as part of FEED design. In contrast, there is no volumetric change in the NLD clay above due to its undrained behaviour, causing minimal displacement at the crown of the tunnel and thus, small change in displacement at the base of the sea defence. This indicates that during the tunnel construction the settlement can be anticipated to be minimal due to the undrained behaviour the clay above the tunnel. However, in the long-term behaviour of the clay, where the dissipation of excess pore pressure occurs in the clay layer over time, settlement would occur gradually. With all excess pore pressure dissipated, the maximum settlement at the base of the sea defence is shown to be about 14mm. With the design life of 30 years, the excess pore pressure dissipation in the NLD clay may not have been completed. Therefore, it should be noted that the asset owner should also consider the gradual dissipation of the excess pore pressure, i.e. gradual increase in deformation over the time.

Figure 4-8 - Vertical settlement from tunnel construction for volume loss 1% (LM model).



4.3 Discussion of results

The ground settlement has been calculated using XDisp and Plaxis 2D software to capture the range of possible settlement of sea defence due to tunnelling.

From the analysis of XDisp results of the settlement for up to 2% volume loss, the settlement remains similar to the 30mm referenced in the Basis of Design [1], except at locations where the tunnel and the sea defence are in close proximity such as sections SB-3b (39mm), SB-4b (34mm), and CC (30mm). It should be noted that 2% volume loss is a moderate value in the context of current-day TBM performance, with many able to operate at lower values. Even though the analyses have considered the 3% volume loss, its occurrence in practice, is highly unlikely.

In addition to the XDisp analytical method, the Plaxis 2D numerical analysis method was also performed, which considers a range of soil parameters to analyse their individual effects on the settlement. The settlement obtained from Plaxis 2D analysis is lower than the settlement obtained from XDisp, as expected (see Table 4-5). This is due to the method of calculation that takes into account the stress redistribution around the tunnel excavation in Plaxis 2D, which better captures the deformation and stress distribution around the excavation. In addition, possible construction staging has been simulated in Plaxis 2D with 1 to 3 % volume loss, including the build-up of the sea defence. The construction history of the sea defence at the site allows better modelling of the ground stresses. Furthermore, it was observed that where a thin and soft NLD clay layer is present above the tunnel, there is minimal settlement at the base of the sea defence.

Overall, the settlement induced from the tunnel excavation, between 11mm to 32mm from XDsip and 6mm to 14mm from Plaxis 2D, should not impact the structure, however, potential crack propagation from existing cracks could occur. This requires a further understanding of the design of the sand core and the effect of cracks within the structure.

Without the allowable deformation limit, the early onset of cracking which occurs in concrete at a strain of about 0.03 up to 0.05% has been considered as a benchmark for the performance of sea defence. Here, a corresponding crack width of up to 0.1mm would be expected. For volume loss up to 1%, the strain at the base of sea defence is about at the strain where early onset of crack could occur. In addition, assessment of seasonal effects, e.g storm season, on crack propagation, and pre and post condition survey should also be considered in the next phase of the design.

Table 4-5 - Xdisp & Plaxis comparison summary

	Vertical Settlement (mm)		
Volume Loss	1%	2%	3%
Xdisp (SB-2B)	11	21	32
Plaxis (SB-2B)	6	10	14

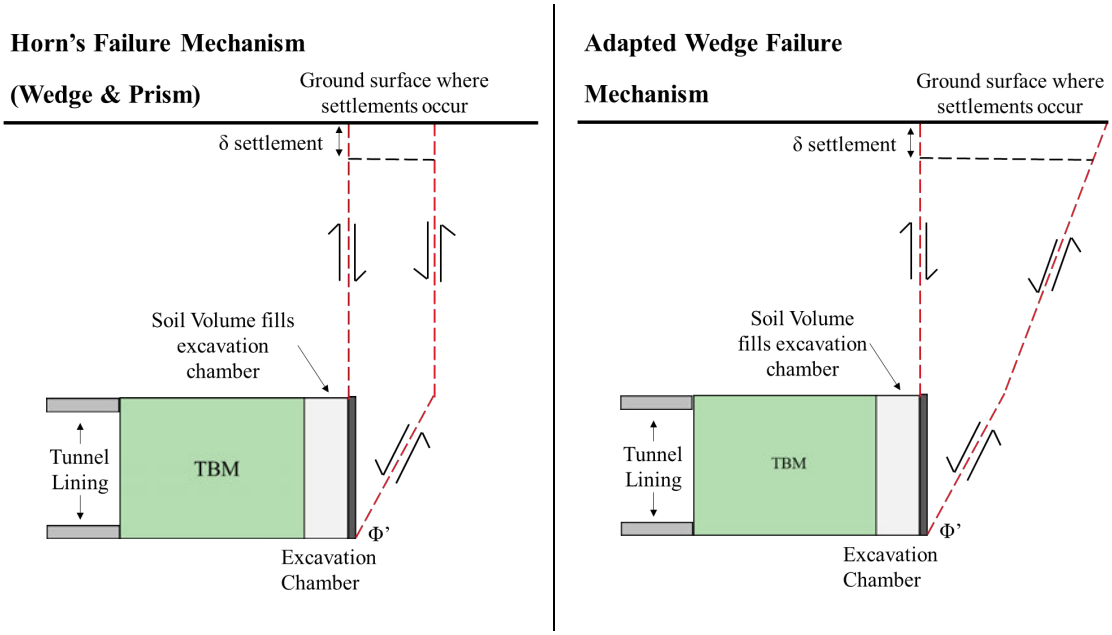
4.4 Extreme Failure Cases

For the reader's information, an additional set of assessments were completed to review the potential impact of tunnel face collapse or complete tunnel collapse on settlement at the sea defence. These are not typical design cases but are provided for information to inform the reader's understanding of a potential worst-case scenario in which the tunnel collapses in the vicinity of the sea defence, to allow further risk assessment and risk evaluation if required. It is noted that many control measures will be in place to control the process of tunnelling to reduce the likelihood of this happening, which is considered to be very low. This includes definition of allowable operating pressure ranges derived in order to maintain face stability.

The estimate of settlement that may occur in the face collapse scenario was undertaken with reference to the typical method used to determine face pressure requirements (Horn's mechanism [10]), with additional sensitivity testing on the angle of ground failure (allowing for an increased "spread" of the failure), see Figure 4-9. It is noted that a more concentrated area (as considered in plan view) leads to a higher calculated settlement above the tunnel for the same volume of material lost, thus the scenario on the left of Figure 4-9 (rectangular prism) leads to a higher calculated settlement that the scenario on the right (adapted wedge failure allowing for spread).

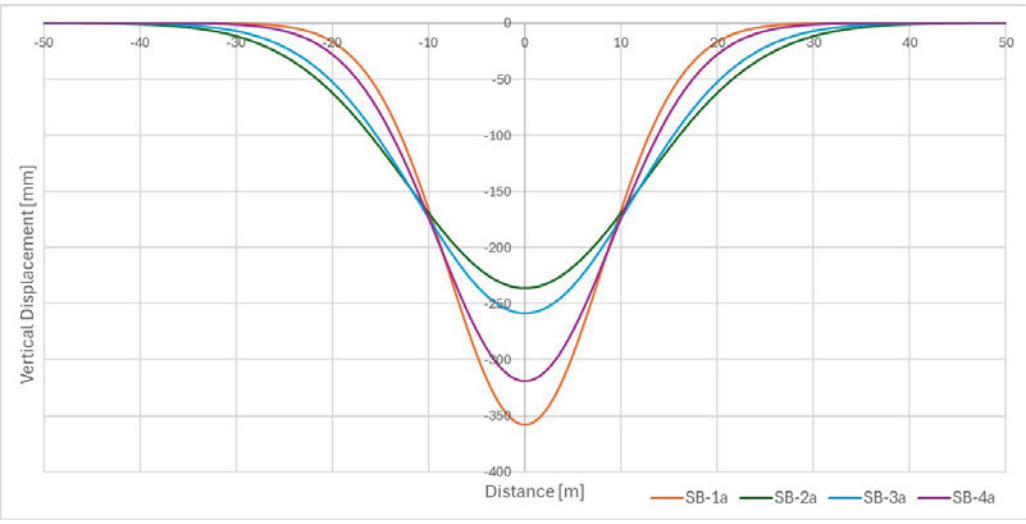
Location SB-3b was considered to provide a relatively conservative result, being the shallowest point considered (base of sea defence) relative to the tunnel. At deeper locations, soil arching behaviour is likely to develop, reducing surface settlements, as any spread of the failure allows for the volume lost to be distributed over a larger area (reducing the average vertical settlement).

Figure 4-9 - Mechanisms Assessed for Face Collapse
(L) Failure Mechanism based on Horn [10]; (R) Adapted Mechanism for sensitivity check



To assess the potential impact of a full tunnel collapse, settlements at the top of the seawall were evaluated (see Figure 4-10) in line with O'Reilly and New [9]. A volume loss of 62% was used in this assessment, calculated by deducting the cross-sectional area of concrete from the overall cross-sectional area of the tunnel, thus assuming that under collapse, the concrete parts of the tunnel partially fill the tunnel void and that the remaining volume is filled by surrounding soil.

Figure 4-10- Extreme tunnel failure case vertical displacement (62% VL)



A summary of the settlements produced as a result of these extreme failure cases is presented in Table 4-6 below.

Table 4-6 - Extreme failure case displacements

Extreme Case ID	Maximum Settlement [mm]
Face Collapse – SB-3b	245
Adapted Face Collapse – SB-3b	90
Tunnel Failure – SB-1a	358
Tunnel Failure – SB-2a	236
Tunnel Failure – SB-3a	258
Tunnel Failure – SB-4a	319

In these extreme cases, vertical displacements are larger than for the typical 1% to 3% VL cases with up to ~360mm settlement in the worst case tunnel failure considered. This is noted to be significantly less that the full diameter of the tunnel (c.3.8m outer diameter accounting for over-excavation). Discussion on the TBM control measures that may provide confidence in tunnelling operations in order to reduce likelihood of these extreme scenarios occurring are included in Section 5.

For information, the magnitude of settlement (up to c.360mm) in these scenarios remains small in relation to the height of the sea defence which has levels of c.9mLAT at top of concrete blocks and c.16mLAT at top of bund. Thus the sea defence, after settling, would remain significantly above the estimate 40-year tide level maximum (4.5mLAT) described in Section 2.2.

In the permanent case it is proposed the tunnel is backfilled, thus these extreme scenarios would only be relevant to construction phase. Note if these scenarios are considered further at detailed design, 3D analysis (allowing for more detailed analysis of 3D settlement shape) could be undertaken by EPC Contractor for further investigation.

5 INSTRUMENTATION AND MONITORING

The following are the proposed instrumentation and monitoring rationale for the RWS sea defence. This methodology is to be confirmed and agreed with the Company in the next stage of design.

Monitoring of assets is to be carried out to:

- Establish baseline condition to assess behaviour of assets pre-construction.
- Control and provide confidence in the design and construction process.
- Verify that both ground and structures are behaving as predicted.
- Identify any unexpected trends, anomalies, and trigger responses.
- Enable post-construction state to be compared with initial baseline measurements if required by design.

5.1 Condition Surveys

It is proposed that pre and post condition surveys are undertaken by the EPC contractor to assess the current condition of the structure and demonstrate that the condition of the asset has not been altered because of tunnelling, excavation or other construction works, or to determine extent of changes that have occurred because of such works.

Pre-construction condition surveys

Pre-construction condition surveys: It is assumed that a photogrammetry survey and or/LIDAR survey will be made available pre-construction to capture the current visual exterior condition of the asset.

The pre-construction survey would also be used to assess the baseline condition. A typical period of 12 months is recommended to capture seasonal effects and allow for better understanding of monitoring data.

Post-construction condition surveys

Post-construction condition surveys: Use of automated capture is recommended for monitoring of ground movements. It is recommended that the extent of monitoring will continue until differential settlement of 2mm per day or less is measured.

Use of high-resolution imagery for baseline records will allow for future revisit and recapture of data. Use of an aerial drone may permit a pre-programmed route to be repeated. These condition surveys may be complemented through integration of object recognition to compare photographic data captured in condition surveys.

Table 5-1 - Overview of proposed condition surveys

Condition Survey	Objective	Survey Method	Extent of Monitoring
Pre-Construction	Establish baseline condition to assess behaviour of assets pre-construction. Confirm and refine material and geometry assumptions.	Photogrammetry survey and or/LIDAR Visual inspection	Offset approx. 50m both sides from the centreline of the tunnel alignment.
During Construction	Control and provide confidence in the design and construction process. Verify that both ground and structures are behaving as predicted. Identify any unexpected trends, anomalies, and trigger responses.	Refer to Section 5.3	
Post-Construction	Enable post-construction state to be compared with initial baseline measurements if required.	Photogrammetry survey and or/LIDAR Visual inspection	Offset approx. 50m both sides from the centreline of the tunnel alignment.
* The cessation of monitoring will be subject to agreement with the third party and design in the next stage of design.			

5.2 Protection measures and diversion

Protection measures are to be discussed with Company and involved parties.

Details and methodology of protective measures are to be confirmed by TBM operator. This should be adjusted based on the actual ground conditions. Particular attention during tunnelling under the sea defence should be given to monitoring of pipe or ring deformations etc as required within the below referenced specifications, as any deformations to the tunnel itself may provide indication of settlement behaviour around the tunnel.

The following list of protection measures is indicative and not exhaustive:

- Control and monitoring of TBM operation parameters as per TBM Specification [11] and Tunnelling and Shaft Specification for EPC [12].
- Injection of pressurised lubrication medium in the anulus gap to support tunnel bore to limit volumetric ground loss to 2% or lower as per Tunnelling and Shaft Sinking Specification for EPC [12].
- Monitoring of joint tolerances, to limit probability of ingress under the sea defence.

Control and monitoring techniques are to be further developed by EPC contractor during detailed design.

The tunnel horizontal alignment is considered fixed and therefore diversion methods are not considered. Should significant settlement be observed during tunnelling, potential grouting could be considered to prevent progressive settlement

5.3 Construction Stage

The following instrumentation and monitoring are proposed for the construction stage of the tunnel associated with the RWS sea defence. This aims to provide an indication of the effect of construction on the sea defence structure in the context of settlement.

5.3.1 Proposed methods

An outline of the suggested monitoring is available below. The equipment will provide an indication of the behaviour of the sea defence at surface level. The following are suggestions and are to be agreed with Company and third party as necessary.

Table 5-2 - Outline of proposed construction stage monitoring

Instrumentation	Extent of Monitoring	Frequency of Monitoring	Monitored parameters
Precise Levelling Monitoring points	5m c/c along centreline with transects at 10m intervals spanning 50m either side at 10m c/c	Automatic – during TBM excavation within 100m of sea defence.	X, Y, Z at the ground surface

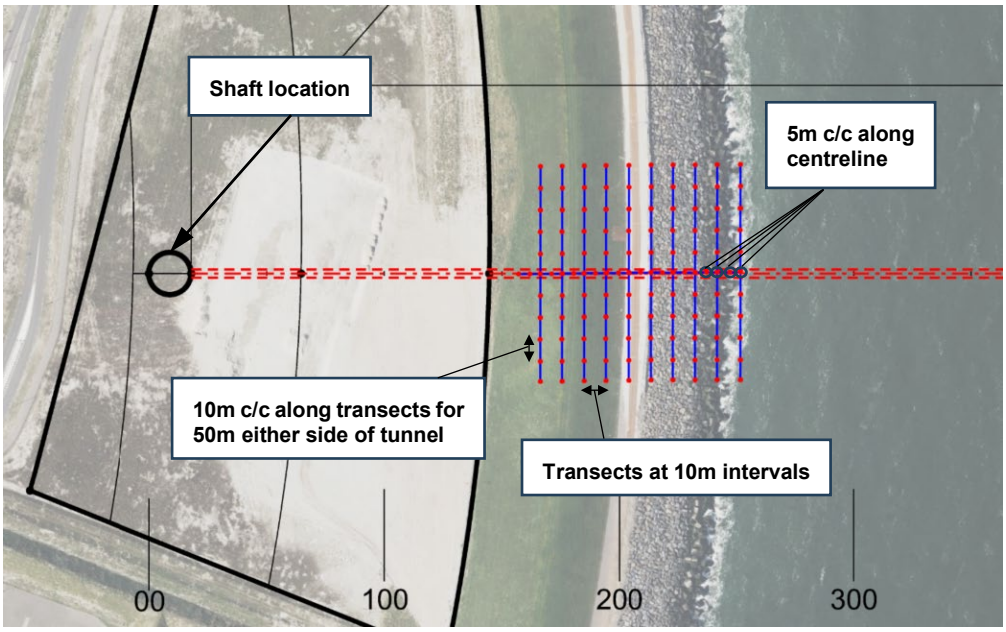


Figure 5-1 - Proposed construction stage monitoring points

5.3.2 Trigger levels, mitigation measures and monitoring frequency

Based on Section 4, comparing the numerical and analytical methods, the following trigger levels have been proposed. These trigger levels use the values generated by XDisp for 2% volume loss providing a comparative upper bound when compared to the Plaxis 2D calculated values of Plaxis 2D. Respective actions associated with the trigger levels have been provided. The use of automated monitoring points allows for adjustment of monitoring frequency dependent on context i.e. increase of reading frequency in the event of abnormal readings. Noted acceptable ground settlements are to be agreed and defined with the asset owner at a later stage.

It is noted that there is potential ongoing settlement of the sea defence of c.3-4mm per year, based on review of satellite data as described in the GDR [3], which should be considered in any comparison of data, and ideally verified in the pre-construction phase.

Table 5-3 - Table of settlement trigger levels for sea defence

Location	Measurement	Trigger values [mm]			Max. calculated displacement [mm]
		Green[1]	Amber[2]	Red[3]	
Sea defence	Vertical displacement	20	27	39	39

1. Green trigger set to 50% of SLS value.
2. Amber trigger set to 70% of SLS value.
3. Red trigger set to SLS value.

Table 5-4 - Table of actions linked to trigger levels

Location	Trigger Level	Observation / measurements	Actions
Sea defence	Green	Measured vertical displacement	- Inform the project manager within 24 hours. - Perform inspection of the surface of the sea defence. - Prepare and submit a written report reviewing all total and differential movements to date. Assessing the effects of the movements on the sea defence and predicting further movements and their effect on the sea defence based on trend to date. - Where it is considered and agreed by the project manager that movement trends indicate that “Amber level” may be reached during the course of works, submit proposal for remedial measures to limit further movement for the approval of the project manager.

Location	Trigger Level	Observation / measurements	Actions
Sea defence	Amber	Measured vertical displacement	- Inform the project manager immediately. - Immediately implement the approved remedial measures in accordance with the contractor's method of construction and temporary works agreed at the "Green level" status. - Work may only proceed if the remedial measures have been implemented and are in the opinion of the project manager to be effective. - Submit an updated report to reviewing the movements including total and differential movements. - Assess the effects on the sea defence and predict further movement and subsequent effects on the sea defence. - Submit revised "Green level" and "Amber level" values which have taken into account the implemented remedial works to project manager for approval before works are resumed.
Sea defence	Red	Measured vertical displacement	- Inform the project manager immediately. - Immediate cessation of work. - Immediately implement requirements for any emergency measures. - Submit a report detailing full history of movements and remedial measures adopted in relation to actual construction sequence. The report shall include interpretation of events with recommendations for enabling work to proceed. - Work may resume upon written instruction of the project manager.

5.4 Operation Stage

Due to the proposed backfill of the tunnel, consideration for operation stage monitoring has been omitted.

6 CONCLUSION

A settlement assessment has been performed to provide an insight into the behaviour of the sea defence associated with the proposed tunnel excavation. This report outlines the expected theoretical ground movements and proposed instrumentation and monitoring relating to the ground settlement.

Limitations in available information from ground investigation and the material properties of the sea defence structure have led to several assumptions being introduced for this assessment.

The integration of an analytical and numerical approach has provided a fuller and more robust perspective of the influence of the tunnel construction on the sea defence by allowing for comparable approaches and account for stress redistribution.

The results from both the analytical and numerical methods indicate that the settlement at the base of the sea defence will be within the assumed 30mm allowable limit with an assumed volume loss of 2% with exception of points with relatively close proximity to the tunnel excavation along the bottom of the sea defence as outlined in section 4.3. It should be noted, a volume loss of less than 2% is common and likely achievable in this context and may provide considerably reduced settlement.

The 1-3% volume loss is used to provide an indication of typical operational performance of a slurry TBM and under beneficial conditions, may perform at even lower volume loss. However, risk management / assessment such as TBM monitoring, surface monitoring, pressurised lubrication medium in the annulus, and controlled excavation should be undertaken to ensure the ground movement is minimised. Mitigation plans should also be made to provide for situations where limits are breached. A proposed instrumentation and monitoring strategy has been discussed within this document.

The assessment indicates potential for hairline cracks (<0.1mm) to materialise on the surface with potential for existing cracks to propagate further. Drawings and satellite imagery indicate remedial works access for clay capping and asphalt concrete area does not have many barriers and may be repaired as necessary.

Key Risks and Failure Mechanisms

Excessive settlements resulting from increased volume loss may introduce cracks at the asphalt surface and generate paths for water flow towards the sand body of the sea defence. It is to be noted, the presence of the clay layer may act as a protective medium.

Instrumentation & Monitoring

Based on the failure mechanisms identified, an instrumentation and monitoring strategy has been proposed to provide prompt warning of excessive settlement and allow for appropriate and timely response.

The instrumentation and monitoring for the sea defence has been proposed and discussed in section 5, it is anticipated that pre-condition and post-condition surveys will provide adequate information to determine the possible impact on the sea defence due to tunnelling. The frequency of monitoring proposed can be adjusted based on actual recorded movement and accommodate different stages of excavation during tunnelling. The use of automated monitoring points will allow for faster response to unexpected ground movements through more frequent monitoring.

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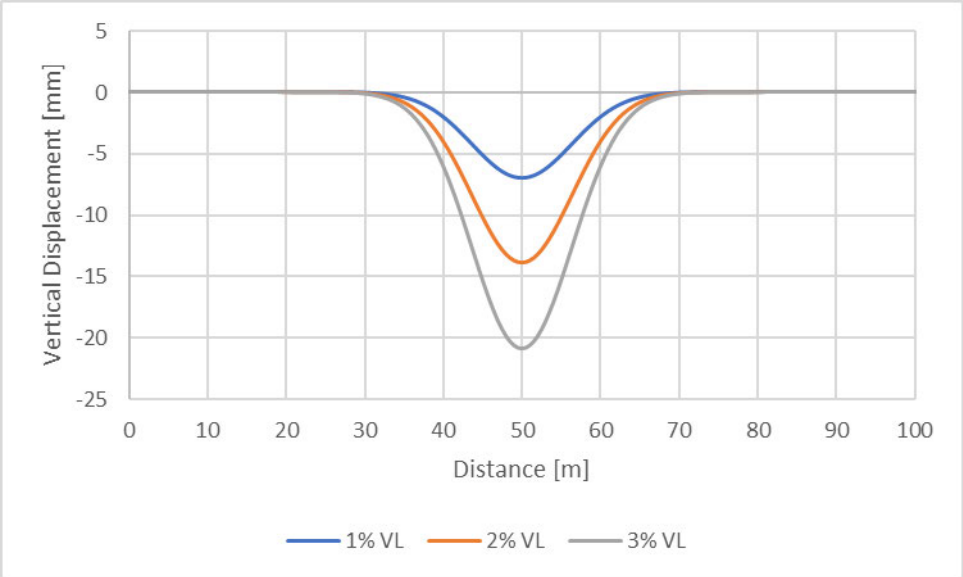
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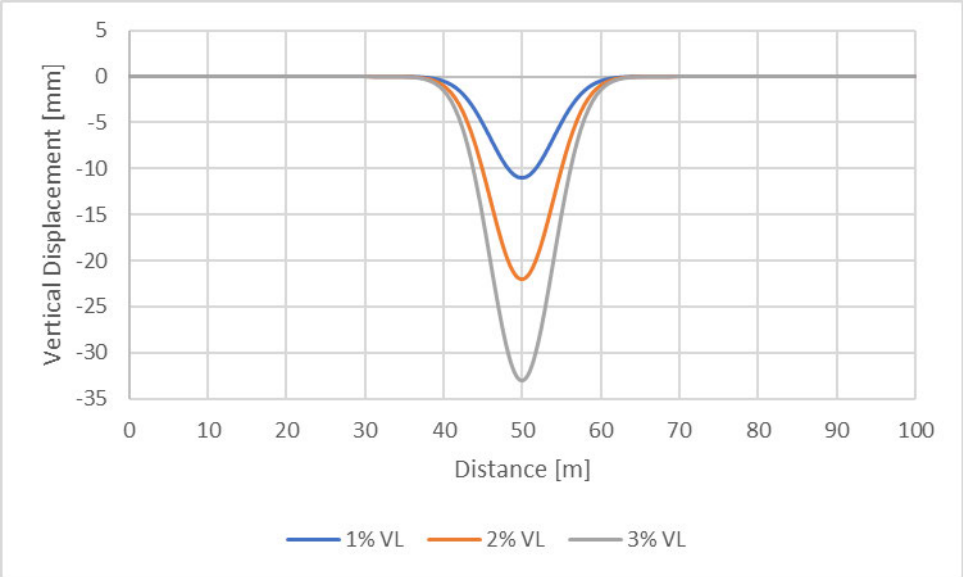
APPENDIX A XDISP RESULTS

A.1 XDisp results for vertical displacement in section

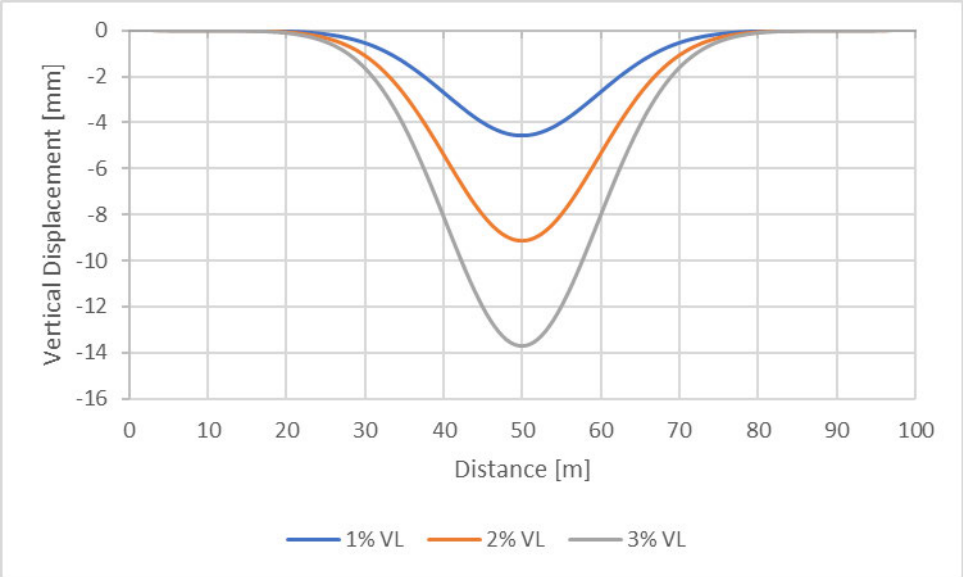
Vertical Displacement at Section SB-1a



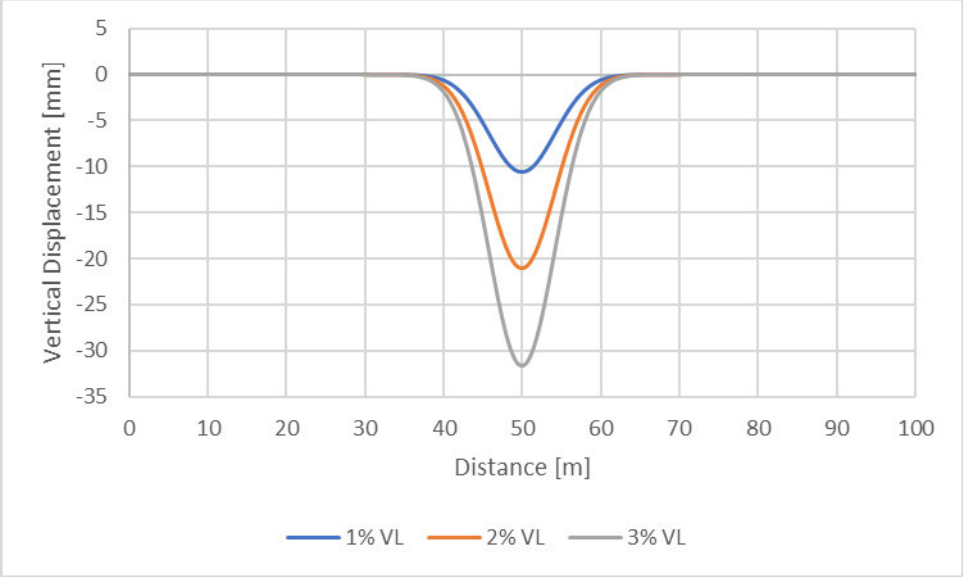
Vertical Displacement at Section SB-1b



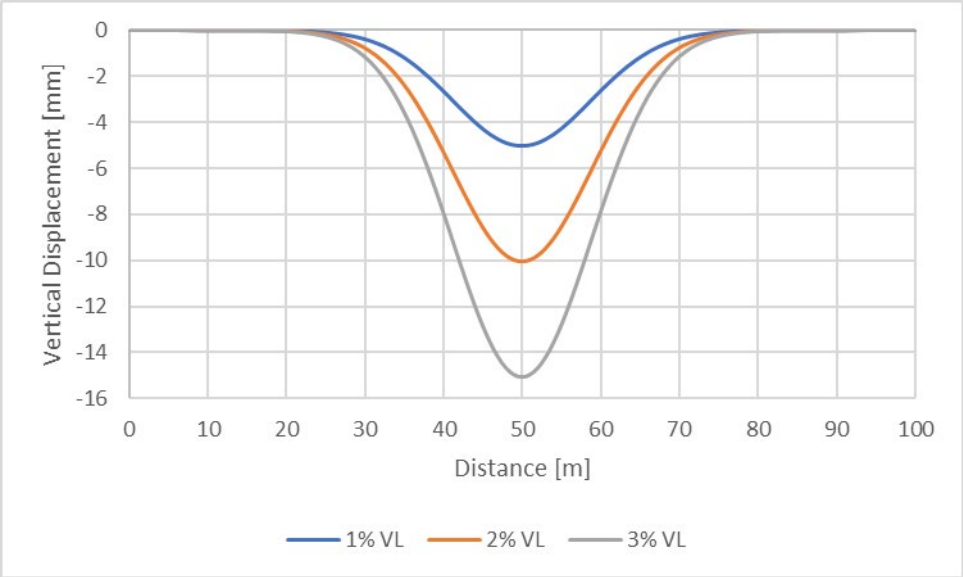
Vertical Displacement at Section SB-2a



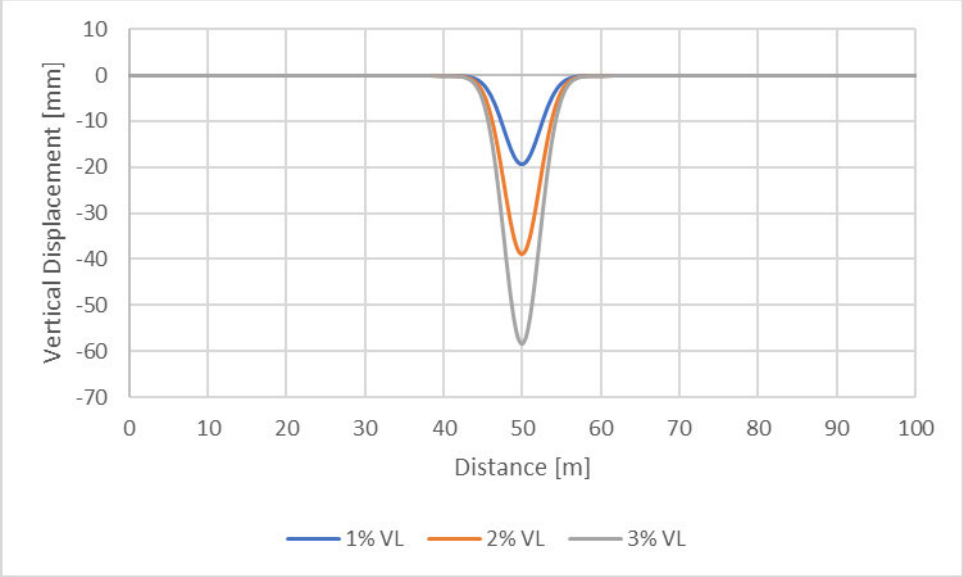
Vertical Displacement at Section SB-2b



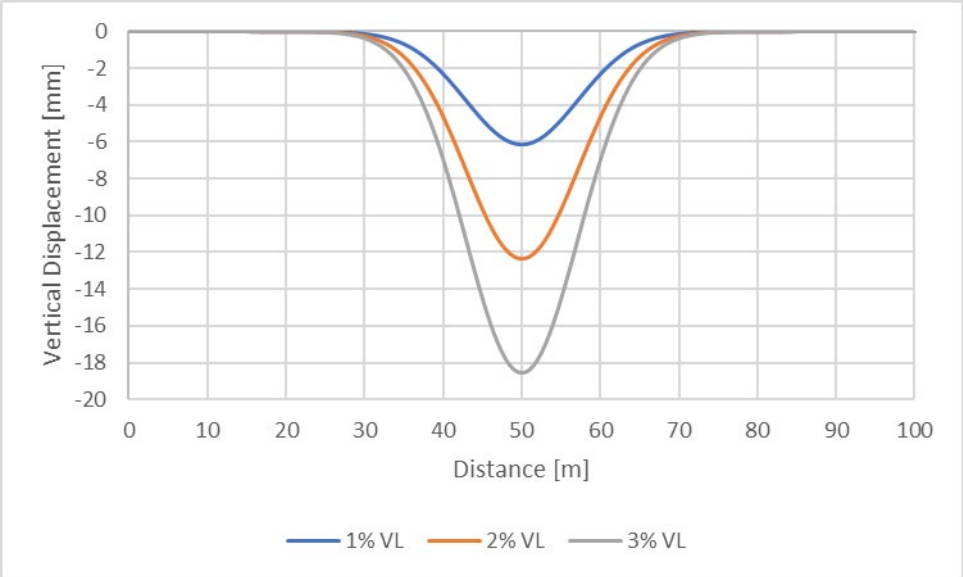
Vertical Displacement at Section SB-3a



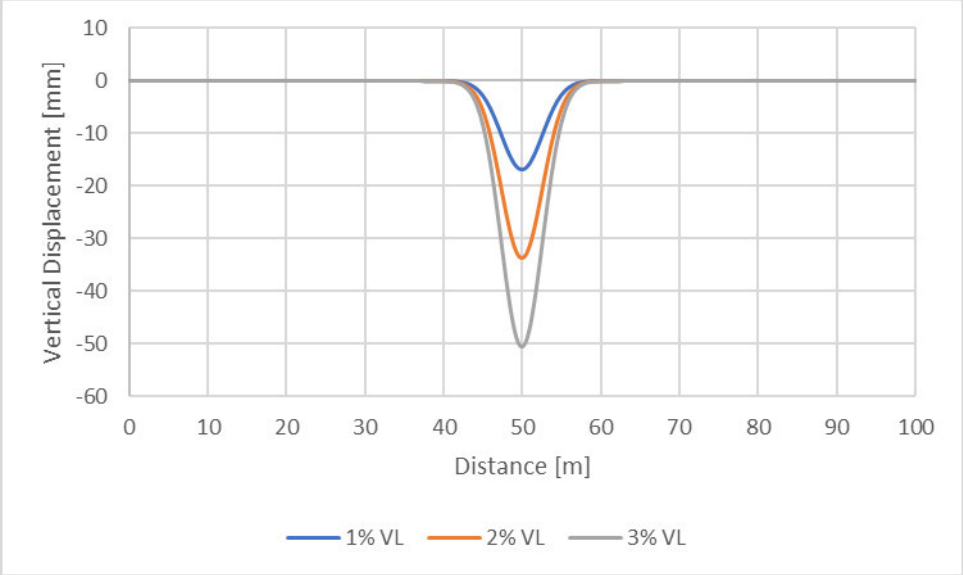
Vertical Displacement at Section SB-3b



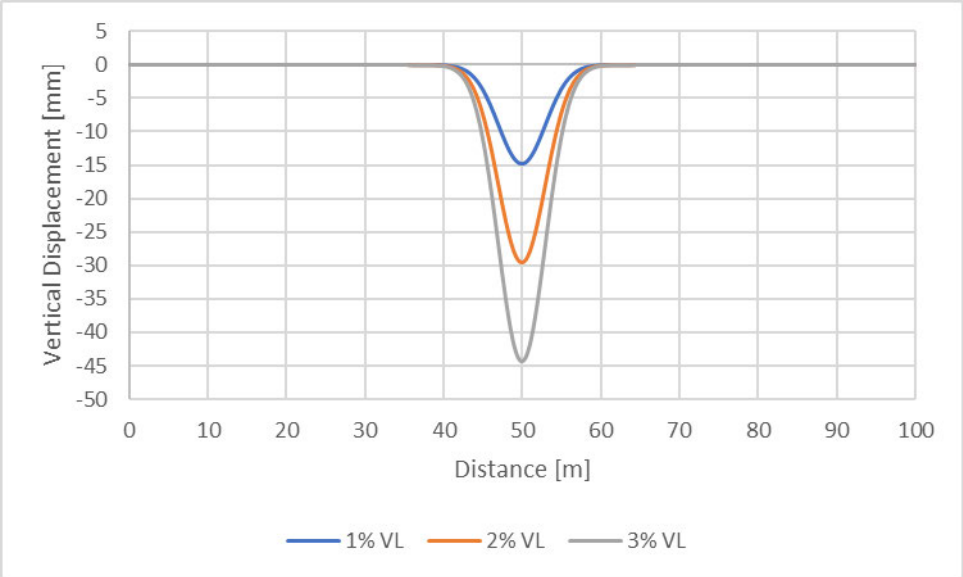
Vertical Displacement at Section SB-4a



Vertical Displacement at Section SB-4b

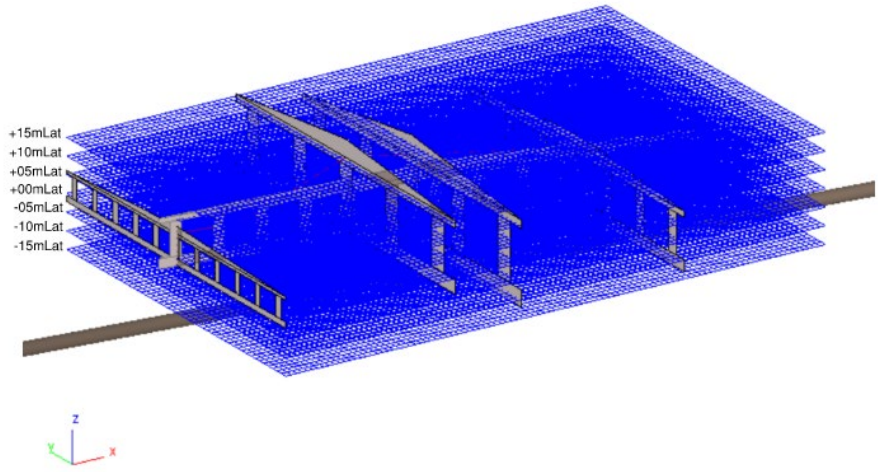


Vertical Displacement at Section CC

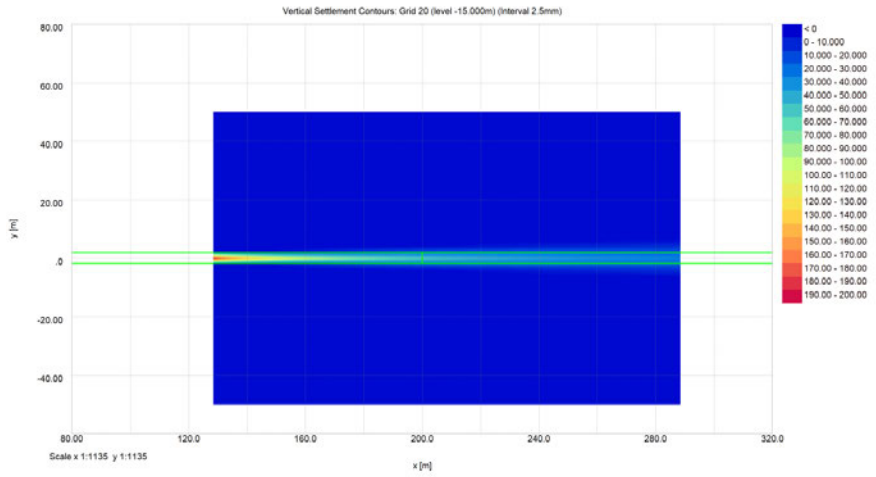


A.2 XDisp result contours for vertical displacement in plan

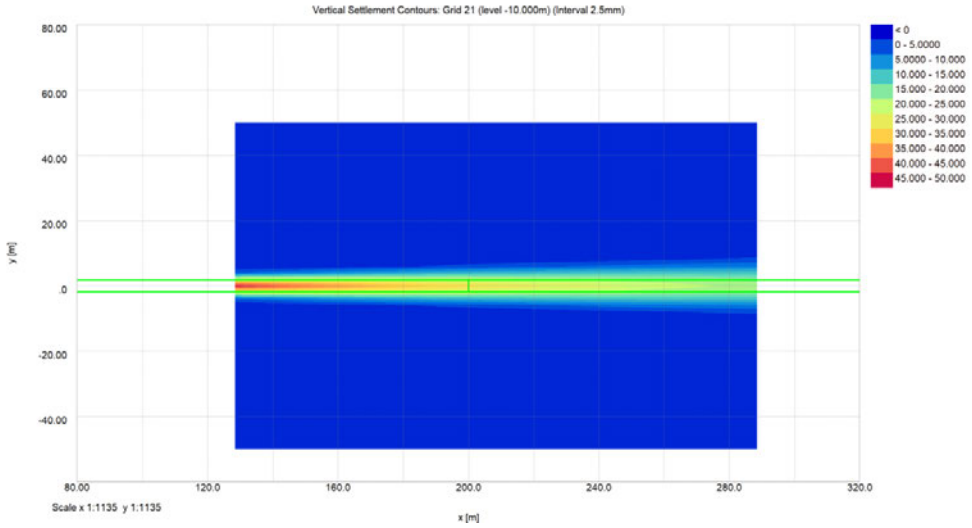
Outline of contours of the sea defence taken at 5m vertical intervals



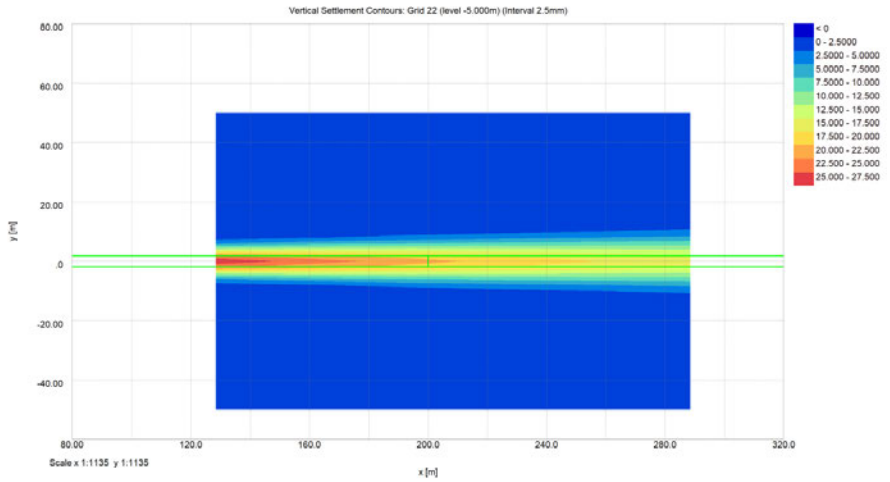
Contour of sea defence at -15mLat for 1% Volume Loss from plan view



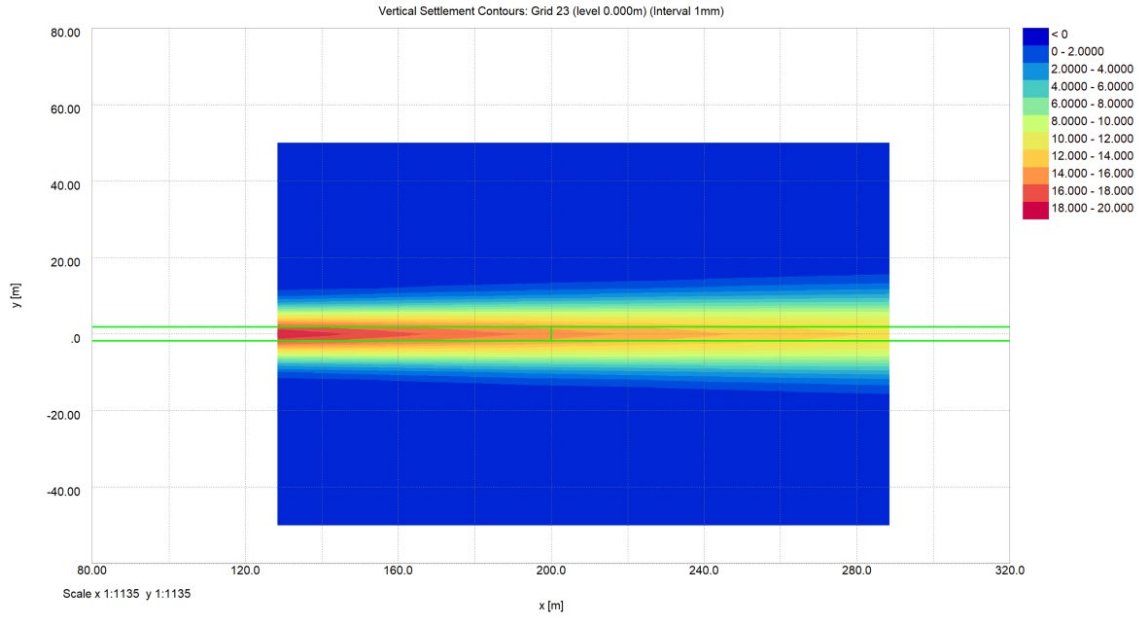
Contour of sea defence at -10mLat for 1% Volume Loss from plan view



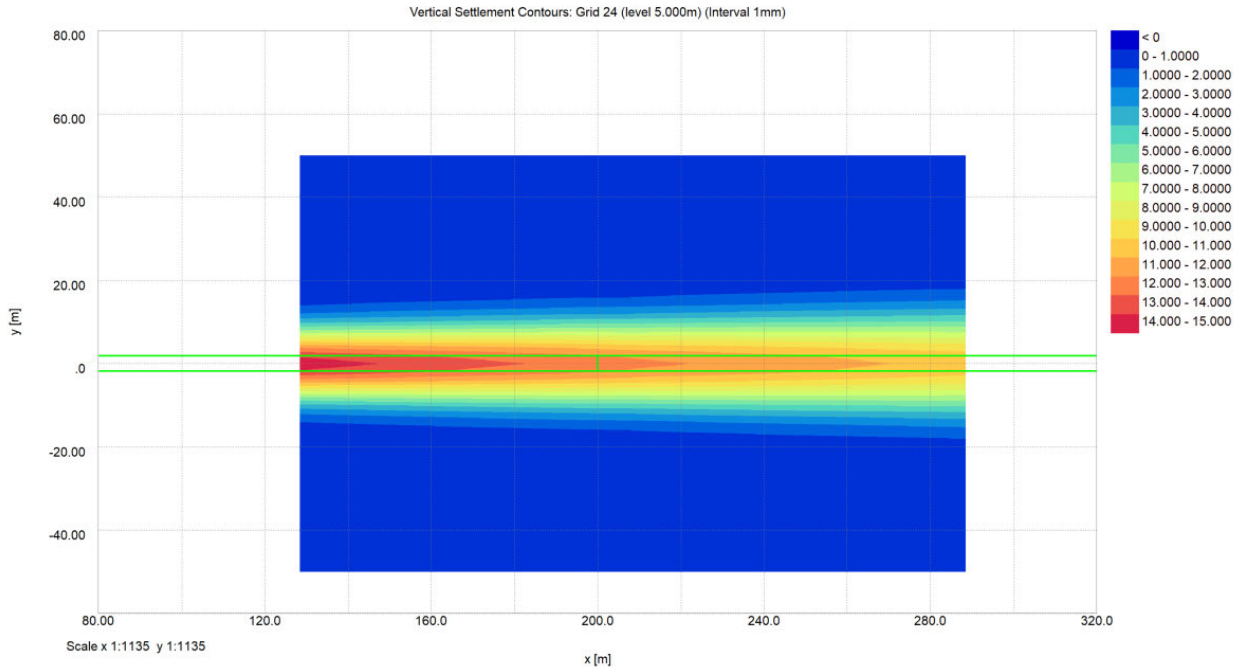
Contour of sea defence at -05mLat for 1% Volume Loss from plan view



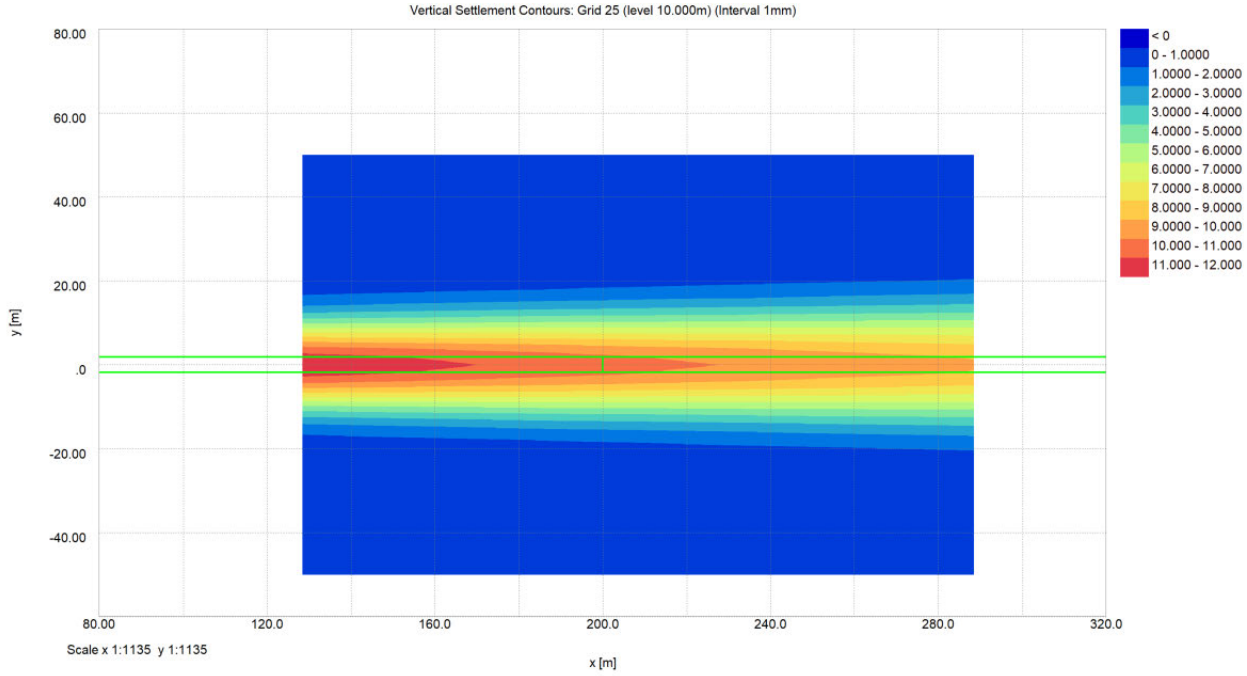
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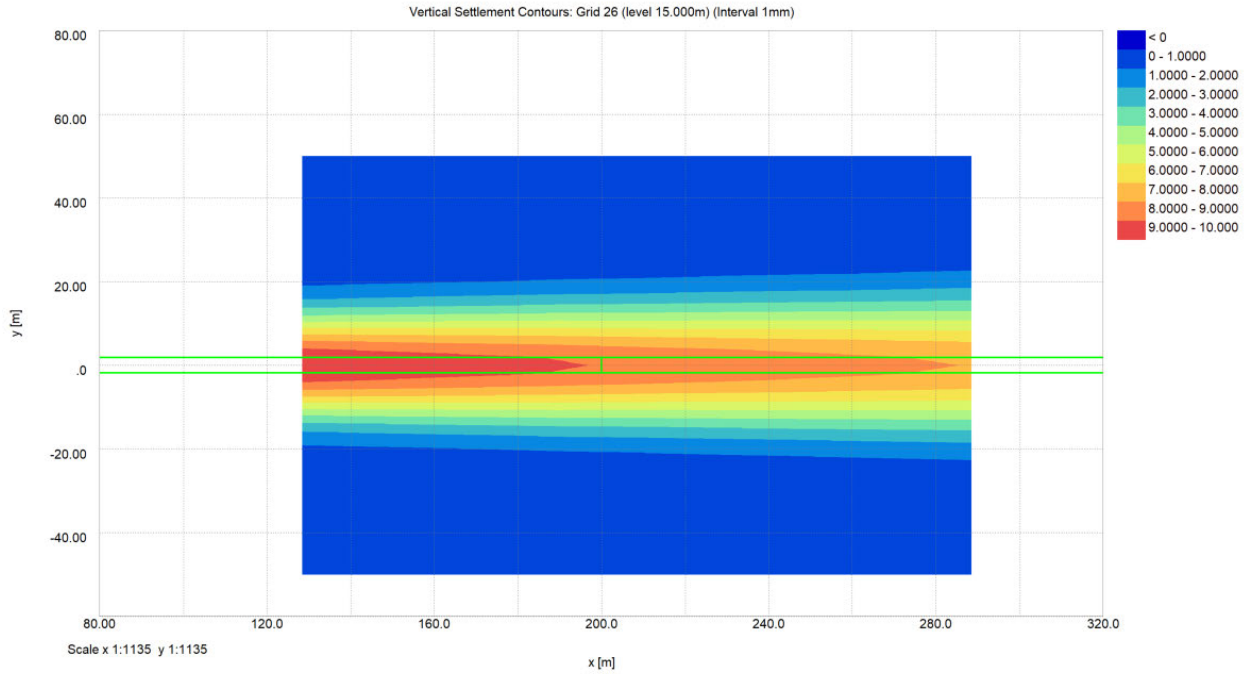
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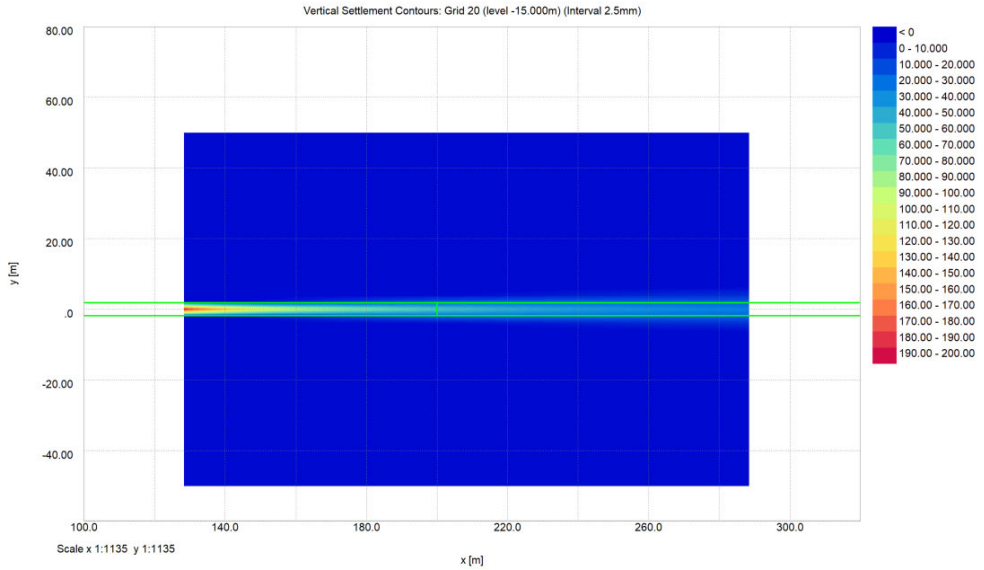
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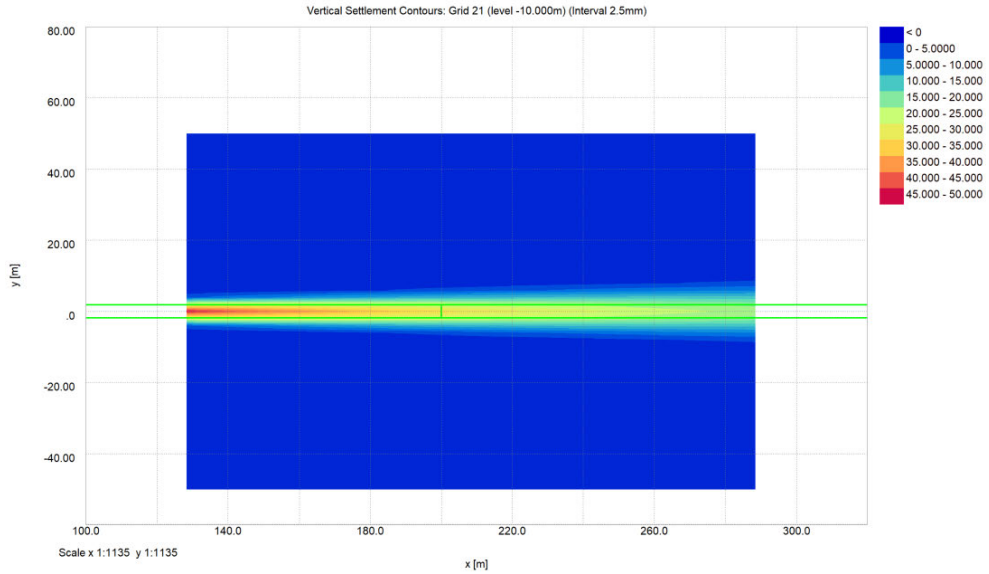
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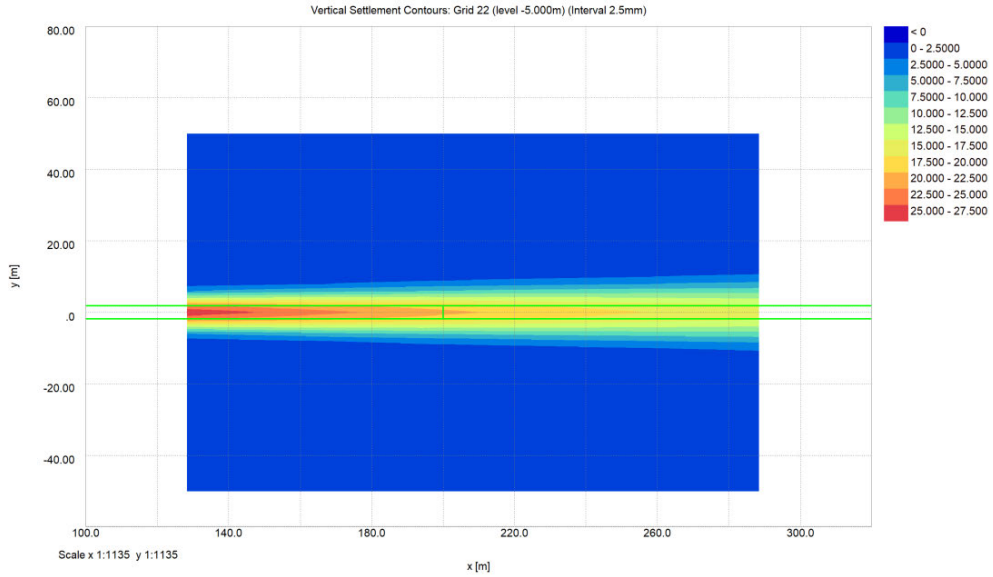
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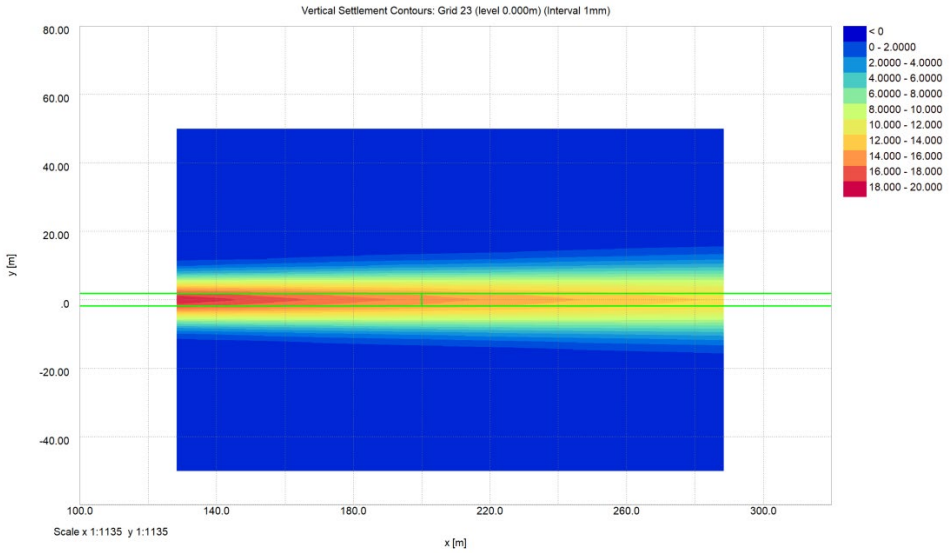
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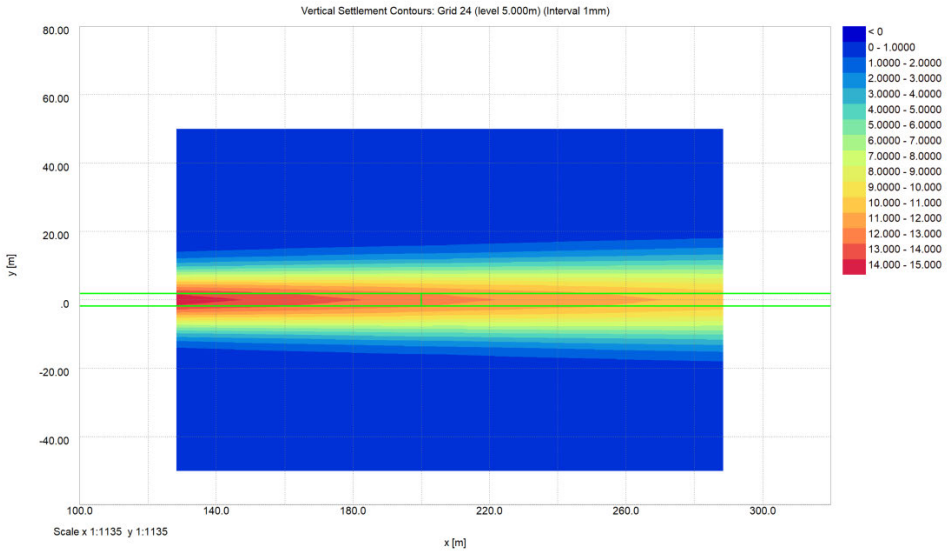
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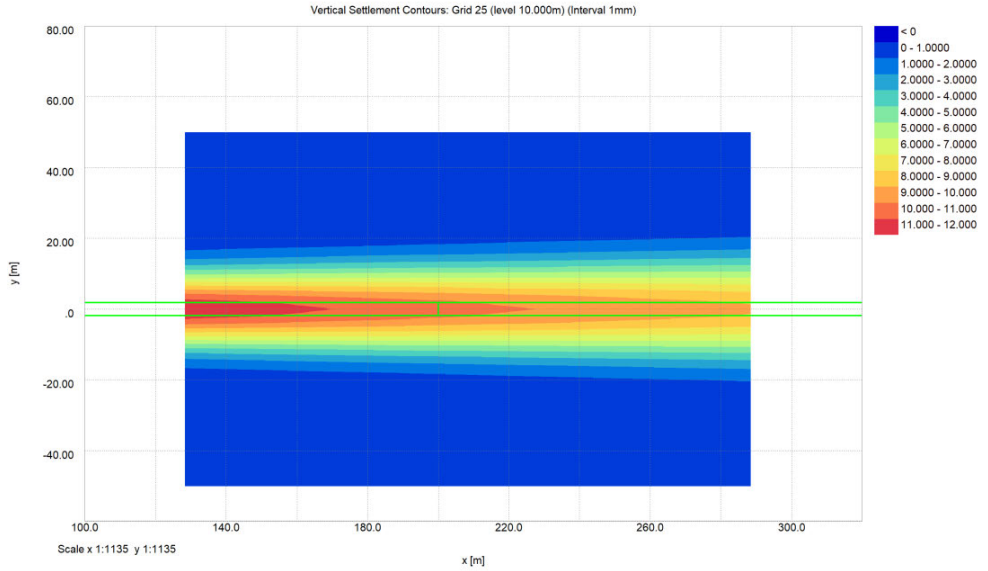
Contour of sea defence at +00mLat for 2% Volume Loss from plan view



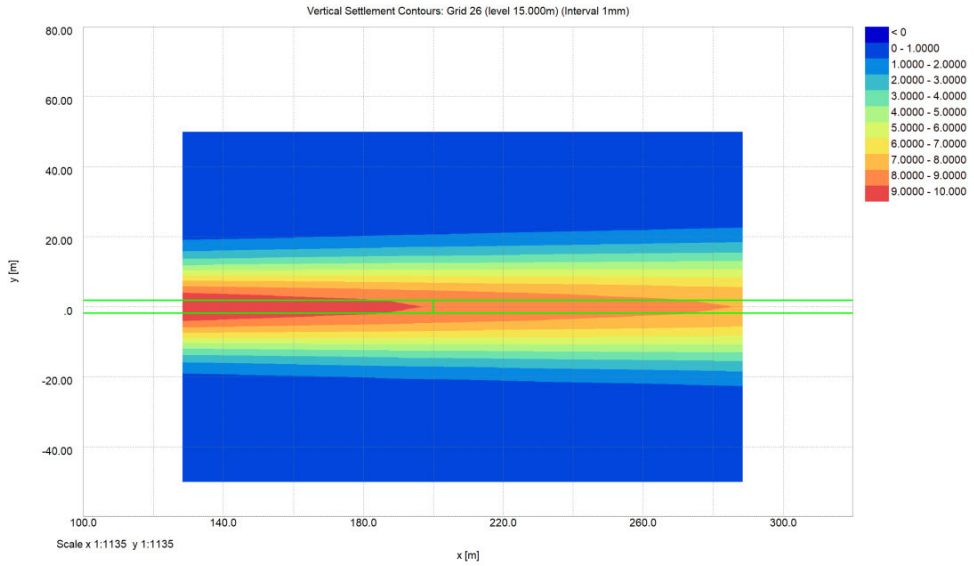
Contour of sea defence at +05mLat for 2% Volume Loss from plan view



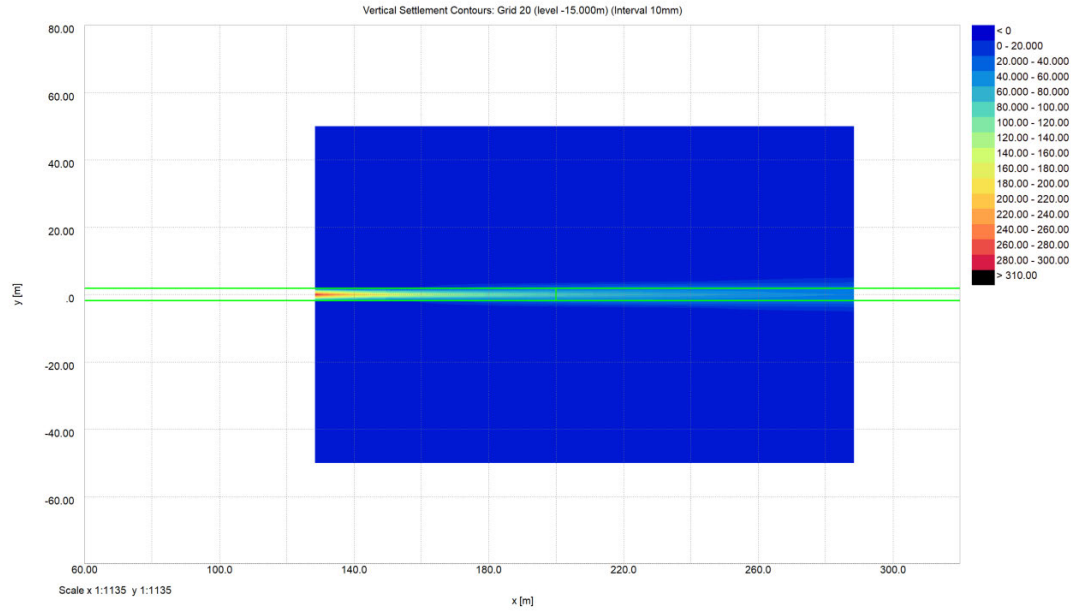
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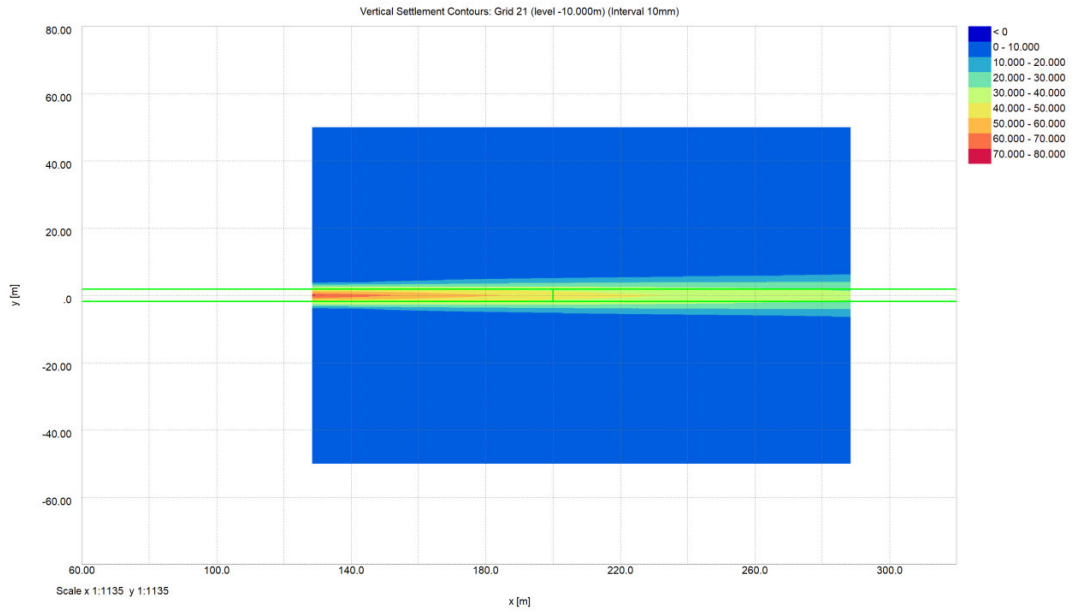
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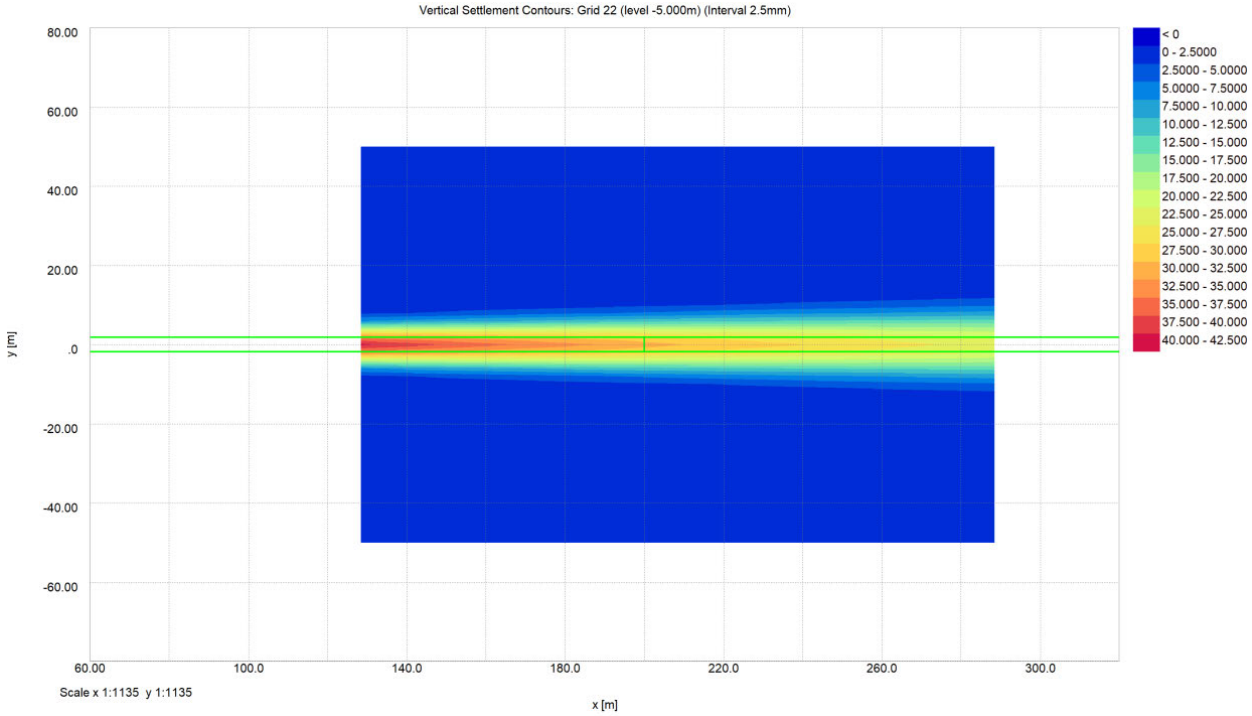
Contour of sea defence at -15mLat for 3% Volume Loss from plan view



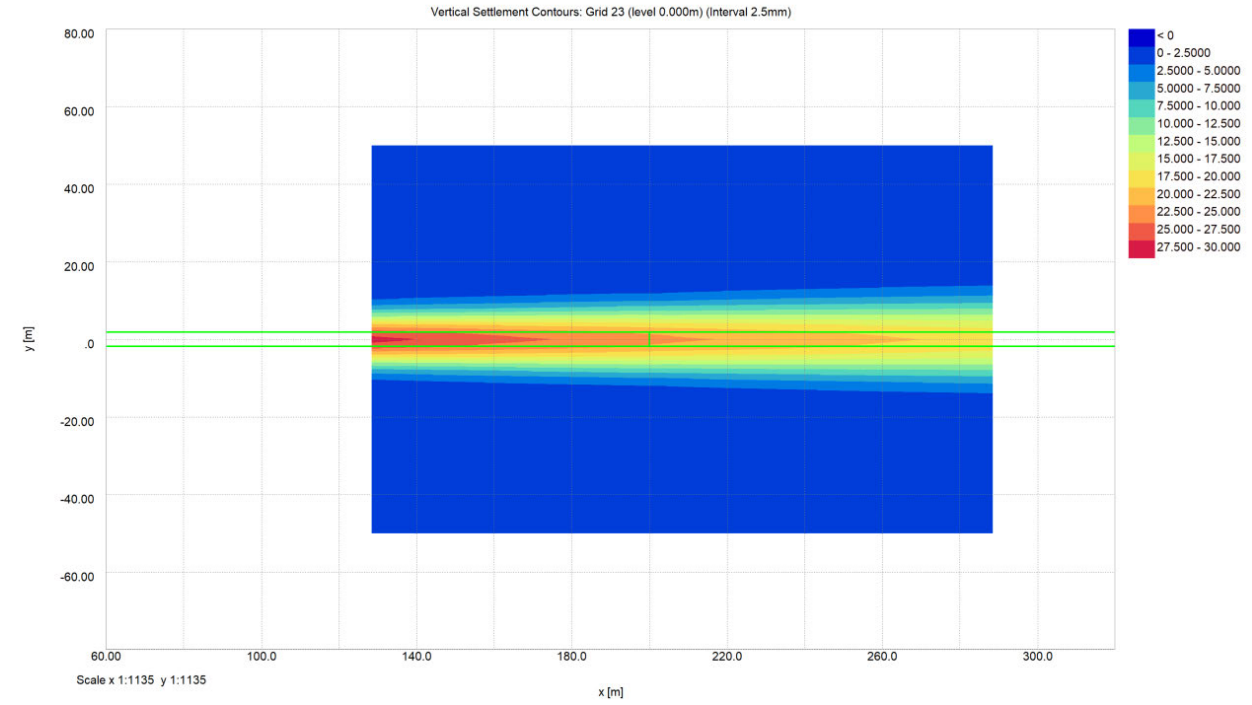
Contour of sea defence at -10mLat for 3% Volume Loss from plan view



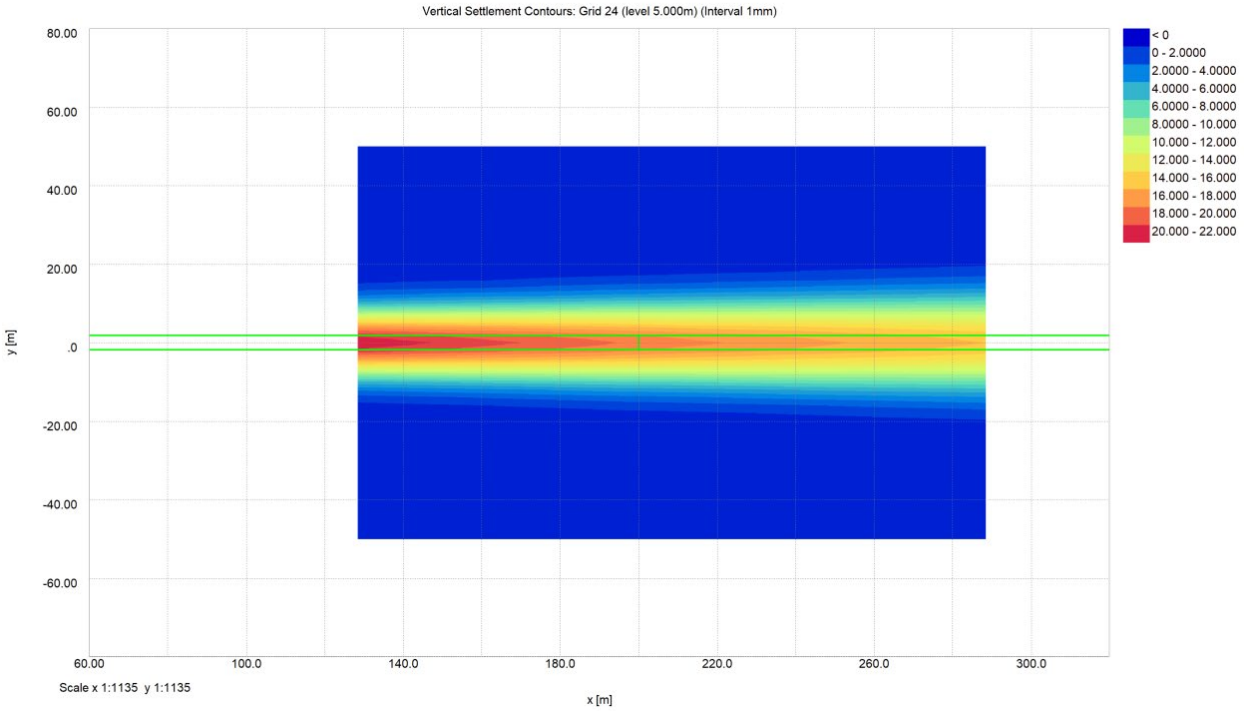
Contour of sea defence at -05mLat for 3% Volume Loss from plan view



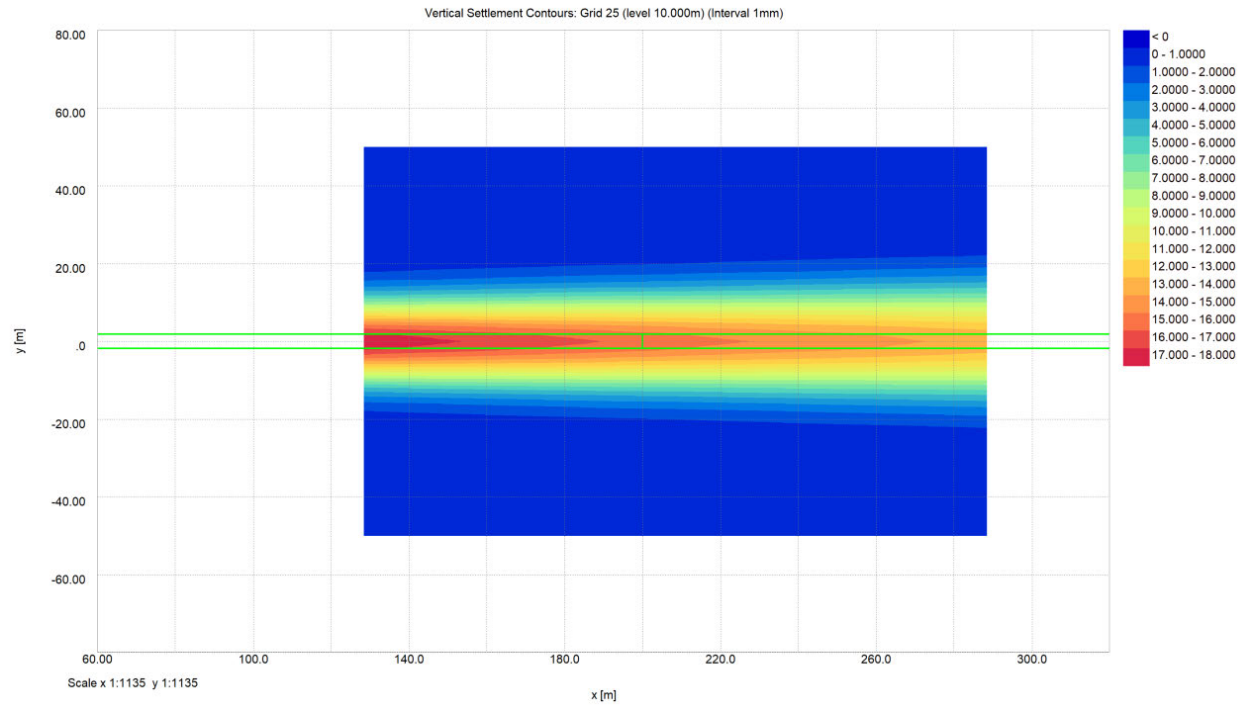
Contour of sea defence at +00mLat for 3% Volume Loss from plan view



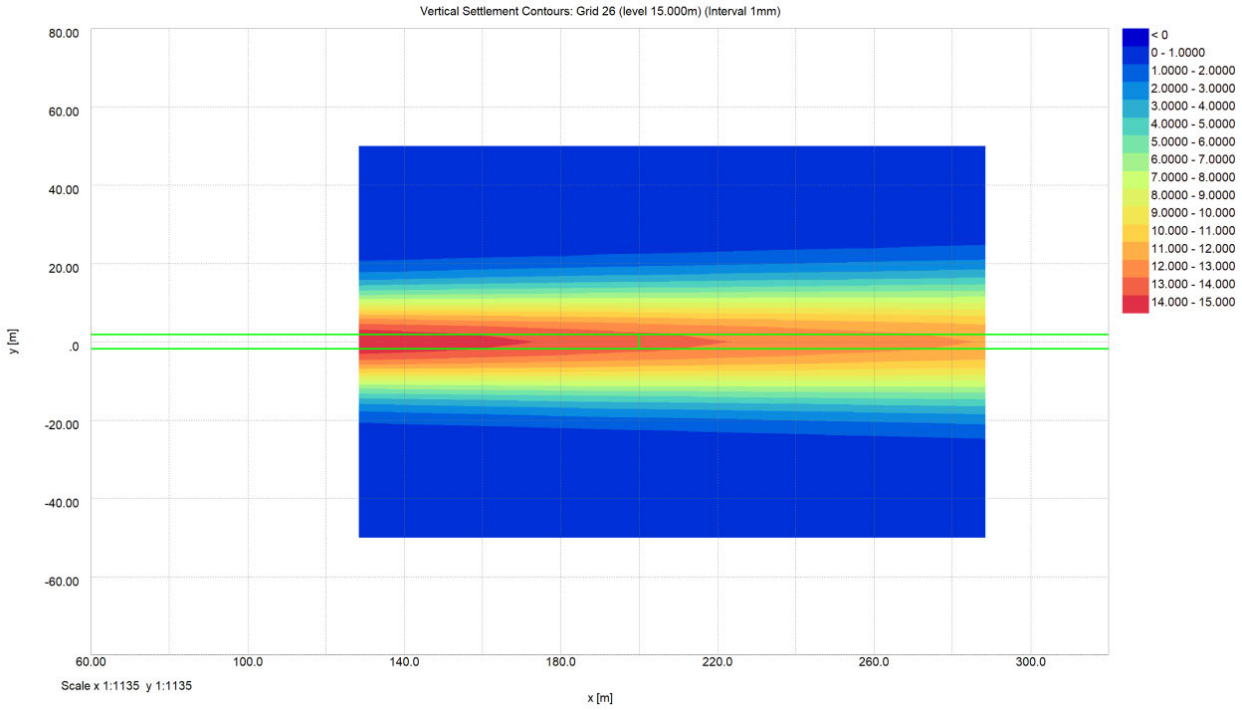
Contour of sea defence at +05mLat for 3% Volume Loss from plan view



Contour of sea defence at +10mLat for 3% Volume Loss from plan view



Contour of sea defence at +15mLat for 3% Volume Loss from plan view


Vertical Settlement Contours: Grid 26 (level 15.000m) (Interval 1mm)
80.00
60.00
40.00
20.00
.0
-20.00
-40.00
-60.00
y [m]
60.00
100.0
140.0
180.0
220.0
260.0
300.0
x [m]
Scale x 1:1135 y 1:1135
< 0
0 - 1.0000
1.0000 - 2.0000
2.0000 - 3.0000
3.0000 - 4.0000
4.0000 - 5.0000
5.0000 - 6.0000
6.0000 - 7.0000
7.0000 - 8.0000
8.0000 - 9.0000
9.0000 - 10.000
10.000 - 11.000
11.000 - 12.000
12.000 - 13.000
13.000 - 14.000
14.000 - 15.000

APPENDIX B PLAXIS RESULTS

ARUP

Total Aramis Micro Tunnelling Design

Settlement analysis- Plaxis 2D



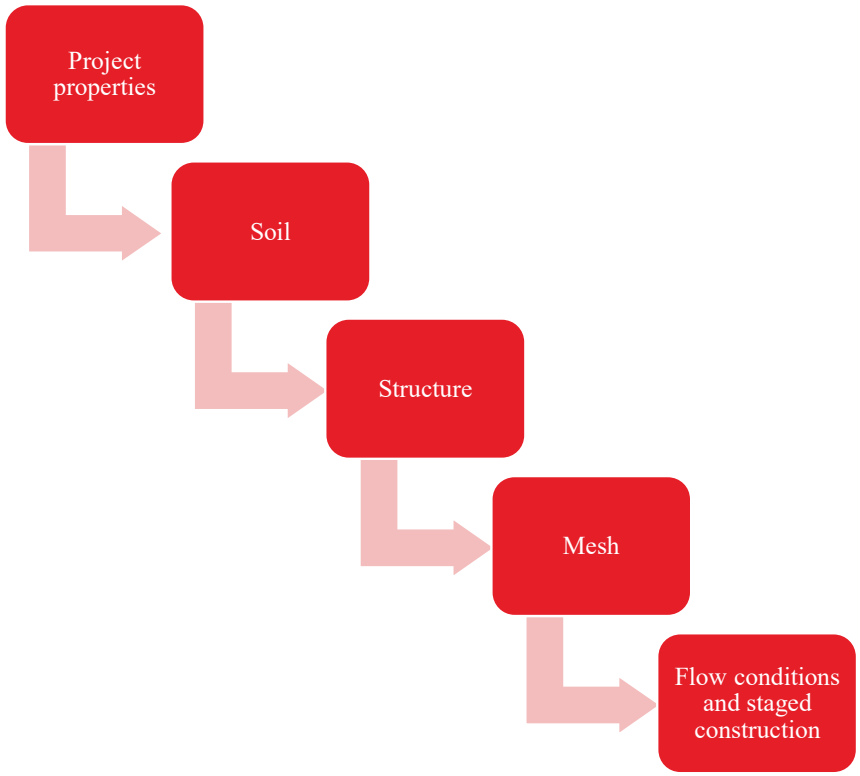
[redacted]

May 2024

Plaxis 2D model

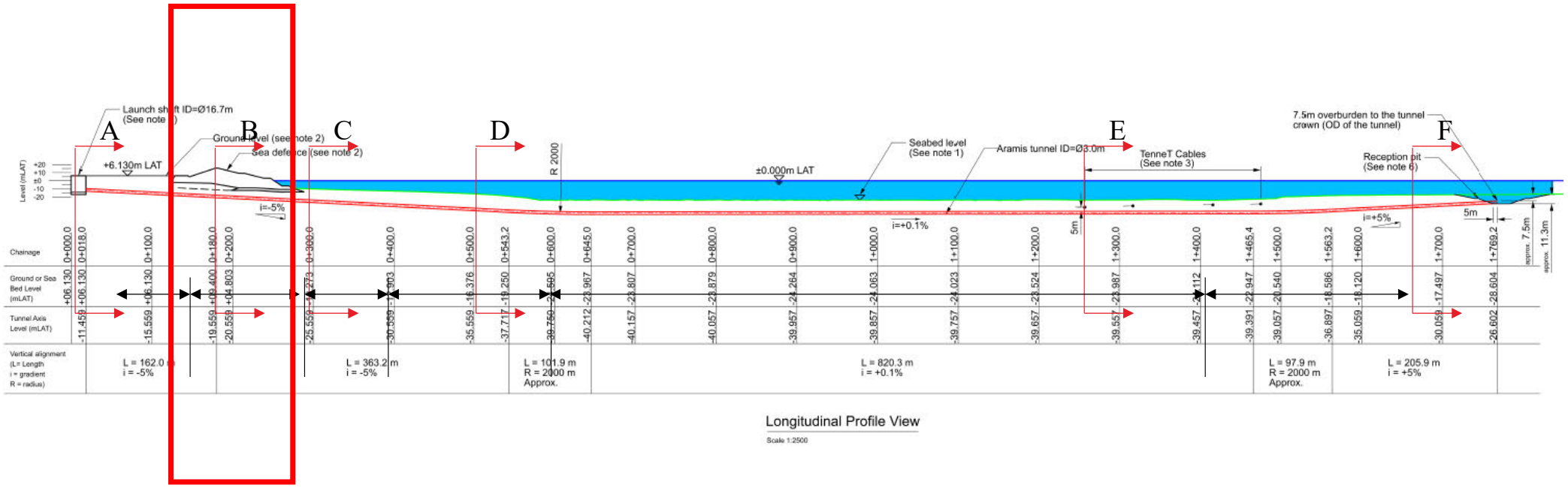
Scope

Aramis tunnel is modelled in Plaxis 2D to observe the settlement analysis of assets near the tunnel (Sea Defence).



Plaxis Analysis

Sections considered- Section B



Longitudinal Profile View

Scale 1:2500

Total Aramis Micro Tunnelling Design

Plaxis 2D

ARUP

Assumptions and Properties

Below are the assumptions made:

- Plain strain model ie. strains can only take place in the xy plane, $\varepsilon_z = 0$.
- At Section B, NLD clay is modelled as 'Undrained B' for the short-term behaviour.
- For soil-structure interaction the interface value of R_{inter} is assumed to be 0.67.
- Tunnel diameter is 3.5 O.D.
- 1m in model is equivalent to 1mLAT.
- Assuming 250mm segmental lining;
- Stratigraphy and geotechnical parameters based on the FEED GIR and divided in lower and upper bound.

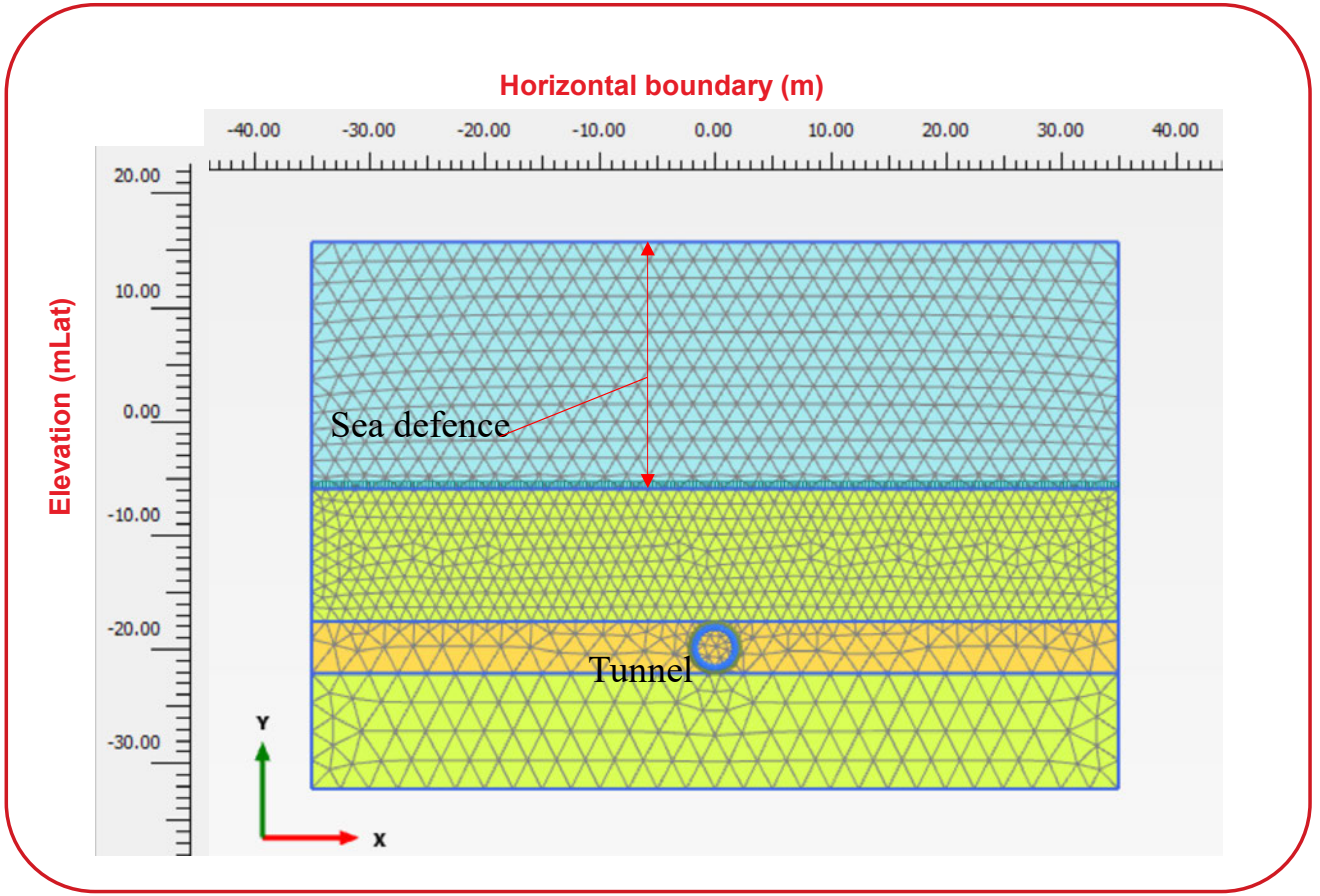
ARUP

Model properties

Plain strain model- Section B

Model boundaries	
x_{min}	-35m
x_{max}	35m
y_{min}	-35m
y_{max}	35m

Model boundaries	
Noded	15
γ_{water}	10kN/m3

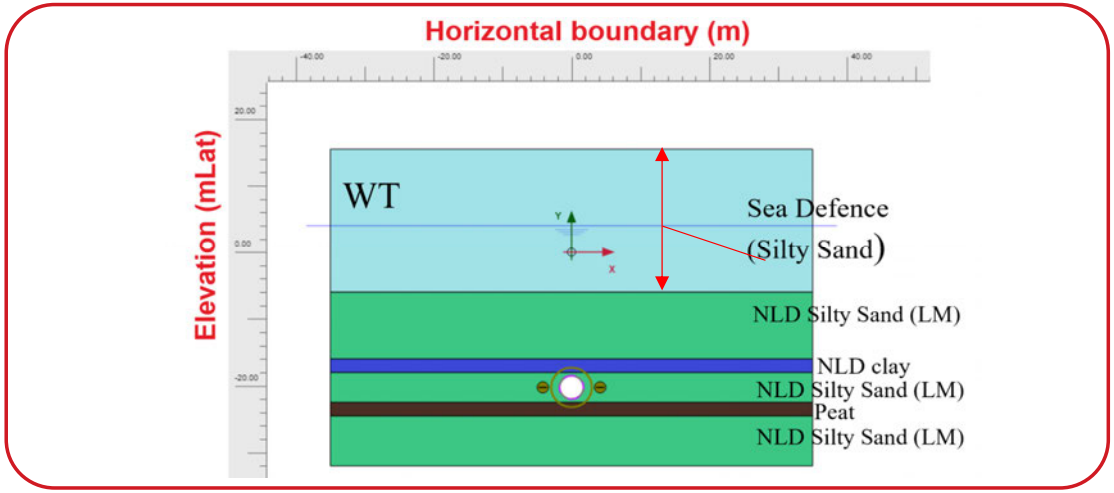


ARUP

Soil Parameters

Soil strata- Section B

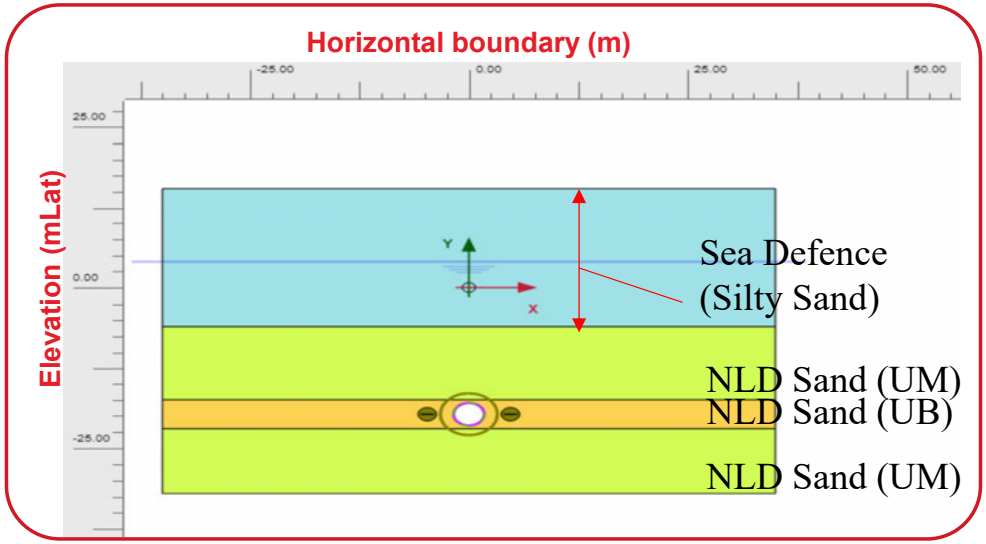
Lower model (LM)	Layer thickness	Unit wt	Poisson's Ratio	Friction angle	Cohesion	Undrained shear str	Avg. Stiffness	EP coeff.	Permeability	Soil model	Drainage type	Classification type	Soil class	Interface strength determination
		γ	ν	ϕp	c'	Su	E'	K0	k	-				
	m	kN/m3	-	degrees	kPa	kPa	kN/m2	-	m/s	-				
MG Silty sand (LM)	21.5	18.5	0.25	29	0	-	40000	0.52	0	HS small	Drained	Standard	Coarse	1
NLD Silty sand LM	10	17.5	0.25	29	0	-	30000	0.52	1E-5	HS small	Drained	Standard	Coarse	1
NLD clay	2	17	0.35	-	0	72	12960	0.5	1E-9	Mohr-Coulomb	Undrained B	Standard	Coarse	1
NLD Silty sand LM	4.5	17.5	0.25	29	0	-	30000	0.52	1E-5	HS small	Drained	Standard	Coarse	1
Peat	2	15	0.45	15	0	-	1500	0.67	1E-7	Mohr-Coulomb	Drained	Standard	Coarse	1
NLD Silty sand LM	7.5	17.5	0.25	29	0	-	30000	0.52	1E-5	HS small	Drained	Standard	Coarse	1



Soil Parameters

Soil strata- Section B

Upper model (UB)	Layer thickness	Unit wt	Poisson's Ratio	Friction angle	Cohesion	Undrained shear str	Avg. Stiffness	EP coeff.	Permeability	Soil model	Drainage type	Classification type	Soil class	Interface strength determination
		γ	ν	ϕp	c'	Su	E'	K0	k	-				
	m	kN/m3	-	degrees	kPa	kPa	kN/m2	-	m/s	-				
MG Sand (UM)	21.5	20	0.2	38	0	-	62000	0.67	0	HS small	Drained	Standard	Coarse	0.67
NLD Sand (UM)	31.6	20	0.2	38	0	-	60000	0.67	0	HS small	Drained	Standard	Coarse	0.67
NLD Sand (UB)	4	20	0.2	42	0	-	76000	0.63	0	HS small	Drained	Standard	Coarse	0.67
NLD Sand (UM)	10	20	0.2	38	0	-	60000	0.67	0	HS small	Drained	Standard	Coarse	0.67



ARUP

Tunnel lining

Properties

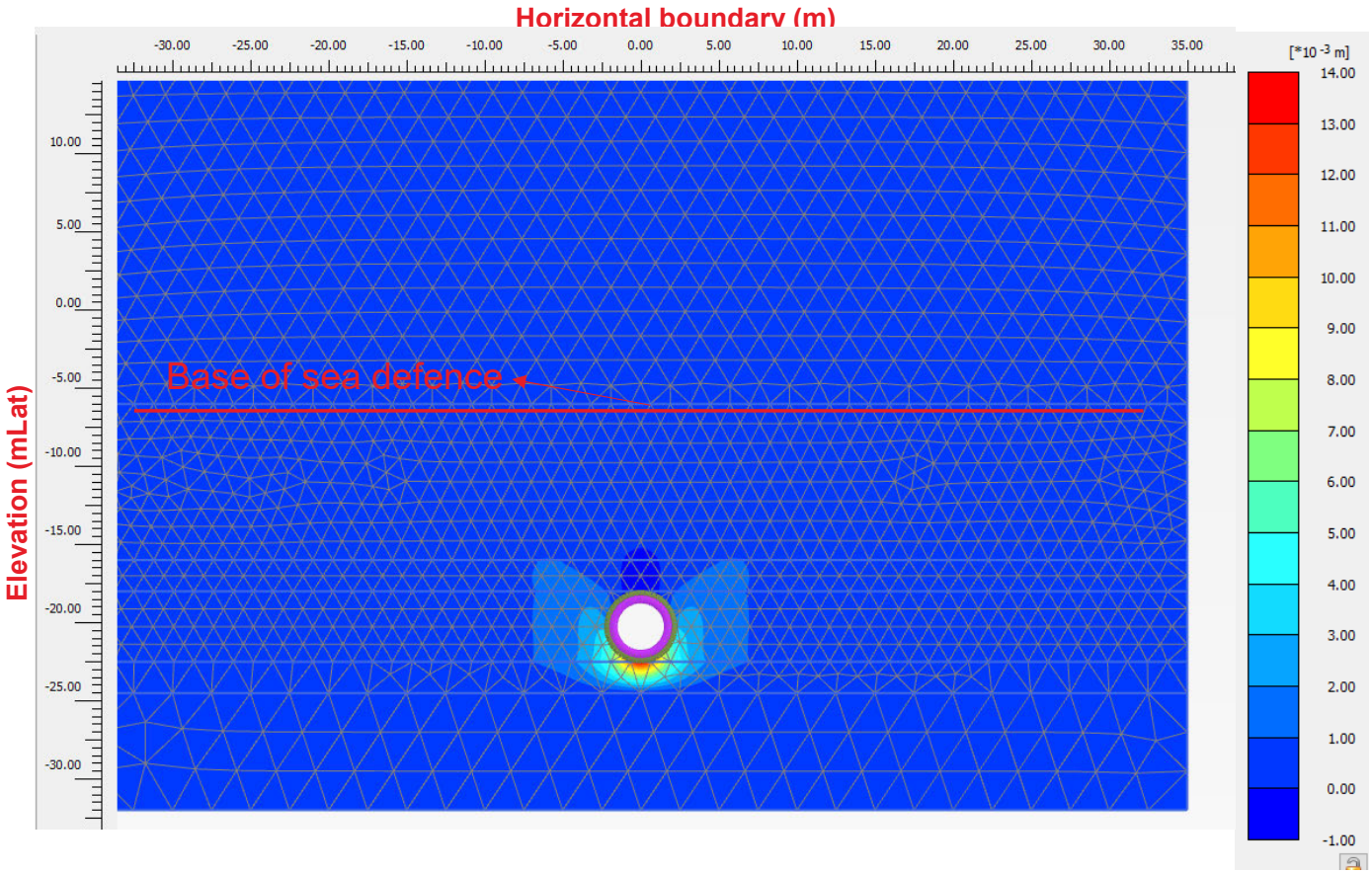
	Material type	Equivalent Unit weight	Mechanical	Section area	Second moment of area	Equivalent depth of lining	Elastic modulus	Stiffness		Poisson's Ratio	d	Segment Radius	Mesh Options
		w		A1	I	d		EA1	EI	ν			
		kN/m/m		m^2/m	m^4/m	m	MPa	kN/m	kNm2/m	-	m	m	-
250mm lining	Elastic	4.8	Isotropic	0.1983347	0.00065015	0.198	37278	7393519.7	24236.35	0.2	0.25	1.75	Fine

Note: Assumed concrete Class is C50/60 and unit weight of the concrete is 24kN/m3.

Results

Section B (LM)

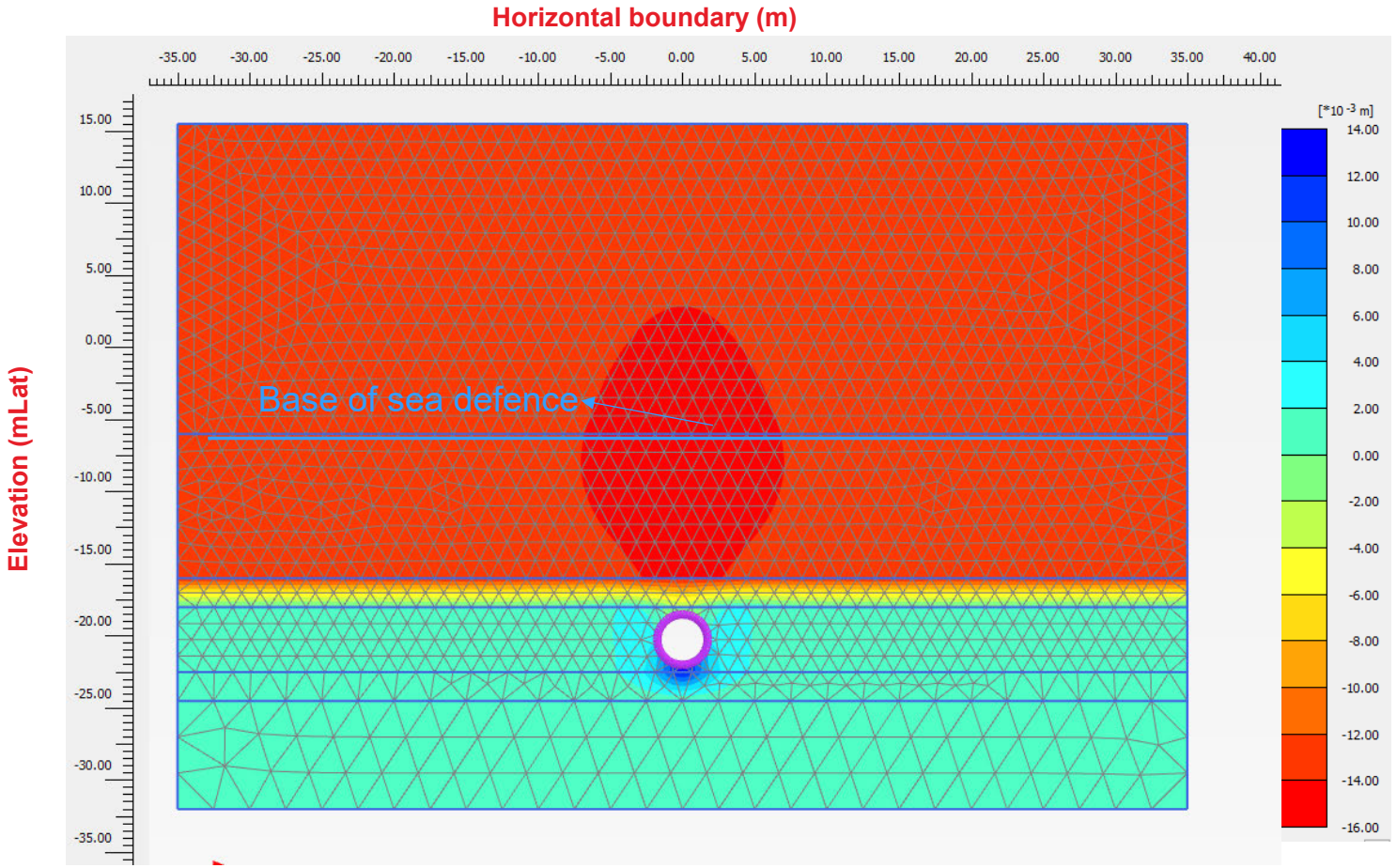
Lower model (LM)	
Volume loss (%)	Maximum Vertical displacement [mm]
1%	0.3
2%	0.3
3%	1



Results

Section B (LM)

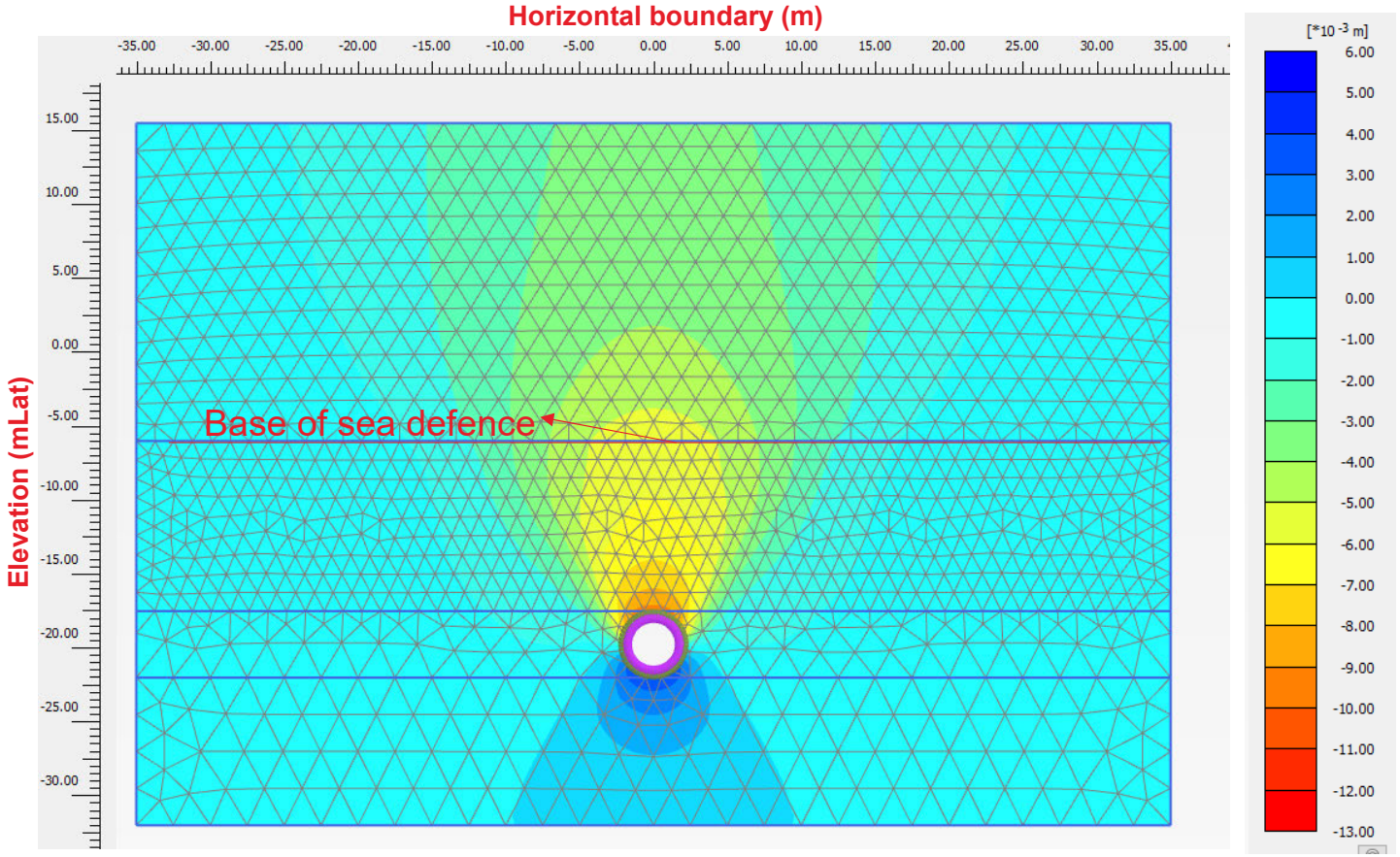
After dissipation of excess pore pressure over time, the maximum settlement at the base of sea defence is 14mm.



Results

Section B (UB)

Upper model (UB)	
Volume loss (%)	Maximum Vertical displacement [mm]
1%	6
2%	10
3%	14



ARUP

APPENDIX C RWS MEETING PRESENTATION - 13 MAY 2024

ARAMIS

A large-scale, flexible CO_2 transport and storage solution

Aramis Landfall versus Sea Defence

RWS Meeting

13 May 2024

Confidential / Restricted

Aramis Landfall - Sea Defence

Agenda

- Introduction
- Current Project Status + Recap and Overview
- Landfall Design (Petrofac/OI)
  - Design tunnel/ landfall
  - Landfall (Shark fin) layout – temporary phase and permanent phase
  - Pipeline design near the sea Defence - NEN 3651 (Petrofac/OI)
    - Safety zones, Wall thickness and Strength test
- Tunnel Design (Petrofac/Arup)
  - Trajectory
  - Settlement (+ comparison to pre-FEED results from MMD)
- Construction (Petrofac/Arup/OI)
  - Storyline Installation Pipeline Through tunnel
  - Temporary use of the Shark fin (all construction phases)
  - Tunnelling schedule (all phases)
  - Filling of the tunnel (grouting + alternatives)



Aramis Landfall - Sea Defence

Current Project Status

- Permit Application Submitted to MEAC (EZK) FEB24
- Overall FEED Progressing (started NOV23)
  - Petrofac – FEED Contractor
- Tunnel FEED commenced end JAN24
  - ARUP – Subcontractor from Petrofac

Recap

- Meetings held:
  - Last meeting held **19SEP23**
- Surveys
  - Preparations ongoing for (PO placement):
    - Topographical survey and monitoring of the Sea Defence
    - Groundwater level monitoring
  - UXO extended desk-top exercise ongoing
- Objective of this Meeting
  - Present Aramis' latest design status of the system near the Sea Defence



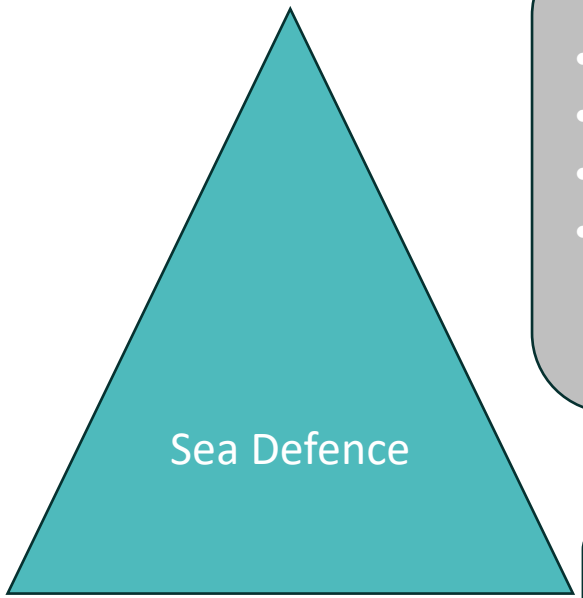
Aramis Landfall - Sea Defence

Overview

- Aramis System in relation to the Sea Defence

Construction Works @ Sharkfin
- *Tunnel shaft*
- *Onshore pipeline (incl BVS)*
- *Pipe pull-in*
- *Pre-commissioning*

Permanent Facilities @ Sharkfin
- *Tunnel shaft*
- *Onshore pipeline (incl BVS)*
- *Analyser Station (TBC)*



Critical information
- *As-built data of the seaDefence:*
- *Maasvlakte 1*
- *Maasvlakte 2*
- *Native files*

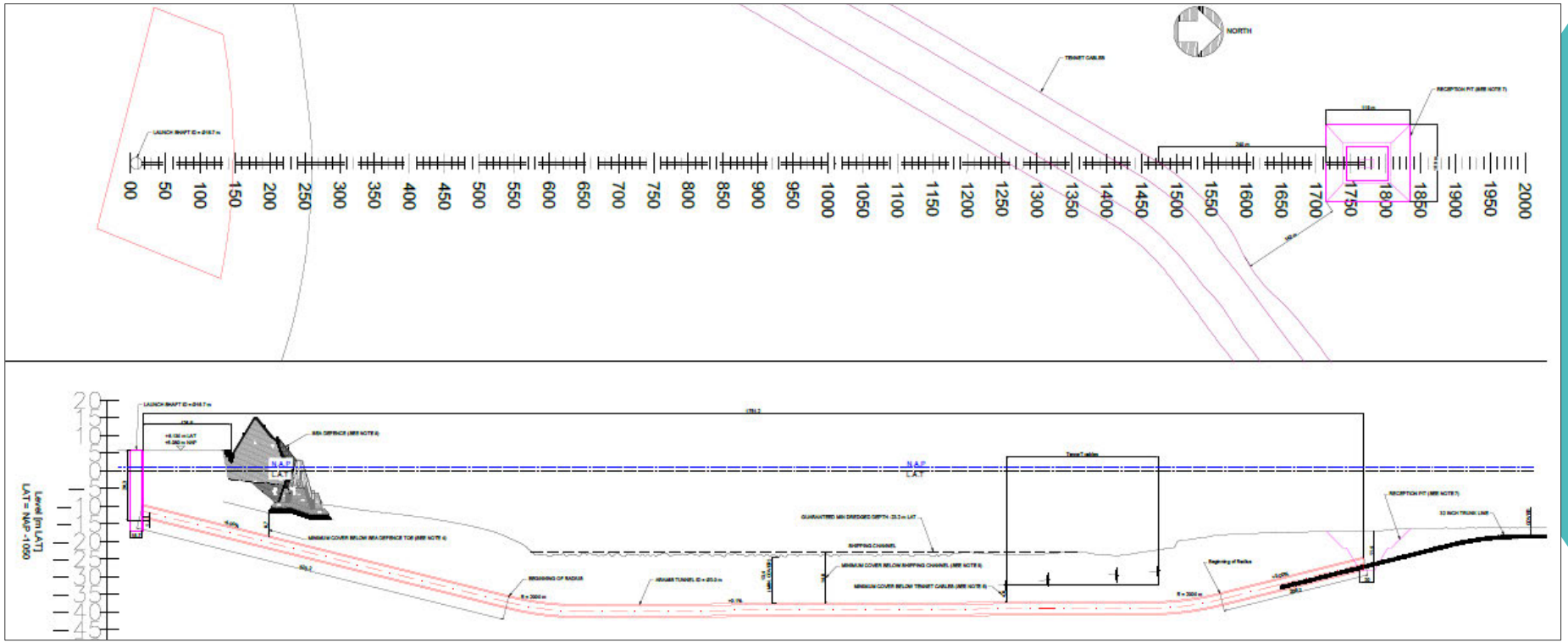
Can RWS still supply this?

Maasgeul

Tunnel and Pipeline Crossing
- underneath SeaDefence and Maasgeul

Aramis Landfall - Sea Defence

Landfall Design- profile tunnel/ landfall

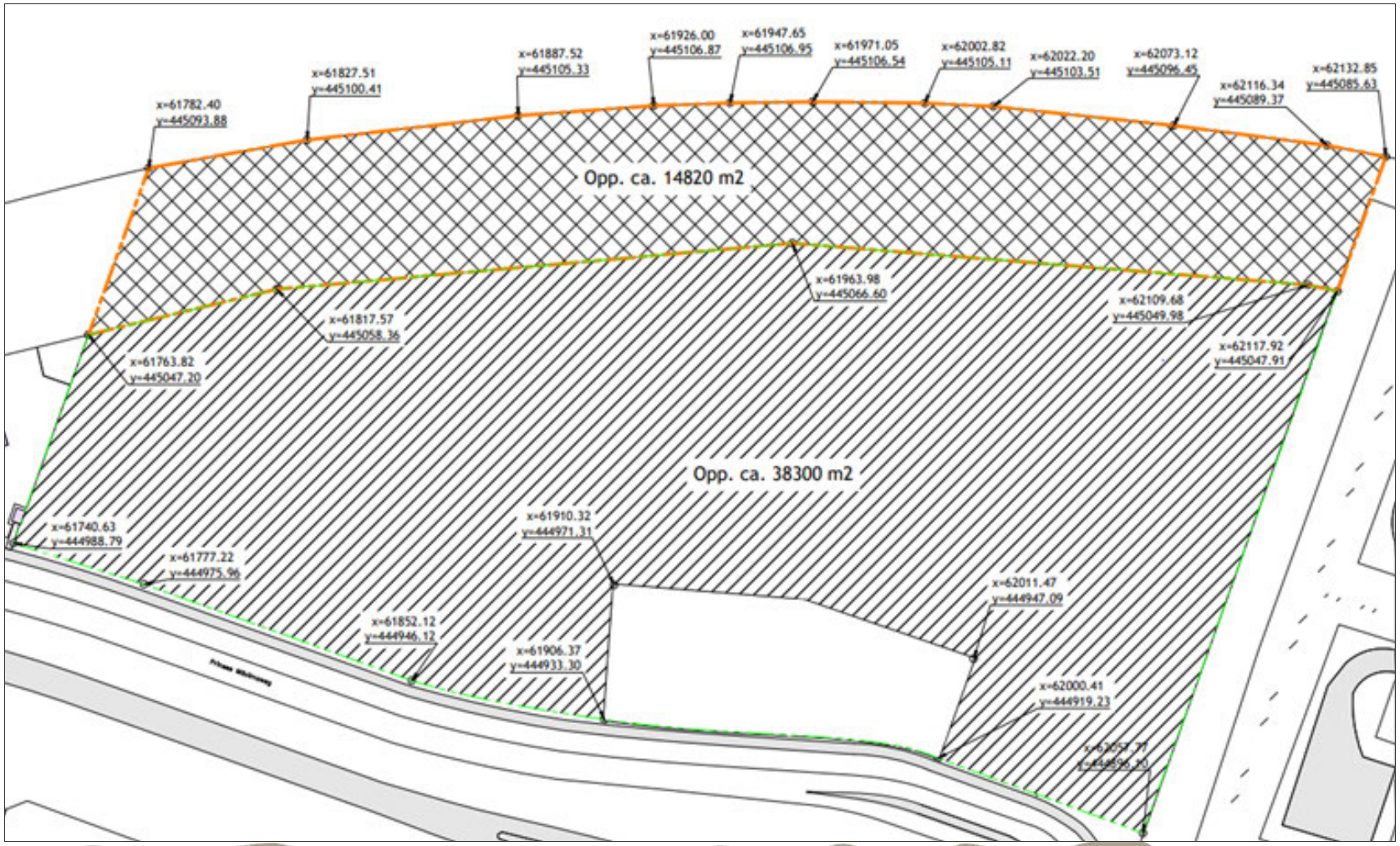


Landfall:

- Tunnel: 1.751 m ID 3m, OD 3.8 m
- Pipeline: DN750, CO2 Dense Phase

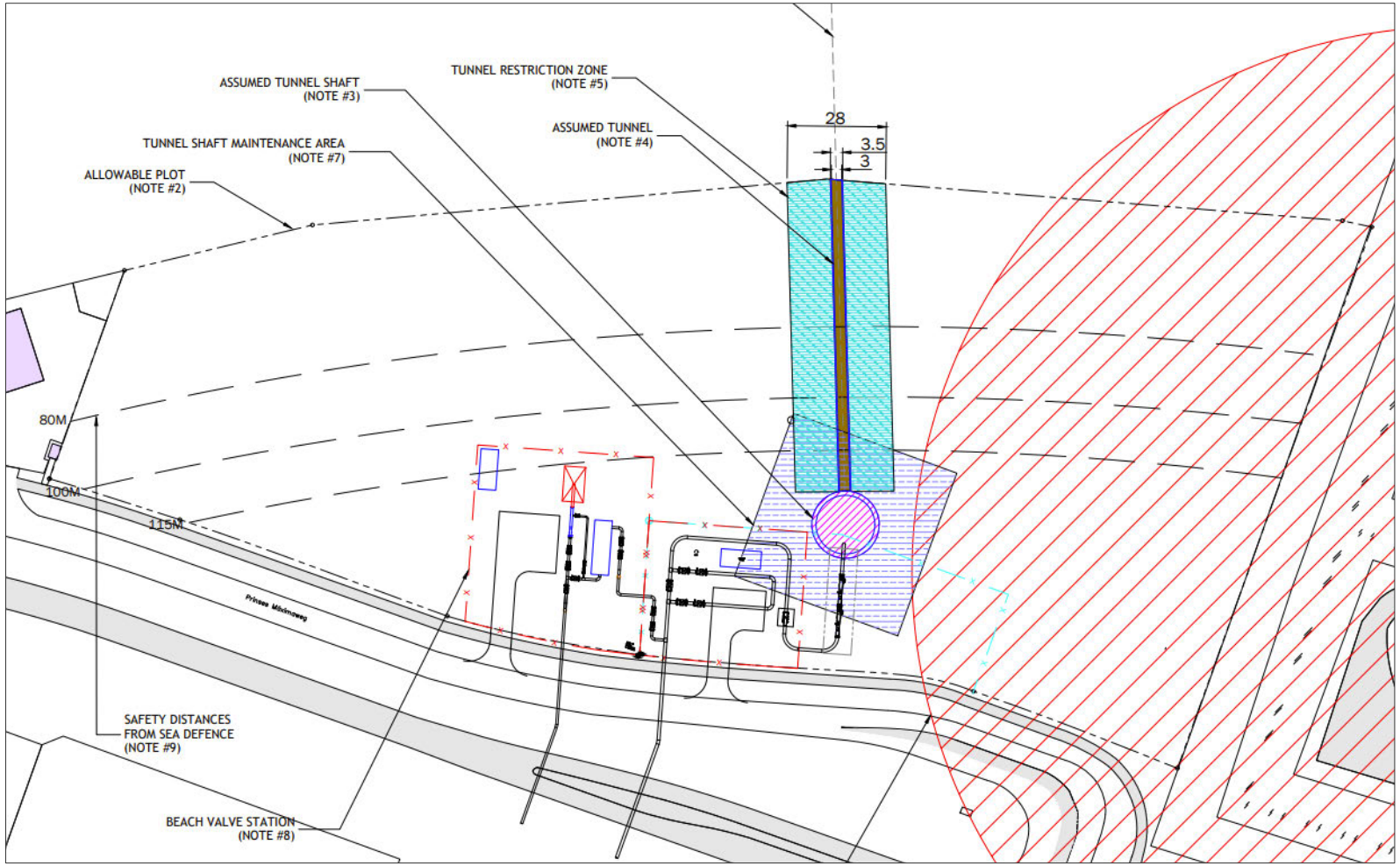
Aramis Landfall - Sea Defence

Landfall Design- Landfall (Shark fin) layout – temporary phase



Aramis Landfall - Sea Defence

Landfall Design- Landfall (Shark fin) layout – permanent phase



Landfall Design- Pipeline design

- Pipeline: DN750;
- Medium: CO2 Dense Phase;
- Design Pressure: 200 bar;
- Design Temperature: - 20 °C - +70 ° C;
- Steel quality: API 5L X65;
- Design according to:
  - Basis of Design Aramis project;
  - NEN-3650/ **3651**/ 3655;
  - the **NEN-3651** is the leading code for the design of the landfall
  - Local regulation of Rotterdam (HBOR);
  - DNV-codes for the offshore pipeline;
  - Total-specifications.

Aramis Landfall - Sea Defence

Landfall Design- Pipeline design crossing sea Defence according NEN 3651

- Safety zone determination
- Wall thickness calculation
- Determination test pressures (strength and tightness)

Aramis Landfall - Sea Defence

Landfall Design- Pipeline design crossing sea Defence according NEN 3651

Safety zone determination:

- Assessment safety zone based on gas phase;
- Assessment location Shaft for tunnel and valve station made on constructability and land use;
- Minimum distance Shaft from sea Defence 80 m;
- Minimum distance parallel pipelines to sea Defence 115 m;

NEN 3651 is not clear about the geotechnical impact in case of a CO2 Dense Phase

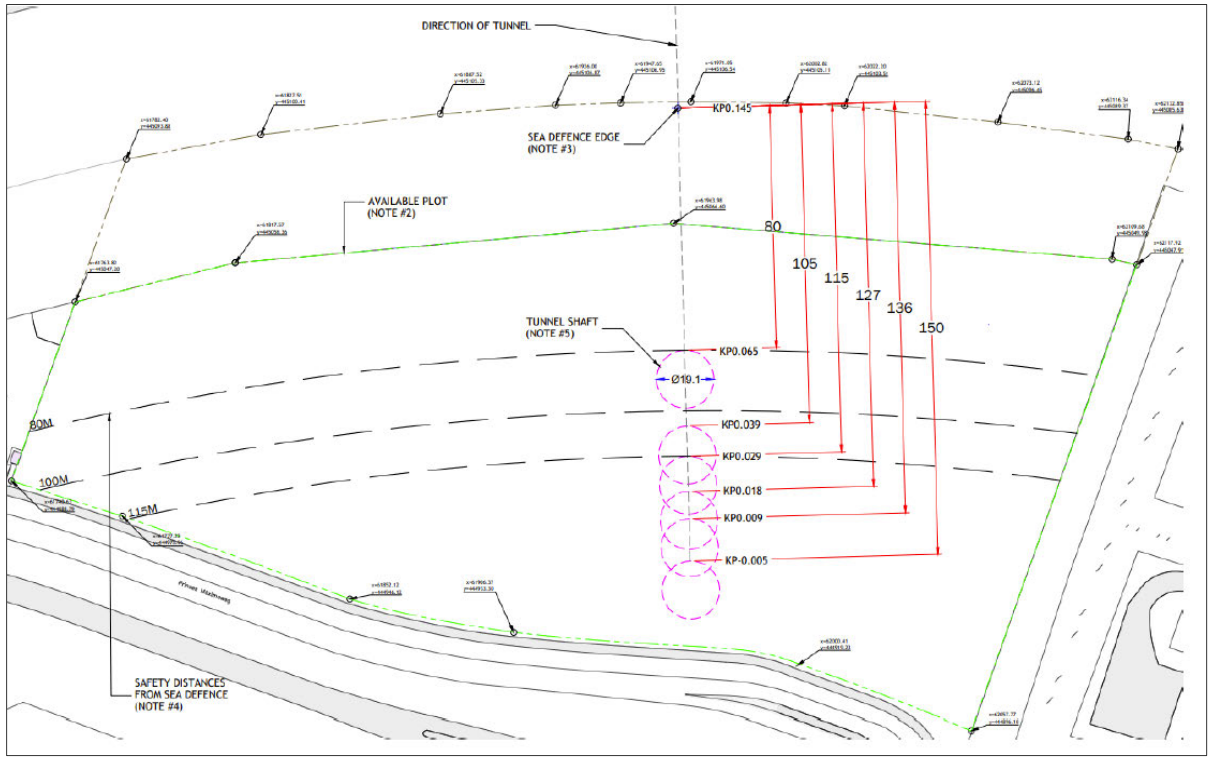
Mitigations:

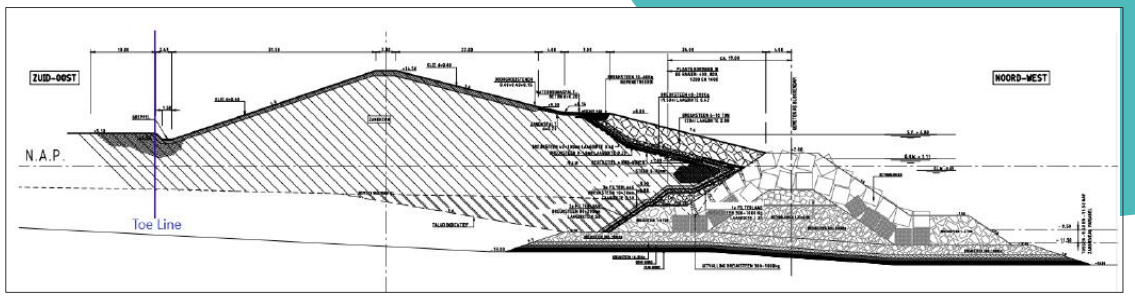
- The shaft for the tunnel construction has been moved to the south at 127 m (+ 47 m);
- Deltares has been approached to execute an assessment to analyze the geotechnical impact in case of a leak with CO2 Dense Phase;
- The wall thickness for the Onshore pipeline and valve station has not been reduced south of the landfall but will be based on the higher requirements according to the HBOR;

Aramis Landfall - Sea Defence

Landfall Design- Pipeline design crossing sea Defence according NEN 3651

Safety zone determination:





Aramis Landfall - Sea Defence

Landfall Design- Pipeline design crossing sea Defence according NEN 3651

Wall thickness determination:

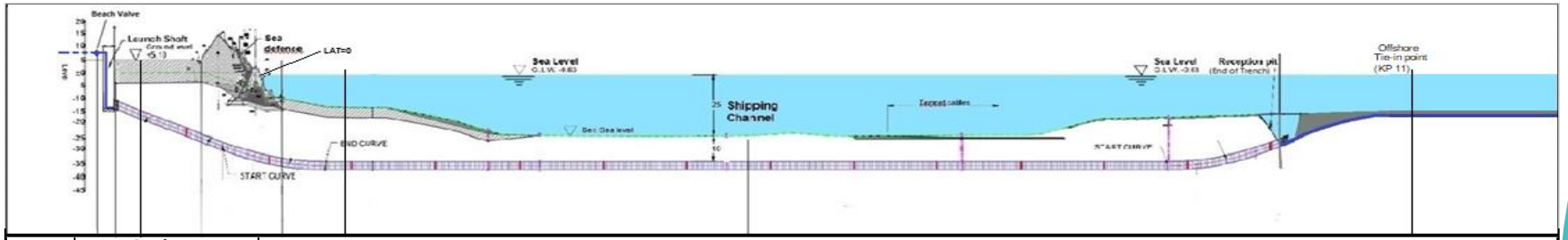
- The wall thickness for the landfall will be 35.3 mm (including 3 mm corrosion allowance and 1 mm production allowance

Hydrostatic test landfall:

- Strength test for minimum 15 min.: 371 barg;
- Tightness Test for minimum 24 hours: 220 barg

Aramis Landfall - Sea Defence

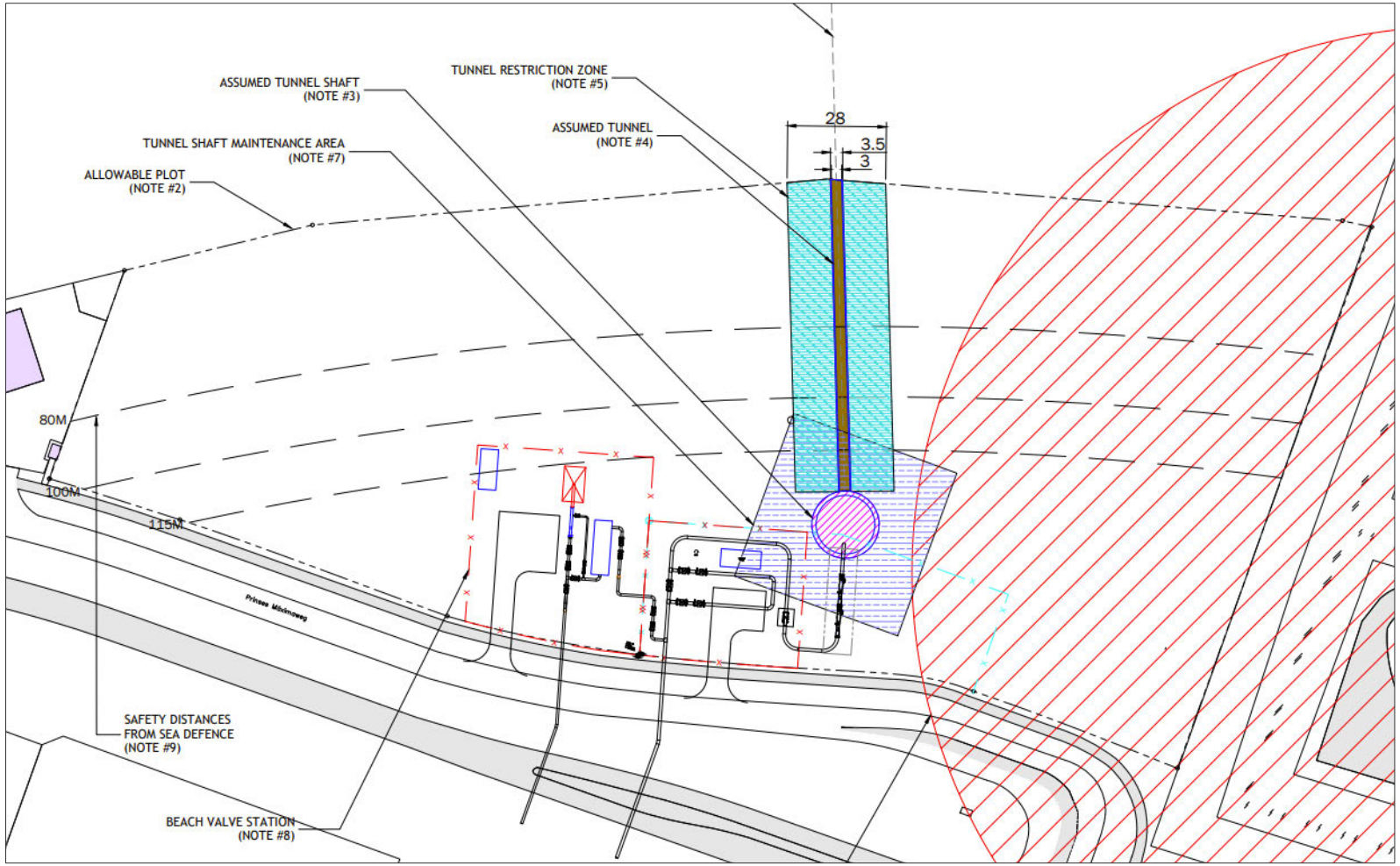
Landfall Design- Pipeline design: summery design



NEN Code Break	**Onshore**				**Offshore**		
	Landfall						
	Normal Zone	Safety Zone of Sea Defence	Sea Defence	Safety Zone of Sea Defence	Normal Zone		
DNV Code Break	**Onshore**	**Offshore**					
		High Safety Zone (Zone II-500 m)			Medium Safety Zone (Zone I)		
Applicable Codes	NEN 3650, GS GS EP PLR 104 & HBOR	NEN 3650-1 , 2 & DNV-ST-F101, GS EP PLR 100	NEN 3650-1&2 and NEN 3651, DNV-ST-F101, GS EP PLR 100	NEN 3650-1&2, NEN 3651, NEN 3656, DNV-ST-F101, GS EP PLR 100	NEN 3656, GS EP PLR 100, DNV-ST-F101		
Minimum WT based on NEN Codes (mm)	35.8	25.2	31.3		25.2		
Minimum WT based on DNV Codes (mm)		27.6			23.9		
Minimum WT +CA + Fab. Tol. (mm)	39.8	31.6	35.3	31.6	29.2		
WT selected (mm)	**39.8**	**35.3**					**29.2 (Till KP 98)**

Aramis Landfall - Sea Defence

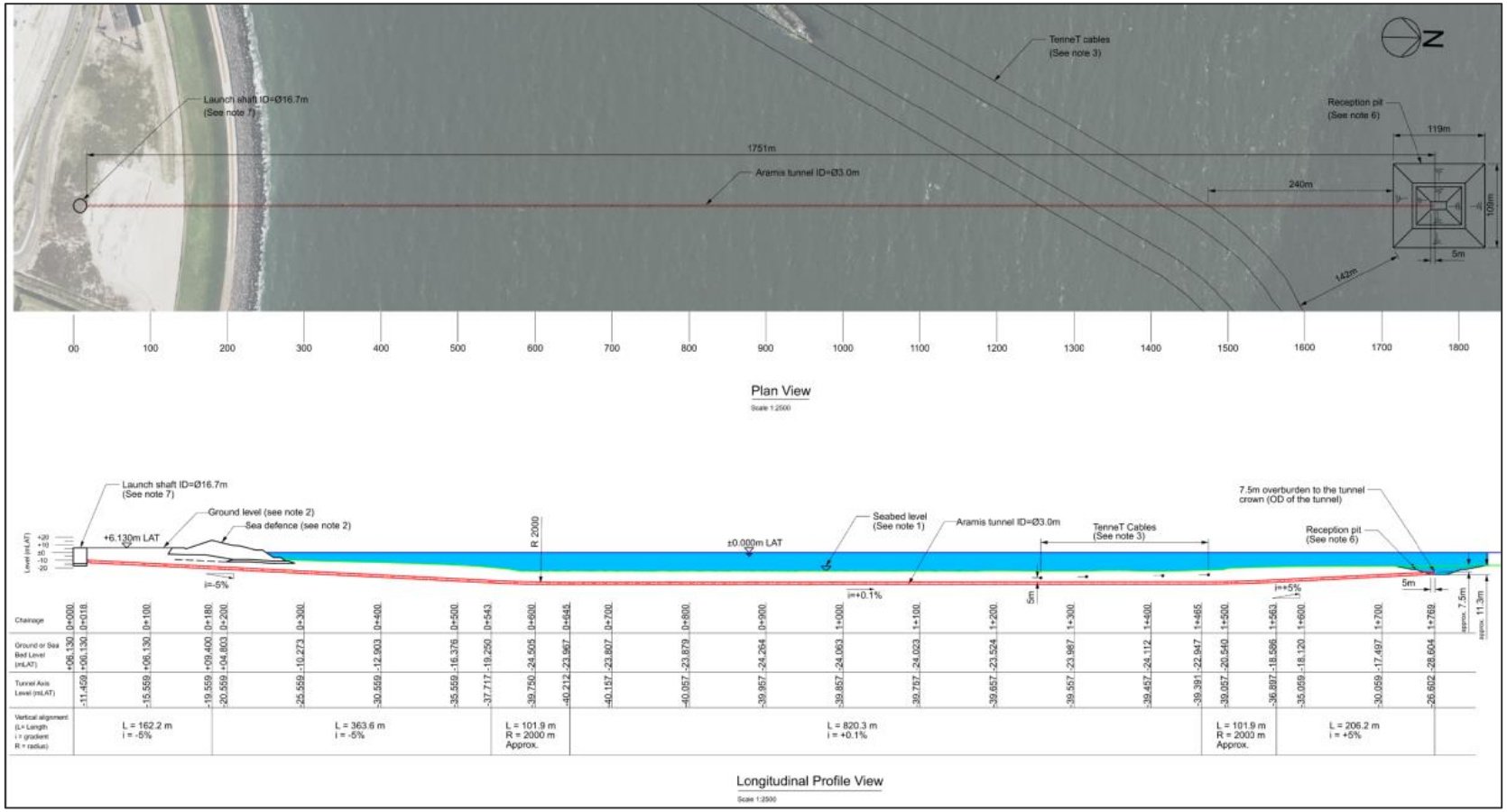
Landfall Design lay-out Shark Fin



Aramis Landfall - Sea Defence

Tunnel Design

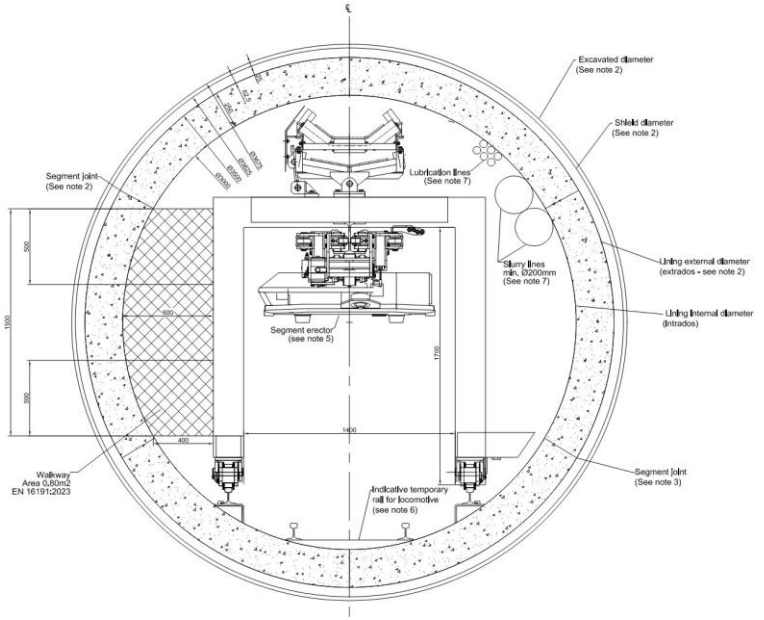
- Trajectory

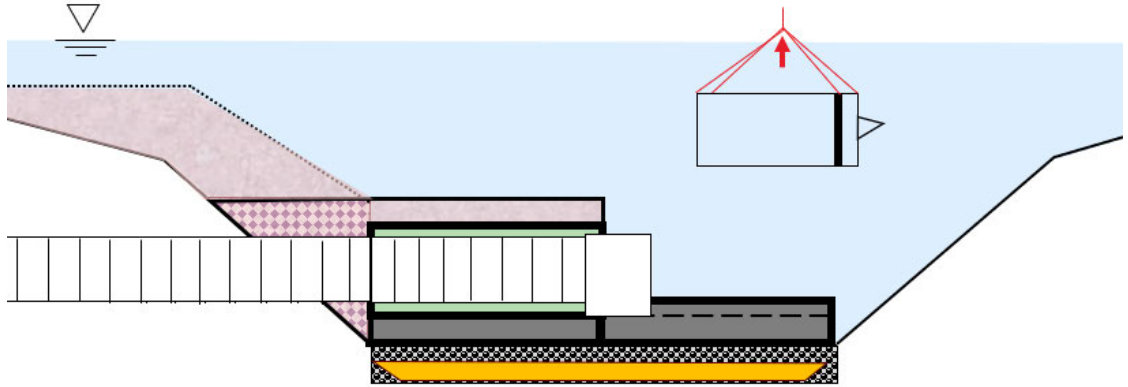


Aramis Landfall - Sea Defence

Tunnel Design

- 1.7 km tunnel under the seabed (shipping channel)
- Internal diameter 3.0m;
- Ground conditions: brown/grey gravelly silty SAND
- Hybrid excavation method:
  - Pipejacking + segmental lining tunnel;
- Max overburden range around 13m - 35m;
- 1 launch shaft (~17m ID) @ 20m depth.
- 1 reception pit for the TBM retrieval offshore

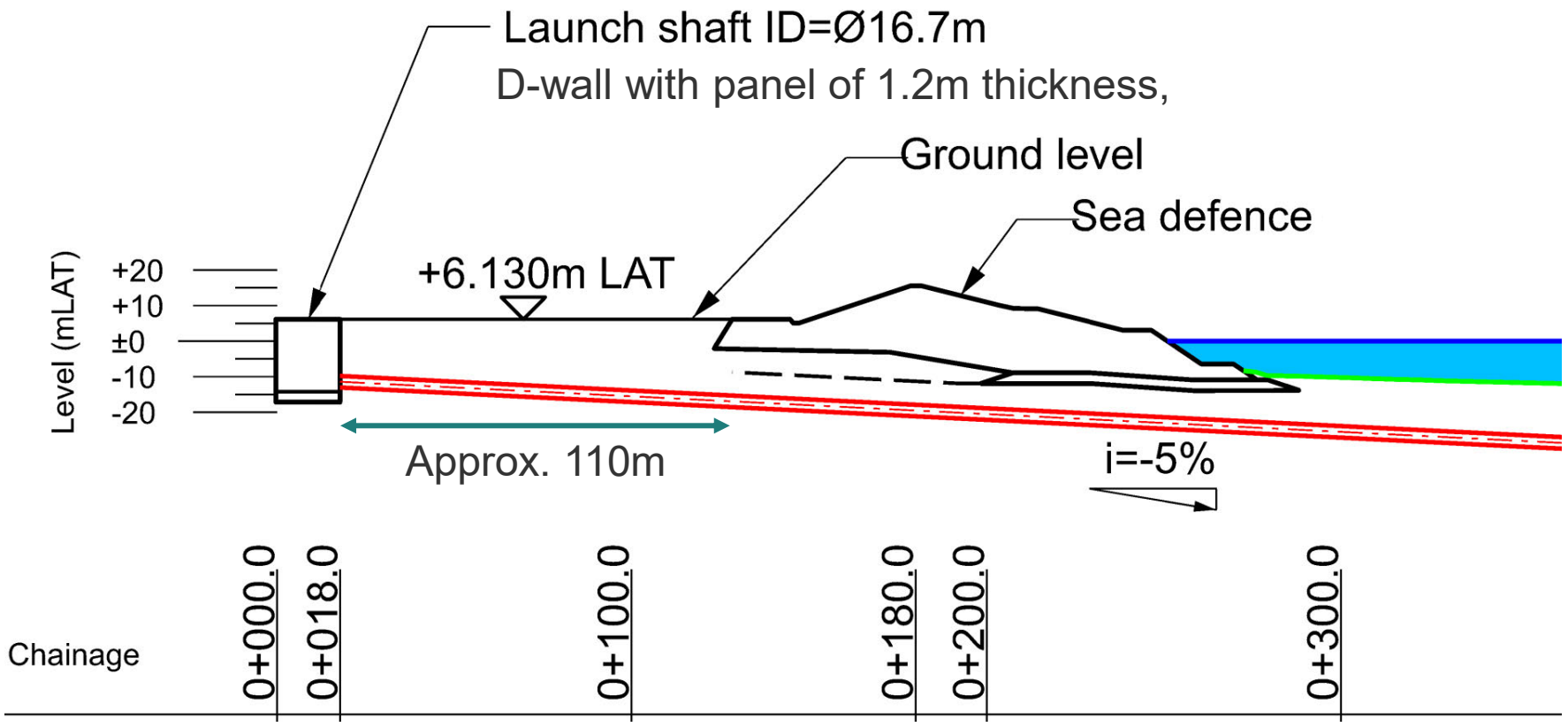




Aramis Landfall - Sea Defence

Tunnel Design

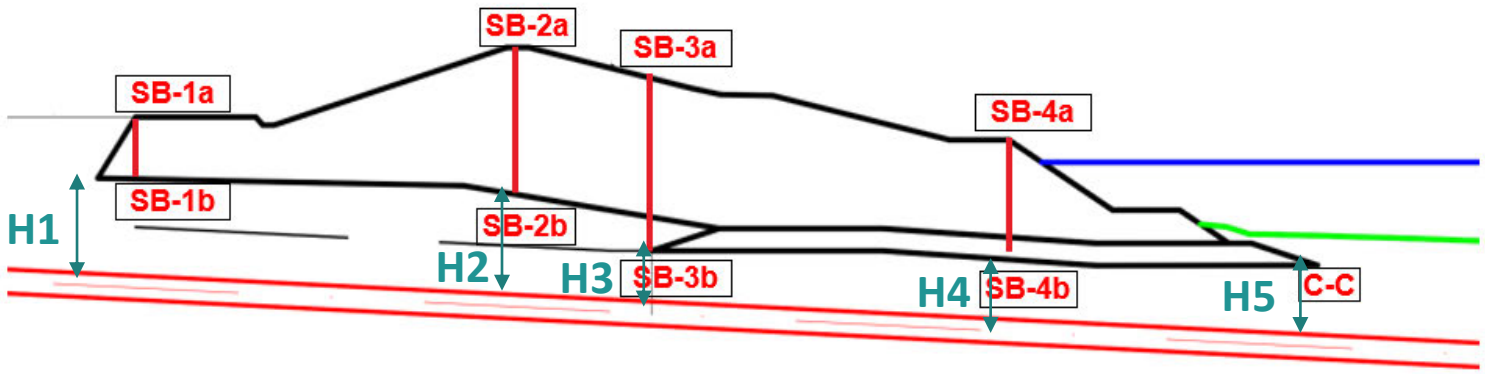
- Launch Shaft in relation to Sea Defence



Aramis Landfall - Sea Defence

Tunnel Design

- Tunnel in relation to Sea Defence



	Overburden from the sea defence base to the tunnel crown [m]
H1	12.8
H2	13.5
H3	6.7
H4	7.8
H5	9.2

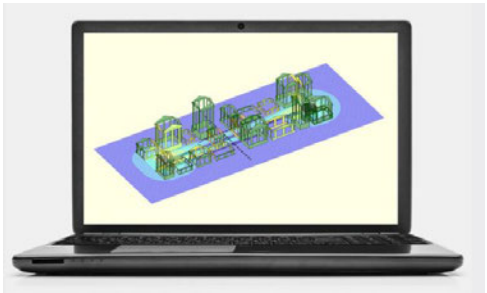
Aramis Landfall - Sea Defence

Xdisp Software

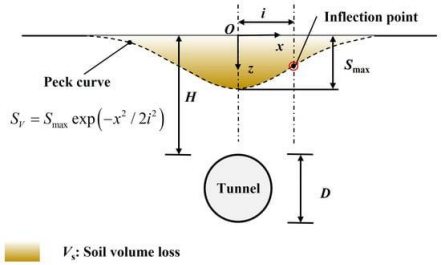
- Settlement analysis - Sea defence

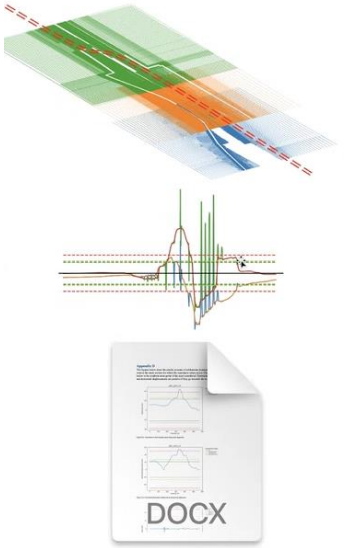
Calculation Models → Analytical solutions for settlement analysis → Data analysis → Output plot

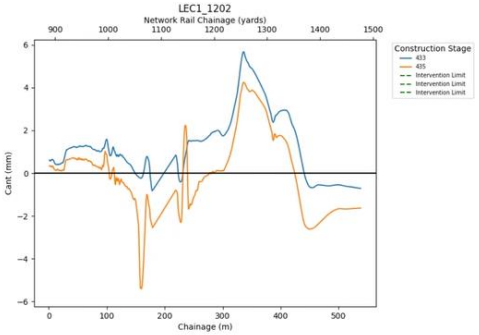
XDisp



Oasys







Settlement analysis

- Analytical solution
- Assessment of the ground movements based on analytical solutions.
- Volume Loss (sensitivity analysis): 1%, 2%, 3% and tunnel excavation diameter 3.755m (Internal diameter 3.0m)
- The cross-sectional shape of the settlement trough is assumed to correspond approximately to an inverted normal Gaussian distribution form, as proposed by **Peck (1969).**
- **O'Reilly and New (1982)** proposed the empirical formula between settlement trough width and the tunnel depth for cohesive soil and sand.

$$s = s_{\max} \exp\left(\frac{-x^2}{2i^2}\right)$$

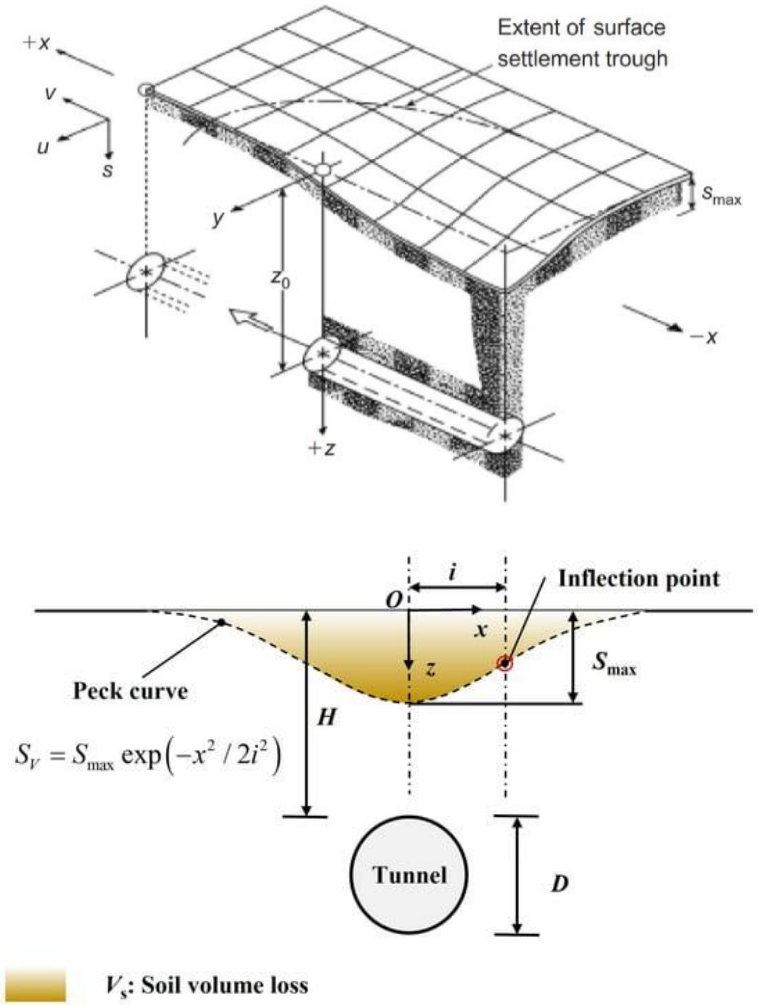
$$V_s = \sqrt{2\pi} i s_{\max}$$

$$i = K \cdot Z_0$$

Clays K=0.4-0.6 (average 0.5)
Sands K=0.25-0.45 (average 0.35)

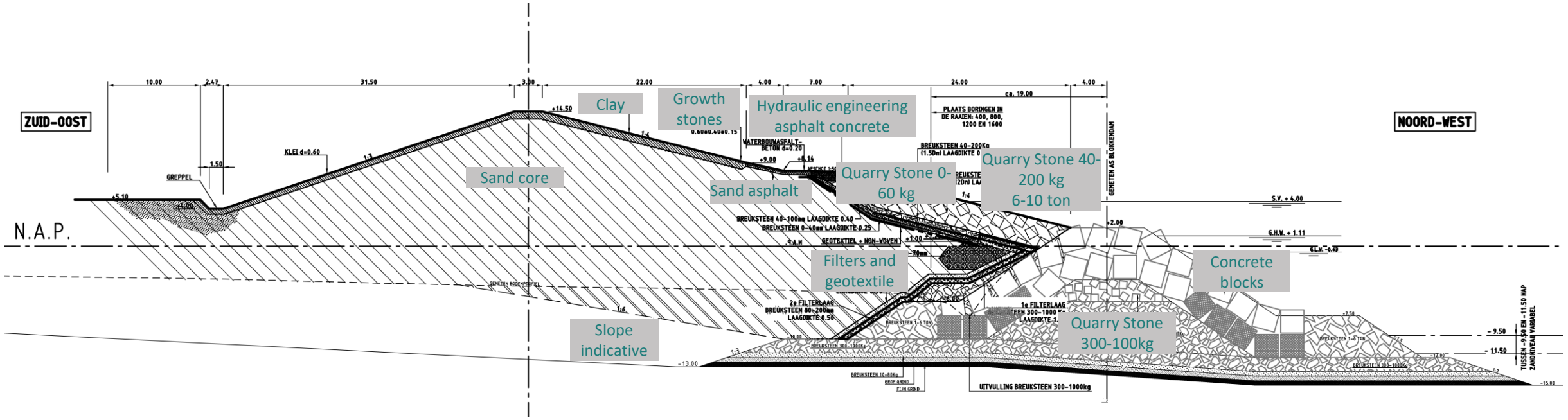
Cohesive soils: $i = 0.43z_0 + 1.1$ metres
Granular soils*: $i = 0.28z_0 - 0.12$ metres

(O'Reilly and New, 1982)



Settlement analysis

- Settlement analysis - Sea defence



Seadefence properties			
Elastic modulus of the made ground/seadefence	E	30	MPa
Poisson modulus of the made ground/seadefence	ν	0.25	-
Shear modulus of the made ground/seadefence	G	18.75	MPa

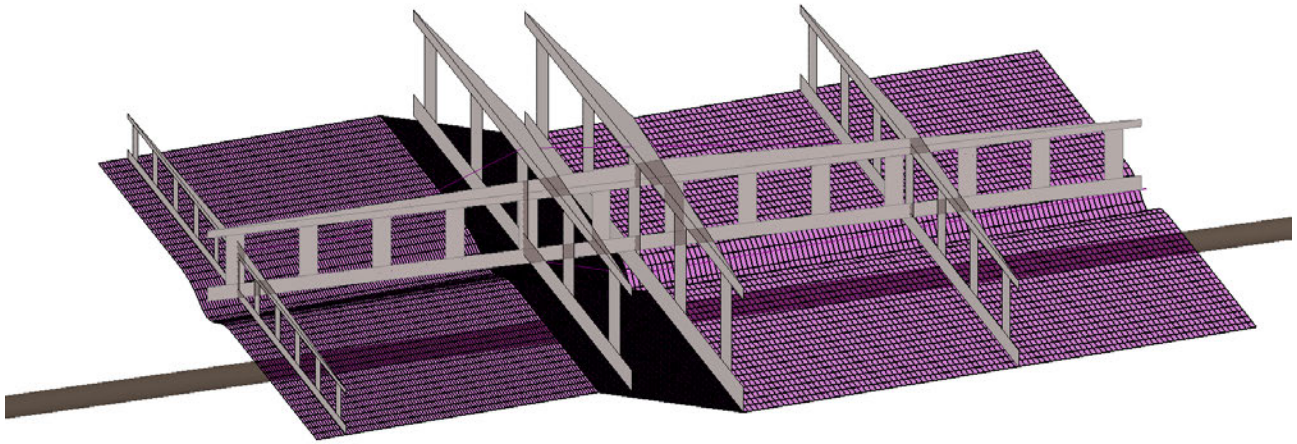
$$E = 2G(1+\nu)$$

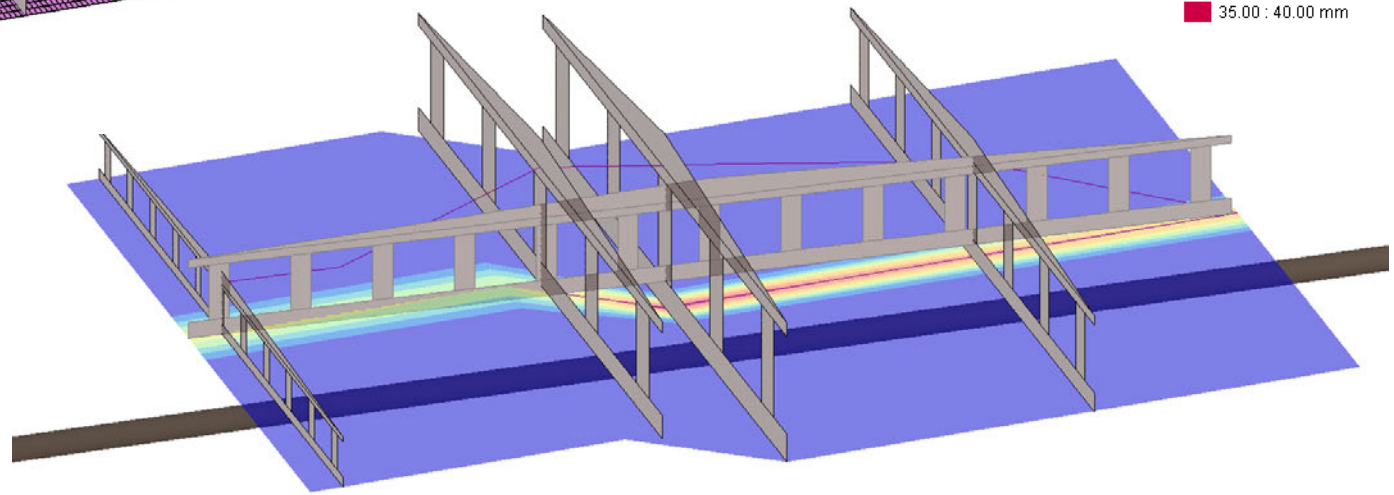
Note: Sea defence analysed as a mass and the mechanical parameters has been assumed in according to the properties of the made ground material

Aramis Landfall - Sea Defence

Settlement analysis - Sea defence

- Surface of Sea Defence - Longitudinal profile displacements

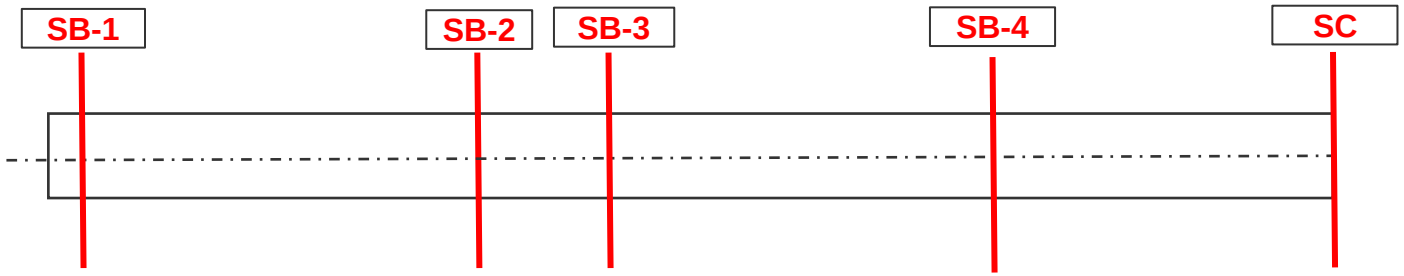




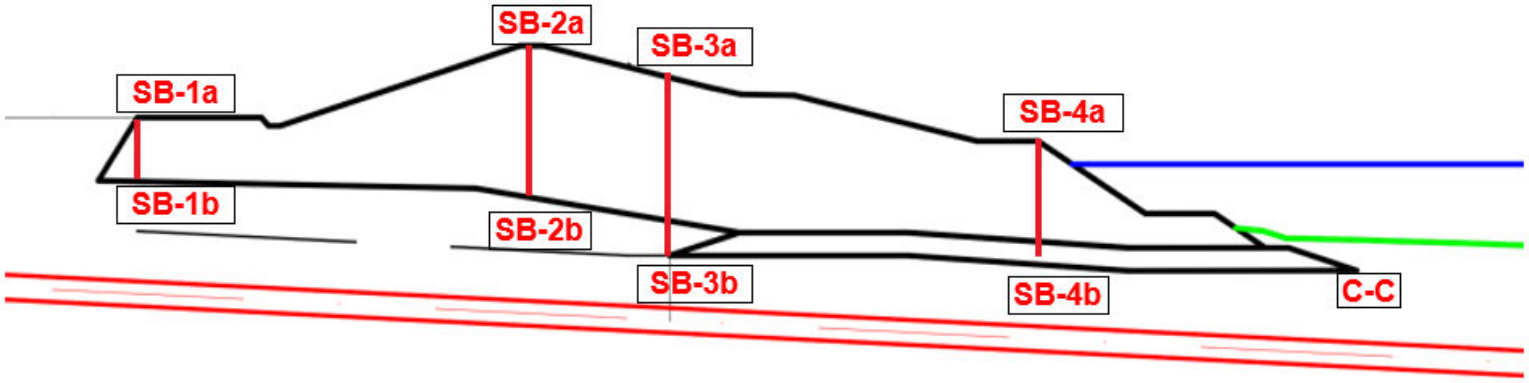
Aramis Landfall - Sea Defence

Sections analysed

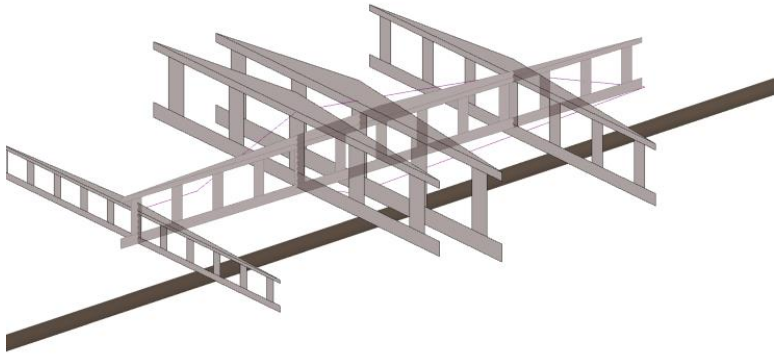
- Settlement analysis - Sea defence



Plan view



Cross section

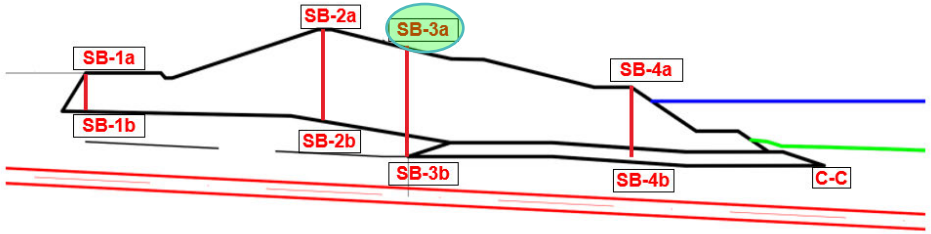


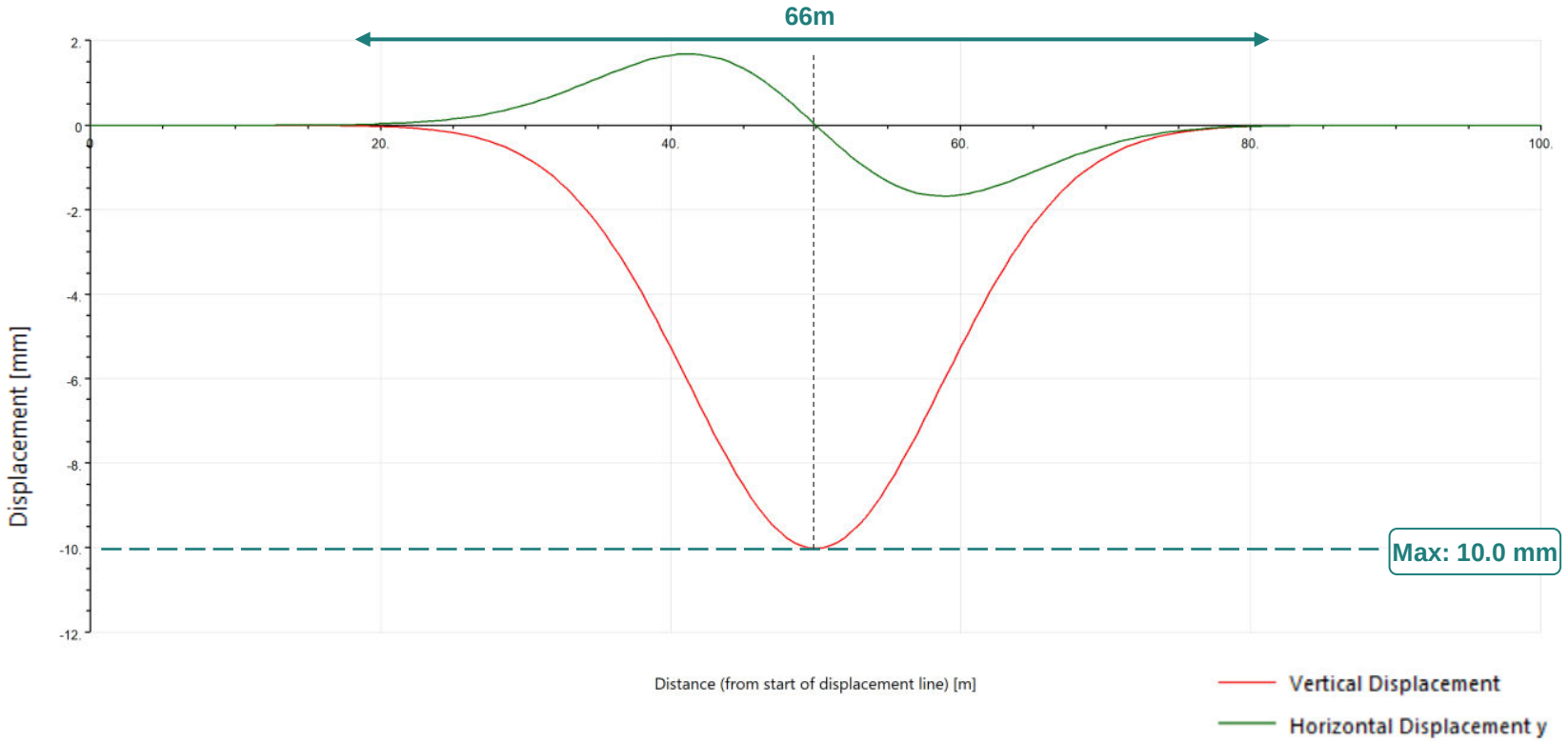
3D Model Along Sea Defence

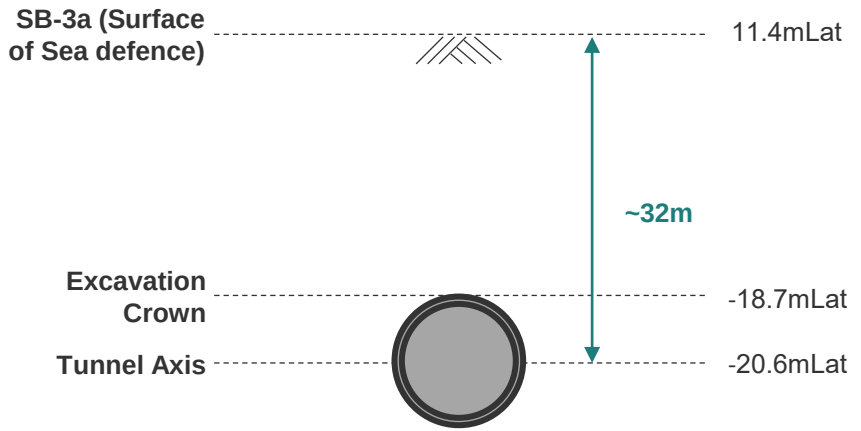
Aramis Landfall - Sea Defence

Sea Defence Results

- Section SB-3a: Surface Sea Defence



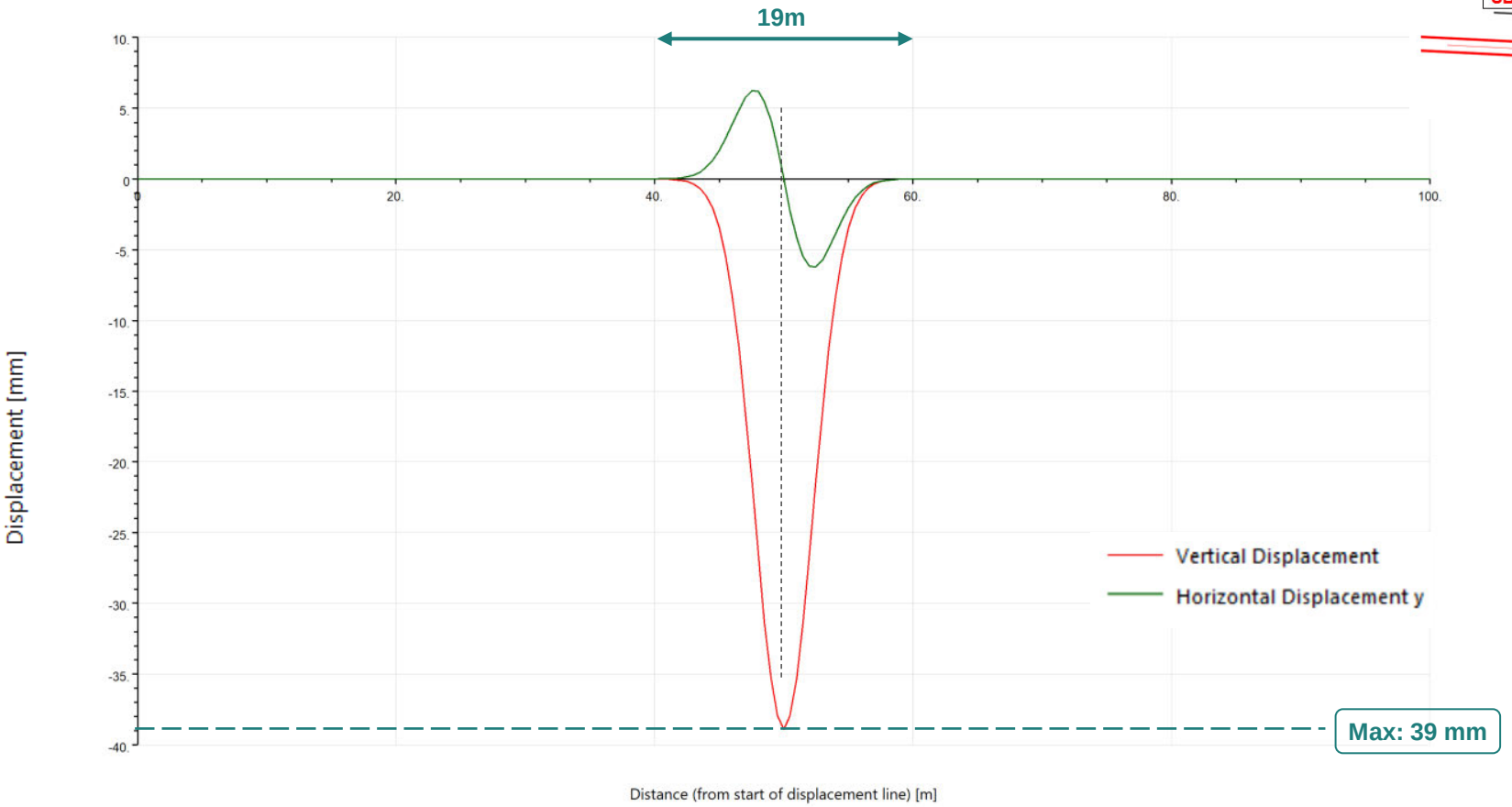


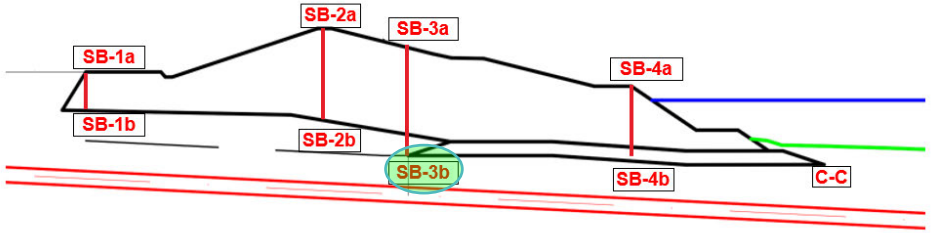


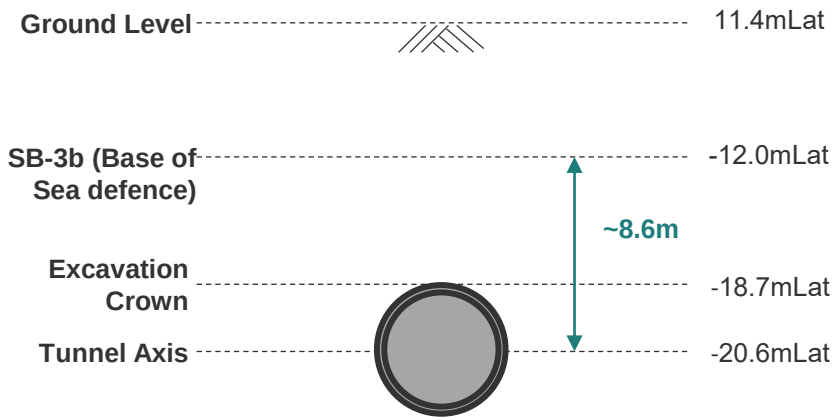
Aramis Landfall - Sea Defence

Sea Defence Results

- Section SB-3b: Base of Sea Defence



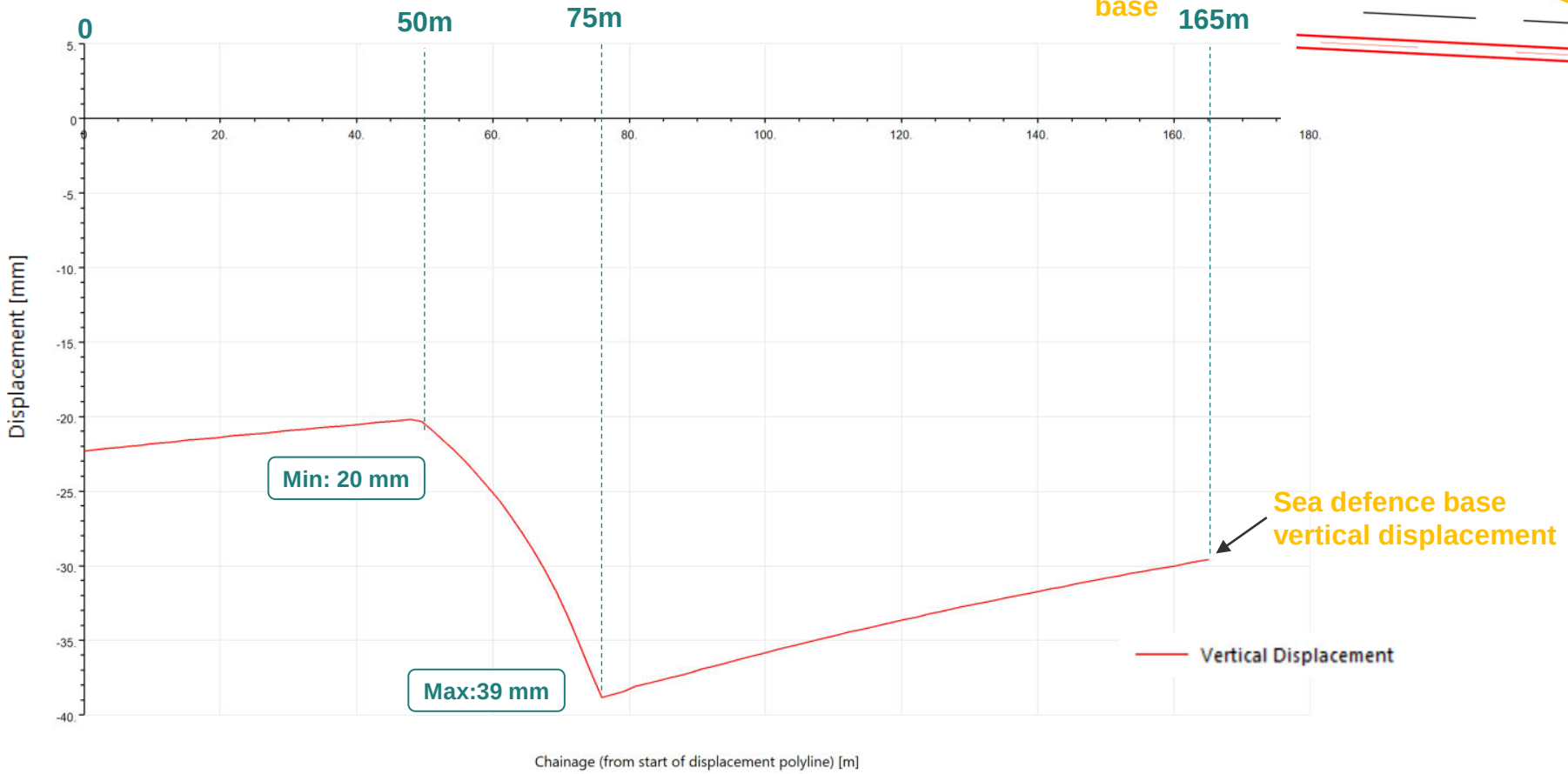


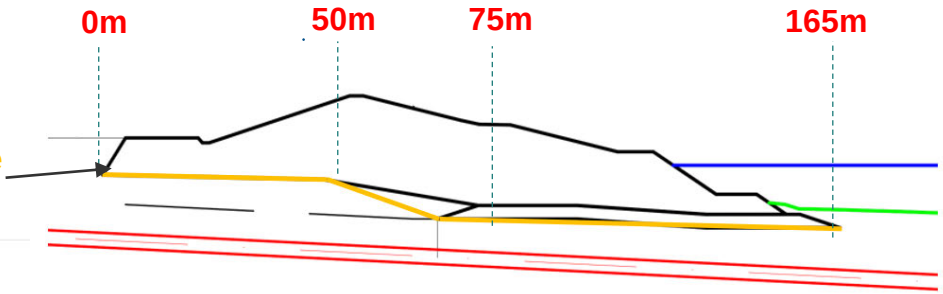


Aramis Landfall - Sea Defence

Sea Defence Results

- Base of Sea Defence - Longitudinal profile displacements

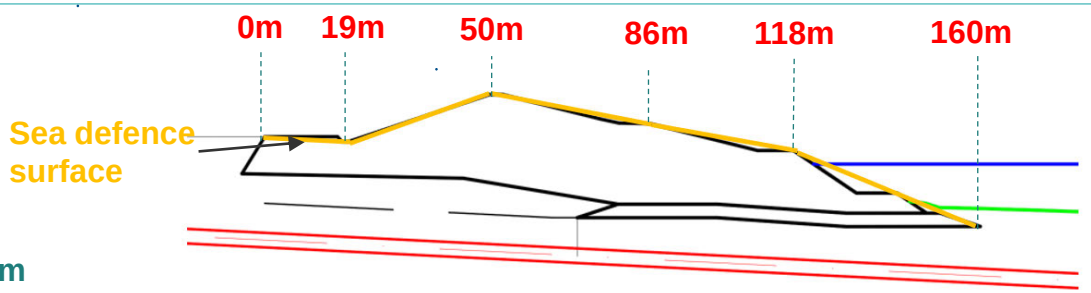


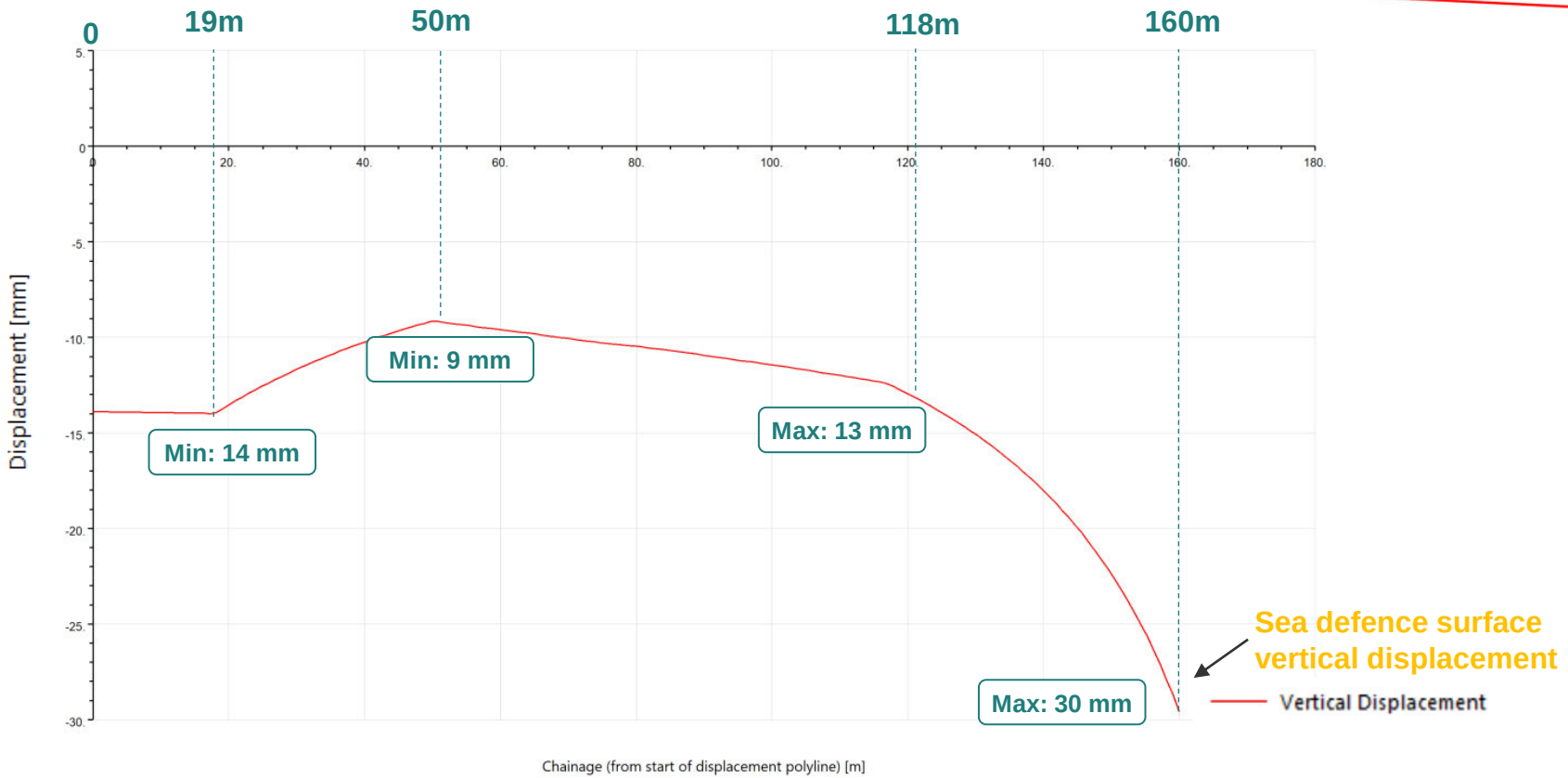


Aramis Landfall - Sea Defence

Sea Defence Results

- Base of Sea Defence - Longitudinal profile displacements





Sea Defence Results

- Base of Sea Defence - Longitudinal profile displacements
- The results of the tensile strains have been evaluated considering the damage criteria of Burland for building structures.
- Tensile strains have been estimated based on the elastic properties assumed for the sea defence.
- The category of damage could be localized between class 2 and 3 for the maximum critical tensile strain estimated.
- However, we consider that the sea defence it's a flexible structure and the tensile strain obtained from the analysis are very small respect the maximum limits expected.

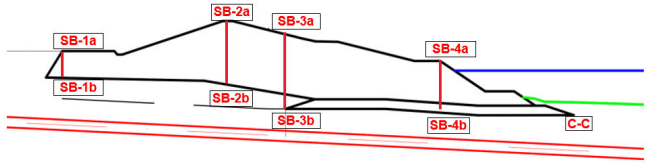
Building and Structure Classification (after Burland et al, 1977 and Boscardin and Cording, 1989)				
Category of Damage	Degree of Severity	Description of Typical Damage and Repair	Approx Crack Width (mm)	Limiting Tensile Strain, ε_{lim} (%)
0	Negligible	Hairline cracks	<0.1	0-0.05
1	Very Slight	Fine cracks that are easily treated during normal decoration. Damage generally restricted to internal wall finishes. Cracks may be visible on external brickwork or masonry.	0.1 to 1	0.05-0.075
2	Slight	Cracks easily filled. Redecoration probably required. Recurrent cracks can be masked by suitable linings. Crack may be visibly externally and some repointing may be required to ensure weather-tightness. Doors and windows may stick slightly.	1 to 5	0.075-0.15
3	Moderate	The cracks require some opening up and can be patched by a mason. Repointing of external brickwork and possibly a small amount of brickwork to be replaced. Doors and window sticking. Service pipes may fracture. Weather-tightness often impaired.	5 to 15 or several (>3)	0.15-0.30
4	Severe	Extensive repair work involving break-out and replacing sections of walls, especially over doors and windows. Windows and door frames distorted, floor sloping noticeably[1]. Walls leaning or bulging noticeably; some loss of bearing in beams. Utilities disrupted.	15 to 25 but also depends on number of cracks	>0.3
5	Very Severe	This requires a major repair job involving partial or complete rebuilding. Beams lose bearing; walls lean badly and require shoring. Windows broken with distortion. Danger of instability.	>25 but also depends on number of cracks	>0.3

Assessment of risk of damage to buildings due to tunnelling and excavation (Burland,1995)

Sea Defence Results

- Summary table – Volume loss VL=2%

Sea defence – Ground movements			
Section	**Vertical displacement [mm]**	**Angular distortion range**	**Maximum tensile strain [%]**
Transversal sections			
SB-1a	14	1/1799	0.05
SB-1b	22	1/727	0.10
SB-2a	9	1/3846	0.02
SB-2b	21	1/782	0.07
SB-3a	10	1/3300	0.03
SB-3b	39	1/244	0.27
SB-4a	12	1/2177	0.05
SB-4b	34	1/325	0.20
S C-C	30	1/400	0.30
Longitudinal section			
Base	39	1/1316	-
Surface	30	1/2471	-



Traversal sections

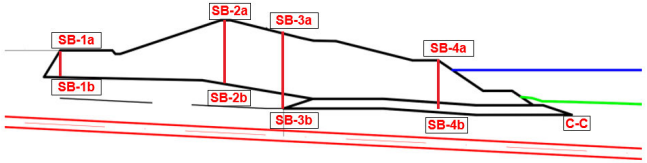


Longitudinal Sections

Sea Defence Results

- Summary table – Volume loss VL=3%

Sea defence – Ground movements			
Section	**Vertical displacement [mm]**	**Angular distortion range**	**Maximum tensile strain [%]**
Transversal sections			
SB-1a	21	1/1190	0.07
SB-1b	33	1/500	0.15
SB-2a	14	1/2643	0.03
SB-2b	32	1/516	0.11
SB-3a	15	1/2267	0.05
SB-3b	58	1/164	0.41
SB-4a	19	1/1556	0.07
SB-4b	51	1/216	0.30
S C-C	44	1/284	0.30
Longitudinal section			
Base	58	1/926	-
Surface	44	1/1680	-



Traversal sections



Longitudinal Sections

Comparison between FEED and Pre-FEED

- Settlement analysis - Assumptions

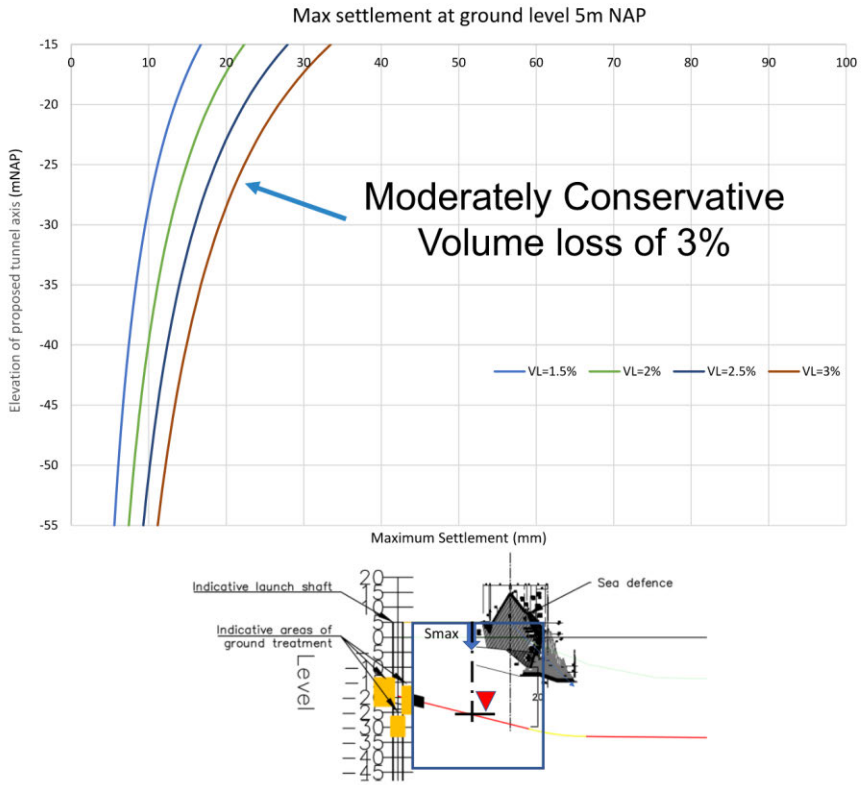
1. Both settlement assessments are based on analytical solutions and similar methods: Mair (2013), Peck (1969), O'Reilly and New (1982).
2. Similar calculation assumptions have been considered for the FEED and Pre-FEED:
    - In similar ground conditions, the expected or likely design range for volume loss, VL, would be between 1.0 and 3.0%. At the Pre-FEED several volume losses has been considering, however VL=3% has been considered the moderately conservative case for the project.
    - K, trough width parameter. It varies from 0.25 to 0.45 for granular soils and its function of the vertical location. Both cases has considered similar values for the analysis. Pre-FEED **k=0.2** and FEED mean value around **k=0.3** (function of the vertical location).
    - Different section have been analysed along the sea defence with similar results for the same volume loss and similar distances between the tunnel crown and the sea defence.
3. The structure properties and current conditions of the sea defence have been assumed. This information needs to be confirmed.

Comparison between FEED and Pre-FEED

- Settlement analysis – Vertical displacements
- **Pre-FEED**

Max settlement of restricted to 20-30mm

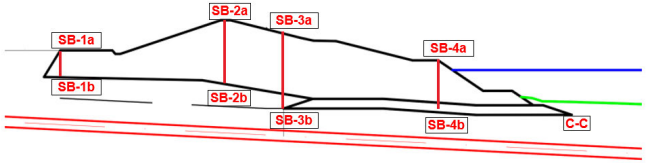
Figure 4-7 Settlement variation at 5mPD with respect to tunnel axis elevation for volume losses, V_l, of 1.5%, 2%, 2.5% and 3%



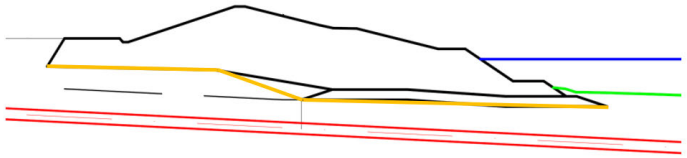
Comparison between FEED and Pre-FEED

- Settlement analysis – Vertical displacements **FEED (3%)**

Sea defence – Ground movements		
Section	**Vertical displacement range [mm]**	**Angular distortion range**
Transversal sections		
SB-1a	21	1/1190
SB-1b	33	1/500
SB-2a	14	1/2643
SB-2b	32	1/516
SB-3a	15	1/2267
SB-3b	58	1/164
SB-4a	19	1/1556
SB-4b	51	1/216
S C-C	44	1/284
Longitudinal section		
Base	58	1/926
Surface	44	1/1680



Traversal sections

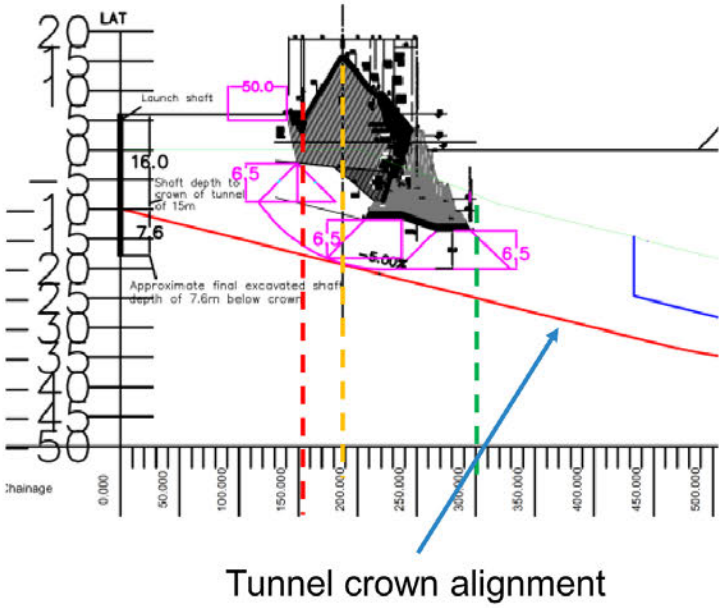


Longitudinal Sections

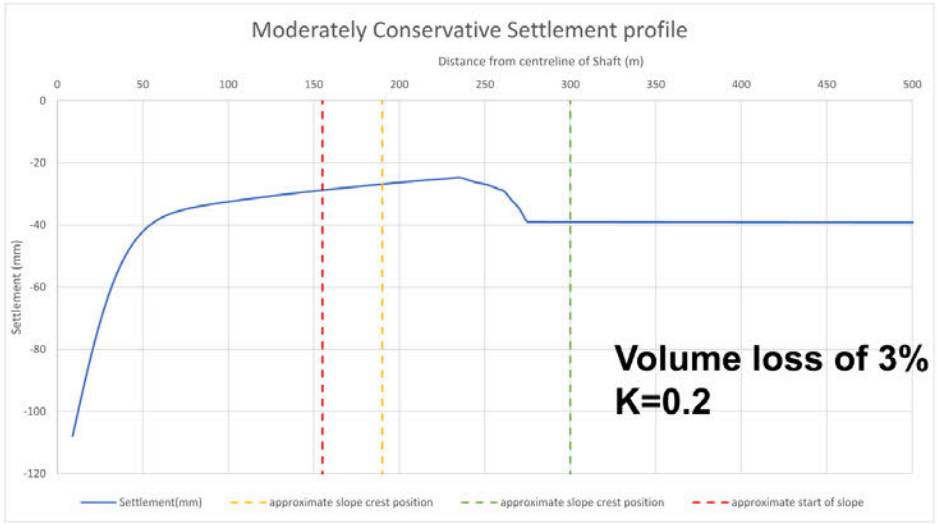
Comparison between FEED and Pre-FEED

- Settlement analysis – Vertical displacements
- **Pre-FEED**

Figure 4-11 Latest Updated Tunnel Profile - Moderately Conservative ground surface settlement profile at sea defence slope



Tunnel crown alignment

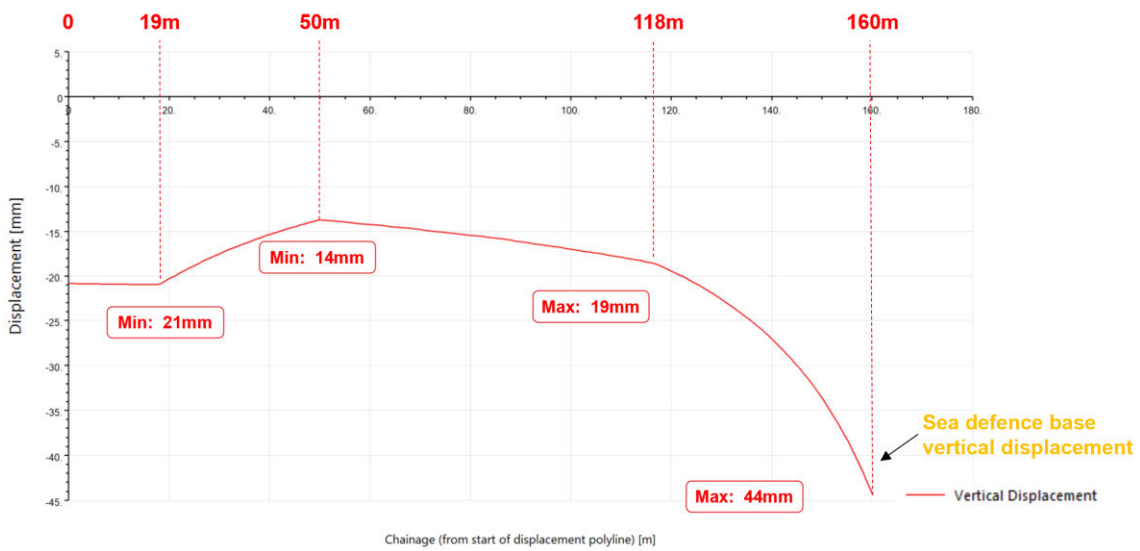


Angular distortion < 1/700

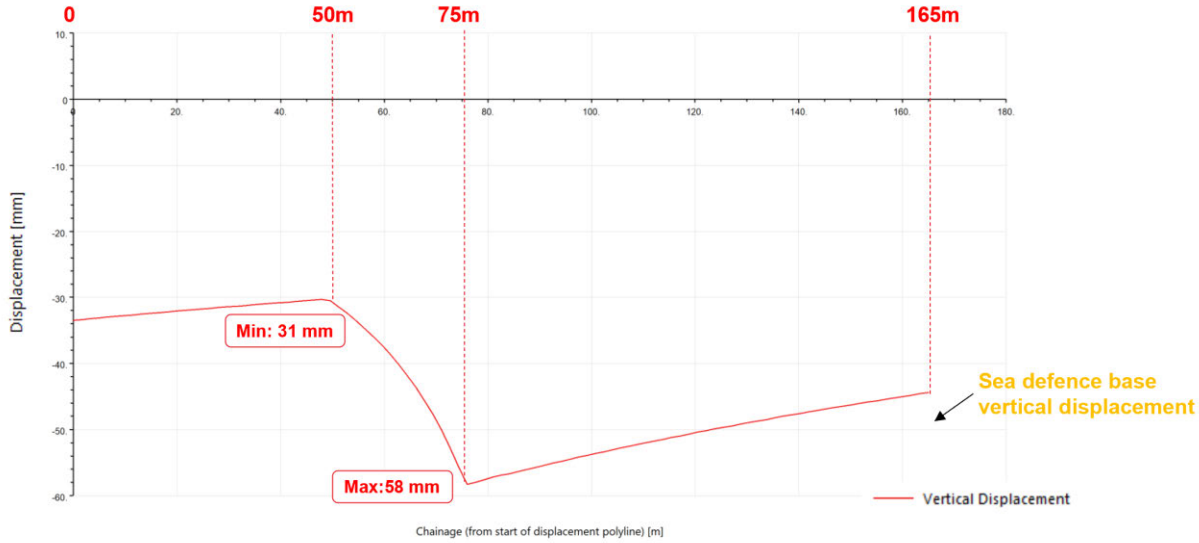
Comparison between FEED and Pre-FEED

- Settlement analysis – Angular distortion

FEED (3%)



Angular distortion < 1/1680



Angular distortion < 1/926

Construction – Preliminary schedule FEED

- Main construction activities

A. Site set up and enabling works
- Clear site, create working platform or slabs, establish compound, hoarding ,offices, facilities, power and water set up.

B. Shaft sinking *(assumed 5 day/wk. - 12 hr/day)*
- D-wall installation and ring beam
- Set up of shaft sinking and ground treatment
- Shaft sinking *(assumed 0.3 – 0.5 m/day)*
- Base slab installation and uplift mitigation works
- Shaft dressing and eye seals and pipe brake and eye grouting

C. Tunnelling (*7 day/wk. 24 hr/day)*
- Mobilise TBM & setup equipment:
- Pipejack excavation (approx. 700-1000 m, 4 pipes per day – 2m long)
- Change over to segmental method: install backup decks, rail, vent and TBM services and power
- Segmental lining excavation (approx. 800m-1000m) (segment 1m long – 6 segments/ring– learning curve 1 month – 200m , 600-800m remain – advance rate 13 to 14m per day)

Construction – Preliminary schedule FEED

- Main construction activities

D. Marine works and TBM recovery (assumed 7 day/wk. - 24 hr/day)

- Dredging the reception pit excavation (around 130000 cu m, assuming 500 cm per day, no UXO / archaeology risks has been assumed)
- Base platform installation, tube installation and rock boulders/stone bags installation
- Excavation of the last meters of tunnel
- Bulkheads installation and TBM retrieval

All necessary consents and permissions for the reception pit dredging activities and supporting borrow / disposal area activities must be obtained by OI . An allowance of 20% is made for downtime due to weather and mechanical breakdowns etc.

E. Landfall installation*(assumed 7 day/wk. - 12 hr/day)*

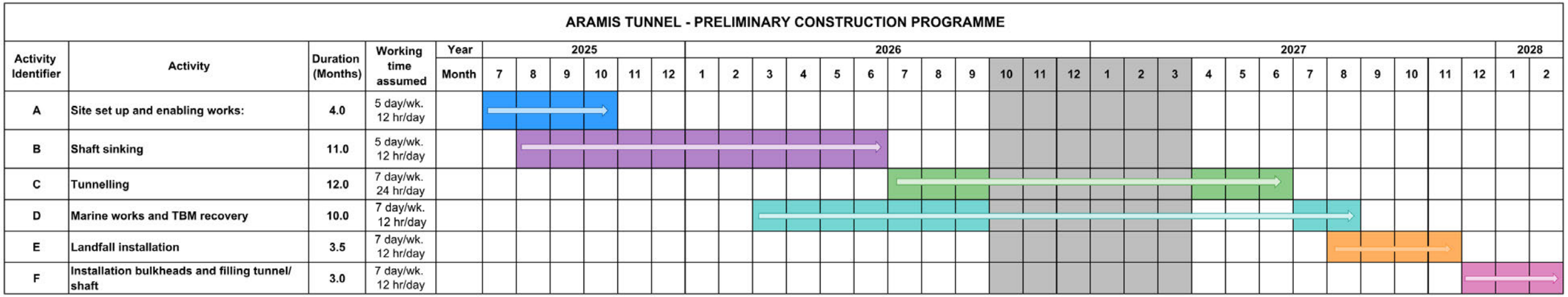
- Installation 800ton winch and sheeves
- Dredging work for pipeline installation
- Pipeline installation landfall (around 2.2 km)
- Pipeline installation until KP 11
- Close of tunnel and installation Riser
- Pre-commissioning landfall

F. Installation bulkheads and filling tunnel/ shaft *(assumed 7 day/wk. - 12 hr/day)*

- Installation bulkheads
- Filling tunnel
- Filling shaft

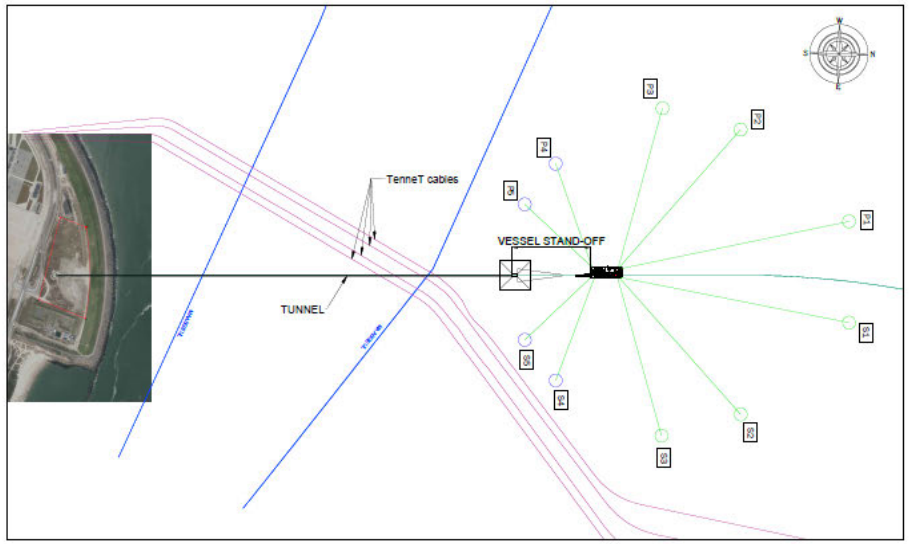
Aramis Landfall - Sea Defence

Construction – Preliminary schedule FEED

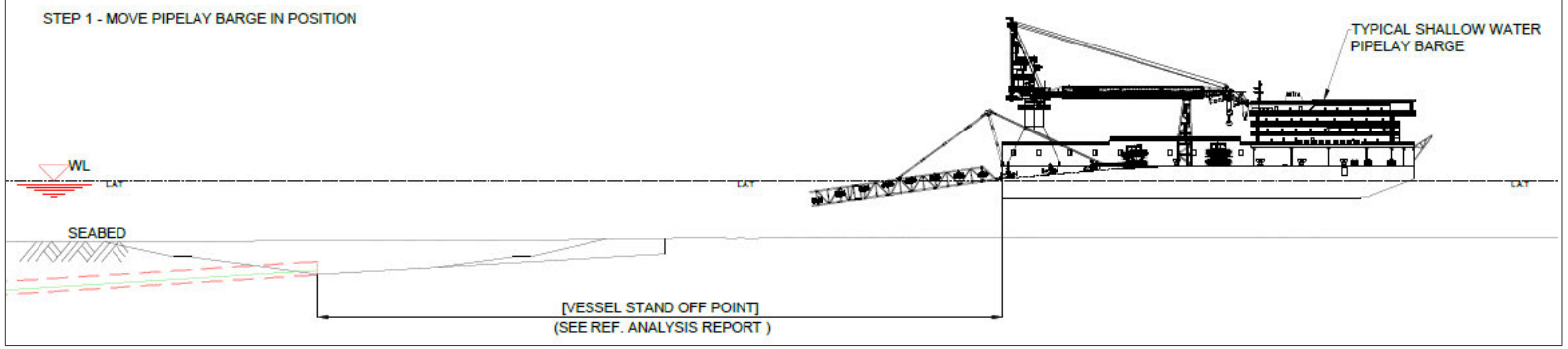


Aramis Landfall - Sea Defence

Construction- Storyline Installation Pipeline Through tunnel

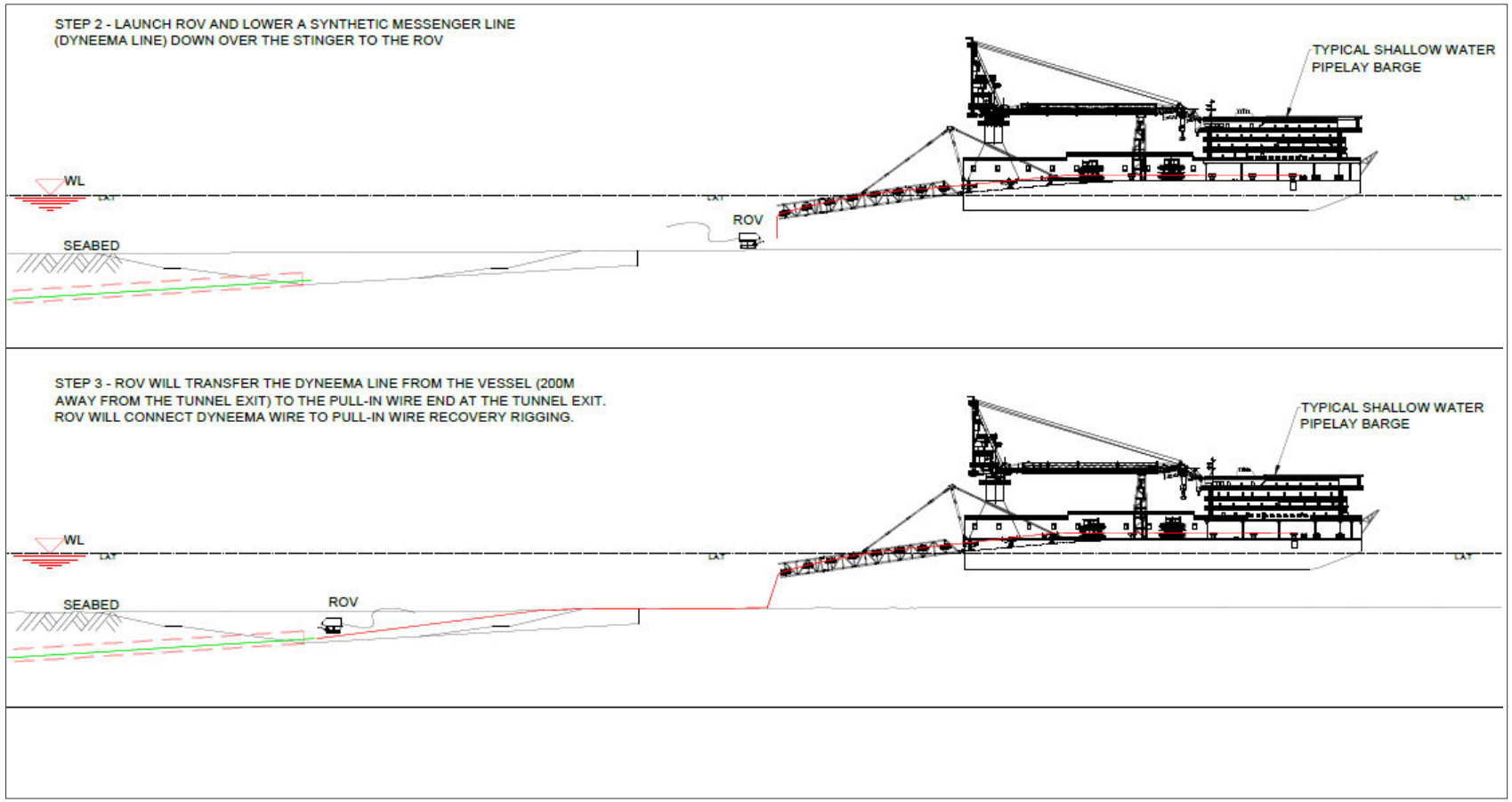


Anchor	UTM31N - ETRS89		Length [m]	Type
	Easting [m]	Northing [m]		
P1	570,638.31	5,762,995.98	891	Delta
P2	570,299.39	5,762,530.25	699	Delta
P3	570,226.42	5,762,227.27	627	Delta
P4	570,448.80	5,761,822.65	417	Suction Pile
P5	570,606.79	5,761,708.16	390	Suction Pile
S1	571,026.06	5,762,966.54	895	Delta
S2	571,385.77	5,762,559.97	701	Delta
S3	571,675.25	5,762,257.37	628	Delta
S4	571,275.31	5,761,845.28	423	Suction Pile
S5	571,170.87	5,761,727.32	394	Suction Pile



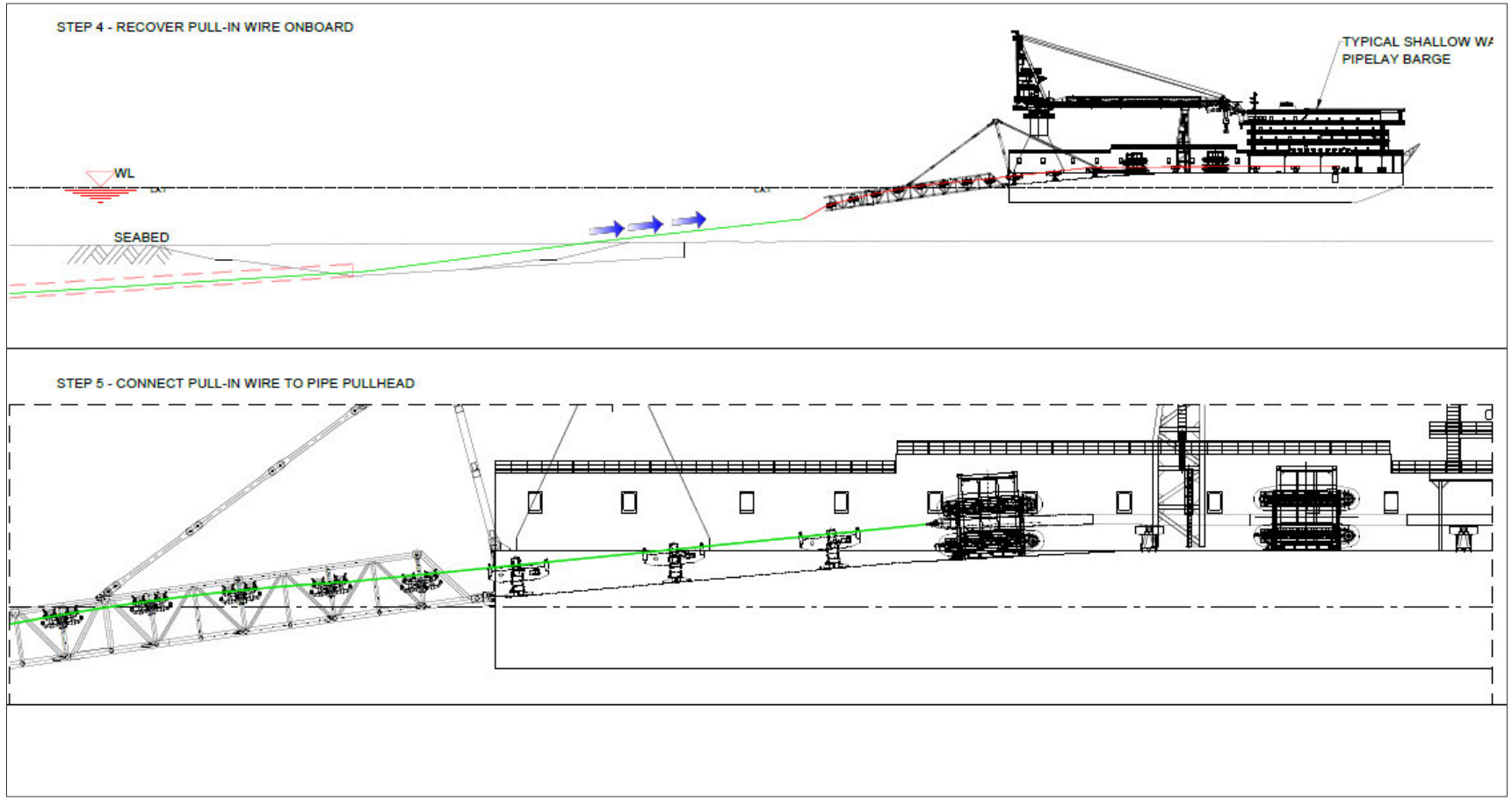
Aramis Landfall - Sea Defence

Construction- Storyline Installation Pipeline Through tunnel



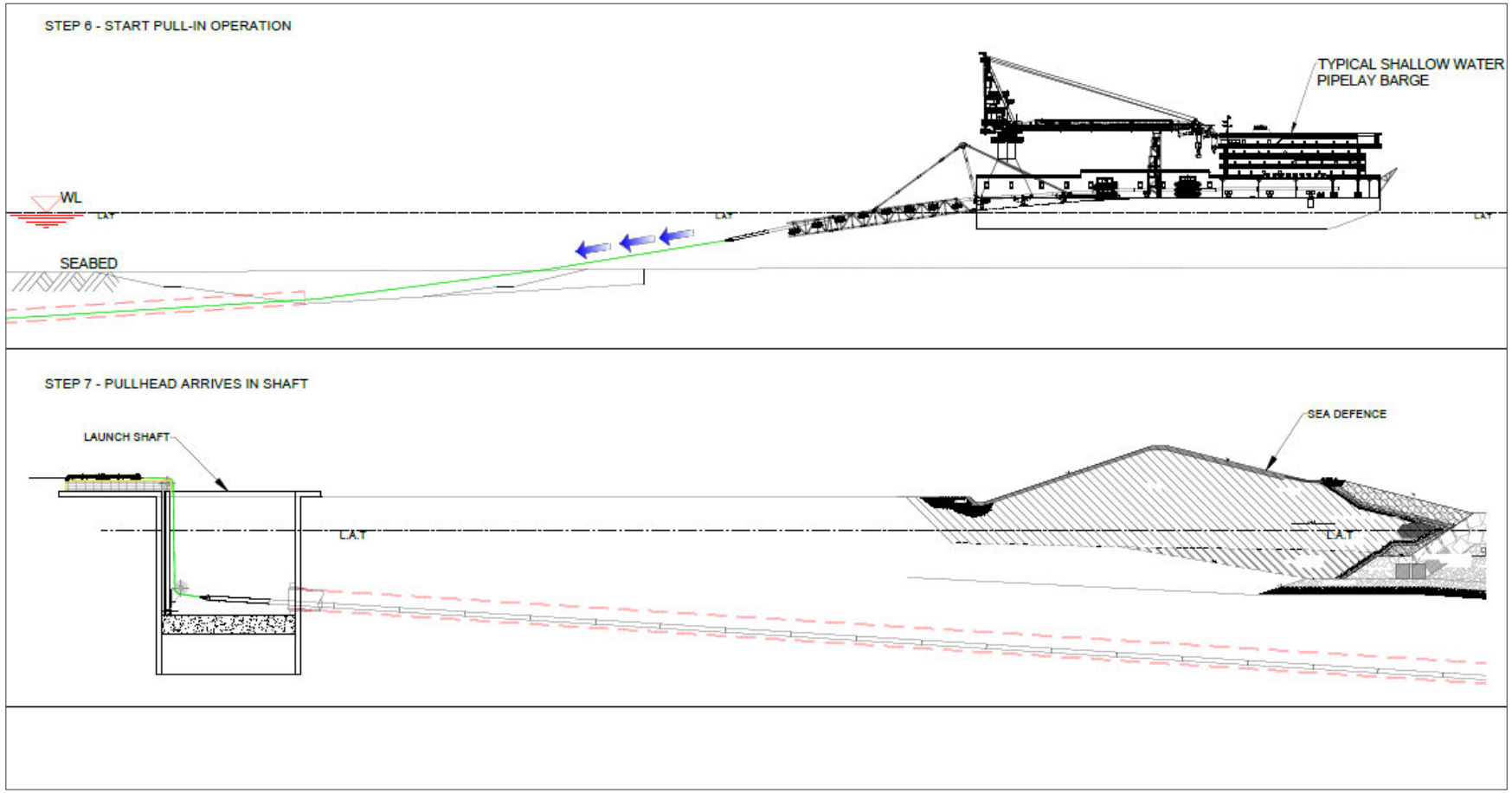
Aramis Landfall - Sea Defence

Construction- Storyline Installation Pipeline Through tunnel



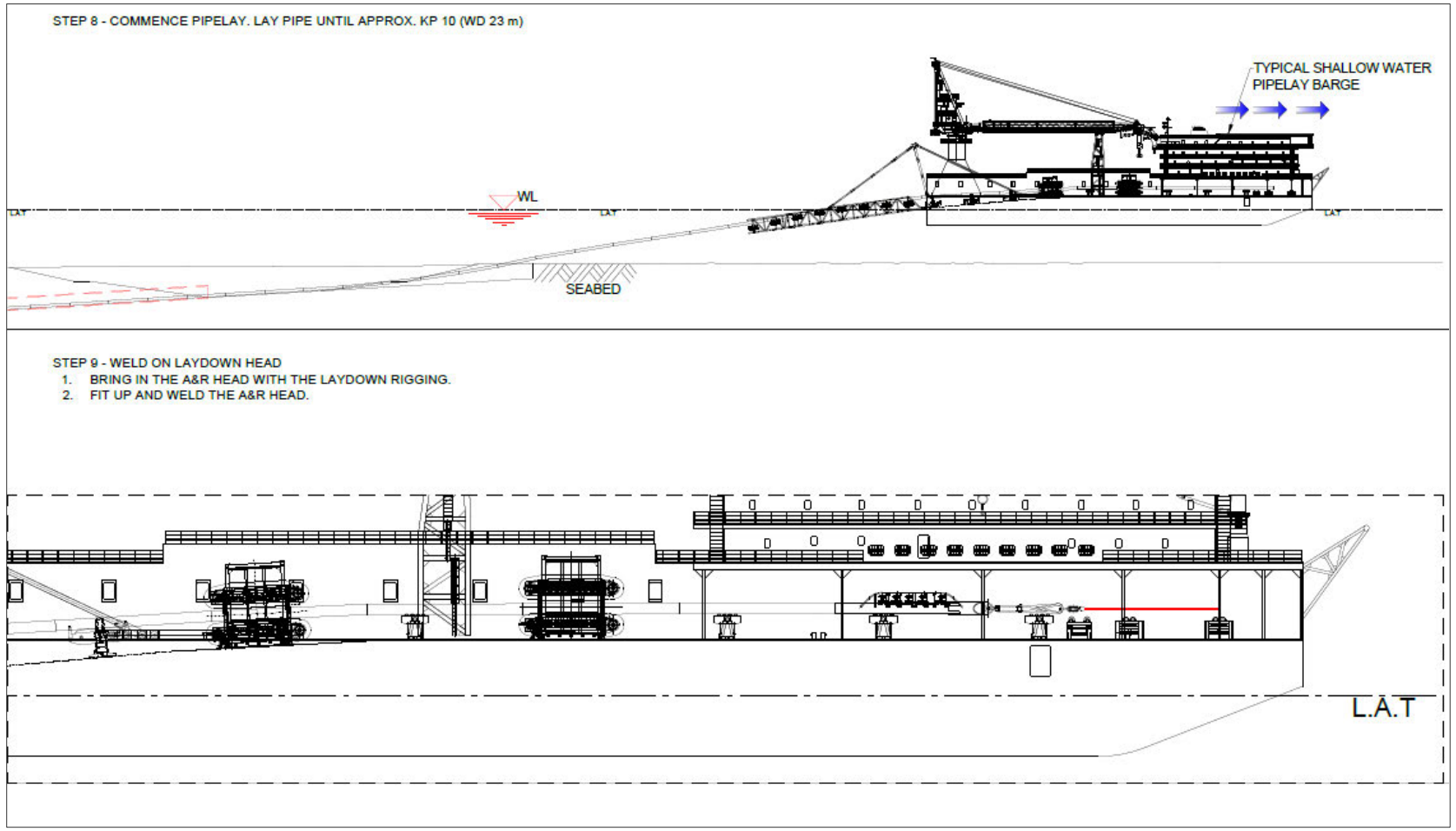
Aramis Landfall - Sea Defence

Construction- Storyline Installation Pipeline Through tunnel



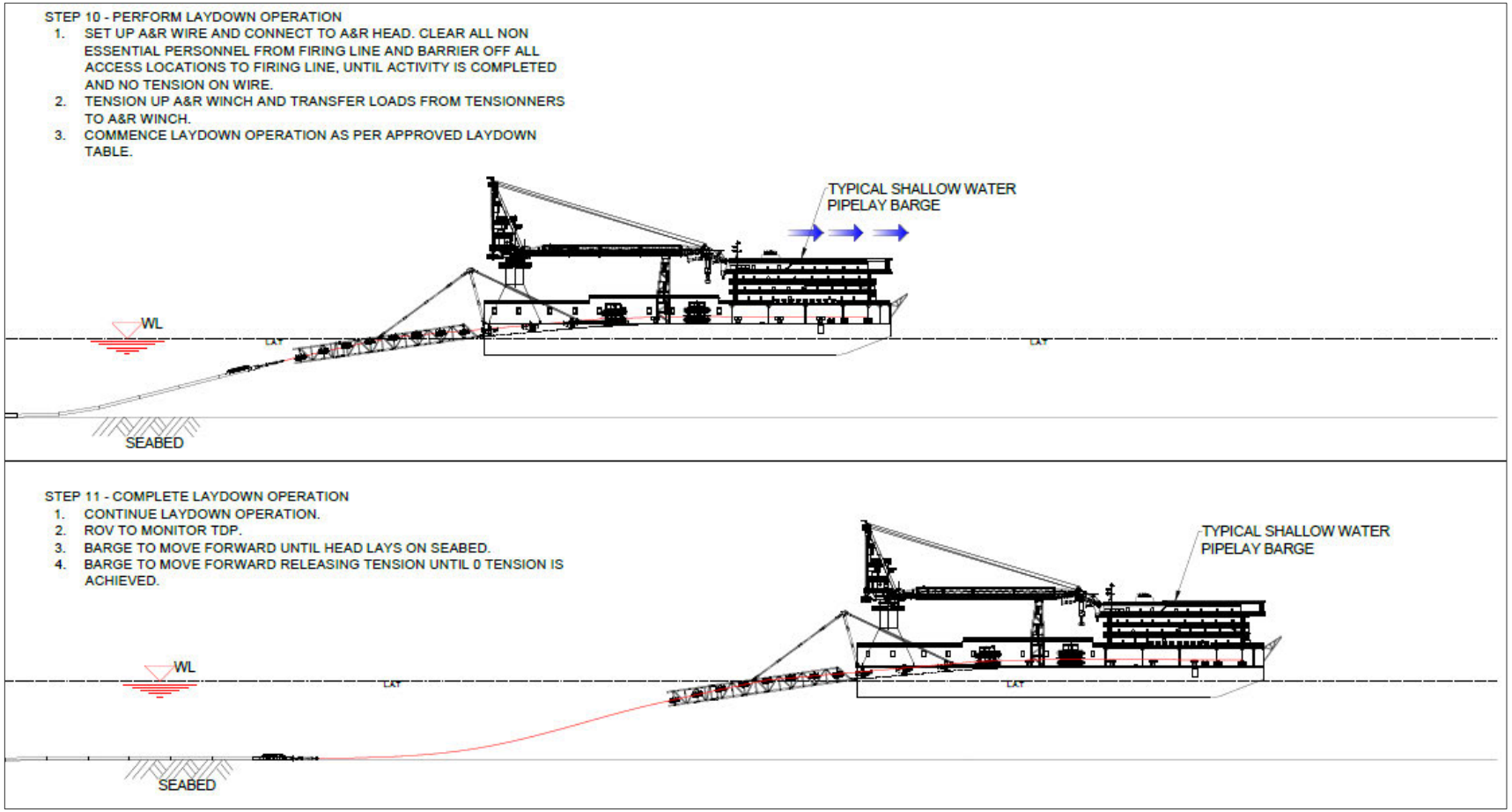
Aramis Landfall - Sea Defence

Construction- Storyline Installation Pipeline Through tunnel



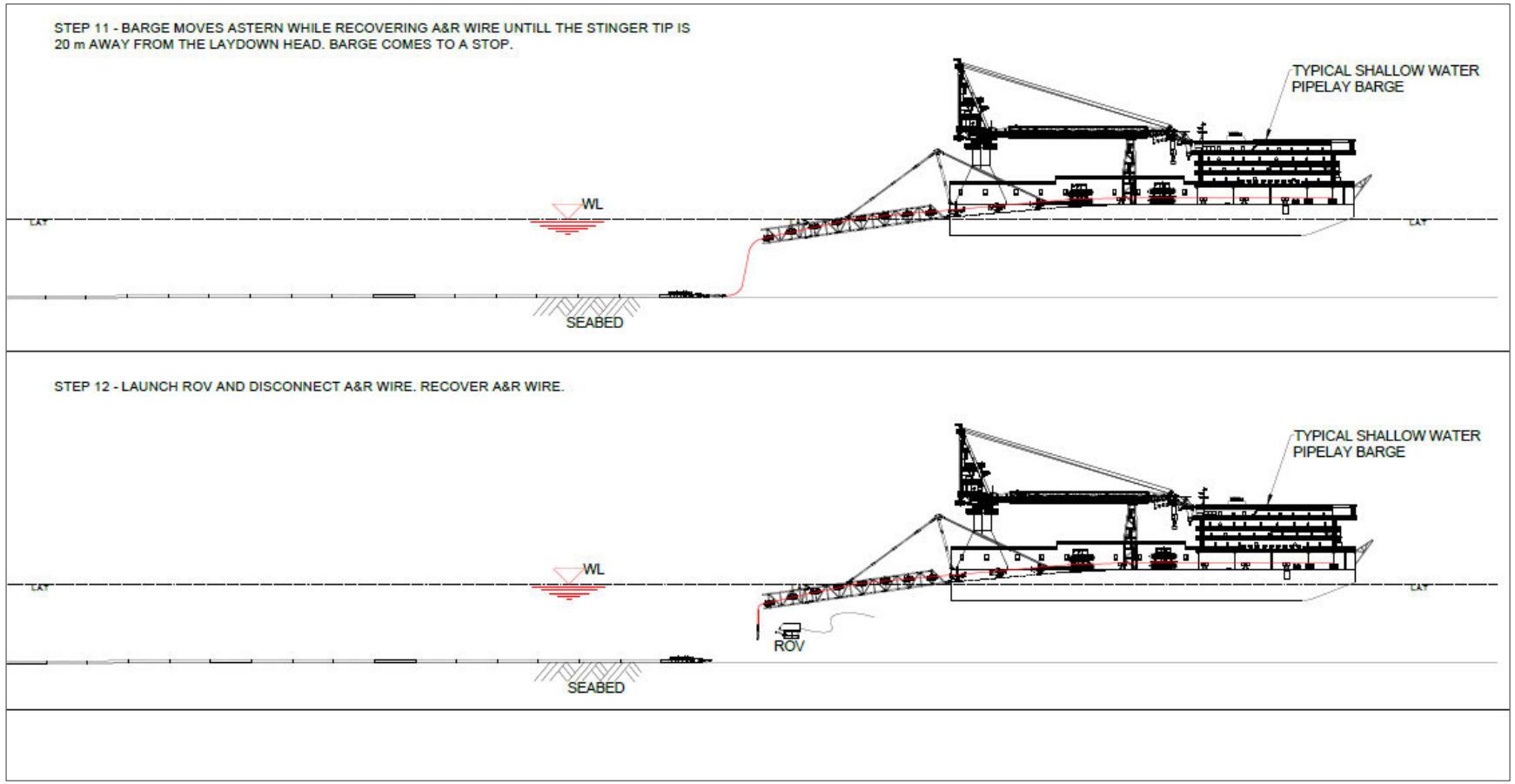
Aramis Landfall - Sea Defence

Construction- Storyline Installation Pipeline Through tunnel



Aramis Landfall - Sea Defence

Construction- Storyline Installation Pipeline Through tunnel



Aramis Landfall - Sea Defence

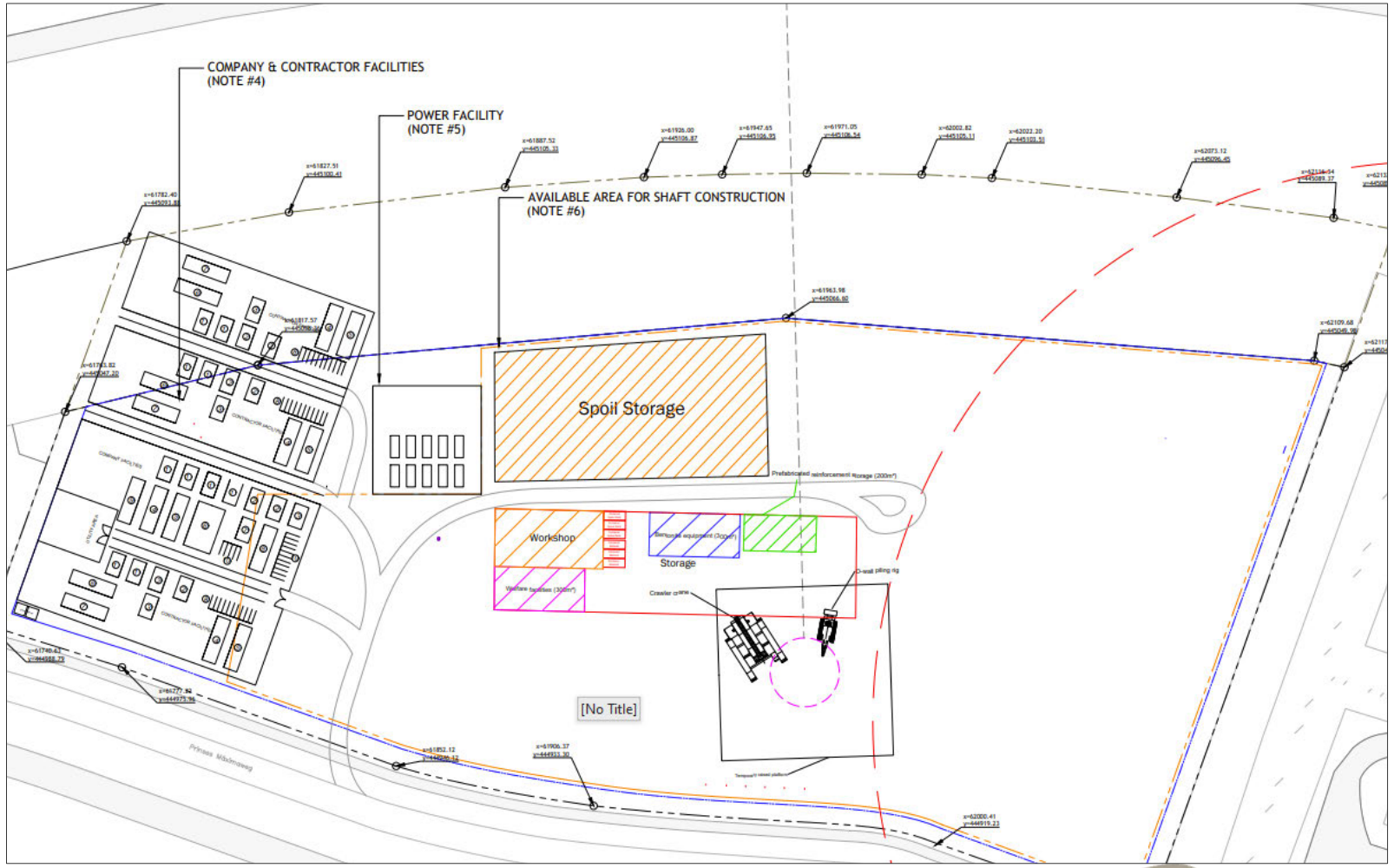
Construction- Storyline Installation Pipeline Through tunnel

After the pipeline has been installed the following activities will take place:

- Blocking the tunnel at the shaft location and installing the riser;
- Precommisioning the landfall according the NEN 3651 requirements;
- Filling the tunnel;

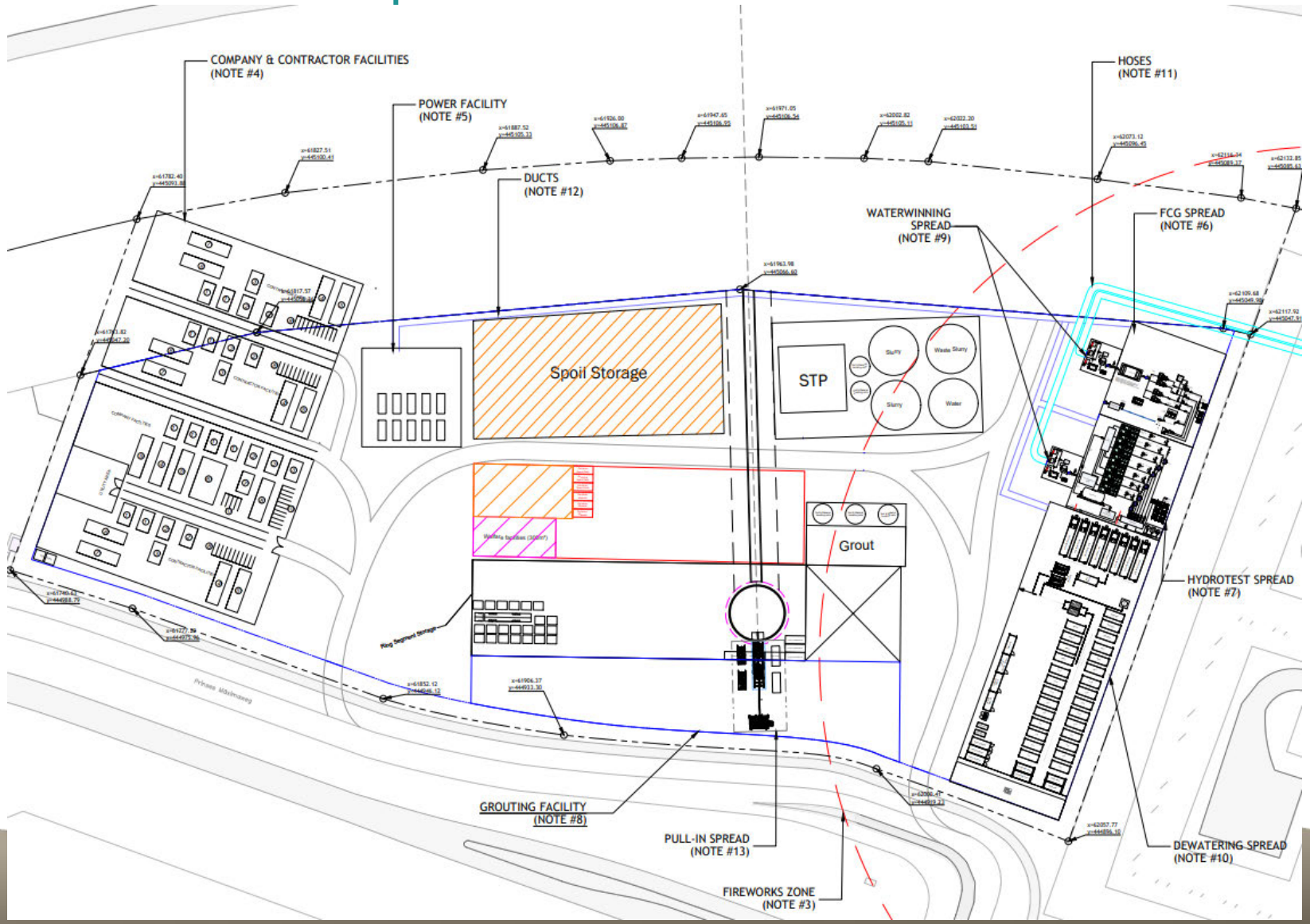
Aramis Landfall - Sea Defence

Construction-phases: Shaft Construction:



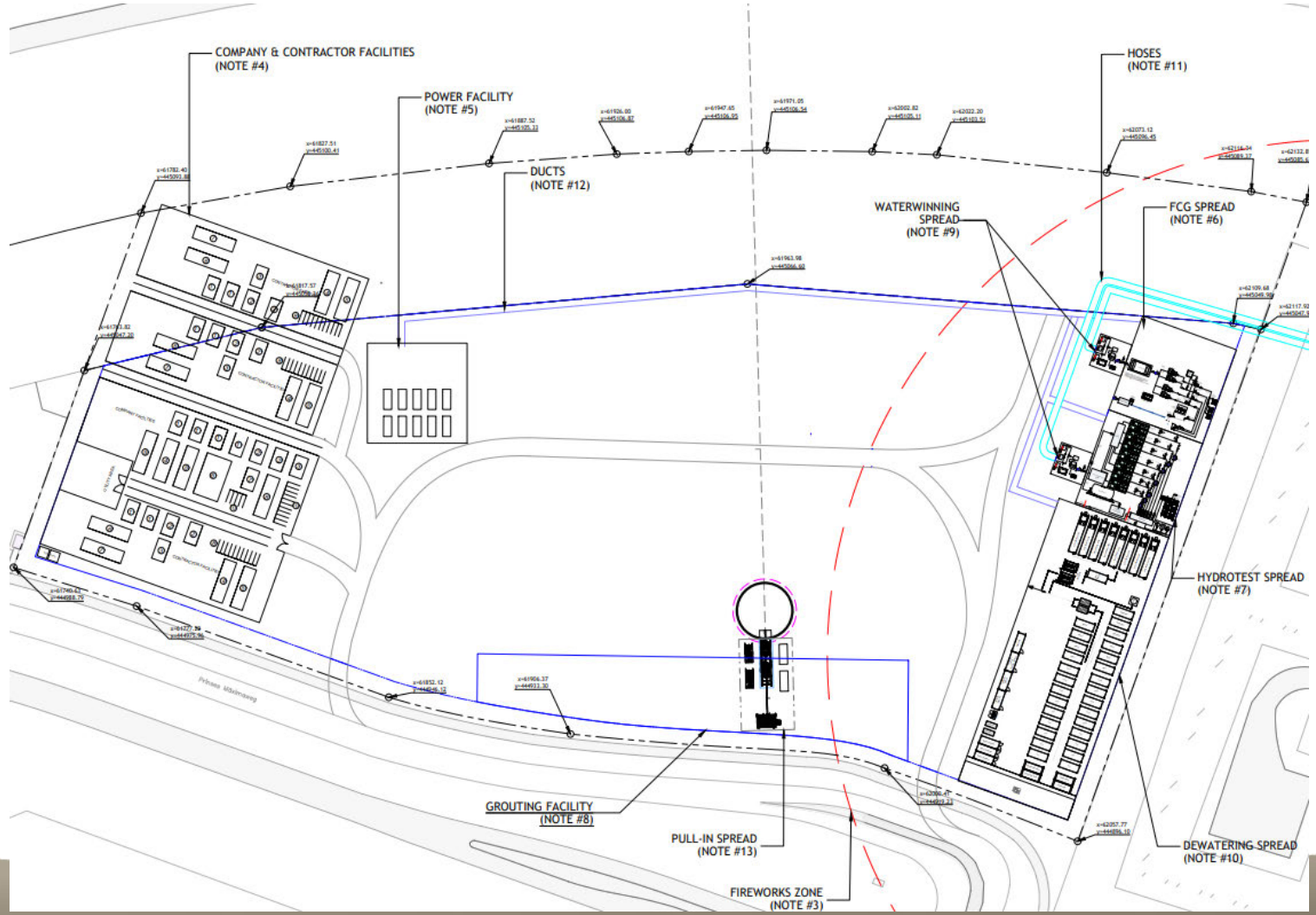
Aramis Landfall - Sea Defence

Construction-phases: Tunnel Construction:



Aramis Landfall - Sea Defence

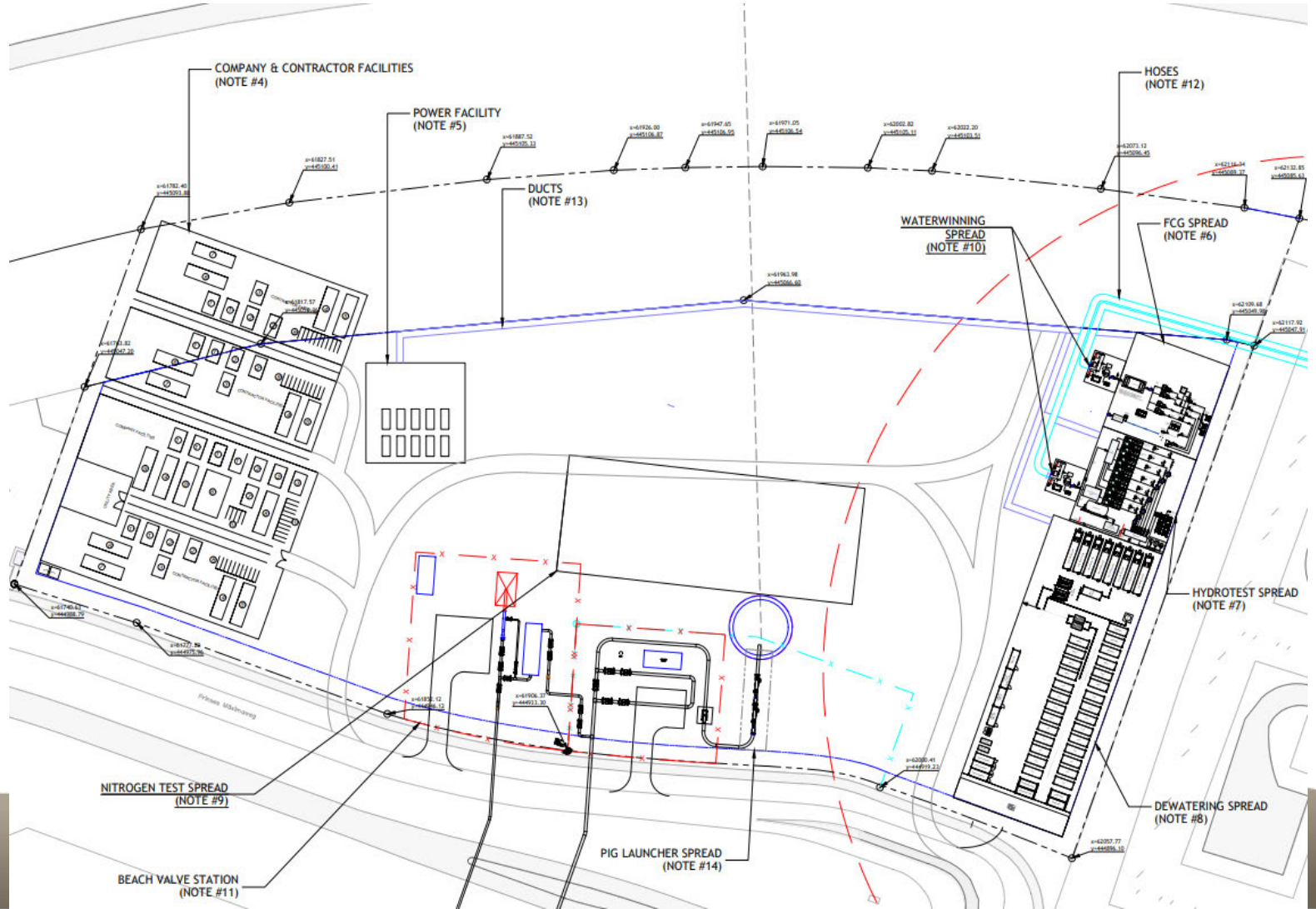
Construction-phases: Landfall (pipeline pull-in, tunnel backfilling, pre-commissioning):



Aramis Landfall - Sea Defence

Construction-phases

- Pre-commissioning:



Aramis Landfall - Sea Defence

Construction

Filling of the tunnel (grouting + alternatives)

- Filling of tunnel will depend on pipeline design and requirements;
- Options are filling with sand or grout (or combinations of both);

ARAMIS

Disclaimer

EBN, Gasunie, Shell and TotalEnergies entered into a cooperation agreement to explore the possibility of setting up a joint venture to jointly develop a CO2 transport activity unlocking a large Dutch offshore storage area. The present documentation and related discussions are entirely prospective and non-binding. They create no obligations on EBN, Gasunie, Shell, TotalEnergies or the prospect.

Bijlage

8. Risicoanalyse

NL-ARM-040-ARM1-000005

Royal HaskoningDHV

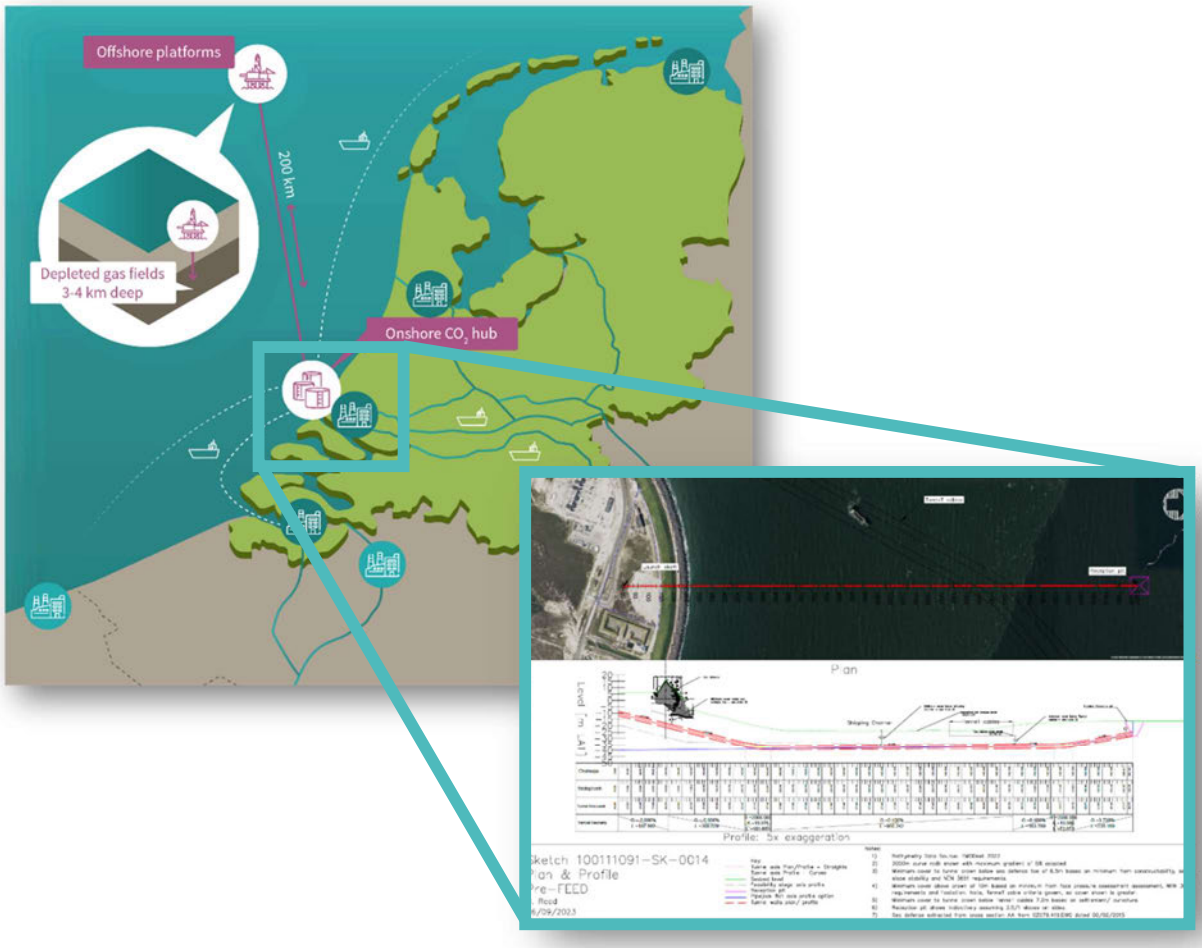
Aramis Tunnel vs Sea Defence

Document Title:	**Aramis Tunnel vs Sea Defence**		
Document Number:	NL-ARM-040-ARM1-000005	Revision:	03
Date:	16 December 2024		

Note: a translated version (NL) of this report is included in Appendix C.

Introduction

As part of the Aramis Project and in order to connect the onshore trunkline to the offshore trunkline, the Project will construct a shore crossing. For this shore crossing, the Aramis Project has selected a tunnelling method, utilizing a tunnel boring machine (TBM).



Tunnelling Method

The tunnel face will be stabilized using a pressurized slurry medium at the face of the Tunnel Boring Machine (TBM). For the Aramis project, a modern TBM equipped with advanced mechanical and instrumentation systems will be utilized.

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This TBM will be a closed full-face machine with a hydraulic slurry circuit, capable of maintaining face pressure during both boring operations and production stops. It will also be fitted with face pressure monitoring instruments to ensure precise control of face pressure during operation and stoppages.

The tunnel structure will be constructed and installed using the pipe jacking method while boring under the sea defence. Heavy-duty reinforced concrete jacking pipes will be pushed behind the TBM in the bored tunnel. A small annulus gap between the bored tunnel and the jacking pipe will be filled with a bentonite medium to stabilize the tunnel, prevent collapse, and minimize friction between the soil and concrete. Upon completion of the pipe jacking, the bentonite medium will be replaced with a non-shrinking cementitious grout.

After the installation and pre-commissioning of the CO2 trunkline in the tunnel, the gap between the pipeline and the tunnel interior will be filled with either a cementitious grout or sand. The backfilling of the tunnel anulus is required to restrict the lateral movement of the trunkline inside the tunnel.

Further details on the planning associated with the tunnelling as well as further explanation on the tunnelling method, reference is made to the presentation held to Rijkswaterstaat on 19 November 2024. A copy of the presentation is included within Appendix B.

Risks

Construction Phase

Aramis conducted a risk assessment to evaluate the potential risks posed by the tunnelling works to the sea defence. This risk assessment demonstrated that the risk of any significant effect from the tunnelling works on the sea defence is acceptable (ALARP).

The low risk profile is mainly the result of the remote likelihood of an extreme failure during the tunnelling works and consequently significant effects from the tunnelling works on the sea defence, whereby the frequency of failure is concluded to be below 10^{-5}. It shall be noted that this frequency of failure may even be less than the frequency of failure considered during the design of the respective sea defence, which may be in the region of 2.5×10^{-4}.

The risk assessment, as was also presented during the meeting with Rijkswaterstaat on 10 October 2024, has been included within Appendix A.

Within the section on settlements an additional evaluation is presented to demonstrate that, even after the remote likelihood of developing any significant settlement, the functionality of the sea defence will not be jeopardized.

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Aramis Tunnel vs Sea Defence

The flowchart presented below (Figure 1) provides some further clarification to the risks to the sea defence and the relationship between the direct and consequent risks due to the tunnelling works.

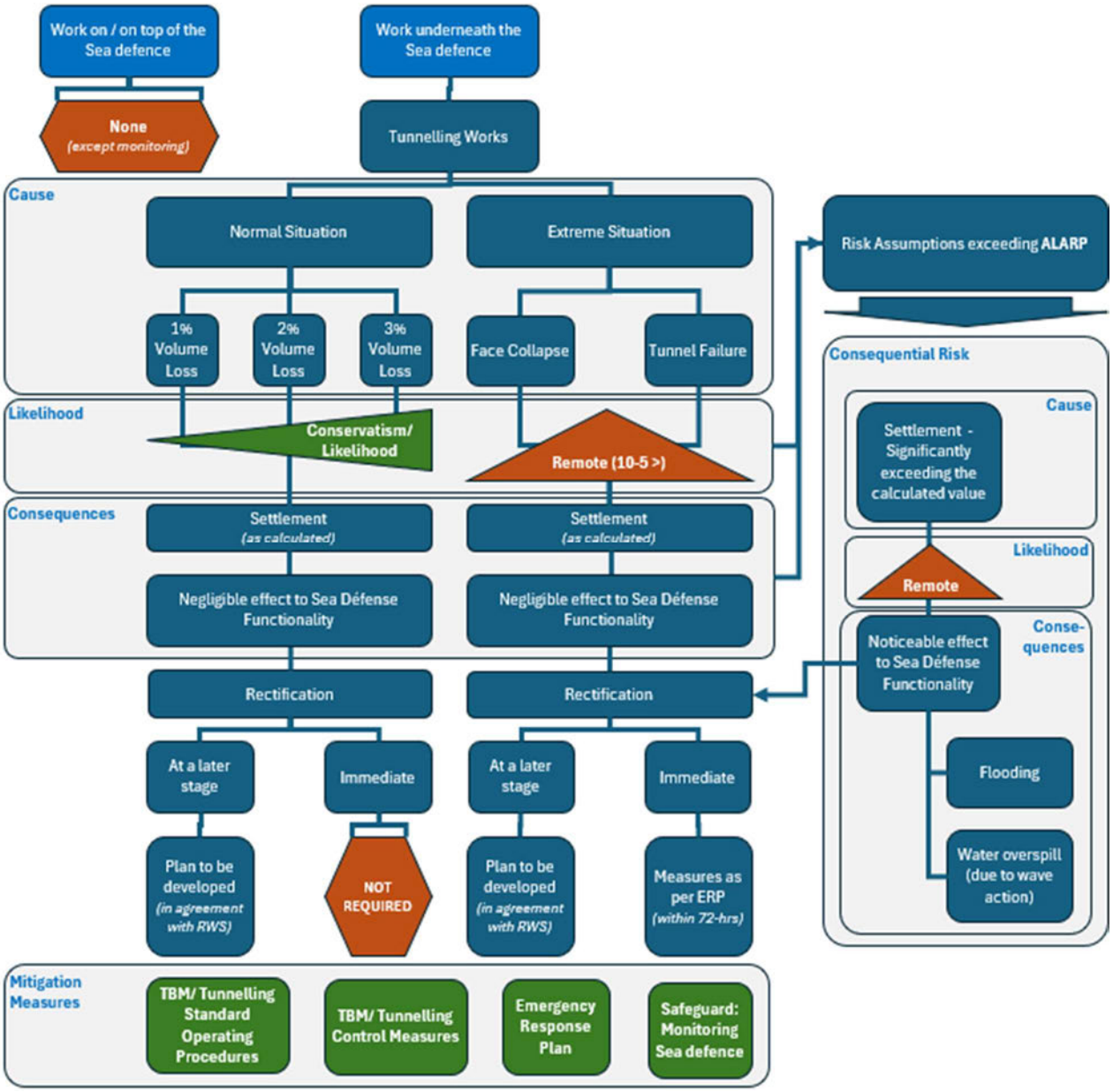


Figure 1 – Flowchart Risk Assessment

The flowchart above (Figure 1) demonstrates that the risk of a flooding and/or water overspill is a consequential risk resulting from a tunnel failure scenario.

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Operational Phase

The above-described risks all relate to the construction of the tunnel. For the risks associated with the tunnel during to operational phase of the tunnel any settlement due to a tunnel failure is eliminated considering that the tunnel will be fully backfilled with grout or sand.

With regards to the trunkline and its operational phase the risk of an explosion and/or large release of CO2 has been eliminated by applying the respective specifications from the NEN 3651. As such, the trunkline section under the sea defence will have a larger wall thickness (20% increase) and will be hydrotested to a significant larger test pressure. It is furthermore noted that each joint of the trunkline section under the sea defence will be hydrotested at the pipe mill and each girth weld will be 100% examined by non-destructive testing.

Once the full NEN 3651 has been applied, the full trunkline will meet the objective of the NEN 3651, which is that in case of an increased pressure (i.e. exceeding the systems design pressure) it is ensured that the parts adjacent to the section underneath the sea defence (with a smaller wall thickness) will fail and not the part(s) underneath or directly adjacent to the sea defence. It is noted that the section with the larger wall thickness will be installed as from the entry shaft, underneath the sea defence and Maasgeul and up to KP11.

Sea Defence Functionality

Although the sea defence is to be considered a primary sea defence for design and maintenance purposes, the actual purpose and/or functionality of the sea defence cannot be to prevent flooding due to high water levels. In case high water levels above 5 m NAP are experienced, the Sharkfin (and other Maasvlakte areas behind the sea defence) would flood from the high-water levels entering through the Maasmond, Calandkanaal, Beerkanaal/Nijlhaven and the Yangtzekanaal.

Settlement

In addition to the risk assessment (ref.: Appendix A) a settlement impact analysis was conducted whereby the settlement of the sea defence due to normal tunnelling operation was predicted as well as an extreme case of the tunnel failure, although such failure scenarios are considered to be a remote likelihood. The settlement assessment in relation to the sea defence is presented in the Aramis FEED document: NL-ARM-040-ARU1-110381, *'Tunnel FEED - Settlement Impact Analysis and Monitoring Plan - Sea Defense'*.

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A summary of the settlement assessment results is presented within Table 1 below.

		Maximum Settlement [mm]
Normal tunnelling	1% volume loss	19
	2% volume loss	39
	3% volume loss	58
Extreme failure	Face collapse	245
	Tunnel failure	360

Table 1 – Summary Settlement Assessment

With respect to the findings of the above assessment, and particularly the settlements as a result of the normal tunnelling operations, the anticipated settlements are considered to be negligible compared to the size of the sea defence and the way it is constructed at the sea-side (i.e. quarry stones, concrete blocks, asphalt, etc.). This also applies to the settlements concluded from the extreme failure load cases.

To assess the effect on the settlements against the sea defence functionality, a simplistic dimensional check was carried out. Hereby the maximum defined settlement was set against the total height of the sea defence (above NAP). The remaining height was then set against the maximum crest and high water (GWH). The resulting clearance, denoted 'freeboard', is concluded to range between 4.94 m and 8.70 m, depending on the wave direction selected. It is noted that this is considering a local settlement/deformation of the sea defence, and that the sea defence will maintain its overall stability. The assumption is made that this approach is justified due to the construction of the sea defence at the seaside (i.e. quarry stones, concrete blocks, asphalt, etc.).

The results of this simplistic dimensional assessment are presented in Table 2 below.

		Value [m]
Top of sea defense		14.5 (NAP)
High water (GWH)		1.1 (NAP)
Metocean Data – Maximum Crest (10,000-yrs return period)	NW	8.10
	NNW	7.28
	N	4.34
Freeboard: max. crest minus min. sea defense height (considering tunnel failure)	NW	4.94
	NNW	5.76
	N	8.70

Table 2 – Dimensional Assessment

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It shall be noted that the impact of the tunnelling works on the sea defence is limited to the aforementioned settlement. The tunnel boring operation is the only activity occurring adjacent to (underneath) the sea defence. It is noted that no construction activities will be conducted on the surface or directly on the sea defence throughout the entire period of landfall and/or tunnel construction works. The only activities envisaged on the sea defence is the installation of the survey base stations, i.e. through stakes and base plates, for further details reference is made to the monitoring section below.

Monitoring

In order to monitor the settlements during the tunnelling works, Aramis will set-up a monitoring campaign. Therefore, and as part of a larger monitoring scope in relation to the Sharkfin, Aramis will also conduct a survey and set-up a monitoring campaign of the sea defence prior to, during and after completion of the tunnelling works.

The objective of this monitoring campaign is to identify any settlement of the sea defence and ensure the settlements are maintained within the expected values, whereby the settlement related to the 1% volume loss will be the base value. Any settlement exceeding such base value will instigate the necessary measure to bring settlement values within the base values. This can be achieved by either reducing or increasing the face pressures, and/or increasing or reducing the tunnelling speed. Such controlling measures will be captured in the tunnelling procedures to be developed by the tunnelling contractor and is already a crucial part of the tunnel contractor's standard operating procedures.

In case of an extreme tunnel failure (i.e. exceeding a 3% volume loss), besides the necessary control measures, the monitoring measurements will also trigger the emergency response plan in relation to any necessary rectification of the sea defence.

Survey Works

Before the start of the tunnelling works a baseline condition survey will be conducted on the sea defence and its immediate surroundings, which will be established by a LIDAR Digital Terrain Model (DTM). Two methods are considered, being: (i) use of a drone survey on an area of 400 x 500 meters or (ii) by terrestrial laser scanning (NavVis mobile scanning system) capturing the sea defence only on an area of 100 x 150 meters.

After completing of the tunnelling works a post-construction condition of the sea defence will be captures by the same method selected for the baseline condition survey.

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Monitoring Works

To monitor the sea defences during the construction of the shaft and the tunnelling works a combination of two methods are considered: (i) manual and (ii) automatic deformation measurements, as detailed below:

- *Manual deformation measurements:*
  For the manual deformation measurements 24 measurements points (stakes with survey marker) will be placed in 4 rows. One row will be placed at the top of the sea defences. The other 3 rows will be placed perpendicular to the first (one on top of the tunnel trace and one on each side).

  After installation a baseline measurement in RD/NAP coordinates will be carried out by tachymeter and precision levelling in which the measurement points will also be connected to at least two government benchmarks. The deformation points will be surveyed at as a set frequency before, during (with an increased frequency) and after the tunnelling works have passed the sea defence.

- *Automatic deformation measurements:*
  For the automatic monitoring 9 GNSS receivers will be installed on the sea defences in three rows (one on top of the tunnel trace and one on each side). An additional GNSS reference point will be installed outside the influence zone of the tunnelling. The GNSS receivers determine their own position in three directions (X, Y and Z) every minute. The data is collected by a gateway once every ten minutes.

  The system allows for trigger levels to be set for the measurements, which will set-off an alarm when trigger levels are exceeded.

Rectification

At the moment the contractor for the tunnelling works have not yet been appointed, whereas the rectification methodology is to be developed by the tunnelling contractors as part of their overall scope of work. It is noted that the rectification works can be divided into two scenarios, namely:

- Rectification after completing tunnelling works
- Immediate rectification.

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Rectification after completing tunnelling works

Rectification of settlements due to normal tunnelling works, i.e.: settlement due to volume loss, will be executed after completing the tunnelling works. The settlement values as presented in the FEED report: NL-ARM-040-ARU1-110381, *'Tunnel FEED - Settlement Impact Analysis and Monitoring Plan - Sea Defense'*, have been considered as the selected threshold, whereby any experienced settlement, not exceeding the anticipated settlements as per the respective FEED report, will not require rectification.

Any such rectification shall be developed by the tunnelling contractor, whereby Rijkswaterstaat is to be presented the work protocol for review.

Immediate rectification

Rectification due to an extreme failure, whereby an immediate and potential intermediate rectification (expected within 72-hrs) is required. Like the rectification described above, the necessary procedure is to be developed by the tunnelling contractor. Such, rectification will be covered in a so-called Emergency Response Plan (ERP), which will also cover all other emergencies related to the tunnelling works. Nevertheless, in principle it can be concluded already that any ERP will specify that the necessary materials (i.e. big bags, etc.), together with the soil distracted by the tunnelling and/or grout, will be on site to ensure timely rectification of the sea defence if deemed required. In addition, the ERP will consider multiple scenarios to capture the rectification of any significant settlement on different locations at the sea defence, which may require different methods due to the nature of the sea defence at that particular location (i.e. asphalt, concrete blocks, clay, grass, etc.).However, as already concluded from the risk assessment (ref.: Appendix A), the actual necessity of such rectification procedure and/or ERP is considered remote.

Like the above referenced work protocol for rectification works, the ERP will be presented to Rijkswaterstaat for review.

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Appendix A

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Aramis Tunnel vs Sea Defence

Title: Risk Assessment Crossing the Sea Defence
Date: 08/10/2024

Introduction

As part of the Aramis Project and in order to connect the onshore trunkline to the offshore trunkline, the Project will construct a shore crossing. For this shore crossing, the Aramis Project has selected to utilise a tunnelling method (closed, full face excavation principle) to facilitate the offshore trunkline crossing with the Maasgeul and respective sea defence.

To determine and evaluate the risks introduced by the tunnelling and affecting the sea defence, a risk assessment has been executed and is presented hereto.

Basis

For the risk assessment two mechanisms have been identified whereby the tunnel works could affect the sea defence. It is noted that these two mechanisms are related to a catastrophically event. The effect on the sea defence during normal tunnelling works are negligible and have already been presented to Rijkswaterstaat on 12MAY24 and 08JUL24.

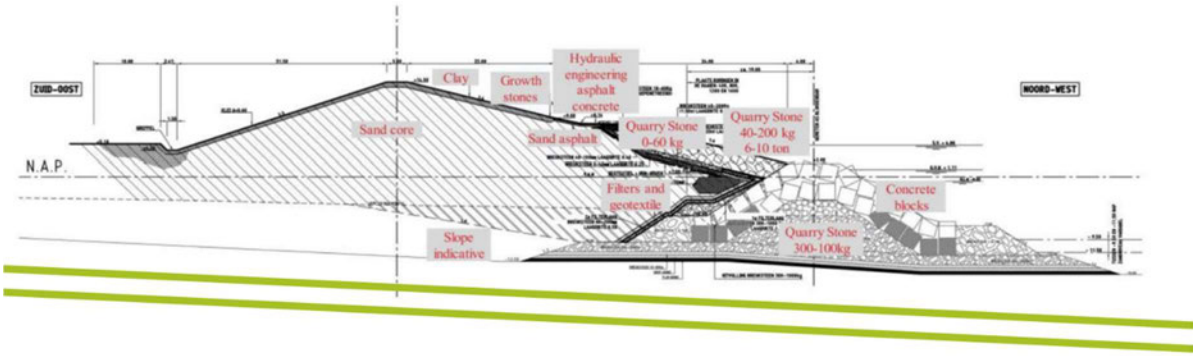
These two mechanism/events are as follows:

- Face instability – full face collapse
  Such event is an instability in front of the TBM caused by an operator failure (not maintaining the necessary pressure), whereby the face pressure is dropped significantly, causing the void in front of the TBM to collapse.
- Full tunnel collapse
  Hereby a tunnel segment would fully fail resulting in a local collapse of the tunnel.

It is noted that the affect from above listed catastrophes will be a local effect only, limited to the diameter of the tunnel (3.6m) and over a limited length, between 2m (for a full face collapse) and 4m (full tunnel collapse, considering the tunnel segment lengths are maximum 3 m).

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Aramis Tunnel vs Sea Defence



Risk Assessment

The risk assessment has been conducted in accordance with TTE General Specification: GS-GR-HSE-312, *"Technological Risk Assessment"*. The results of the risk assessment are presented below:

ID	Deviation/ What if	Causes	Consequences	Likelihood[1]	Severity [2]	Risk [3]
1	Excessive ground movements at the ground surface	Face instability - full face collapse	Sea defence settlement - 0.5m	Remote	(i) Moderate (ii) Serious	**Acceptable**
2		Full tunnel collapse (i.e. collapsing tunnel elements)	Sea defence settlement - conservatively chosen to be equal to the tunnel outside diameter, i.e.: 3.6 m	Remote	(i) Moderate (ii) Serious	**Acceptable**

Notes:

(1) **Likelihood:**

The likelihood classification is based on Table 7 of GS-GR-HSE-312, *"Technological Risk Assessment"*, of which an extract is included below:

Table 7 - Damage Frequency Categories

Frequency Category	Definition for Qualitative Assessment	Frequency Scale (Likelihood of Occurrence per Year)
Likely	Could occur several times over plant life	$\geq 10^{-2}$
Unlikely	Could occur once for every 10 to 20 similar plants over 20 to 30 years of plant lifetime	$10^{-2} > \geq 10^{-3}$
Very Unlikely	One time per year for at least 1 000 units. One time for every 100 to 200 similar plants in the world over 20 to 30 years of plant lifetime	$10^{-3} > \geq 10^{-4}$
Extremely Unlikely	Has already occurred in the industry but corrective action has been taken	$10^{-4} > \geq 10^{-5}$
Remote	Event physically possible but has never or seldom occurred over a period of 20 to 30 years for a large amount of sites (above few thousands, e.g. wagons, process vessels)	$10^{-5} >$

(2) **Severity:**

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In relation to severity the following catastrophes have been considered: (i) physical injury to people (external) and (ii) material damage in terms of asset replacement cost.

It is noted that the severity is based on the respective consequence having no affect to the Sharkfin, i.e. no expected over-topping of the sea defence and/or flooding of the Sharkfin due to an unfavourable combination of waves and tidal ranges. It is noted that based on the maximum sea level peak height (10.7m NAP - which is based on: max. tide - 4.71 m at 1,000-yrs return period and Hmax 11.9 m at 10,000 return period) not exceeding the existing sea defence height (14.5m NAP) minus the expected settlement due to the tunnelling works.

The severity classifications are based *on Table 8 and Table 10 of* GS-GR-HSE-312, *"Technological Risk Assessment"*, of which extracts are included below:

Table 8 - Damage Severity to People

Severity	Description	Parameter (note 2)	Number of people exposed (note 3)		
			Onsite	External	Both onsite + external
Moderate	Onsite: Non-permanent effect External: No effect	SEI	0	0	
Serious	Onsite: Permanent effects External: Non-permanent effects	SEI	1 to 99	1 to 9	
Very serious (note 1)	Onsite: Permanent disabilities and/or one fatality External: Permanent effects	SEI	100 to 499	10 to 99	< 500
		LC1%	1 to 99	1 to 9	
		Fatality	1	0	
Catastrophic (note 1)	Onsite: Several fatalities External: Many physical injuries and/or one fatality	SEI	500 to 999	100 to 999	< 1000
		LC1%	100 to 499	10 to 99	< 500
		Fatality	2 to 5	1	
Disastrous (note 1)	Onsite: Many fatalities External: Several fatalities	SEI	> 999	> 999	>= 1000
		LC1%	> 499	> 99	>= 500
		Fatality	> 5	> 1	

Note 1 : Consider the worst-case combinaison for establishing the severity
Note 2 : Effect thresholds for SEI, LC1% and fatality parameters are specified in appendix 11
Note 3 : Outside or inside buildings
SEI : Irreversible effect threshold
LC1% : 1% lethality threshold

Table 10 - Damage Severity to the Environment for DRA Selection Phase, E&P branch

Severity Level	Pollution Event	Intensity Expressed as Spill Volume (in bbl)
Moderate	Very localised pollution with minimal impact on the environment	Coastal or fragile area < 0.1 bbl Onshore < 1 bbl Offshore < 10 bbl
Serious	Slight pollution with limited consequences on the environment	0.1 ≤ Coastal or fragile area < 10 bbl 1 ≤ Onshore < 100 bbl 10 ≤ Offshore < 1,000 bbl
Very Serious	Pollution with significant consequences on the environment	10 ≤ Coastal or fragile area < 200 bbl 100 ≤ Onshore < 2,000 bbl 1,000 ≤ Offshore < 20,000 bbl
Catastrophic	Extensive pollution affecting ecosystems of recognized ecological interest	200 ≤ Coastal or fragile area < 2,000 bbl 2,000 ≤ Onshore < 20,000 bbl 20,000 ≤ Offshore < 200,000 bbl
Disastrous	Pollution having massive, lasting consequences on vast ecosystems of high ecological interest	2,000 bbl ≤ Coastal or fragile area 20,000 bbl ≤ Onshore 200,000 bbl ≤ Offshore

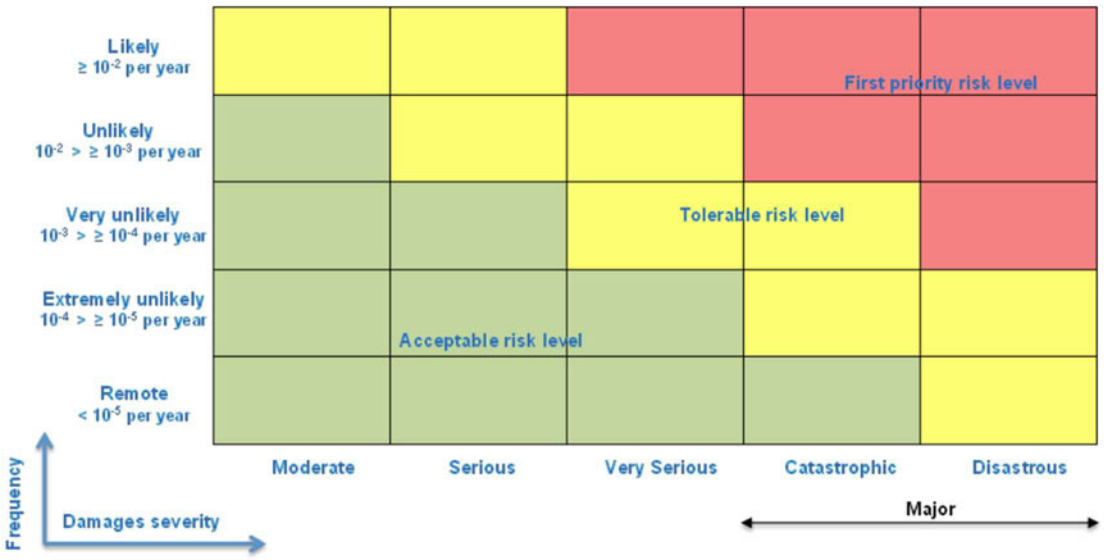
(3) **Risk:**

The likelihood classification is based on the risk matrix, Figure 10 of GS-GR-HSE-312, *"Technological Ris Assessment"*, of which an extract is included below:

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Aramis Tunnel vs Sea Defence

Figure 10 - Company Risk Matrix



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Appendix B

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Aramis Tunnel vs Sea Defence

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A large-scale, flexible CO_2 transport and storage solution

Aramis Project – Tunnel Works

RWS Overleg

19 november 2024

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1

ARAMIS Aramis Project – Tunnel Works

Agenda

- Introductie
- Huidige project status
- Tunnel layout
- Constructie sequence
- Haaienvin layout
- Werkzaamheden schema
- Jacking-pipe method / segmental method
- AOB



2

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Aramis Tunnel vs Sea Defence

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ARAMIS Aramis Project – Tunnel Works

Huidige Project Status

- Systeem voorontwerp fase (FEED) afgerond oktober 2024
- Aanbesteding fase (CFT) voor de tunnel in uitvoering



Tunnel werkzaamheden (EPC):

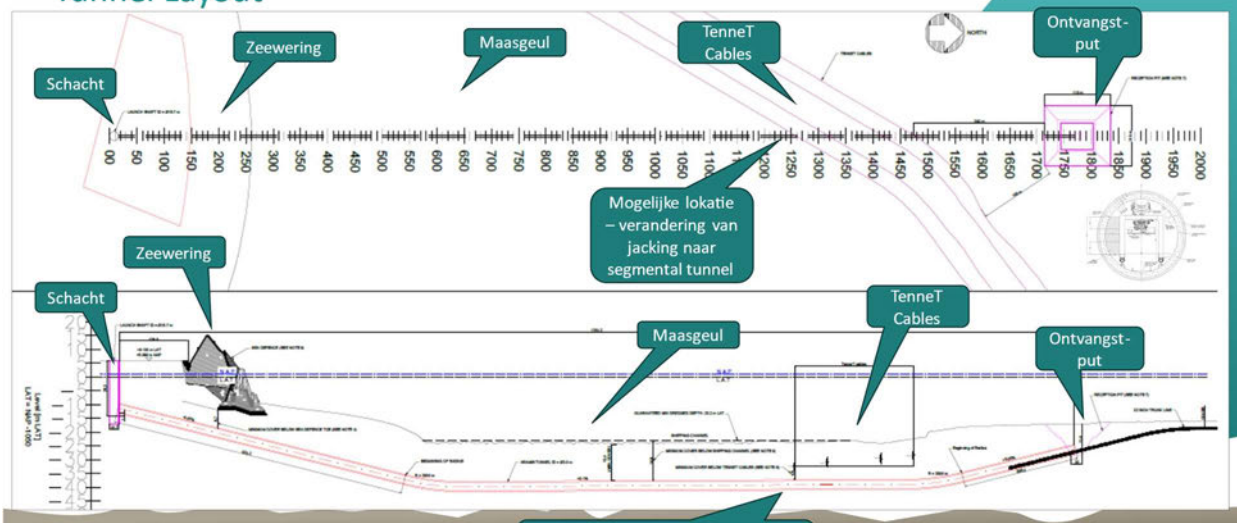
- Engineering (*detailed engineering*)
- Aankoop:
  i. Schacht materialen
  ii. Tunnel elementen (jacking pipe/segmental elementen)
  iii. Tunnel boring machine (TBM) – aanpassing/nieuw
- Constructie:
  i. Schacht
  ii. Tunnel

Note: TBM recovery en bagger werkzaamheden voor ontvangstput door EPC Offshore Trunkline aannemer.

3

ARAMIS Aramis Project – Tunnel Works

Tunnel Layout

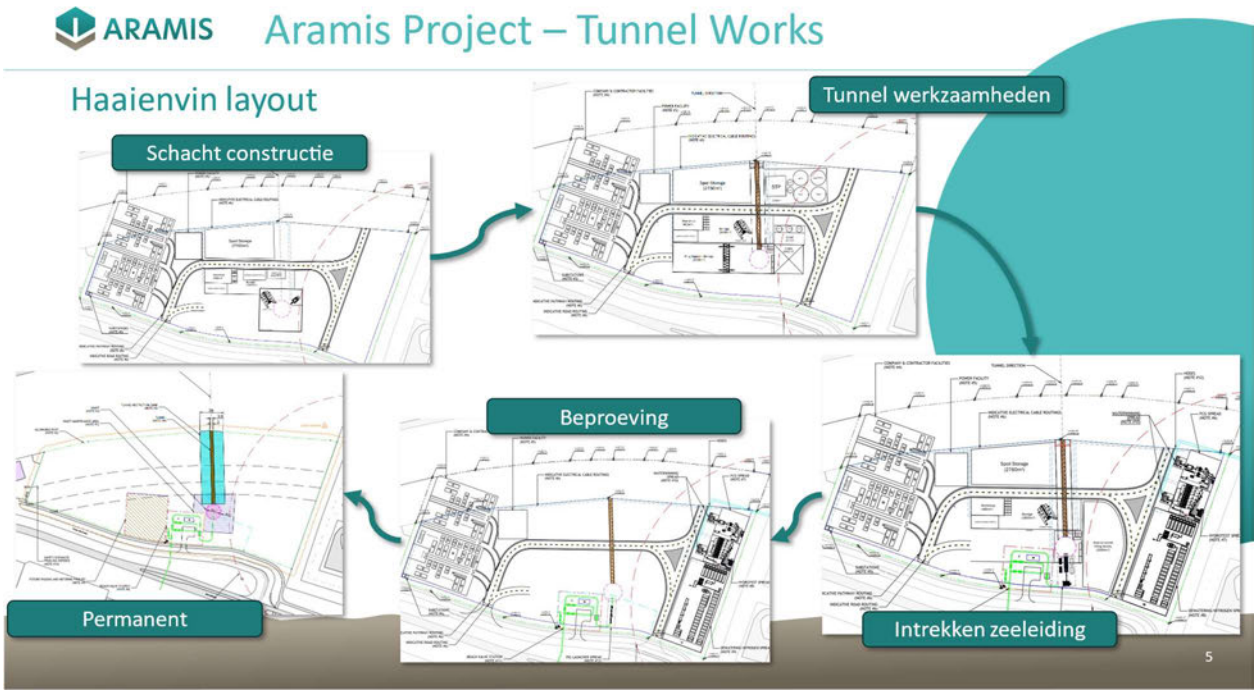


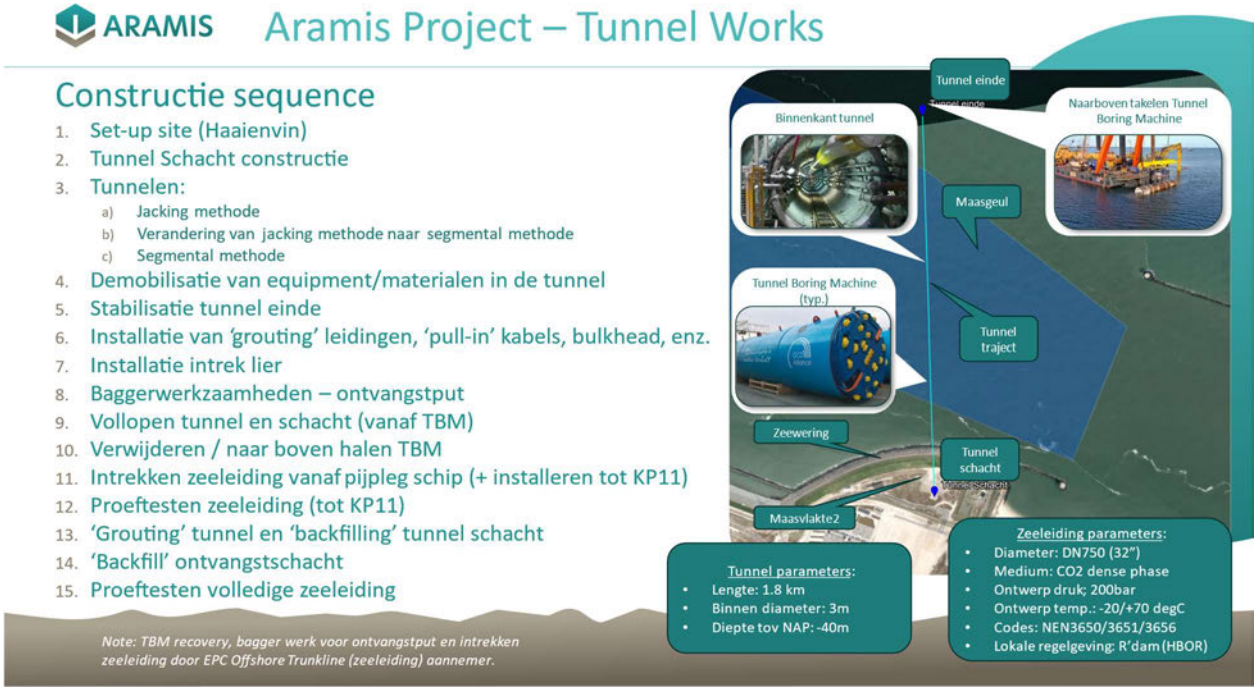
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Aramis Tunnel vs Sea Defence

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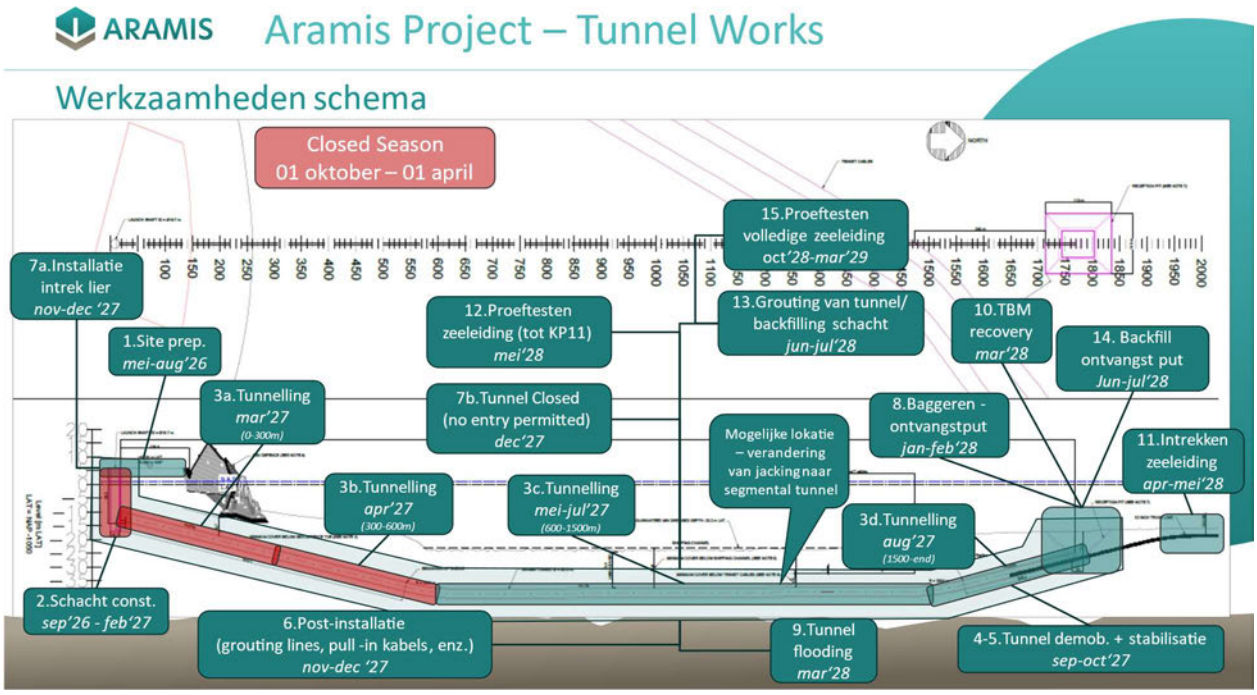


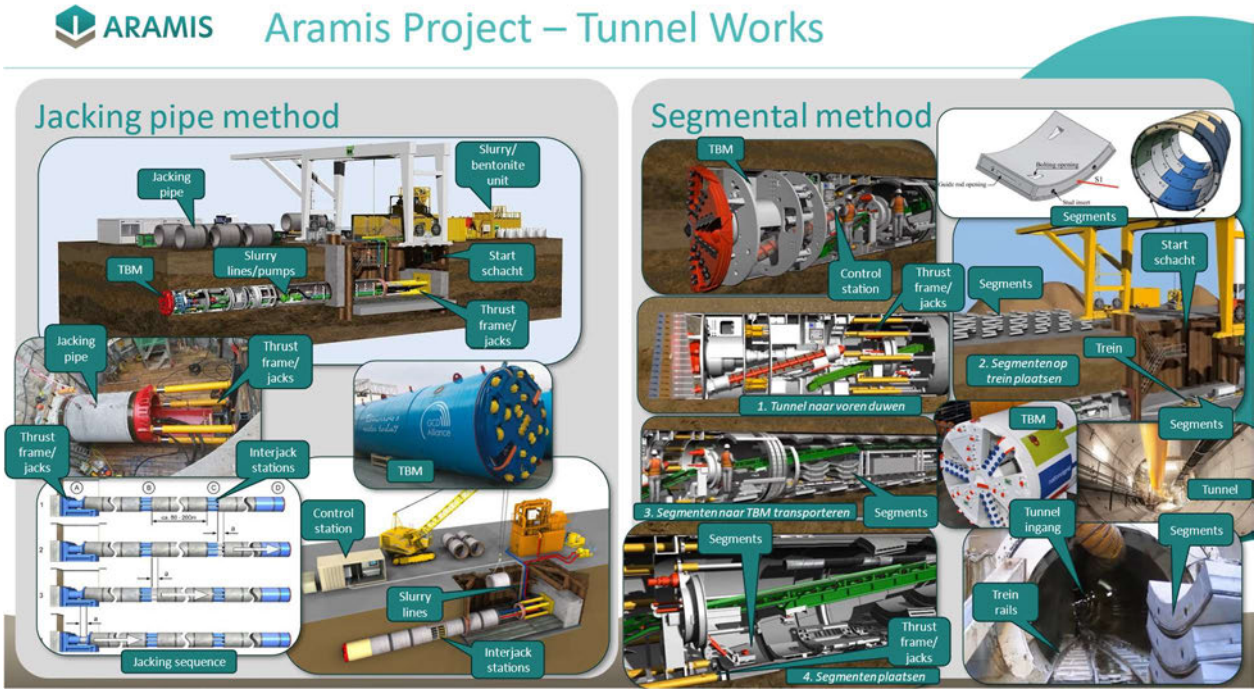


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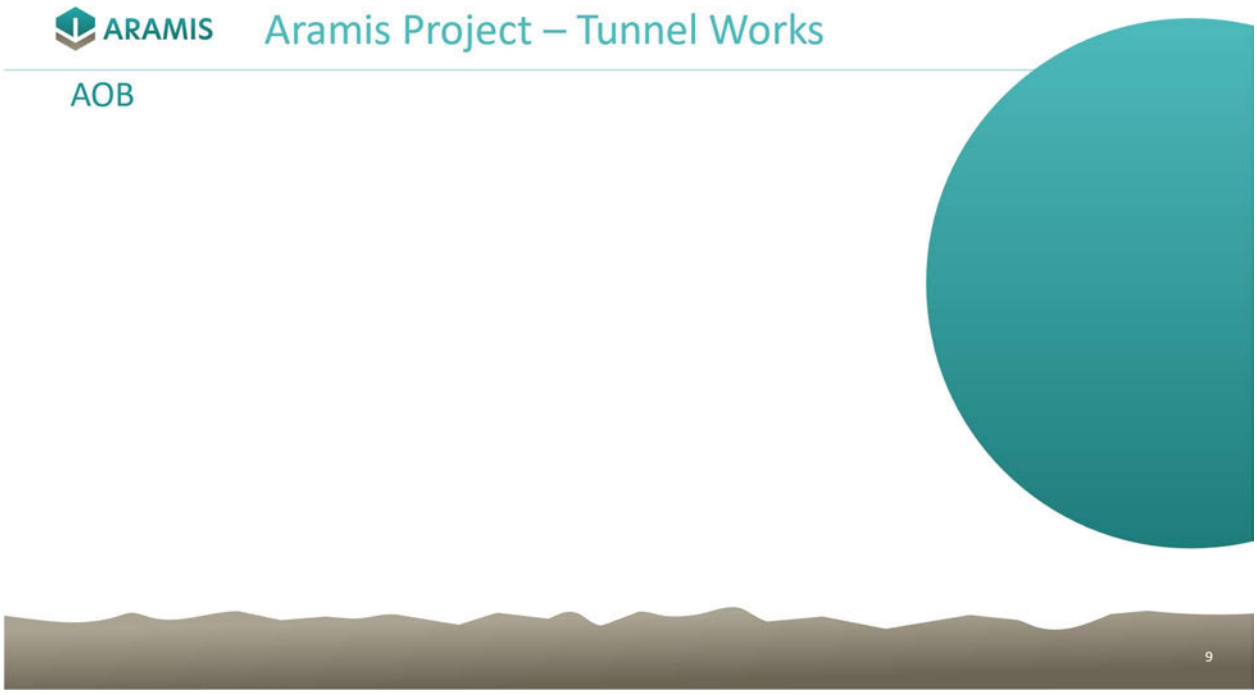




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Appendix C

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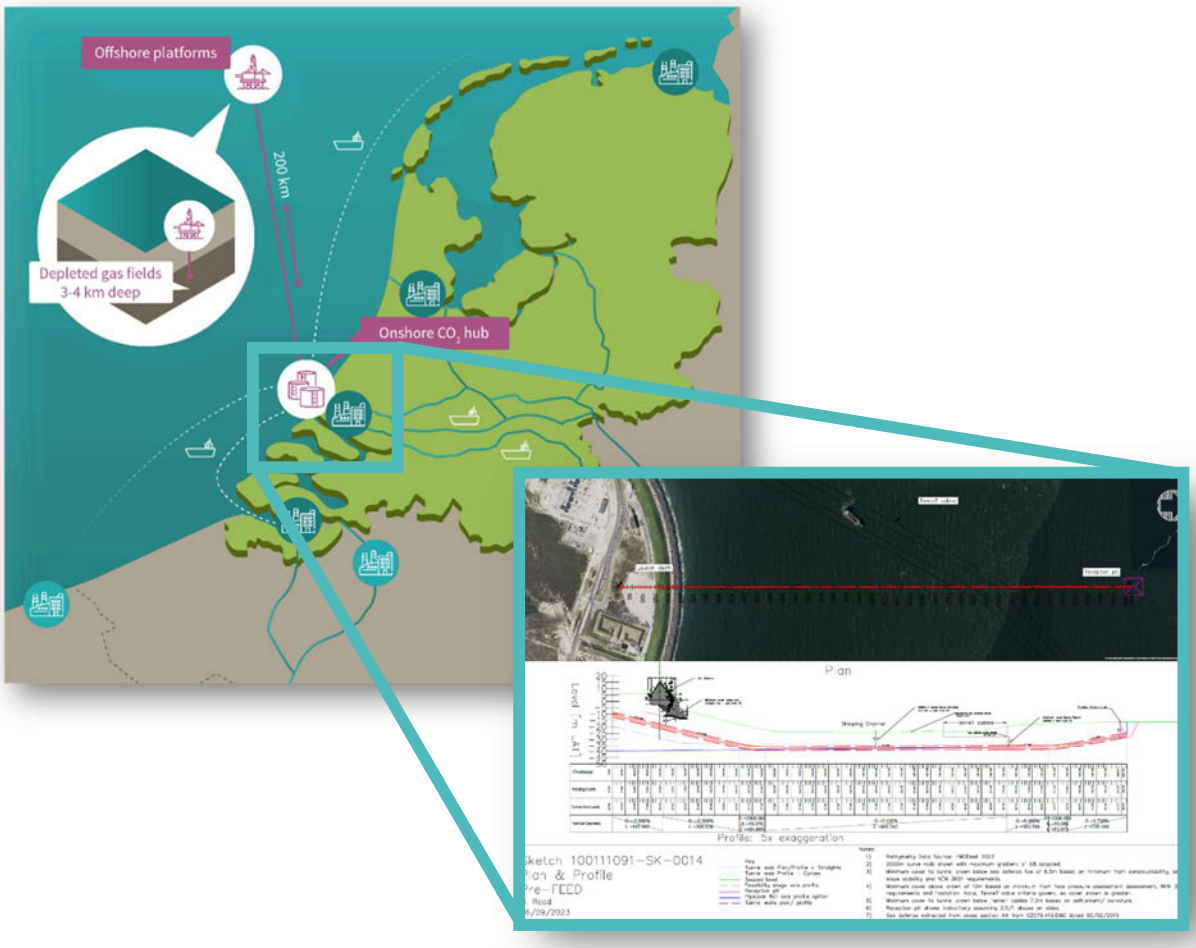
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Aramis Tunnel vs Sea Defence

Documenttitel:	**Aramis Tunnel vs Zeewering (NL versie)**		
Documentnummer:	NL-ARM-040-ARM1-000005	Herziening:	03
Datum:	16 december 2024		

Inleiding

Als onderdeel van het Aramis Project en om de onshore leiding te verbinden met de zeeleiding, zal het Project een kustovergang (‘shorecrossing’) bouwen. Voor deze kustovergang heeft het Aramis Project een tunnelmethode gekozen, waarbij gebruik zal worden gemaakt van een tunnelboormachine (TBM).



Tunnelmethode

De open tunnel (zonder bekleding), ‘tunnel face’, wordt gestabiliseerd met behulp van een onder druk staand slurrymedium aan de voorzijde van de tunnelboormachine (TBM). Voor het Aramis Project wordt gebruikgemaakt van een moderne TBM welke is uitgerust met geavanceerde mechanische- en instrumentatiesystemen.

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Deze TBM zal een gesloten ‘full-face’ machine zijn met een hydraulisch slurry circuit, welke de ‘face’ druk kan handhaven tijdens zowel booroperaties als productiestops. De TBM zal ook worden uitgerust met ‘face’ druk controle instrumentatie om nauwkeurige controle van de ‘face’ druk te garanderen tijdens zowel operaties als enige onderbrekingen.

De tunnel wordt gebouwd en geïnstalleerd met behulp van de ‘pijpjackingmethode’ voor het gedeelte wat onder de zeewering wordt geboord. Hierbij worden zware, gewapende betonnen ‘jacking’ buizen achter de TBM in de geboorde tunnel geduwd. Een kleine ringvormige opening tussen de geboorde tunnel en de ‘jacking’ buis wordt gevuld met een bentonietmedium om de tunnel te stabiliseren, instorting te voorkomen en de wrijving tussen de grond en het beton te minimaliseren. Na voltooiing van de ‘pijpjacking’ wordt het bentonietmedium vervangen door een niet-krimpende cementmortel.

Na de installatie en pre-commissioning van de CO2 zeeleiding in de tunnel, zal de anulus tussen de zeeleiding en de tunnel binnenkant worden opgevuld met of een cementgebonden grout of zand. Het opvullen van de tunnel anulus is vereist om de laterale beweging van de zeeleiding in de tunnel te beperken.

Voor nadere details over de planning van de tunnel bouw en een nadere toelichting op de tunnelbouwmethode wordt verwezen naar de presentatie welke op 19 november 2024 is getoond aan Rijkswaterstaat. Een kopie van deze presentatie is opgenomen in Bijlage B.

Risico's

Bouwfase

Aramis heeft een risicobeoordeling uitgevoerd om de potentiële risico's van de tunnelwerkzaamheden ten opzichte van de zeewering te evalueren. Deze risicobeoordeling toonde aan dat het risico van enig significant effect op de zeewering, voortkomend uit de tunnelwerkzaamheden, acceptabel is (ALARP).

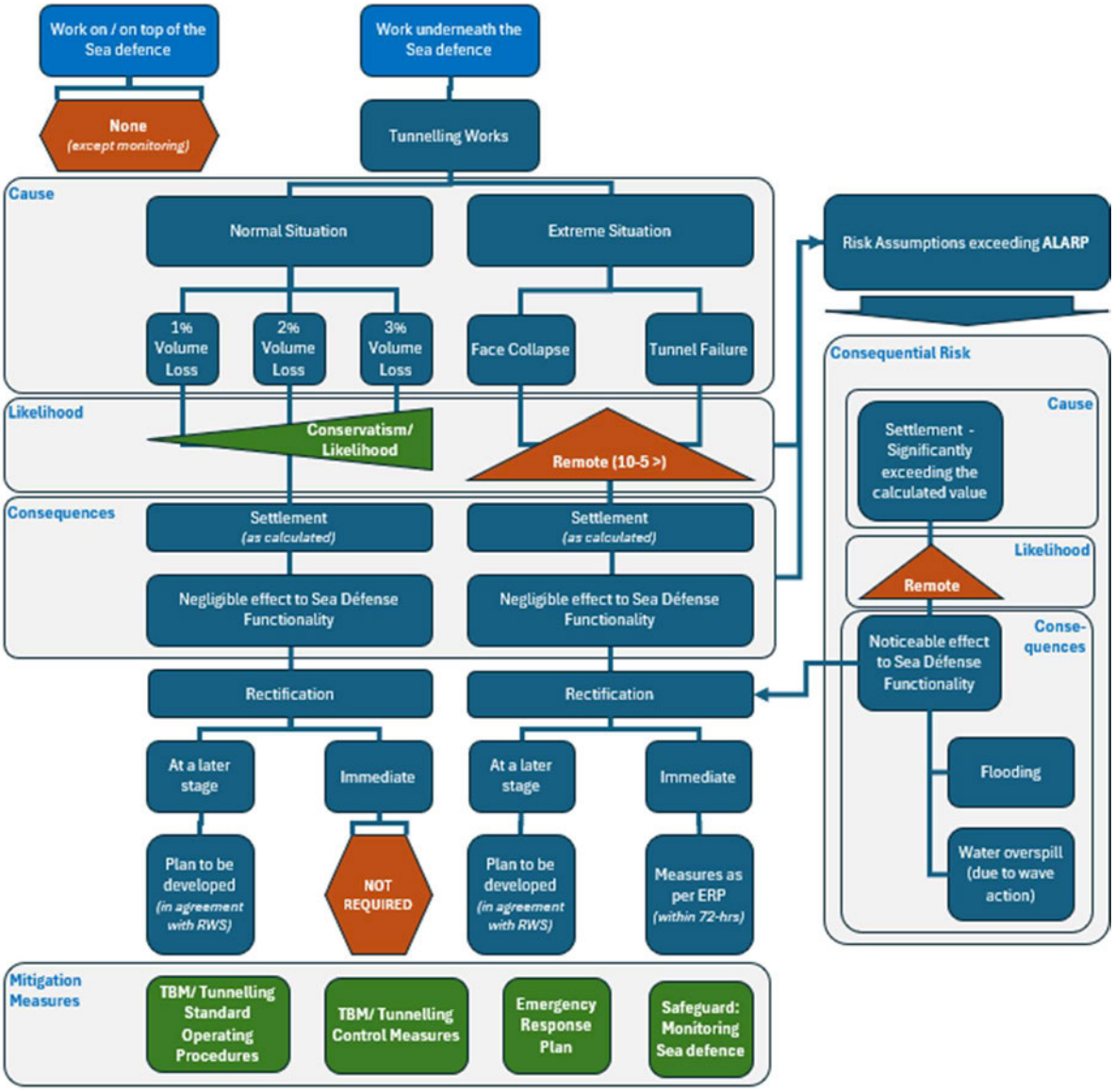
Het lage risico niveau is voornamelijk het resultaat van de geringe waarschijnlijkheid van een extreem falen scenario tijdens de tunnelwerkzaamheden en daaropvolgende significante effecten van de tunnelwerkzaamheden op de zeewering, waarbij de faalfrequentie lager is dan 10^{-5}. Er dient te worden opgemerkt dat deze faalfrequentie zelfs lager kan zijn dan de faalfrequentie die wellicht is aangenomen tijdens het ontwerp van de betreffende zeewering, die waarschijnlijk in de regio van $2,5 \times 10^{-4}$ zal liggen.

De risicobeoordeling, zoals ook gepresenteerd tijdens het overleg met Rijkswaterstaat op 10 oktober 2024, is opgenomen in Bijlage A.

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In het gedeelte over verzakkingen en zettingen wordt een aanvullende evaluatie gepresenteerd om aan te tonen dat zelfs bij de geringe waarschijnlijkheid van het ontstaan van een significante verzakking en/of zetting, de functionaliteit van de zeewering niet in gevaar komt.

Het onderstaande diagram (figuur 1) geeft meer duidelijkheid over de risico's voor de zeewering en de relatie tussen de directe en de indirecte risico's als gevolg van de tunnelwerkzaamheden.



Figuur 1 – Diagram Risicobeoordeling

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Het bovenstaande diagram (Figuur 1) laat zien dat het risico op overstromingen en/of wateroverlast een gevolgrisico is dat voortvloeit uit een scenario waarin de tunnel faalt.

Operationele fase

De hierboven beschreven risico's hebben allemaal betrekking op de bouw van de tunnel. Voor de risico's die verband houden met de tunnel tijdens de operationele fase van de zeeleiding/tunnel, wordt elke zetting als gevolg van de tunnel voorkomen doordat de tunnel anulus volledig wordt opgevuld met grout of zand na het intrekken van de zeeleiding.

Met betrekking tot de zeeleiding en de operationele fase is het risico op een explosie en/of grote CO2-uitstoot ondervangen door toepassing van de desbetreffende specificaties uit de NEN 3651. Als zodanig zal het zeeleiding-gedeelte onder de zeewering een extra grote wanddikte hebben (20% toename) en zal het deel beproefd worden tot een aanzienlijk hogere testdruk dan de rest van de zeeleiding. Verder wordt opgemerkt dat elk pijp stuk ('joint') van het zeeleiding-gedeelte onder de zeewering zal worden beproefd in de pijpen-fabriek en dat elke verbinding/rondlas 100% zal worden onderzocht door middel van niet-destructief onderzoek ('NDT').

Wanneer de volledige NEN 3651 is toegepast, voldoet de volledige zeeleiding aan de doelstelling van de NEN 3651, namelijk dat bij een verhoogde druk (dus overschrijding van de ontwerpdruk van het systeem) wordt gewaarborgd dat de delen buiten het gedeelte onder de zeewering (met een kleinere wanddikte) zullen bezwijken en niet het/de deel(en) onder of direct grenzend aan de zeewering. Opgemerkt wordt dat het gedeelte met de grotere wanddikte wordt geïnstalleerd vanaf de inlaatschacht, onder de zeewering en Maasgeul, tot aan KP11.

Functionaliteit van zeeverdediging

Hoewel de zeewering voor ontwerp- en onderhoudsdoeleinden als primaire zeewering moet worden beschouwd, kan het werkelijke doel en/of de functionaliteit van de zeewering niet aannemelijk zijn om overstromingen door hoge waterstanden te voorkomen. In het geval van hoge waterstanden, boven 5 m NAP, zullen de Sharkfin (en andere Maasvlaktegebieden achter de zeewering) overstromen door de hoge waterstanden welke via de Maasmond , het Calandkanaal , het Beerkanaal / Nijlhaven en het Yangtzekanaal binnenkomen .

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Zetting

Naast de risicobeoordeling (ref.: Bijlage A) is een zettingseffectanalyse uitgevoerd waarbij de zetting van de zeewering als gevolg van normale tunnelwerkzaamheden is bepaald, evenals een extreem geval door volledig falen van de tunnel, hoewel dergelijke faalscenario's een verwaarloosbare waarschijnlijkheid hebben. De zettingsbeoordeling met betrekking tot de zeewering is gepresenteerd in het Aramis FEED-document: NL-ARM-040-ARU1-110381, *'Tunnel FEED - Settlement Impact Analysis and Monitoring Plan - Sea Defense '* .

Een samenvatting van de resultaten van deze zettingsbeoordeling is weergegeven in Tabel 1 hieronder.

		Maximale zetting [mm]
Normaal tunnel werkzaamheden	1% volumeverlies	19
	2% volumeverlies	39
	3% volumeverlies	58
Extreem falen	'Face' instorting	245
	Tunnelfalen	360

Tabel 1 – Samenvatting van de zettingsbeoordeling

Met betrekking tot de bevindingen van de bovenstaande beoordeling, en met name de zettingen als gevolg van de normale tunnelwerkzaamheden, worden de verwachte zettingen als verwaarloosbaar beschouwd in vergelijking met de omvang van de zeewering en de manier waarop deze aan de zeezijde is opgezet (d.w.z. breukstenen, betonblokken, asfalt, enz.). Dit geldt ook voor de zettingen die zijn bepaald als gevolg van de extreme faalbelastinggevallen.

Om het effect op de zettingen ten opzichte van de functionaliteit van de zeewering te beoordelen, is een simplistische dimensionale controle uitgevoerd. Hierbij is de maximaal bepaalde zetting afgezet tegen de totale hoogte van de zeewering (boven NAP). De resterende hoogte is vervolgens uitgezet tegen de maximale golfkam ('wave-crest') en hoogwaterstand (GWH). De resulterende speling, aangeduid als 'vrijboord', variërt tussen 4,94 m en 8,70 m, afhankelijk van de geselecteerde golfrichting. Er wordt opgemerkt dat dit alleen rekening houdt met een lokale zetting/vervorming van de zeewering, en dat de zeewering zijn algehele stabiliteit zal behouden. Er wordt aangenomen dat deze aanpak gerechtvaardigd is vanwege de constructie van de zeewering aan de kust (d.w.z. breukstenen, betonblokken, asfalt, enz.) en het tunnel ontwerp.

De resultaten van deze simplistische dimensionale beoordeling worden hieronder in Tabel 2 gepresenteerd.

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		Waarde [M]
Top van de zeewering		14,5 (NAP)
Hoogwater (GWH)		1.1 (NAP)
Metocean-gegevens – Maximale piek (10.000 jaar terugkeerperiode)	NW	8.10
	NNW	7.28
	N	4.34
Vrijboord: max. kruin minus min. zeeweringhoogte (rekening houdend met tunnelfalen)	NW	4.94
	NNW	5.76
	N	8.70

Tabel 2 – Dimensionale beoordeling

Opgemerkt dient te worden dat de impact van de tunnelwerkzaamheden op de zeewering beperkt is tot de eerdergenoemde verzakking/zetting. De tunnelbooroperatie is de enige activiteit die plaatsvindt naast en onder de zeewering. Tevens dient opgemerkt te worden dat er gedurende de gehele periode van aanlanding en/of tunnelbouwwerkzaamheden geen bouwactiviteiten aan het oppervlak of direct op de zeewering zullen worden uitgevoerd. De enige activiteiten die op de zeewering worden voorzien, zijn de installatie van de onderzoeksbasisstations, d.w.z. via palen en grondplaten. Voor meer details wordt verwezen naar het onderstaande gedeelte over monitoring.

Toezicht

Om de verzakkingen/zettingen tijdens de tunnelwerkzaamheden te monitoren, zal Aramis een monitoringcampagne opzetten. Daarom, en als onderdeel van een grotere monitoringscope met betrekking tot de Haaienvin (‘Sharkfin’), zal Aramis ook een onderzoek uitvoeren en een monitoringcampagne opzetten van de zeewering vóór, tijdens en na de voltooiing van de tunnelwerkzaamheden.

Het doel van deze monitoringcampagne is om eventuele verzakkingen van de zeewering te identificeren en ervoor te zorgen dat de verzakkingen binnen de verwachte waarden blijven, waarbij de verzakking gerelateerd aan het 1% volumeverlies de basiswaarde zal zijn. Elke verzakking die deze basiswaarde overschrijdt, zal de nodige maatregelen in gang zetten om de verzakkingswaarden binnen de basiswaarden te brengen. Dit kan worden bereikt door de frontdrukken te verlagen of te verhogen en/of de tunnelsnelheid te verhogen of te verlagen.

Dergelijke controlemaatregelen worden vastgelegd in de tunnelprocedures die door de tunnelaannemer moeten worden ontwikkeld en vormen al een cruciaal onderdeel van de standaardwerkprocedures van de tunnelaannemer.

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Bij een extreme tunnelbreuk (d.w.z. een volumeverlies van meer dan 3%) zullen de monitoringsmetingen, naast de noodzakelijke beheersmaatregelen, ook het noodplan in werking stellen met betrekking tot eventueel noodzakelijke herstelwerkzaamheden aan de zeewering.

Onderzoek/Survey werk

Vóór de start van de tunnelwerkzaamheden wordt een baseline conditie survey uitgevoerd op de zeewering en de directe omgeving, die wordt vastgesteld door een LIDAR Digital Terrain Model (DTM). Er worden twee methoden overwogen, namelijk: (i) het gebruik van een drone survey op een gebied van 400 x 500 meter of (ii) door middel van terrestrische laserscanning (NavVis mobiel scanning systeem) waarbij alleen de zeewering wordt vastgelegd op een gebied van 100 x 150 meter.

Nadat de tunnelwerkzaamheden zijn afgerond, wordt de toestand van de zeewering na de bouw vastgelegd met behulp van dezelfde methode die is gekozen voor het basisconditieonderzoek.

Monitoringwerkzaamheden

Om de zeewering tijdens de bouw van de schacht en de tunnelwerkzaamheden te monitoren, wordt een combinatie van twee methoden overwogen: (i) handmatige en (ii) automatische vervormingsmetingen, zoals hieronder gedetailleerd:

- *Handmatige vervormingsmetingen:*
  Voor de handmatige vervormingsmetingen worden 24 meetpunten (palen met meetmarkering) in 4 rijen geplaatst. Eén rij wordt bovenaan de zeewering geplaatst. De andere 3 rijen worden loodrecht op de eerste geplaatst (één boven op het tunneltracé en één aan elke kant).

  Na de installatie wordt een nulmeting in RD/NAP-coördinaten uitgevoerd door middel van tachometer en precisie-nivellering waarbij de meetpunten ook worden verbonden met ten minste twee Overheidsbenchmarks. De vervormingspunten worden gemeten met een vaste frequentie vóór, tijdens (met een verhoogde frequentie) en nadat de tunnelwerkzaamheden de zeewering zijn gepasseerd.

- *Automatische vervormingsmetingen:*
  Voor de automatische monitoring worden 9 GNSS-ontvangers in drie rijen op de zeewering geïnstalleerd (één boven op het tunneltracé en één aan elke kant). Een extra GNSS-referentiepunt wordt buiten de invloedszone van de tunneling geïnstalleerd. De GNSS-ontvangers bepalen elke minuut hun eigen positie in drie richtingen (X, Y en Z). De gegevens worden elke tien minuten door een gateway verzameld.

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Het systeem biedt de mogelijkheid om drempelwaarden voor de metingen in te stellen, waardoor er een alarm afgaat wanneer de drempelwaarden worden overschreden.

Rectificatie

Op dit moment is de aannemer voor de tunnelwerkzaamheden nog niet aangesteld, terwijl de rectificatiemethodologie door de tunnelaannemers moet worden ontwikkeld als onderdeel van hun algehele werkterrein. Er wordt opgemerkt dat de rectificatiewerkzaamheden in twee scenario's kunnen worden verdeeld, namelijk:

- Herstel na voltooiing van tunnelwerkzaamheden
- Onmiddellijke herstel/rectificatie.

Herstel na voltooiing van tunnelwerkzaamheden

Herstel van verzakkingen als gevolg van normale tunnelwerkzaamheden, d.w.z. zetting door volume verlies, wordt uitgevoerd na voltooiing van de tunnelwerkzaamheden. De verzakkingswaarden zoals gepresenteerd in het FEED-rapport: NL-ARM-040-ARU1-110381, *'Tunnel FEED - Settlement Impact Analysis and Monitoring Plan - Sea Defense ',* zijn beschouwd als de geselecteerde drempelwaarde, waarbij elke ervaren verzakking, die de verwachte verzakkingen/zettingen volgens het betreffende FEED-rapport niet overschrijdt, geen rectificatie/herstel behoeft.

Een dergelijke rectificatie wordt door de tunnelbouwer ontwikkeld, waarbij het werkprotocol ter beoordeling aan Rijkswaterstaat wordt voorgelegd.

Onmiddellijke herstel/rectificatie

Herstel als gevolg van een extreem falen, kan leiden tot een onmiddellijke en potentiële tussentijdse herstelmaatregel (verwacht binnen 72 uur). Net als bij de hierboven beschreven herstelmaatregel moet de tunnelbouwer de benodigde procedure ontwikkelen.

Een dergelijke herstelmaatregel wordt opgenomen in een zogenaamd noodplan (‘Emergency Response Plan’ - ERP), dat ook alle andere noodgevallen met betrekking tot de tunnelwerkzaamheden omvat.

Niettemin kan in principe al worden geconcludeerd dat de ERP zal specificeren dat de benodigde materialen (d.w.z. big bags, enz.), samen met de grond die door de tunnelbouw is afgevoerd en/of grout, ter plaatse zullen zijn om te zorgen voor tijdige herstelmaatregel van de zeewering indien nodig geacht. Bovendien zal de ERP meerdere scenario's in overweging nemen om de rectificatie van significante verzakkingen op verschillende locaties bij de zeewering vast te leggen.

Intern/Internal

Hiervoor zijn mogelijk verschillende methoden nodig vanwege de aard van de zeewering op die specifieke locatie (bijv. asfalt, betonblokken, klei, gras, enz.). Zoals echter al is geconcludeerd uit de risicobeoordeling (zie Bijlage A), wordt de werkelijke noodzaak van een dergelijke rectificatieprocedure en/of ERP als gering beschouwd.

Het ERP wordt, net als het hierboven genoemde werkprotocol voor herstelwerkzaamheden, ter beoordeling aan Rijkswaterstaat voorgelegd.

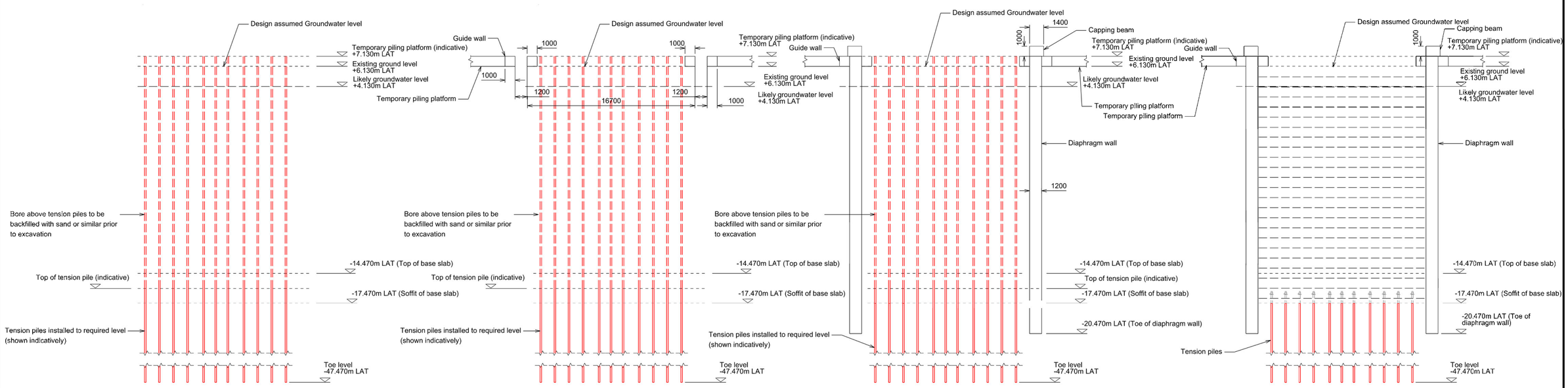
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9. Shaft Construction Sequence

NL-ARM-040-ARU1-110332_01_IFD_20240904_01

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Stage 1 :
Construct Tension piles (in wet)

Stage 2 :
Construct temporary piling platform and guide walls

Stage 3 :
Construct Diaphragm Wall (in wet) and Capping Beam

Stage 4:
Excavate shaft in the wet to the bottom of the shaft and install connection detail (e.g steel plates) at the top of the tension piles

Stage 5:
Cast 1st stage base slab (unreinforced) in the wet

Stage 6:
Cast 2nd stage base slab (reinforced) in the dry

Stage 7:
Construct tunnel-eye and thrust wall

Notes

1. All dimensions are in mm unless stated otherwise.
2. All levels are in metres relative to LAT unless stated otherwise.
3. Thicknesses do not include tolerances.
4. The shaft design and sequence presented in drawings NL-ARM-040-ARU1-110332 to 110338 has been developed for FEED design purposes only. The EPC Contractor shall take full responsibility for validating the shaft design and developing a suitable design to detailed design.
5. For commentary relating to the FEED design please refer to the Tunnel FEED Design Report ref NL-ARM-040-ARU1-110306.
6. For specification requirements, refer to Tunnelling and Shaft Sinking Specification for EPC ref NL-ARM-040-ARU1-110316.

0 5 10 Metres
Scale 1:200

Rev.	Date
01	04/09/24
00	06/06/24

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Project Aramis
Tunnel FEED
Shaft
Construction Sequence

Syst / S-Syst :	N/A	Discipline :	CIV		
NL-ARM-040-ARU1-110332		Rev:	01	Scale:	1:200
NL-ARM-040-ARU1-110332		Format:	A1	Folio:	1 of 1

Bijlage

10. Safety Zone Calculation Sea Defence

NL-ARM-040-PTC1-114043_02_IFD_20241014_02

Safety Zone Calculation Sea Defence				Document Reference NL-ARM-040-PTC1-114043	
				Revision: 02	**Step**: IFD
				Date: 14-10-2024	
Document Type: REP	**Family Equip:** N/A	**Discipline:** PLR	**Class:** 2	Page 1 of 29	

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Offshore Independents Rotterdam

Rev	Step	Date	Revision memo	Issued by	Checked by	Approved by
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ABBREVIATIONS

Abbreviation	Description
CCS	Carbon Capture and Storage
CPT	Cone Penetration Test
D-HUBN	Distribution Hub
FEED	Front End Engineering Design
HDD	Horizontal drill
LAT	Lowest Astronomical Tide
NoBo	Notified Body (e.g. Lloyds for example)
OD	Outer Diameter
SCD	Sonic Core Drilling
ToP	Top of Pipe

DEFINITIONS

Terminology	Definition
EBN	Energie Beheer Nederland
ISO	International Organization for Standardization
MMD	Mott Macdonald
NEN	Nederlandse Normalisatie-instituut
NPR	Nederlandse Praktijk Richtlijn
RWS	Rijkswaterstaat
TTE	TotalEnergies

HOLDS

No.	Section	Hold

1 INTRODUCTION

1.1 Background

The Aramis project is a cooperation between Shell, TotalEnergies, Gasunie and EBN to utilize depleted gas fields and potentially aquifers in the Dutch North Sea to establish an open-access CCS infrastructure for transport and storage, targeting hard-to abate industrial emissions in the Netherlands and Europe.

This part of the project provides the transport of CO_2 from the onshore terminal to an offshore platform, which is used as distribution hub (D-HUBN). Other parts of the project are provided by CO2next and Porthos, while the actual storage in the depleted offshore gas fields under the North Sea is performed by Shell and TotalEnergies as part of their offshore operations.

In Figure 1-1 below an overview of the envisaged CCS infrastructure is given. The left panel illustrates the trunkline that transports the CO_2 from the Maasvlakte to the offshore storage. The middle panels illustrate the Maasvlakte Hub with the Porthos compressor station and the CO2next Terminal. The right panel shows the platforms of TEPNL and Shell.

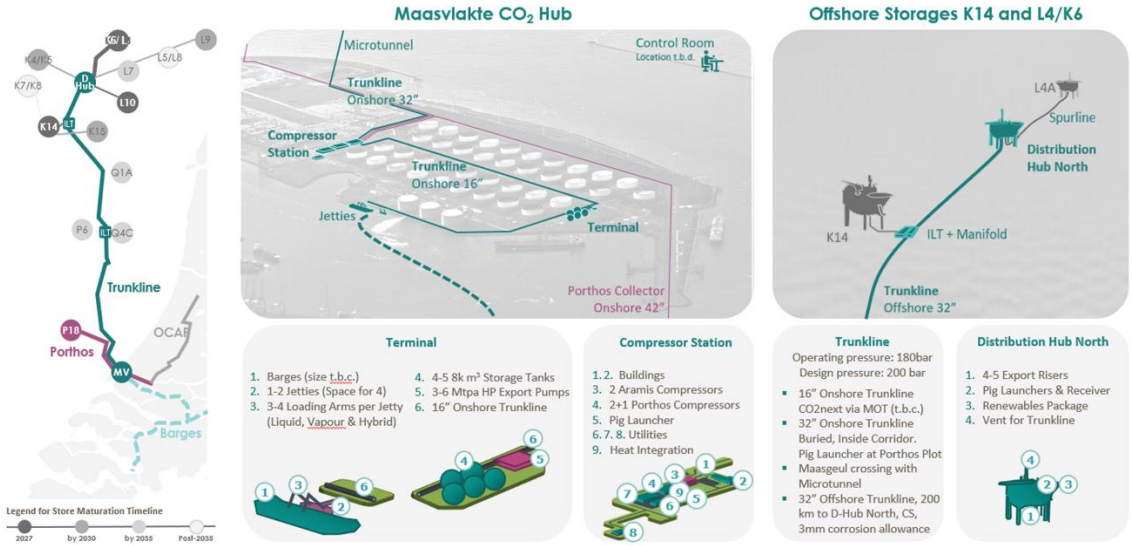


Figure 1-1 - Overview of CCS Infrastructure

1.2 Purpose of Document

The sea defence at Shark fin is a critical structure safeguarding the Maasvlakte area against high waves during storms. While not classified as a formal dike, the sea defence is required by Rijkswaterstaat (RWS) to be evaluated in accordance with NEN 3651 (2020) standard [1]. This report aims to define the safety zone for the sea defence.

The safety zone determination will influence the minimal distance of the shaft to the sea defence, the pipeline section that should be 20% stronger for internal pressure than the adjacent (offshore) section, the pipeline section that should be tested at a higher pressure and the minimum distance from the sea defence regarding the location of the valve-station. This section is included in the Landfall wall thickness report NL-ARM-040-PTC1-114044 [2].

The NEN 3651 (2020), as of the date of issue of this report, includes two limitations. These are:

1) The code does not provide specific guidance for pipelines in a tunnel, i.e. does not account additional protection provided by grouting of the tunnel and shaft;
2) The code does not define how to determine the safety zone for medium in “dense phase”, but only gaseous or liquid.

*__Note__: Dense phase CO_2 is a highly compressed fluid that demonstrated properties of both a liquid and gas. It is called a dense fluid, or supercritical fluid, to distinguish it from normal vapor and liquid.

Due to these uncertainties Deltares has been involved in the determination of the safety zone as they are participating in updating the NEN 3651 to include CO_2. New insights and formulas that are updated in the NEN 3651 code have been implemented in the Deltares study.

The safety zone results of Deltares will be evaluated to the current FEED design of the Shaft location and the onshore pipeline route.

1.2.1 Areas of attention

At the moment the NEN 3651 (2020) does not include guidelines for the erosion, explosion and softening zone for dense phase CO_2. Deltares included calculations that are currently not in the NEN 3651 but will be in the future updated NEN 3651.

Although the sea defence is treated as a primary sea defence, in reality is does not serve the same purpose. The land behind the sea defence is approx. +6.1[mLAT] and is used to ward of waves during storms.

Text literally taken from the NEN 3651 code will be in cursive with the accompanying paragraph behind it.

Prior to the Deltares report, a preliminary safety zone distance was estimated internally using the current NEN code which does not include dense phase CO_2 formulas. The estimated safety zone values were approximately 80m, 100m and 115m depending on the type of disturbance zone and location (perpendicular or parallel to the sea defence) shown in Figure 1-2. The shaft is located as far back (south) as possible in order to leave space for potential future projects located on the shark fin. After the Deltares report, these preliminary values are not used anymore and can be ignored on the drawings.

1.3 Shark fin area and landfall

The tunnel concept follows from the Preliminary Design Report - NL-ARM-040-ARU1-110302 [3] performed by ARUP.

- Crossing length ~1.750[m];
- Internal Diameter tunnel 3.0[m];
- Locations:
  - Entry shaft: Shark fin area
  - Exit pit: beyond the Maasgeul
- Hybrid tunnel solution (first part jacking-pipe and a late switch to segmental)
- Angle of tunnel to shaft 5%
- Depth shaft ~23.3[m] (incl. flat base slab)
- Shaft internal diameter 16.7[m], external 19.1[m]

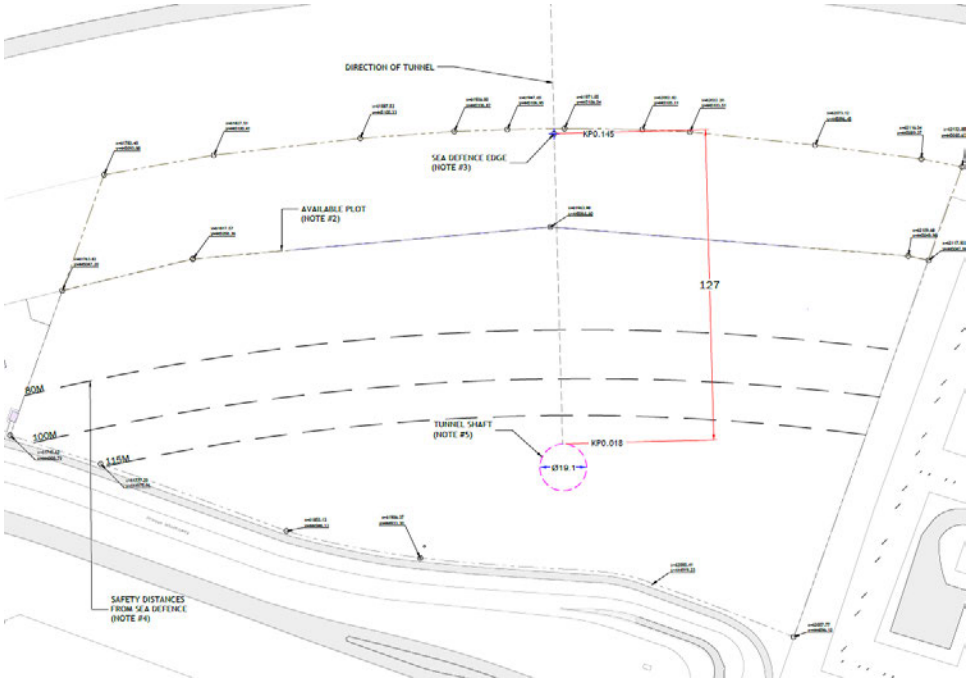


Figure 1-2 - Shaft location 127[m] from sea defence [4]

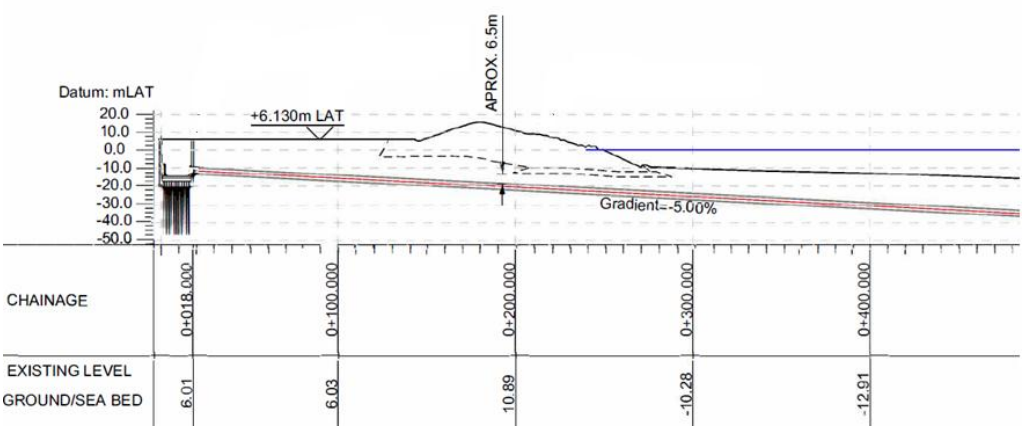


Figure 1-3 - Tunnel approach and shaft design [5]**.**

1.4 Codes & standards

The Project shall be designed, constructed, operated, and maintained and in compliance with the relevant Netherlands HSE guidelines, directives and government statutory guidelines. A systematic review of all applicable standards of each specific topic (e.g. for offshore pipelines, NEN, DNV and General Specifications) will be undertaken to ensure all requirements are fully documented in the project documents (e.g. BoD, design premises and relevant reports). Where there is misalignment between standards, codes, or regulation the most stringent should be adhered to. The misalignment and ultimate design choice shall be reported to Company.

Individual discipline philosophies and design premises will list regulations, codes and standards to be used throughout the design and will be kept regularly updated. At the end of FEED a List of Applicable Regulations, Codes and Standards will be produced.

The order of precedence of codes and standards shall be as follows

1. Statutory Netherlands Laws and Regulations
2. Project Documents (requisitions, specifications, datasheets, basis of design, and philosophies) covering qualifications and derogations to General Specifications
3. Company General Specifications
4. International Codes and Standards

References to General Specifications and Standards, in-country laws, decrees and ministerial orders, standards & regulations, and international codes and Standards shall be understood to be referenced to the latest edition in force on the Effective Date of the Contract commencement (13 November 2023) If substantially changed or new applicable codes and standards come into force after this date, Petrofac shall submit proposals for compliance to the Company's Representative in accordance with the Change Management Procedure.

2 SUMMARY

In order for a pipeline to fall outside the scope and additional safety requirements of the NEN 3651 (2020), the safety zone distance (Column 5) should be smaller than the distance to the toe line of the sea defence (Column 6). As shown in Table 2-1 all safety zones distances are less than the distance of the pipeline to the toe line of the sea defence. As such the conclusion is that the current FEED design of the shaft location and onshore pipeline with respect to the safety zone is fit for purpose and no additional NEN 3651 (2020) requirements are applicable to the onshore pipeline (up to the bottom of the shaft).

Table 2-1 - Summary Disturbance zone (Appendix A)

Location	Zone	Disturbance zone	Stability zone	Total Safety zone	Distance to sea defence
[-]	[-]	[m]	[m]	[m]	[m]
Location 1	Erosion (Gas)	34	40	**73**	146
Location 1	Erosion (Liquid)	50	40	**90**	146
Location 2	Softening (5%)	68	40	**108**	142

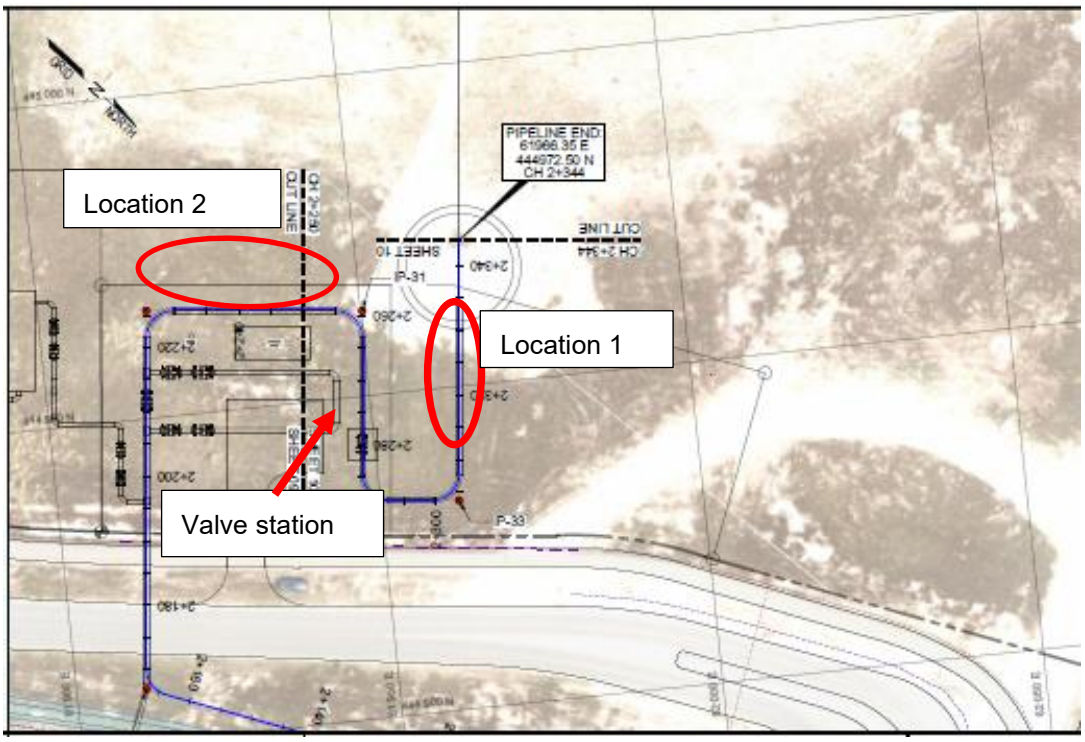


Figure 2-1: Location of ruptures

3 BACKGROUND INFORMATION NEN 3651 (2020)

3.1 Safety zone

"The important public work, together with a corridor of land on either side of the work, the safety zones, forms the area in which the NEN 3651 applies.

The safety zone is measured from the toe line of the embankment of a flood defence or a road. The width of the safety zone is equal to the radius of the disturbance zone (R_k) plus the width of the stability zone as depicted in Figure 3-1. The disturbance zone is determined by scouring and/or disturbance due to leakage, fracture or explosion of the pipeline.

For pipelines parallel to the important public work the requirements of NEN 3651 apply if the stability zone and the disturbance zone overlap one another as depicted in Figure 3-2. NEN 3651 does not apply if the stability zone and the disturbance zone touch one another (Figure 3-3) and for pipelines outside the safety zone (Figure 3-4)." NEN 3651 (2020) §6.2.1

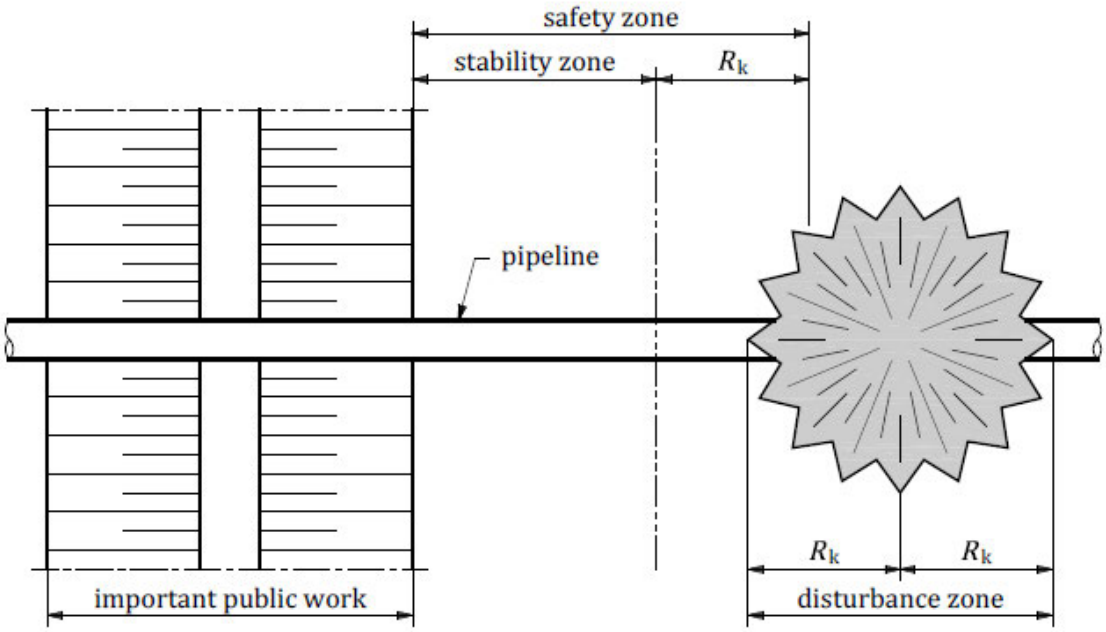


Figure 3-1 – Schematic representation of stability, disturbance and safety zone at a crossing

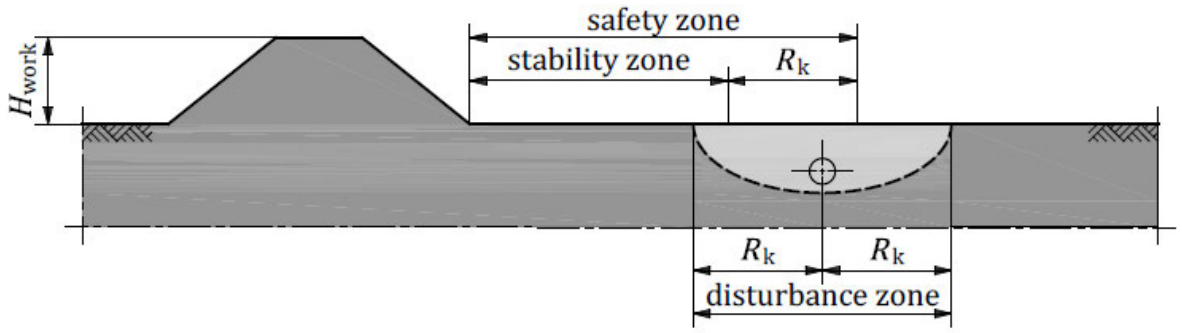


Figure 3-2 – Parallel pipeline lies within the safety zone: NEN 3651 (2020) applies

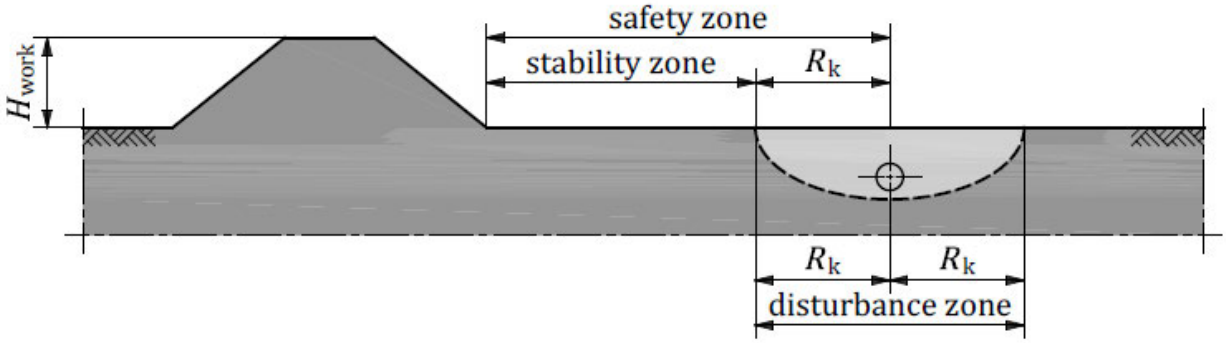


Figure 3-3 – Disturbance zone of the parallel pipeline and stability zone of the flood defence touch one another: NEN 3651 (2020) does not apply

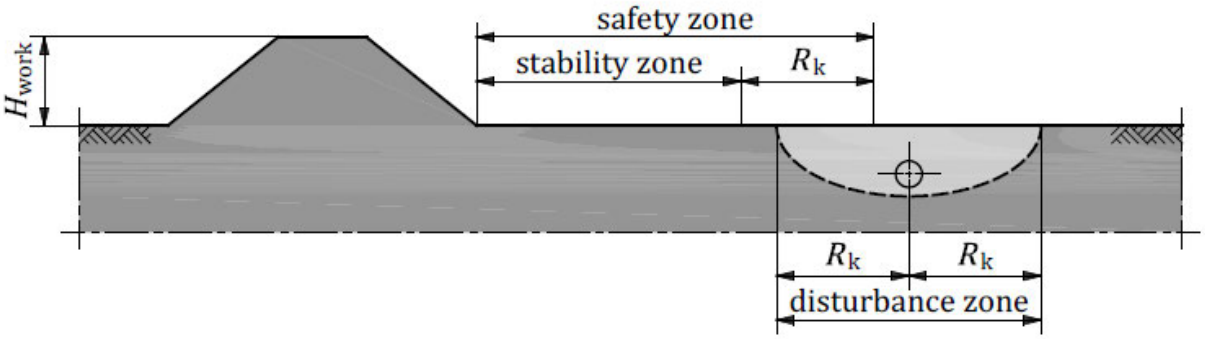


Figure 3-4 – Disturbance zone of the parallel pipeline lies outside the stability zone of the flood defence: NEN 3651 (2020) does not apply

3.1.1 Stability zone

"The width of the stability zone depends on the properties of the sub soil and the dimensions of the work. The width can be determined by a geotechnical survey. However, if it is found that there are no special circumstances that could cause loss of equilibrium of the foundation, the width of this corridor of land can be determined without further survey at four times the height of the work (H_{work}) above ground level." NEN 3651 (2020) §6.2.2.1.

A geotechnical survey can also be useful to establish whether the indicated value of four times the height as a stability zone can be reduced" NEN 3651 (2020) §6.2.2.2. (This option is not further considered in this report. The stability zone is taken as four times the height of the of the sea defence.*)*

3.1.2 Disturbance zone

"The disturbance zone determines the area in which scouring or soil disturbances can occur due to leakage, fracture or explosion of the pipeline. Depending on the type of incident the disturbance zone can be further indicated as the:

- *Erosion zone: Where the leaking fluid erodes a crater around the leak;*
- *Explosion zone: Where a crater occurs due to an exploding pipeline with a plastic zone around it that is regarded as an explosion zone. Even if no crater occurs, there is a plastic zone;*
- *Softening zone, where due to an explosion of a pipeline the shockwaves generated give rise to soil deformation and granular shifting; due to over-pressurized pore water loss of bearing capacity of sandy soil (soil softening) can occur.* NEN 3651 §6.2.3

3.1.2.1 Erosion zone

As per NEN 3651 (2020) the dimensions of the scouring hole with respect to the escape direction can be indicated as depicted in Figure 3-5 and Figure 3-6 for both lateral escape or full shear. NEN 3651 (2020) §A1.1.1.

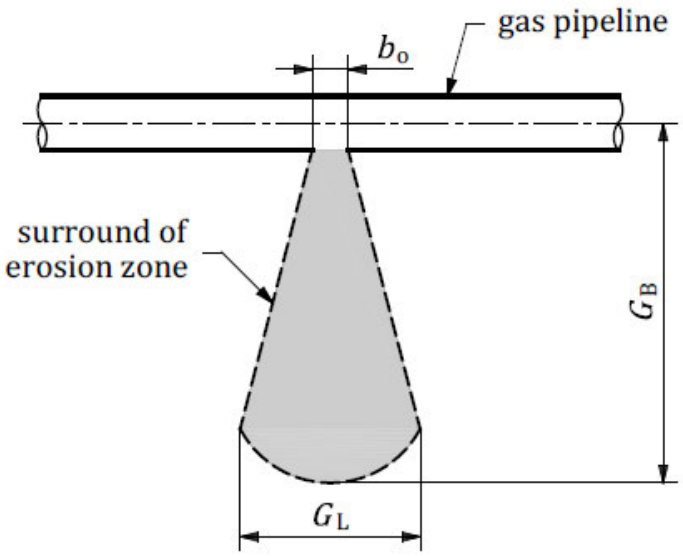


Figure 3-5 – Determination of erosion zone of gas pipelines (top view for lateral escape). For a small hole in the side: $G_l=0.5G_b$

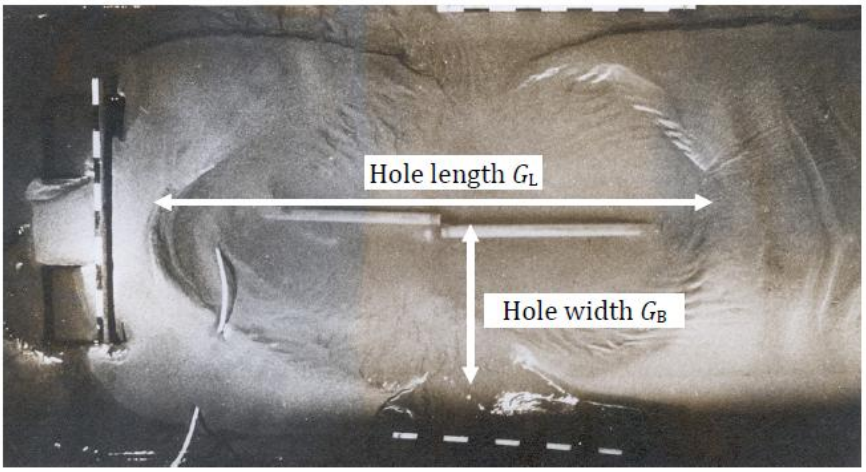


Figure 3-6 – Determination of erosion zone of gas pipelines. For full shear and escape on two sides $G_l=2G_b$. For a large hole in the side on one side $G_l=G_b$.

3.1.2.2 Explosion zone

"An explosion crater can occur due to an exploding gas pipeline. Such a crater only occurs at high gas pressures and is not related to the flammability of the gas. Around the crater a plastic zone occurs that shall be taken into account as a disturbance zone." NEN 3651 (2020) §A.1.2.1

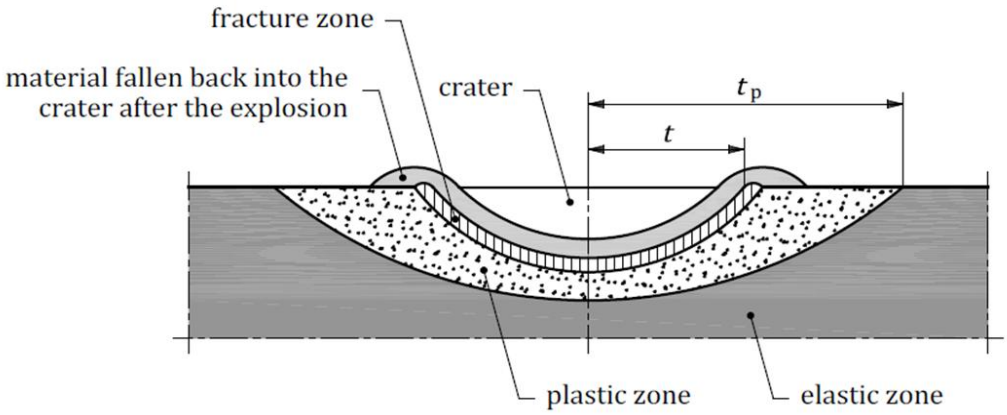


Figure 3-7 – Schematisation of the explosion zone and plastic zone

3.1.2.3 Softening zone

"Soil deformation as a result of horizontally directed shock waves in case of explosion of a gas pipeline can cause granular shifting and changes in packing density (in loosely packed sand) The result is an over-pressure in the pore water and therefore loss of bearing capacity (soil softening), expressed in a reduction in the cone penetration resistance as a percentage of the angle of internal friction." NEN 3651 (2020) §A.1.2.2

3.2 Calculation zone

"*Mechanical effects (displacement, forces) on a pipeline section outside the safety zone can still effect the section within the safety zone.*

The system calculation (strength analysis) of the crossing shall be continued until these effects have been taken into account in practise.

Theoretically all types of dampings are asymptotic processes. Their termination can in practise take place at a physically acceptable point with a replacement precondition (for example axial spring constant for the longitudinal and torsional friction, see D.2.5 of the NEN 3650-1).

The couplings with the site pipeline on either side of the crossing shall lie outside the safety zone and, if possible, outside the site section that has a mechanical effect on the pipeline section in the safety zone. The couplings are the joints made last to join the completed crossing section to the site sections (field sections).

In practice this means that attention only has to be paid to mechanical effects in the lateral direction of the pipeline (movement, settlement). Effects in in the longitudinal direction (temperature variations) have to be taken into account in the strength analysis of the crossing but not in the determination of the site of joining to the field section." NEN 3651 (2020) §6.3.

Due to the fact that the pipeline connection to the valve station is short and the valve station is located outside the 115[m] zone the displacements and forces from the connection is limited and will not affect the safety zone determination.

3.3 Relative strength

"The pipeline section in the crossing with a flood defence including safety zones, shall be 20% stronger for internal pressure than the field section.

A 20% higher relative strength shall be applied in the pipeline route at the point where there are pipeline tunnels and viaducts in roads.

Calculated stresses from internal pressure shall be multiplied by 1.2 and shall be smaller than or equal to the permissible stress using an importance factor S=1.0.

For steel pipelines only the stress due to internal pressure without external loads shall be taken into account.

Pressure reducing facilities are not allowed between crossing and intended pipe length". NEN (2020) 3651 §6.6. This section is included in the landfall wall thickness report NL-ARM-040-PTC1-114044 [2].

3.4 Functional design

3.4.1 General

"Crossing with or parallel laying of pipelines within the scope of the standard for an important public work shall be avoided, unless additional measures are taken according to this standard." NEN 3651 (2020) §7.1

3.4.2 Crossing with an important public work

"The crossing of a pipeline with an important public work shall preferably (excluding deep HDDs) be carried out perpendicular to the longitudinal direction of the important public work in a place where no engineering structures, connections or special provisions are present unless this is not possible for technical, economic or planning reasons." NEN 3651 (2020) §7.1

3.4.3 Parallel position with an important public work

"A pipeline parallel to an important public work is preferably laid outside the safety zone next to the important public work. The pipeline then falls outside the scope of the standard.

If there is insufficient space present near the flood defence, consideration can be given to locating the pipeline outside the profile of the flood defence but within the safety zone, in consolation with the authority responsible for managing the flood defence.

If a part of the pipeline is laid in the safety zone, this part shall meet the requirements for crossings with important public works." NEN 3651 (2020) §7.3

3.5 Closed season

"Installation of crossings with primary flood defences is permissible exclusively outside the closed season. This applies for both open excavation and for trenchless technology." NEN 3651 (2020) §8.1.3.5

The closed season runs from 1 October to 1 April. The execution of the installation of the tunnel is estimated up around 9 months. A meeting with RWS is held on the 8th of July 2024 [7] where RWS agrees that the closed season does not apply to this project. However, studies need to be performed to identify the risks. These studies will not be further elaborated in this report.

3.6 Entry and exit pit

"Entry and exit of a pipeline installed with an HDD technique shall lie outside the safety zones of the important public work." NEN 3651 (2020) §8.6.1

As such it is assumed that it is also **not** allowed to locate the shaft for the hybrid tunnelling solution within the safety zone.

3.7 Testing

"The crossing section shall be separately hydrostatically tested for strength (that is before this is jointed to the adjacent field section). For the execution of the strength test, please refer to 9.16 of NEN 3650-1 and the relevant provisions from NEN 3650-2." NEN 3651 (2020) §9.7.1

The (strength) test pressure is discussed in NL-ARM-040-PTC1-114044 Landfall wall thickness report.

3.8 Casing pipes

"If casing pipes are used as a second fluid barrier… (Not applicable for Aramis)

Relatively thin-walled plastic casing pipes that are intended exclusively for the lead-through of (fibreglass) cables, where applicable bundled, constitute a threat to an important public work due to:

- *Implosion of the casing pipe*
- *Seepage along the casing pipe.*

Implosion can be prevented by manufacturing the casing pipe of pipes (for example steel) that are resistant to implosion or filling them with a fluid (for example ground water)." (Not applicable for Aramis) NEN 3651 (2020) §8.6.3.3

4 DESIGN ASSUMPTIONS

In this chapter the design assumptions will be listed for the onshore pipeline. The pipeline alignment sheets are in NL-ARM-030-PTC1-113764 [8]. The current shaft location is located at 127[m] from the toe line of the sea defence (measured from the front (north) of the shaft, Figure 1-2) and is discussed in the layout arrangement justification note NL-ARM-040-PTC1-110265 [4].

The required safety zone will be calculated for the Erosion zone in location 1 and for the Softening zone (liquefaction) in location 2 which is depicted in Figure 4-1. The considered locations are explained in the Deltares report.

a) Location 1 is positioned downstream from the valve station and as such, ~200km pipeline containing dense phase CO_2 will flow out of the rupture. Location 1 is approximately 144m from the sea defence;

b) Location 2 is the closest parallel pipeline to the sea defence and is positioned approximately 140m from the sea defence.

The pipeline in the tunnel is considered within the safety zone and shall adhere to the NEN 3651 (2020) as explained in NL-ARM-040-PTC1-114044 [2].

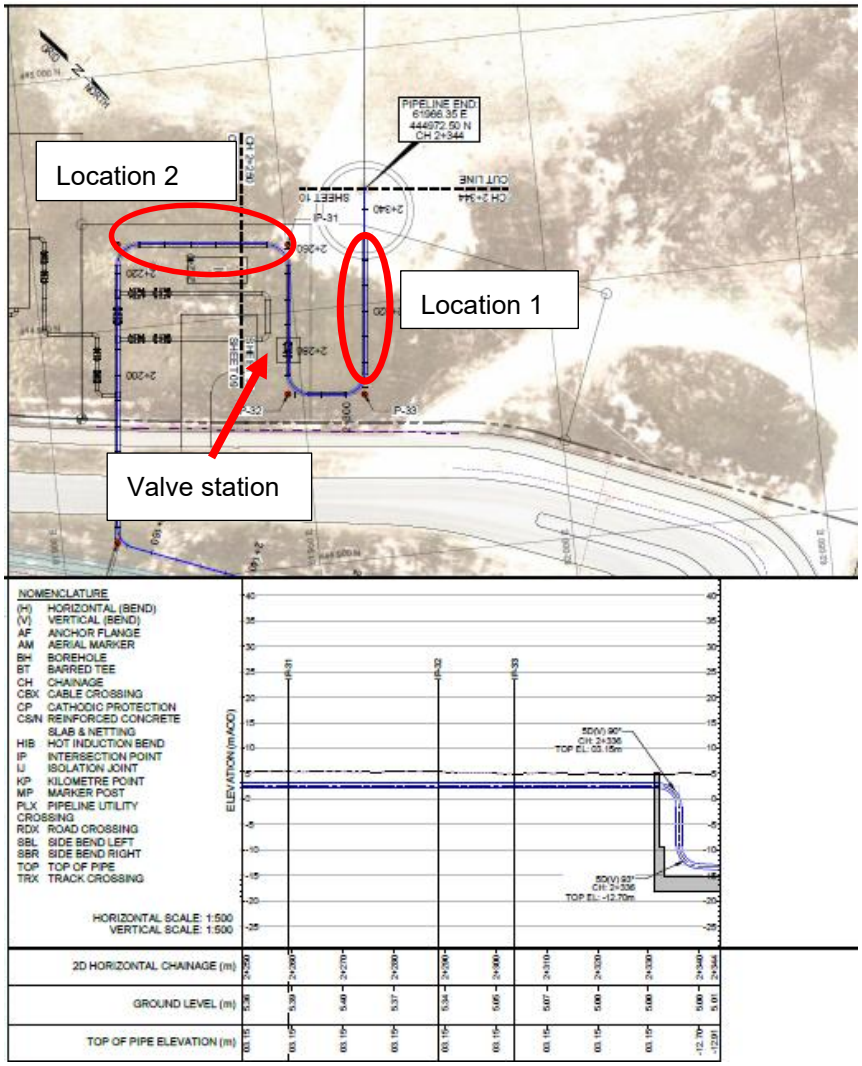


Figure 4-1 – Pipeline route [8]

4.1 Process data

Table 4-1 – Process data [9]

Pipeline	Medium	Operating Pressure	Design Pressure	Minimal Design Temperature	Maximum Design Temperature
[-]	[-]	[barg]	[barg]	[ºC]	[ºC]
Onshore	Dense phase CO2	185	200	-25	70

4.2 Pipeline data

Table 4-2 – Pipeline data [2]

Pipeline	Internal diameter	Isolation and/or coating	Orientation to sea defence	Negative fabrication tolerance	Corrosion Allowance	Nominal Wall Thickness
[-]	[m]	[y/n]	[-]	[mm]	[mm]	[mm]
Location 1	0.7556	Y	Perpendicular	1	3	39.8*
Location 2	0.7556	Y	Parallel	1	3	39.8*

*NL-ARM-030-PTC1-113754 Onshore WT report

4.3 Soil investigation

For the design of the pipeline for the Aramis Project, a soil investigation has been carried out by Fugro along the pipeline route both on- and offshore in accordance with the ISO 19901-8:2014 [10] and GS-EP-GEO-102 [11]. The geotechnical information is found NL-ARM-100-ARM1-100013_01_IFU_02 [12].

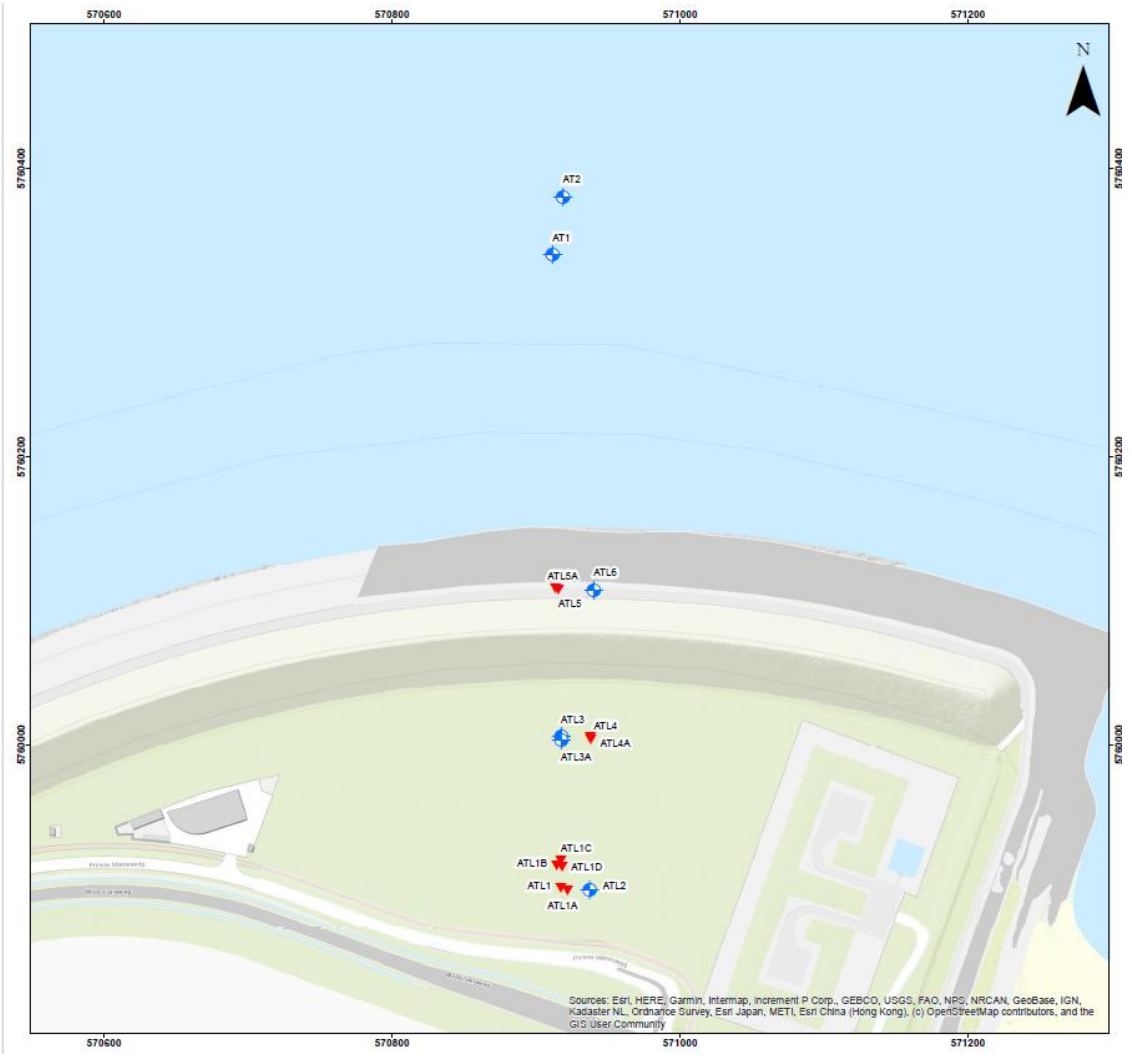
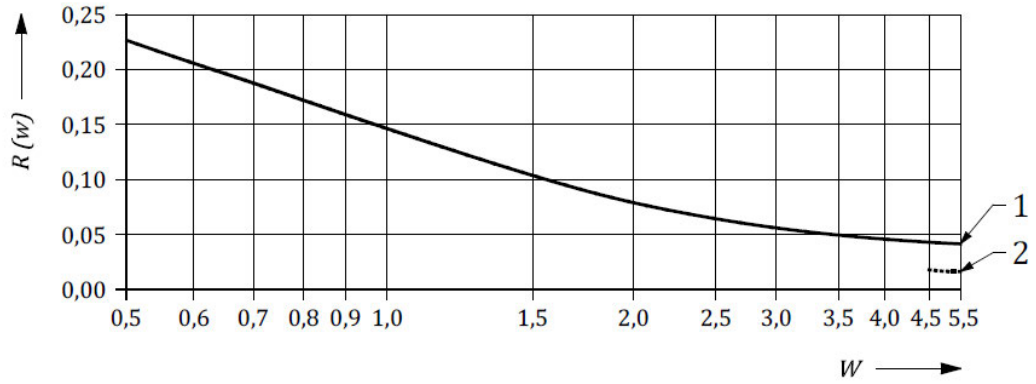


Figure 4-2 – CPT and SCD locations [12]

4.3.1 Moisture dependent constant

The moisture content (w) determines the moisture-dependent constant (R_w) as depicted in Figure **4-3**. R_w is used as a variable in the Erosion zone calculation. Since the pipeline is on a depth where it falls between two Rw value (0.025 and 0.04, explained in Figure 4-3) a worst-case estimate is assumed of 0.04(-).



Key

1	sand $W \leq 5$ %, for a moisture content for the sand >5 % a linear interpolation is made for W
2	saturated wet sand, clay, peat
$R(w)$	is a moisture-dependent constant
W	is the moisture content in % by weight dry matter (%)

NOTE For a moisture content for sand > 5 % and for topsoil and clay a value of 0,025 is used.

Figure 4-3 – Determining R(w) (NEN 3651 (2020) §A.1.1.2)

4.4 Stability zone

The stability zone is required to determine the final safety zone. The height of the important public work is 10[m] as seen in Figure 4-4. The stability zone, as per NEN 3651 (2020) requirements, is taken as four times the height of the sea defence.

Note: Currently no official cross view drawing is available of the sea defence. An AutoCAD drawing was constructed from a PDF file. The sea defence height measured in AutoCAD is 9.65[m]. To account for some small rounding errors due to converting the file, 10[m] is assumed.

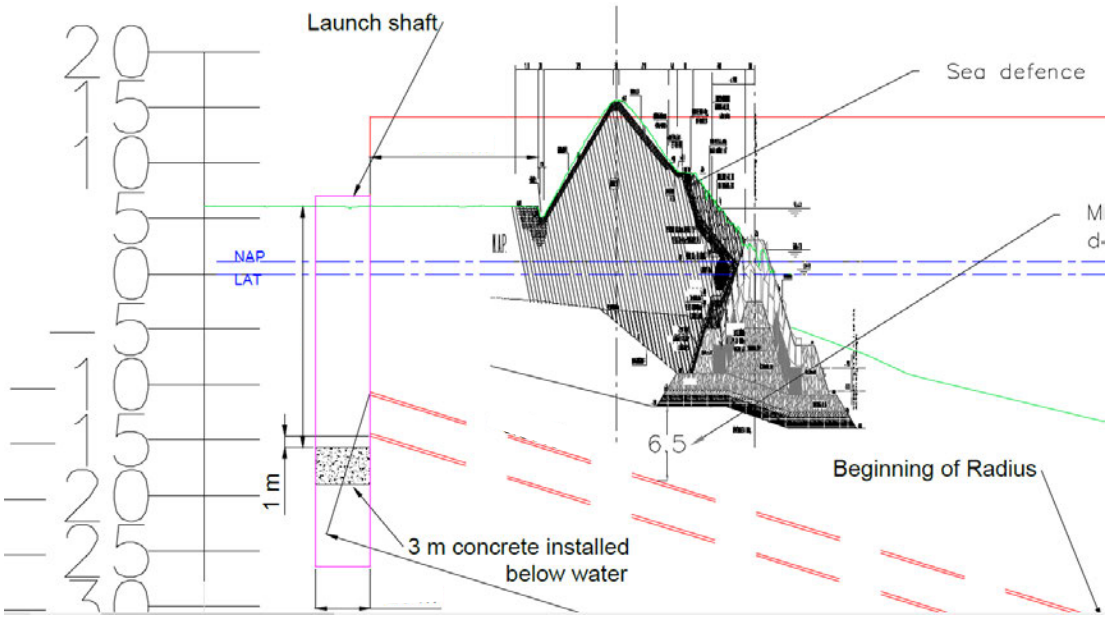


Figure 4-4 - Cross view of the important public work*

***Note**: Shaft and tunnel design outdated. This figure only serves an illustrative purpose of the sea defence.

Table 4-3 - Stability zone

Height Important Public Work	Stability zone
[m]	[m]
10	40

5 SAFETY ZONE RESULTS

The results from the Deltares study are included in Appendix A. A brief summary is provided below. As mentioned in Chapter 3.1 the disturbance zone is determined by the largest of the Erosion and Softening zone. The explosion zone is not included as it is negligible compared to the other disturbance zones.

5.1 Erosion zone (Gaseous phase)

Both the upstream and downstream part of the pipeline will both have a different Erosion zone due to the amount of CO_2 coming from each direction. The largest is 33m coming from the downstream side as shown in Table 5-1.

Table 5-1 – Gaseous phase Erosion zone

Pipeline part	Orientation to sea defence	ToP (below ground level)	Erosion zone
[-]	[-]	[m]	[m]
Upstream	Perpendicular	2	13
Downstream	Perpendicular	2	33

5.2 Erosion zone (Liquid phase)

Two liquid formulas have been investigated. However the initial condition is considered to overestimate the crater dimension since most liquid will transition to gas after 400 seconds due to pressure changes (Figure 5-1). The steady state is considered to be the best estimate. The results are included in Table 5-2.

Table 5-2 – Liquid phase Erosion zone

Situation	Orientation to sea defence	ToP (below ground level)	Erosion zone
[-]	[-]	[m]	[m]
Steady state	Perpendicular	2	50
Initial condition (not realistic)	Perpendicular	2	72

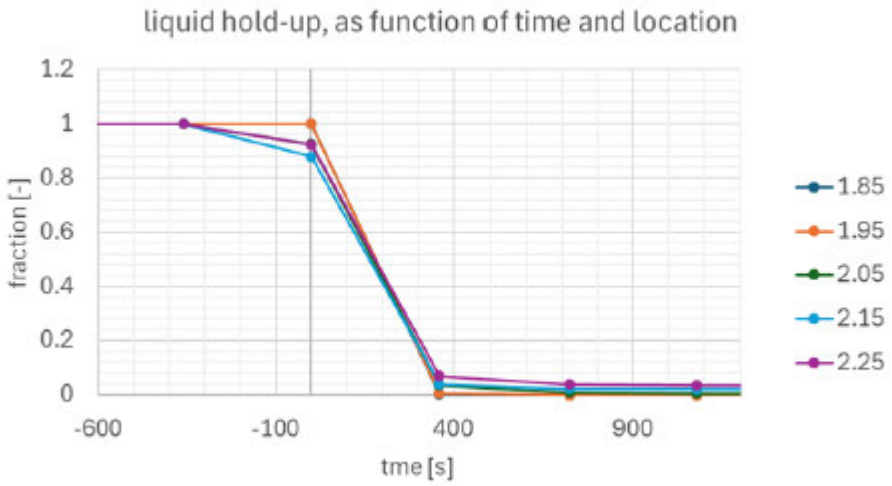


Figure 5-1: Liquid / Gaseous ratio in the pipeline

5.3 Softening zone (Liquefaction)

As per NEN 3651 (2020), the base case is that the loss of strength in the sea defence cannot exceed 5%. However, a sensitivity analyses is performed to include other strength loss zones. These are included in Table 5-3.

Table 5-3 - Softening zone

Pipeline part	Loss of strength	Orientation to sea defence	Softening zone
[-]	[%]	[-]	[m]
Location 2	5	Parallel	68
Location 2	10	Parallel	53
Location 2	25	Parallel	38
Location 2	100	Parallel	23

5.4 Safety zone results and conclusion

The final results are presented in Table 5-4. The total safety zone, which is a combination of the disturbance zone (taken from the Deltares study in Appendix A) and the stability zone, is shown in column 5. In order for a pipeline to fall outside the scope of the NEN 3651 (2020) the total safety zone distance (column 5) should be smaller than the distance to the toe line of the sea defence (column 6).

Table 5-4 - Summary Disturbance zone

Location	Zone	Disturbance zone	Stability zone	Total safety zone	Distance to sea defence
[-]	[-]	[m]	[m]	[m]	[m]
Location 1	Erosion (Gas)	34	40	**73**	146
Location 1	Erosion (Liquid)	50	40	**90**	146
Location 2	Softening	68	40	**108**	142

The current FEED design is fit for purpose. During the detailed design phase, the EPC contractor shall verify the Deltares report on potential design updates.

6 REFERENCES

[1] "Email from RWS - Re-status zeekering - dated 24 May 2023".

[2] "NL-ARM-040-PTC1-114044, Landfall wall thickness report".

[3] "NL-ARM-040-ARU1-110372, Technical Note – Tunnel Alignment," 2024.

[4] "NL-ARM-040-PTC1-110265, Shaft Location Selection Technical Note".

[5] "NL-ARM-040-ARU1-110323, Tunnel FEED - Tunnel Plan view and longitudinal profile".

[6] "NL-ARM-040-ARU1-110381, Tunnel FEED - Settlement Impact Analysis".

[7] "Email: Aramis Landfall - Sea Defense - RWS Meeting 08JUL (FINAL)".

[8] "NL-ARM-030-PTC1-113764 Onshore Pipeline Alignment Sheet".

[9] "ARM-PFE-BB0-APP-BOD-0237 V0," 2023.

[10] International Organization for Standardization, *ISO 19901-8:2014 Petroleum and natural gas industries,* 2014.

[11] TotalEnergies, *GS-EP-GEO-102.*

[12] Fugro, "NL-ARM-100-ARM1-100013_01_IFU_02_Landfall-and-Nearshore-Pipeline-Route_Measured-and-Derived-Geotechnical-Parameters_Final-Results," 2023.

[13] NEN, "NEN 3651: 2020 (en), Additional requirement for pipelines in or nearby important public works.," NEN, 2020.

[14] Petrofac, *Preliminairy release study,* 2024.

[15] NL-ARM-040-PTC1-110261, SharkFin Layout.

[16] "100111091-TN-0005, Aramis Pre-FEED Studies – Profile Design and Constraints, 06.09.2023.pdf".

[17] "NL-ARM-040-PTC1-110262, Layout arrangement justification note," 2024.

APPENDIX A DELTARES SAFETY ZONE CALCULATION

Deltares

Crossing Aramis pipeline with Maasvlakte sea defence

Estimation crater dimensions in case of a rupture



Crossing Aramis pipeline with Maasvlakte sea defence
Estimation crater dimensions in case of a rupture

Author(s)
[redacted]

Crossing Aramis pipeline with Maasvlakte sea defence
Estimation crater dimensions in case of a rupture

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Reference	Quotation 11210760-001-GEO-0001 Purchase order PFML5066052
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Summary

The objective of the Aramis project is to store CO2 in depleted offshore gas fields, under the North Sea. The CO2 will be collected at an onshore collection hub on the Maasvlakte. From there the CO2 will be transported at a high-pressure (about 180 bar) in the so-called dense phase. The length of the offshore pipeline is 194 km. This pipeline will pass underneath the existing flood defence (dike) of the Maasvlakte. A tunnel existing of concrete elements will be constructed for the crossing of the flood defence.

Below the flood defence the pipeline is located at a large depth in the tunnel. Near the entry of the tunnel the pipeline will be installed at shallow depth. The soil cover on top of the pipeline is expected to be 2 m (in this report a possible soil cover of 1.5 m is considered too). At the location of the entry shaft of the tunnel, a pipeline rupture may cause soil erosion due to the outflow of the CO_2 so that a crater can develop. The crater might affect the stability of the flood defence structure in case the crater location is close to the flood defence. Pipeline failure might also cause the possible liquefaction (loss of soil strength due to excess pore pressure) of the granular subsoil, since the behaviour of the sudden CO_2 outflow can generate a shock wave in the subsoil.

According to the regulations for the safety of water retaining structures (flood defences), the influence of a rupture of the pipeline should be considered. The Dutch code NEN-3651 prescribes calculation procedures for the calculation of dimensions of the erosion crater and of the zone with loss of soil strength (liquefaction). The code gives calculation procedures for a gas pipeline and for a liquid pipeline.

In case of a rupture of the Aramis pipeline with dense phase CO_2, the transition of the dense phase (comparable to a liquid phase) to the gaseous phase will occur during the outflow of CO_2. For this situation no calculation procedures are given in NEN-3651. This report describes a first approach to calculate the crater size and the zone with loss of soil strength in case of pipeline rupture near the tunnel entry. For the calculations use is made of the background of NEN3651 and NPR 3659 and the green version of NEN-3651 (2025).

The outflow characteristics of the CO_2 are calculated and reported by Petrofac. The outflow report is included in an Appendix of this report. These calculations yield that the phase of the outflow of the CO_2 at the rupture location is uncertain. The CO_2 next to the pipeline can be gas or liquid. Most likely a combination of gas and liquid can be expected, whereby during time the percentage of liquid decreases and the percentage of gas increases. Since no calculation model for this combined outflow is available in the first step (described in this report), it was decided to consider two situations: (i) fully gas outflow and (ii) fully liquid outflow. The crater dimensions for these two situations and a possible soil cover of 1.5 m or 2.0 m are calculated.

The results of the calculations are:
- Modelling the outflow as gas outflow: maximum crater size: 34 m.
- Modelling the outflow as liquid outflow: maximum crater size: 50 m.
- Zone with loss of soil strength:
  - Zone with 5% or more loss of strength: maximum distance 68 m.
  - Zone with 25% or more loss of strength: maximum distance 38 m.

These results can be used to evaluate the effects of a pipeline rupture on the stability of the sea defence of the Maasvlakte.

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1 Introduction

The objective of the Aramis project is to store CO2 in depleted offshore gas fields, under the North Sea. The CO2 will be collected at an onshore collection hub on the Maasvlakte. From there the CO2 will be transported at a high-pressure (about 180 bar) in the so-called dense phase. The length of the offshore pipeline is 194 km. This pipeline will pass underneath the existing flood defence (dike) of the Maasvlakte. A tunnel existing of concrete elements will be constructed for the crossing of the flood defence.

According to the regulations for the safety of water retaining structures (flood defences), the influence of a rupture of the pipeline should be considered. The regulations are described in NEN 3651.

Below the flood defence the pipeline is located at a large depth in the tunnel. Near the entry of the tunnel the pipeline will be installed at shallow depth. The soil cover on top of the pipeline is 2 m. At this location, a pipeline rupture may cause soil erosion due to the outflow of the CO_2 so that a crater can develop. The crater might affect the stability of the flood defence structure in case the crater location is close to the flood defence.

Pipeline failure might also cause the possible liquefaction (loss of soil strength due to excess pore pressure) of the granular subsoil, since the behaviour of the sudden CO_2 outflow can generate a shock wave in the subsoil.

The Dutch code NEN-3651 prescribes calculation procedures for the calculation of dimensions of the erosion crater and of the zone with loss of soil strength (liquefaction). The code gives calculation procedures for a gas pipeline and for a liquid pipeline.

In case of a rupture of the Aramis pipeline with dense phase CO_2, the transition of the dense phase (comparable to a liquid phase) to the gaseous phase will occur during the outflow of CO_2. For this situation no calculation procedures are given in NEN-3651. Deltares was asked to analyse the outflow conditions and to predict the dimensions of the crater and the loss of strength zone in case of a rupture of the Aramis pipeline. This report describes the performed analysis and calculated size of the crater and the loss of strength zone.

2 Available data

The following data was made available:

- Petrofac. ONSHORE PIPELINE ALIGNMENT SHEET. Drawing NL-ARM-030-PTC1-113764. 2024-06-28 (10 sheets).
- Petrofac. SHARK FIN CONSTRUCTION SITE LAYOUT. Drawing NL-ARM-040-PTC1-110261, issued for design. 2024-07-12 (6 sheets).
- Petrofac. Pipeline Shore Crossing and Tunnel Profile. Drawing NL-ARM-040-PTC1-113853. 2024-07-02.
- Petrofac. Offshore Pipeline Pull In Operation Drawings. Drawing NL-ARM-040-PTC1-113856.
- Petrofac. SHARK FIN CONSTRUCTION SITE LAYOUT. Drawing L-ARM-040-PTC1-110261. 2024-03-07. 2 sheets (and 6 empty sheets).
- Fugro. Measured and Derived Geotechnical Parameters and Final Results. Aramis CCS Project | Port of Rotterdam, The Netherlands. F212522-4 02, 1 November 2023.

In addition, Petrofac performed a detailed study of the outflow of CO_2 at a possible rupture. Files with the main results of these calculations were made available. A description by Petrofac of the performed calculations is attached as Annex A.

3 Approach

In the Dutch code NEN-3651, a calculation procedure is given for the calculation of the dimensions of the crater in case of gas outflow and in case of water (liquid) outflow. The calculation of the liquefaction area is also described in the NEN-3651. The calculation procedures for the crater dimensions are based on experimental research on gas pipelines and on liquid pipelines.

No calculation procedure is available, so far known in the literature, for the rupture of a pipeline containing liquefied or dense phase CO_2. After the pipeline failure, several mechanisms play a role, such as evaporation (flashing) of the CO_2 and the temperature drop. Therefore, the application of the standard calculation method is not possible. A stepwise approach is needed in order to gain insight in the outflow effects of the CO_2.

The first step is to calculate the crater dimensions in a conservative way using the existing gas and liquid models. In case the crater dimensions and the loss of strength zone are unacceptable large a more detailed study is required, and the second step should be performed.

This report concerns this first step. The results of the performed analyses are:
- Size of crater due to erosion.
- Zone with excess pore pressures.

The assessment of the influence of the crater and the liquefaction zone on the existing flood defence is not part of the scope of this report.

The calculations are based on the NEN 3651, and more particular the green version of the code version 2025. In the green version of the code it is explained that the size of the explosion crater is always smaller than the size of the erosion crater.

The green version provides the calculation of the time dependent development of the erosion crater. The time dependent calculation requires the discharge and pressure in time at the location of the rupture in the pipeline.

The assessment of the liquefaction zone is required. This is the zone with excess pore pressures due to the passing of a shock wave, generated by the rupture of the pipeline and release of CO_2. The method given in NEN-3651 will be followed, including the refinements of the green version.

4 Conditions

4.1 Location

The crossing with the sea defence is located at the Maasvlakte, next to the Nieuwe Waterweg. Figure 4.1 shows the location. A detailed view of the pipe lay-out is shown in Figure 4.2.

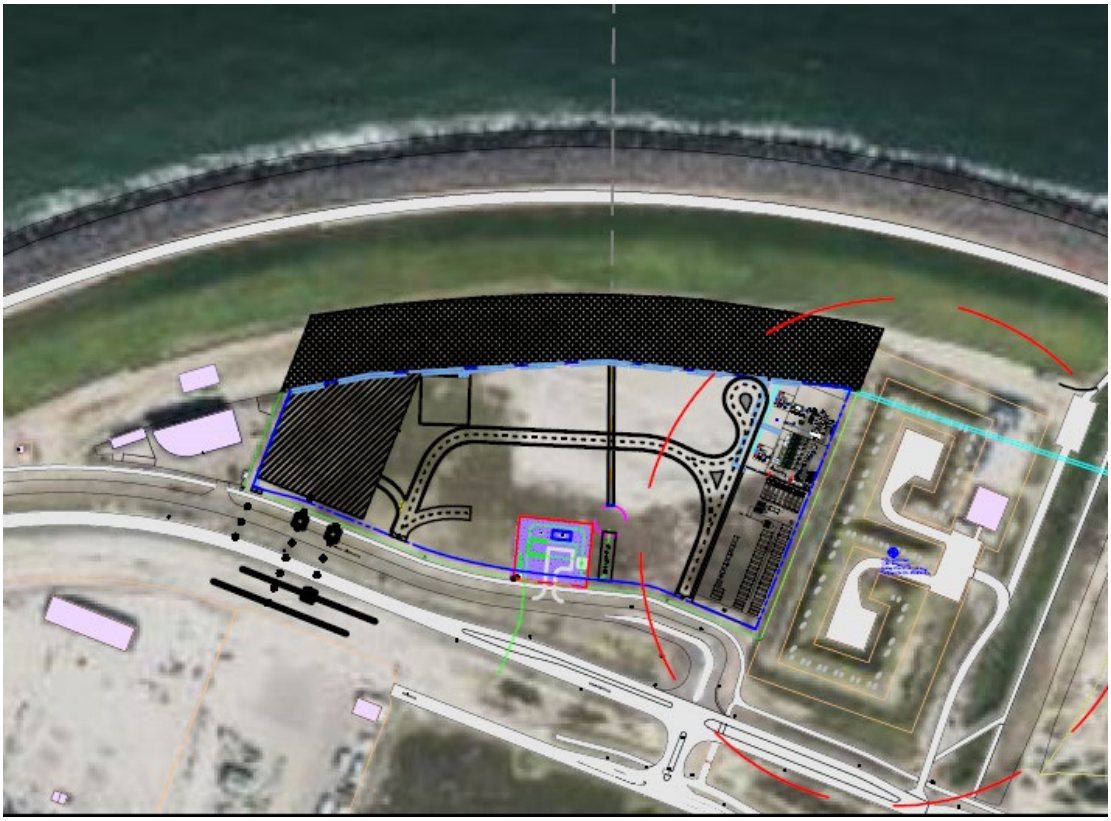


Figure 4.1 Top view location tunnel crossing sea defence, part of drawing NL-ARM-040-PTC1-110261.

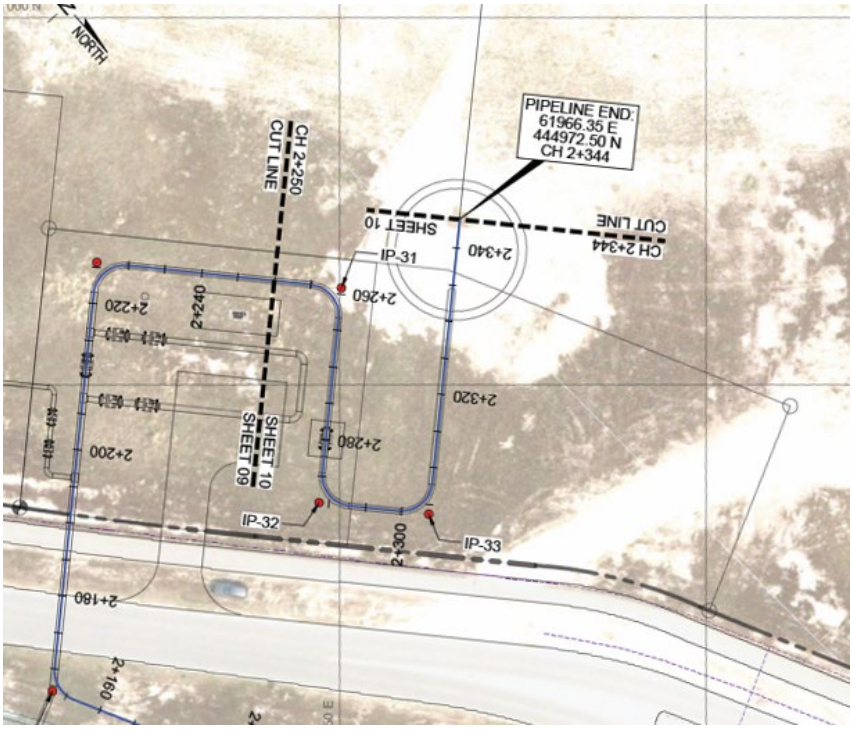


Figure 4.2 Top view location pipeline near the tunnel entrance, part of drawing NL-ARM-030-PTC1-113764.

For the crossing of the sea defence a concrete tunnel with an internal diameter of 3 m is constructed. The pipeline is installed in this tunnel. After installation of the pipeline the void between pipeline and tunnel is filled with low strength concrete.

Figure 4.3 shows the crossing with the sea defence and the shaft were the pipeline is brought towards the surface.

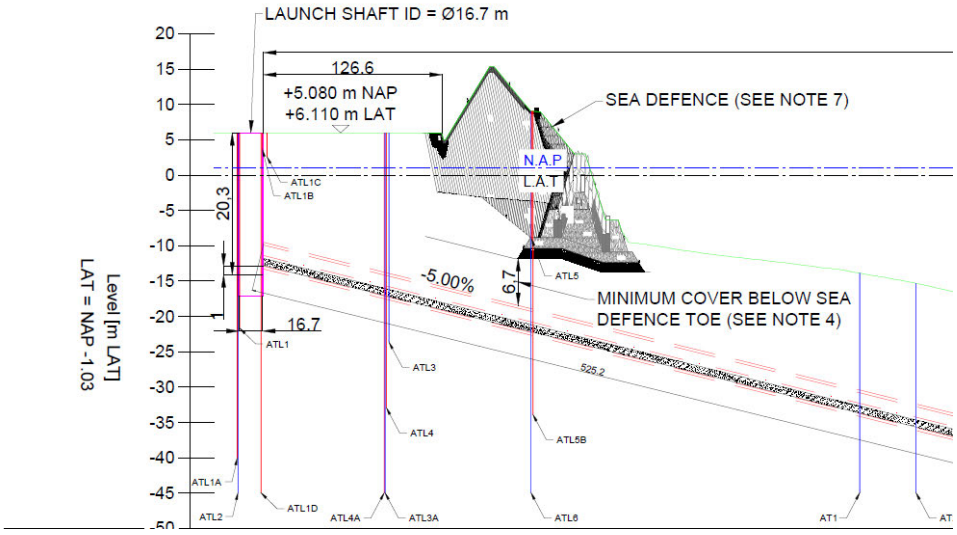


Figure 4.3 Crossing Aramis pipeline with sea defence, part of drawing NL-ARM-040-PTC1-113853.

The concrete tunnel protects the surroundings and the sea defence from the out flow of CO_2. In case of a rupture of the pipeline, effect of the outflow of CO_2 can be expected nest to the tunnel. Therefore, the possible rupture location just outside of the tunnel is considered in this report.

4.2 Definition upstream and downstream

In the remaining of this report, the terms upstream and downstream of the rupture location will be used. Upstream of the rupture is the part of the pipeline between the rupture and the compressor. Downstream of the rupture is the pipeline crossing the sea defence and the offshore pipeline. The upstream and downstream parts are illustrated in Figure 4.4.

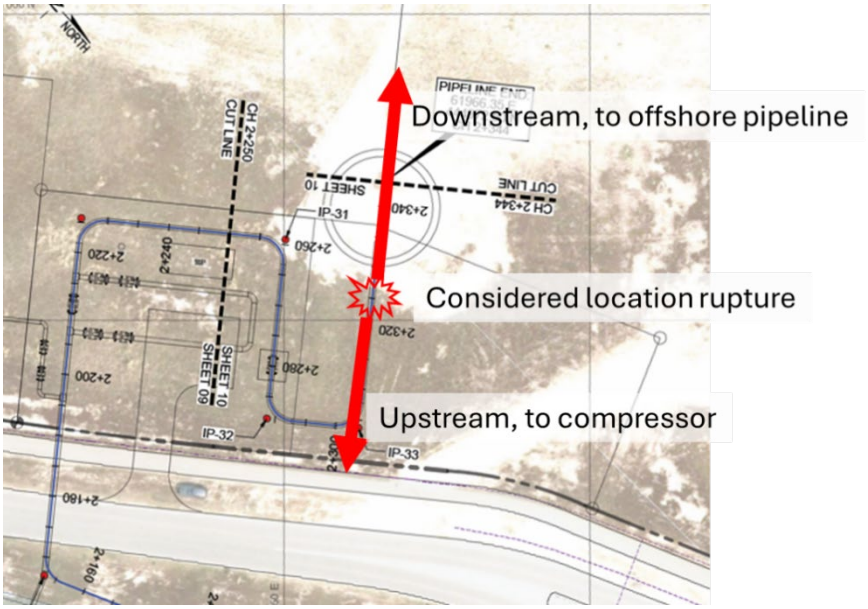


Figure 4.4 Illustration terms upstream and downstream.

4.3 Subsoil conditions

The CPT and boring near the entrance shaft are:

- CPT: ATL4, ATL4A (NB ATL4A is a boring with depth of 2.6 m, presumably a pre-drilled hole for detection of pipeline and cables at CPT location.
- Boring: ATL3 / ATL3A.

The ground level during the soil investigation was at NAP + 3.9 m. During construction it will be raised to about NAP + 5 m near the tunnel shaft.

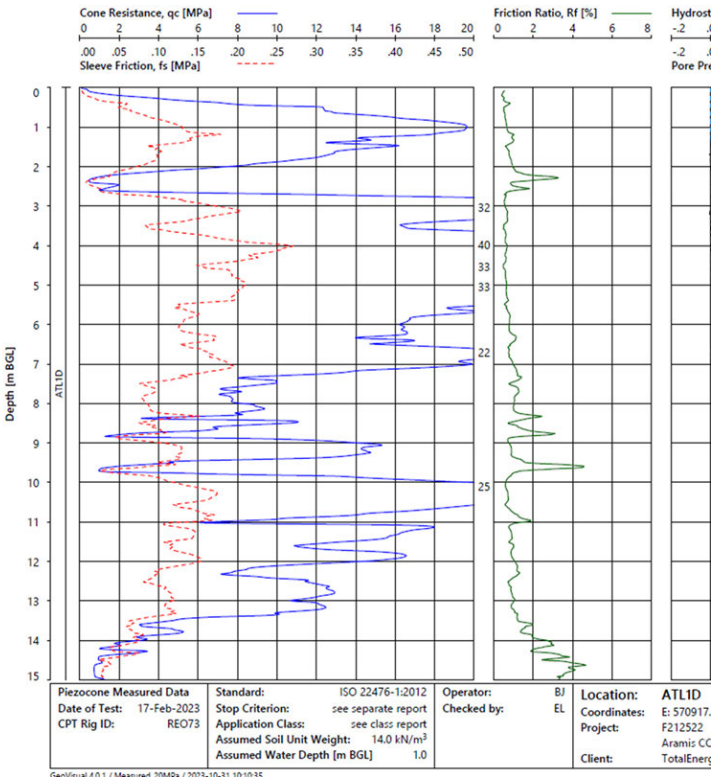


Figure 4.5 CPT ATL1, near shaft at the rupture location.

The subsoil consists mainly of sand. According to the performed bore hole ATL3A the top sand layer reaches a depth of 7 m below GL (NAP – 3.12 m). At that depth a sandy lay layer is encountered. During the performed soil investigation, no ground water level was reported. From local experience a ground water level of NAP + 1 m is expected.

According to the Fugro report the water contents in the Made Ground range between 5% and 34%. In the calculations in chapter 6 a conservative value of 4% was used.

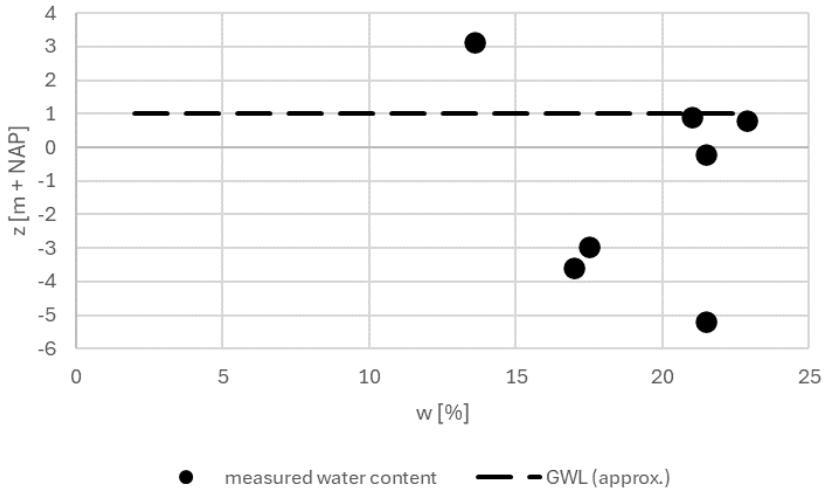


Figure 4.6 Reported water content as function of depth.

4.4 Dimensions pipeline and CO2

The main dimensions of the pipeline are:

- Inner diameter: 755.6 mm.
- Inner cross section: 0.4484 m2.
- Wall thickness: 39.8 mm.

Outer diameter: 835.2 mm.

Top of pipeline: NAP + 3.15 m:

- Soil cover: 2 m (1.85 m - 2.25 m depending on the exact location).
- The soil cover of 1.5 m is considered as an additional option.

The properties of the CO_2 in the pipeline during normal operations at the sea defence location are:

- Pressure:180 bar.
- Temperature: 36 °C.
- Density: 843 kg/$m^{3.}$.
- Mass flow (ρ*u*A): 697.6 kg/s.

5 Outflow of CO_2

5.1 Used calculation method

The rupture of the pipeline and the characteristics of the outflow of CO_2 were calculated by Petrofac with the program OLGA. Details of the performed calculation are given in a memo by Petrofac. This is included as Annex A.

The main results of the calculations made by Petrofac are given in the following paragraphs.

5.2 Mass outflow

The mass flow after pipeline rupture is the mass CO_2 released after pipeline failure. Figure 5.1 shows the calculated mass flow during 72 hours after the rupture. The moment of rupture in the figure is at t = 0 s. Please note that the sign of the flow is positive in the downstream direction, a negative mass flow from the downstream pipe is thus a flow towards the rupture location.

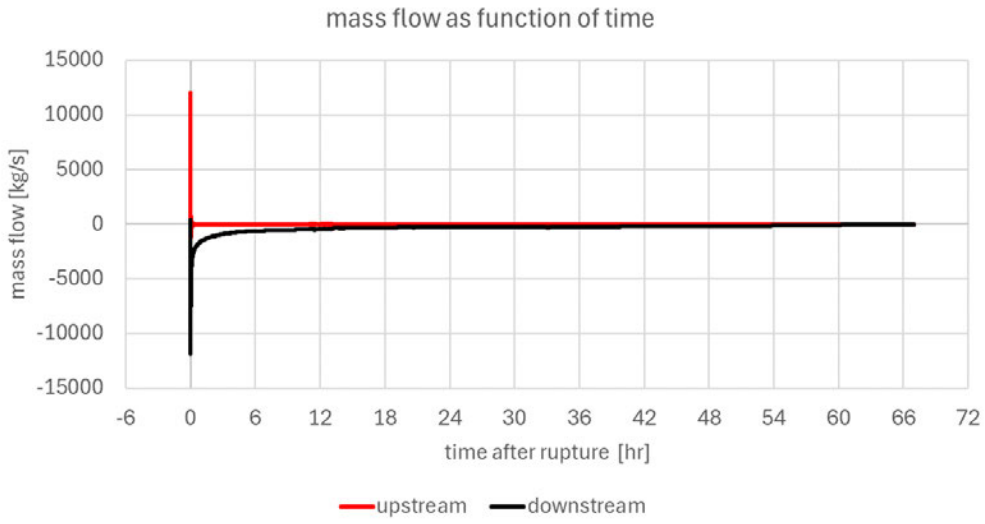


Figure 5.1 Mass flow at rupture.

This figure shows that the mass flow has a high value at moment of rupture. After a few seconds the flow decreases considerably. This is illustrated in Figure 5.2, where the development during the first 2 minutes after rupture is shown.

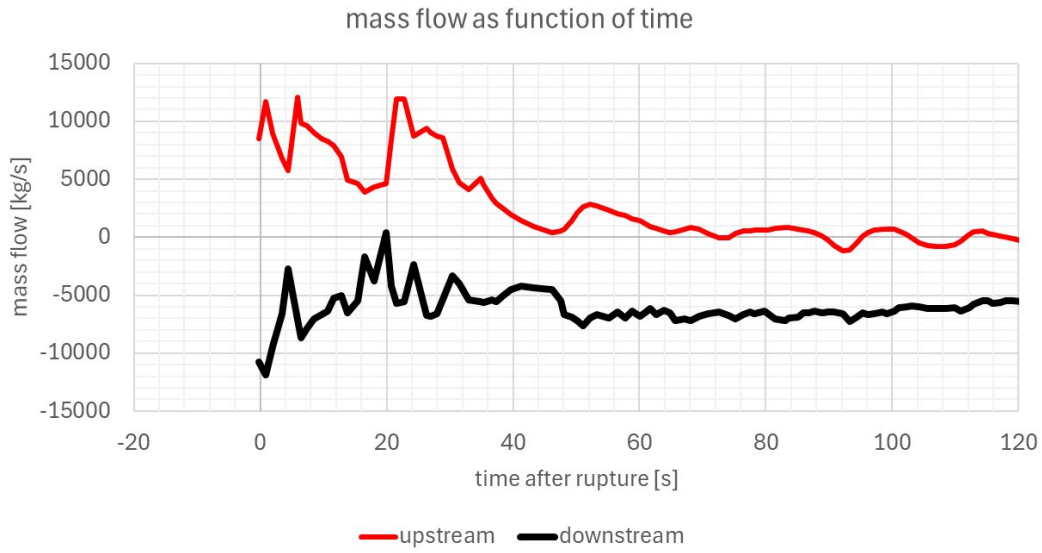


Figure 5.2 Mass flow at rupture (detail).

After 40 s the mass outflow from the upstream pipe is nearly zero. The rapid decrease in mass flow is caused by the limited length of the pipeline upstream. Downstream the outflow drops from about 12 000 kg/s at moment of rupture to about 7000 kg/s after 40 s. The intermittent negative flow after 70 s is a compression effect.

5.3 Pressure

The development of the pressure in the pipeline at the location of rupture is shown in Figure 5.3.

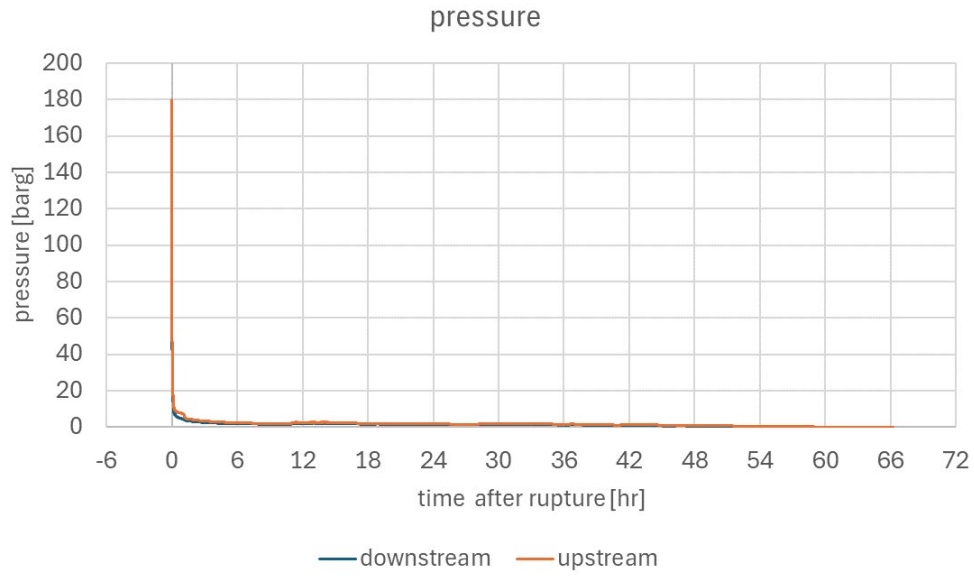


Figure 5.3 Pressure at rupture.

The pressure drops directly after the rupture. This is shown in more detail in Figure 5.4, where the development of the pressure for the first two minutes after the rupture is shown.

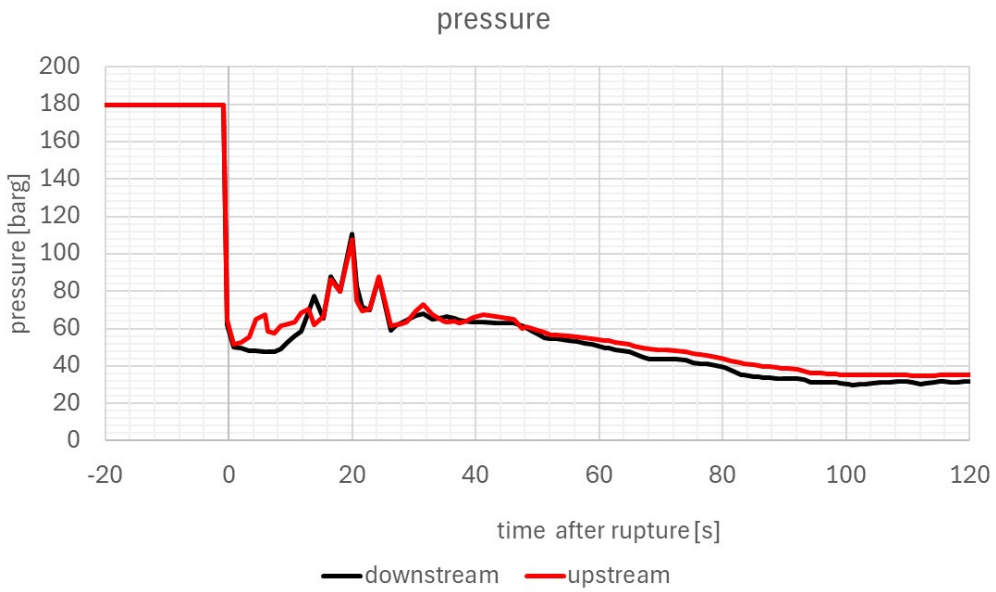


Figure 5.4 Pressure at rupture (detail).

The pressure drops from about 180 bar to about 60 bar, to rise again to 100 bar in the first 20 seconds, followed by a slow further decrease. Pressure rise to 100 bar is due to liquid boil-off. When the CO_2 hits the dew-point line, the liquid starts to boil-off, releasing gas and hence temporary pressure rise in the pipeline at the rupture.

5.4 Liquid hold-up

An important parameter is the phase of the released CO_2 at the rupture. In the pipeline the CO_2 is in a so-called dense phase (comparable to a liquid). At moment of rupture and outflow of the CO_2, the initial dense phase, a gaseous phase, a solid phase, or a combination of these phases might be present next to the pipeline.

The amount of liquid is given by the parameter ‘liquid hold-up. This parameter is defined as: the fraction of the volume which is occupied by liquid at same instant. The value ranges between 1 (only liquid) and 0 (no liquid).

The calculated development of the liquid hold-up as function of time and location is shown in Figure 5.5 and Figure 5.6.

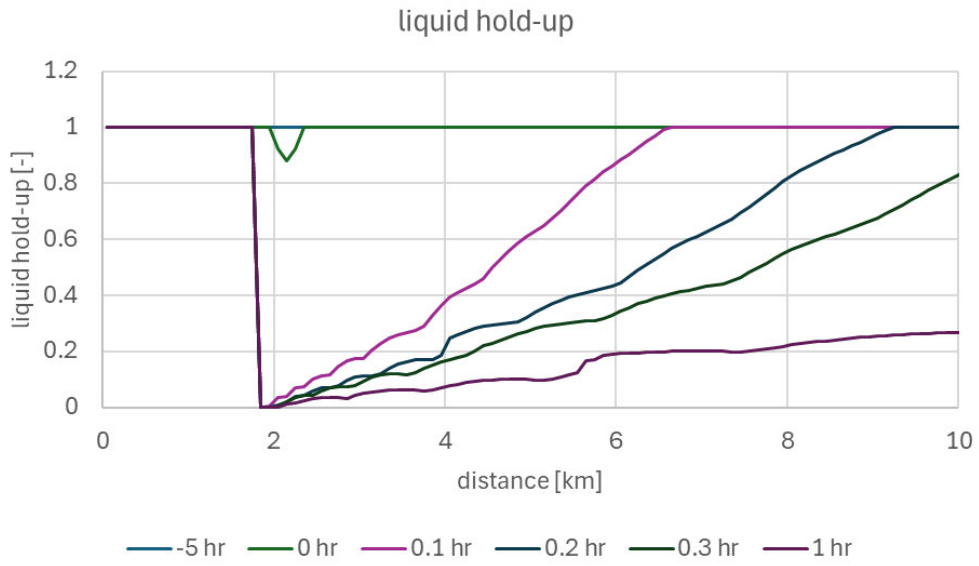


Figure 5.5 Liquid hold-up as function of time after rupture and distance to rupture, for different values of the time after rupture, rupture is at 2 km; the first line (t = -5 hr) is the situation before the rupture.

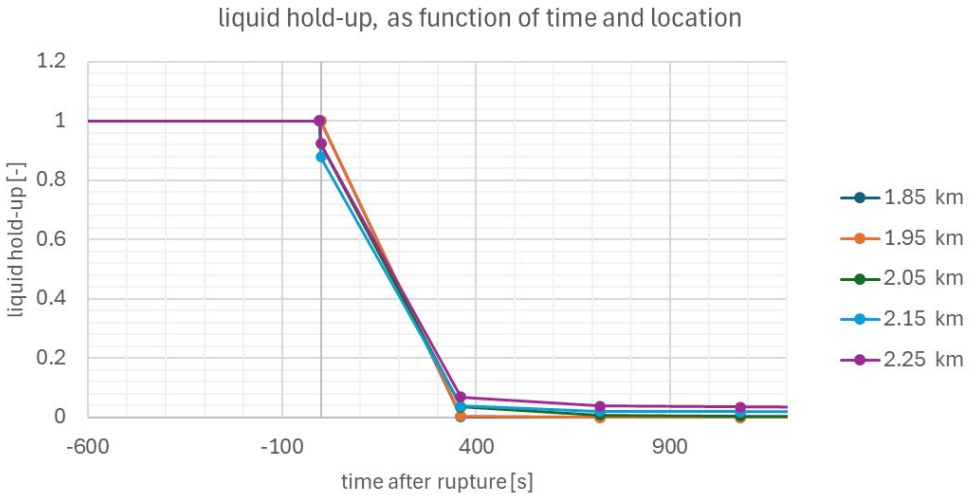


Figure 5.6` Liquid hold-up as function of time after rupture, for different distances to the rupture, rupture is at 2 km. The legend shows the location of for which the liquid hold-up is given. Rupture is at km 2.0.

The graph shows that initially the CO2 is in a liquid phase. During the outflow the amount of liquid CO_2 reduces. Gaseous CO2 is formed. The liquid hold-up decreases to a value of .05 at the rupture location.

The situation outside the pipeline directly after rupture cannot be deduced from these calculations. Firstly, the calculation does only consider flow in the pipeline. Secondly, the time steps in the calculation results in the figure above are to large. Due to the large time steps the decrease rate in the first minute of the liquid hold-up cannot be determined.

At moment of rupture most of the CO_2 is released as a liquid, but within a short time (less than 5 minutes) the situation changes and most of the CO_2 at the rupture is released as a gas.

6 Crater size

6.1 General

The phase of the outflow of the CO_2 at the rupture location is uncertain. The CO_2 next to the pipeline can be gas or liquid. Most likely a combination of gas and liquid can be expected, whereby during time the percentage of liquid decreases and the percentage of gas increases. Since no calculation model for this combined outflow is available in the first step (described in this report), it was decided to consider two situations: (i) fully gas outflow and (ii) fully liquid outflow. The crater dimensions for these two situations are calculated.

6.2 Modelling CO_2 outflow as a gas

6.2.1 Background used calculation model

The present model for the crater dimensions at gas outflow are based on model tests by Delft Hydraulics, performed in 1972 (WL 1972). During these tests a constant outflow of compressed air was used, and the crater dimension measured. Based on these experiments an expression for the growth of the crater dimension with time was derived. The expression is valid for a constant outflow of gas.

$$G_B = R(w)\left\{\frac{g}{D_K^{\,2}}\left(\frac{I}{\rho_{const} g}\right)^3 t^2\right\}^{1/8}$$

A shortcoming of this expression is the assumption of a constant outflow. In reality however the outflow will decrease in time. Therefore, the tests and the derived equation were re-evaluated (Deltares 2023). This resulted in a time dependent formulation taking into account the expected outflow in time.

$$G_{B_{i+1}} = \left[\left(G_{B_i}\right)^4 + \left(R(w)\right)^4 \left\{\frac{g}{D_k^{\,2}}\left(\frac{I_i}{\rho_{const} g}\right)^3\right\}^{1/2} (t_{i+1} - t_i)\right]^{1/4}$$

With:

- G_B Radius of the crater (m).
- $R(w)$ Coefficient related to the moisture content (-).
- I Impulsflux (N).
- Q Discharge (kg/s).
- t Time (s).
- D_k Depth of the crater (m).
- ρ_{const} Constant 0.833 in kg/m^3.
- g Gravity acceleration 9.81 (m/s^2).

This approach will be included in the green version of NEN-3651 (2025).
In this time dependent approach time records of the mass outflow and the pressure at the rupture location are used.

6.2.2 Calculation procedure

The calculations for the crater dimensions are made with the program gas-uitstroom-effecten, v1.1.0 (Deltares 2023). This program requires as input the development in time of the mass outflow and the pressure from the upstream pipeline and from the downstream pipeline. The

data from the Petrofac calculations as described in the previous chapter are used (see Appendix A for the Petrofac report).

6.2.3 Calculation results

The pipeline is situated above the ground water table and 2 m below the surface (2 m overburden according to the design of the shaft location).
At this level the soil will be moist. For the calculations a conservative value for the moisture content (w) of 4% is used. The corresponding value of R(w) is 0.04.

The calculated crater size due to gas outflow is shown in Figure 6.1. In these calculations the full length of the Petrofac records (about 3 days) is used.

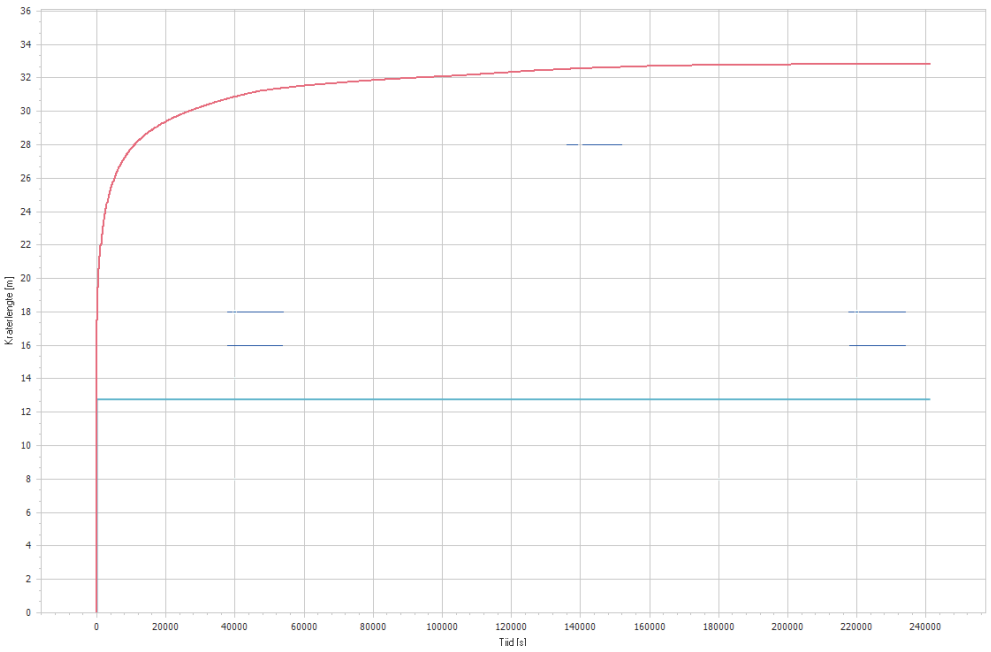


Figure 6.1 Growth of erosion crater, the blue line is the erosion crater due to outflow of the upstream pipe and the red line the erosion crater due to outflow of the downstream pipe.

For a better view of the growth of the crater shortly after the rupture the results are also presents using a logarithmic time axis. (see Figure 6.2).

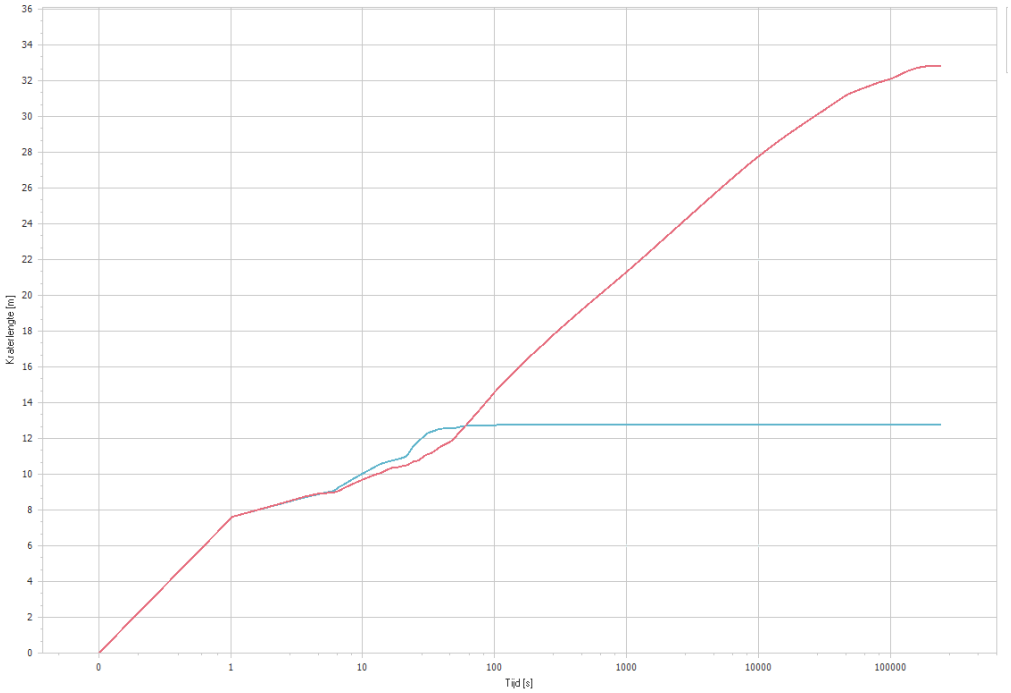


Figure 6.2 Growth of erosion crater, logarithmic time axis.

This shows that during the first 60 s the crater due to the upstream outflow and due to the downstream outflow shows a comparable growth. After about 60 s the crater due to the upstream outflow became nearly stagnant. From the graphs with the mass outflow (section 5.2) it is evident that after this time the mass outflow has become almost zero. This is due to the limited length of the upstream pipeline.

The crater due to the downstream outflow steadily grows in time. The growth rate decreases. The outflow of the downstream pipeline continues for a long time, as it is a long (194 km) pipeline without a valve.

The calculated crater dimensions are:

- For cover depth 2.0 m:
  - Due to upstream outflow: 12.8 m.
  - Due to downstream outflow: 32.9 m.

The parameter G_B gives the length of the crater in case of one-sided outflow (see Figure 6.3 for the definition). In this case, with a different outflow from the upstream and from the downstream part of the pipeline, the length of the crater will be different for the two sides of the rupture. As half width of the crater the value of GB is to be used according to NEN-3651.

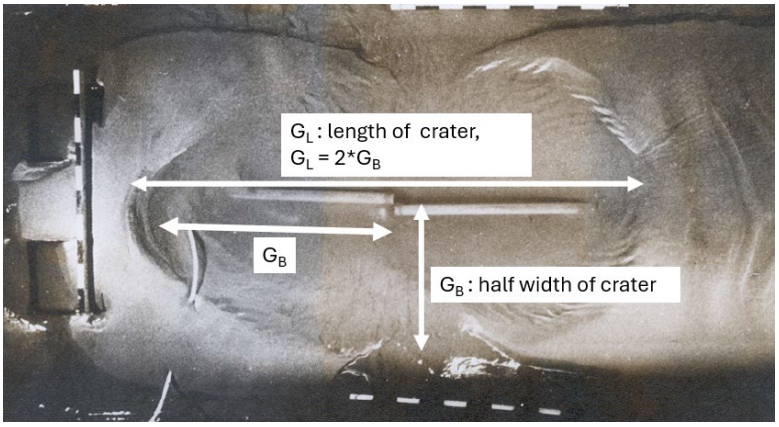


Figure 6.3 Definition crater dimensions, figure taken from NEN-3651.

According to the current design drawings, the soil cover amounts to 2 m at the shaft location where the pipeline enters the tunnel. Possibly the soil cover might be reduced in the final design drawings. Therefore, a calculation with a soil cover of 1,5 m is performed as well. The calculated crater dimensions are:

- For cover depth 1.5 m:
  - Due to upstream outflow: 13.3 m.
  - Due to downstream outflow: 34.2 m.

6.3 Modelling CO2 outflow as a liquid

6.3.1 Background present approach in NEN-3651

The expression for calculating the erosion crater due to a rupture of a liquid pipeline is based on model tests (WL 1969). In those tests a constant discharge was applied. When for the crater an equilibrium state was achieved the crater dimensions were determined. The equilibrium state was reached after approximately 5 minutes. The method was re-evaluated in 1991 (WL 191) and in 2010 (Deltares 2010). No time dependent development of the erosion crater was modelled, and no calculation methods for taking into account a changing discharge in time were developed.

The present approach is based on the measured size of the crater in case of a constant outflow. For the situation with the outflow of CO_2, where the outflow decreases, the present approach yields conservative results.

6.3.2 Calculation procedure

For the outflow of a liquid the size of the erosion crater is calculated using the expression given in NEN-3651.

$$R_B = 7.8 \cdot d_g \cdot \left(\frac{P}{\rho \cdot g^{1.5} \cdot \mu \cdot d_g^{3.5}} \right)^{0.243}$$

With:

- R_B Radius erosion crater [m].
- d_g Diameter hole in pipeline [m].
- Di Internal diameter pipeline [m].
- g Acceleration gravity (g = 9.81 m/s^2).
- μ Contraction coefficient flow at hole [-].
- P Power, P = ρ g Q h [W].
- h Pressure head [m].
- Q Discharge [m^3/s].

As mentioned in the previous paragraph, the situation at rupture of the Aramis pipeline is not constant. Shortly after the rupture a high outflow (approx. 12 000 kg/s) of both upstream and downstream is present. Within about 10 seconds the outflow drops to about 6000 kg/s for the downstream part. At the upstream part, the discharge drops to almost zero after 50 seconds. Since the experiments of the outflow of liquid show that the crater develops in several minutes after the pipeline rupture the calculation with the high initial discharge is conservative.

For the calculation of the crater dimensions due to outflow from the downstream pipe two situations are considered:

- Directly after rupture: Q= 12 000 kg/s, pressure 180 bar.
- After 1 minute: Q = 6000 kg/s, pressure is 60 bar.

The first situation is considered a conservative approach, as it assumes that the situation at rupture is constant for a longer period. The second situation is therefore considered more realistic for the assessment of the crater dimensions.

The critical velocity at the outflow is:

$$v = \sqrt{2\,g\,h}$$

For the diameter of the hole (d_g) a value is selected such that the product of critical velocity times contraction coefficient times area of hole equals the actual discharge Q.

The pressure head h is calculated using the density of the liquid CO_2. This gives a higher-pressure head, and thus outflow velocity, as using the density of water for calculating the pressure head.

Figure 6.4 shows the definition for the crater dimensions R_B and R_L. According to the green version of NEN-3651 (2025) is $R_L = R_B$.

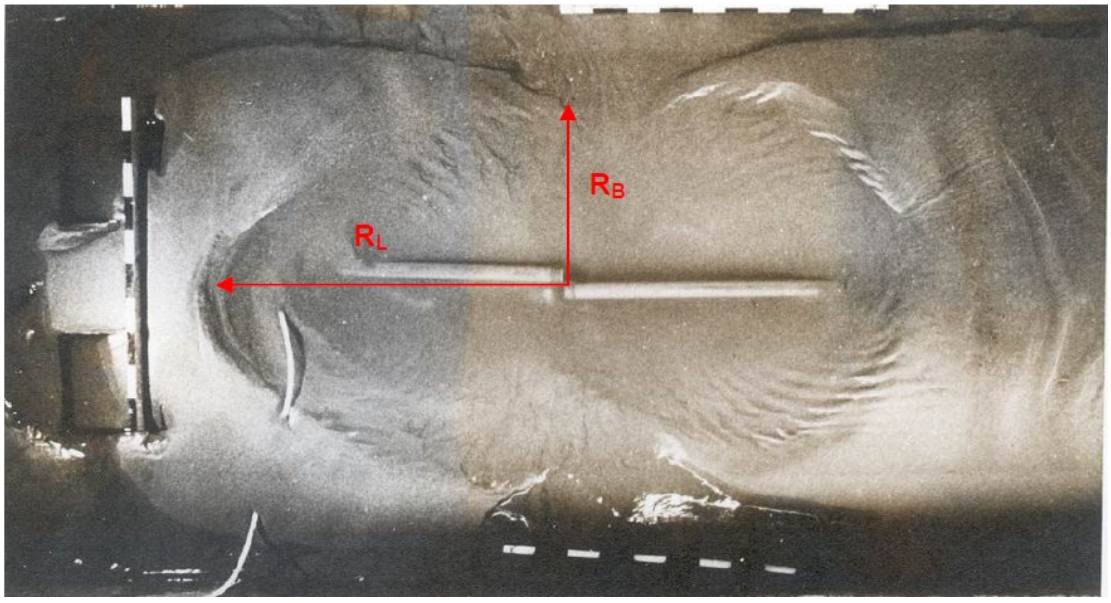


Figure 6.4 Definition R_B and R_L, figure from (WL 2010).

6.3.3 Calculation results

The outflow discharge after 1 minute: Q = 6000 kg/ with a pressure of 60 bar is used for the calculation of the crater size. In addition, a conservative calculation is made using the parameters at moment of rupture, so with a discharge of Q= 12 000 kg/s and a pressure of 180 bar.

The following values are used:

- Density liquid: 843 kg/m3.
- Contraction at outflow: mu = 0.5 (value for h > 50 m).

Results of the calculation are shown in Table 6.1.

Table 6.1 Assessment erosion crater for liquid flow.

Discharge [kg/s]	Pressure at rupture [bar]	Pressure head [m CO2]	Outflow velocity v [m/s]	d_g [m]	R_B [m]	R_L [m]
12000	180	2177	207	0.42	72	72
6000	64	726	119	0.39	50	50

Outflow from the upstream pipe segment occurs for only 50 s. For this situation no realistic calculation of the crater dimensions can be made. Given the short duration of the outflow it

may be expected that the crater dimensions will be much smaller than the crater caused by the outflow from the downstream pipe.

7 Zone with excess pore pressure

The approach for the calculation of the zone with excess pore pressure (denoted as liquefaction zone) is described in NEN 3651. The calculation model is based on observed excess pore pressures after detonating of explosives. The model assumptions are:

- The rupture occurs as a long rupture in the pipeline direction.
- Length of rupture is taken as infinite.
- The content of the pipeline suddenly expands.
- The liquefaction zone is the zone with 5% or more loss of soil strength.

Recently Deltares has refined the approach to take into account the situation of a limited rupture length. Another refinement is to quantify the excess pore pressure as function of distance to the pipeline. This is described in Deltares report 11208847-002-GEO-0001, dated October 2023. The derived expression for the loss of soil strength (c_v in %) as function of length of rupture (L_{scheur} in m), explosive load (W in kg TNT/m) and distance to the pipeline (y in m) is:

$$c_v = \frac{2 * 925 * W}{(0{,}75 * y)^3 * \sqrt{\left(\frac{2}{L_{scheur}}\right)^2 + \left(\frac{1}{0{,}75 * y}\right)^2}}$$

These refinements are incorporated in the green version of NEN-3651 (2025). The formula should be applied on granulair soils such as sand with a relative density which allows densification during the passage of the shock wave.

Due to the configuration of the pipeline near the entrance shaft it is unlikely that a rupture over a large length will occur. The bends in the pipeline will act as crack arrestors. The same holds for the weld seams between pipeline segments. For the CO_2 pipeline it is expected that a rupture will be limited to one or two pipeline segments.

From the pipeline drawing (see Figure 7.1) it is observed that a limited part of the pipeline is parallel to the sea defence. In Figure 7.1 this parallel part is encircled. It is assumed that the rupture occurs in between the bends. The length of the parallel part is taken as the length, resulting in a rupture length of 35 m.

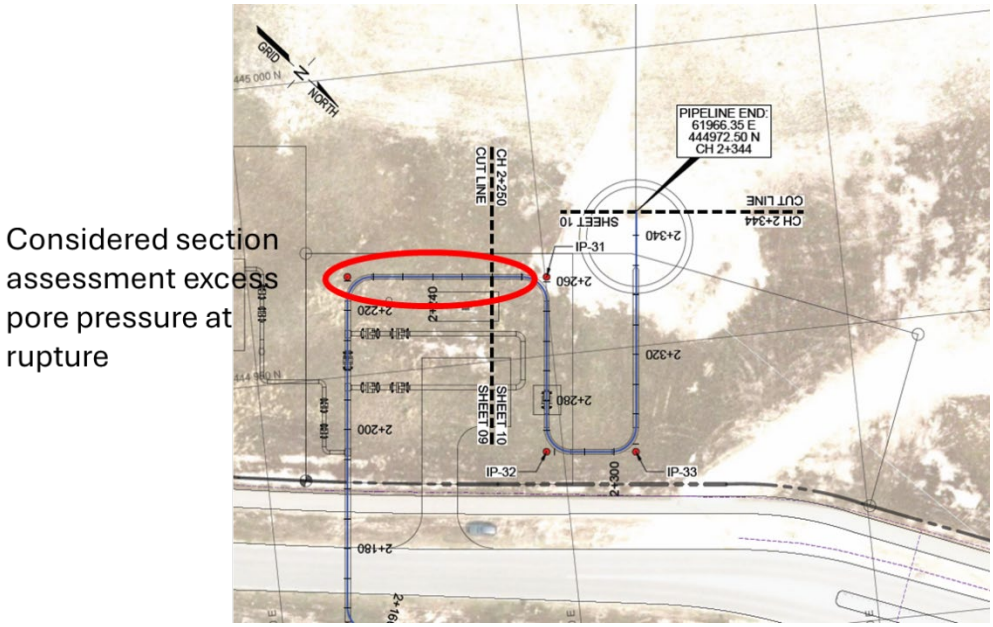


Figure 7.1 Selected section of pipeline for liquefaction assessment.

For the determination of the equivalent explosive load the original expression as described in NPR 3659 is used. One meter of the pipeline contains (cross section times unit weight) 0.4484*843 = 378 kg of CO_2, which equals 192 m^3 of gaseous CO_2 at atmospheric pressure. This value is transferred to an equivalent explosive load according to NPR 3659, L = 19.2 kg TNT/m.

With these parameters the excess pore pressure, which is equivalent to the loss of soil strength (c_v) as function of distance to the pipeline is calculated. Figure 7.2 shows the result. According to NEN-3651 the zone with 5% or more loss of strength is considered to be the so-called liquefaction zone (the zone with non-marginal excess pore pressure). In Figure 7.2 I therefore also a line at c_v = 5% is drawn.

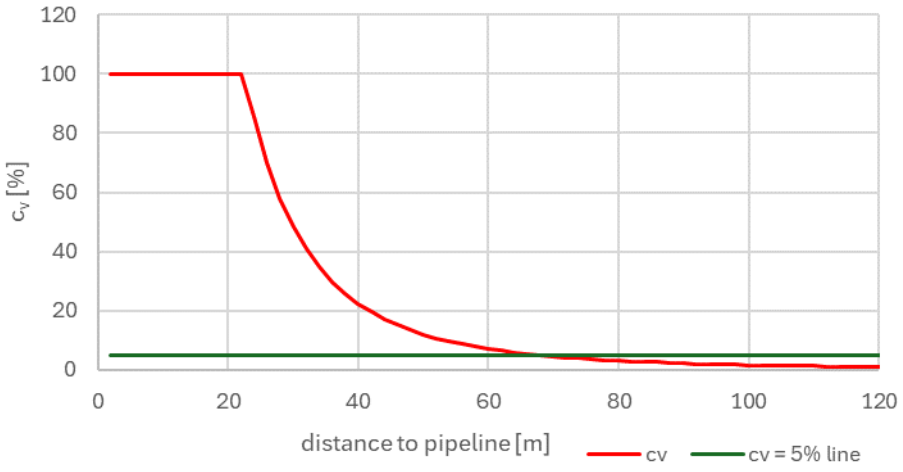


Figure 7.2 Excess pore pressure as function of distance to pipeline, length of rupture 35 m.

It should be noted that c_v = 5 % represents only a marginal loss of soil strength.

For comparison the distance at which a certain value of cv is reached is given:

- cv = 5%: r = 68 m.
- cv = 10%: r = 53 m.
- cv = 25%: r = 38 m.
- cv = 100%: r = 23 m.

It should be noted the characteristics of the soil (such as density of the sand) is not taken into account. In dense packed sand no excess pore pressure can develop due to the dilation.

The results of the CPT near the shaft of the tunnel shown in chapter 4 indicates the presence of dense packed sand. Therefore, an additional analysis might yield the occurrence of dilatation after passage of the shock wave. Dilatation prevents densification and the development of excess pore pressure.

8 Discussion results and conclusions

8.1 Discussion

The phase of the outflow of the CO_2 during rupture of the pipeline is uncertain. The CO_2 next to the pipeline can be gas or liquid. Most likely a combination of gas and liquid can be expected, whereby during time the percentage of liquid decreases and the percentage of gas increases. Since no calculation model for this combined outflow is available in the first step (described in this report), it was decided to consider two situations: (i) fully gas outflow and (ii) fully liquid outflow. The crater dimensions for these two situations are calculated.

The gas outflow is calculated using the refined calculation procedure in the green version of NEN-3651 (2025). This refined calculation procedure allows for taking into account the change in outflow with time. For liquid outflow such a time dependent calculation procedure is not available. This will result is an overestimation of the crater size. To overcome a strong overestimation, not the initial situation, but a situation after rupture was considered.

Another reason that the calculation procedure for liquid will be conservative for this situation is that the outflow will be mainly gaseous a short period after rupture, and only for a limited part liquid.

The zone with excess pore pressure is the zone with loss of soil strength. In the calculation the characteristics of the soil (such as density of the sand) is not taken into account. The results of the CPT near the shaft of the tunnel indicate that the actual loss of soil strength will be probably less than calculated.

8.2 Summary of findings

For the two considered situations and the possible soil cover of 1.5 or 2.0 m, the following maximum crater dimensions (radius) are obtained:

- Modelling as gas:
  - Outflow from downstream pipe: G_B = 34 m.
  - Outflow from upstream pipe: G_B = 13 m.
- Modelling as liquid:
  - R_B = 50 m (from condition at t = 10 s).

For the modelling of the outflow as a liquid, the first condition (initial condition) is considered to overestimate the crater dimension, since as this condition is present for only a few seconds. The initial situation yields R_B = 72 m. The crater dimension R_B = 50 m is considered to be the best-estimate value for the crater dimensions in case the outflowing CO_2 is considered as a liquid.

The results indicate that the crater dimensions after rupture will be in the range of 34 m to 50 m. The 50 m radius is caused by the downstream pipe segment. In case of so-called guillotine failure, the crater radius is shown in the subsequent figure. It should be noted that the crater can also be formed adjacent to the downstream pipe segment in the shaft.

The zone with excess pore pressure or loss of strength due to the explosion is 68 m. This is the zone with 5% or more loss of soil strength. The zone with 25% or more loss of soil strength is only 38 m. Given the high measured cone resistance values in the CPT's at the shaft location, which indicate a high density of the sand, it can be expected that no excess pore pressures (no loss of strength) occur after passage of the shockwave.

The calculation results are schematically shown in Figure 8.1.

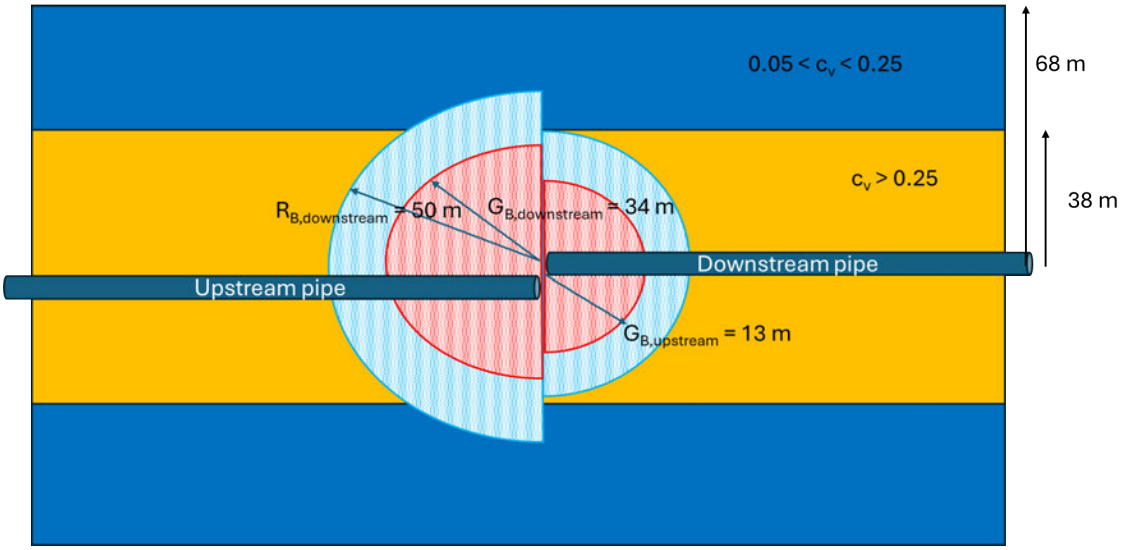


Figure 8.1 Overview zones, red shader area is crater dimension modelling CO_2 outflow as gas, blue shaded area is crater dimension modelling CO_2 outflow as liquid, solid blue and orange areas are zones with loss of soil strength according to NEN-3651 (soil density not considered).

The calculation results as described in this report are based on conservative assumptions and schematisations regarding the outflow. The calculations can be optimised in step 2 which should focus on the transition from liquid to gas during the outflow at the rupture location.

References

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Deltares (2023). Uitstroom effecten van gassen uit leidingen op omringende grond.

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NEN (1996). Ondergrondse pijpleidingen, grondslagen voor sterkteberekening. NPR 3659, Oktober 1996.

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A Memo Petrofac on OLGA calculation

Memo

OLGA 2021.1.0 has been used to carry out the transient thermo-hydraulic simulation for the rupture event. Pure CO_2 instead of the Aramis composition represents the transported CO_2 fluid within the pipeline. It is believed that the depressurization rate will not be affected on whether the fluid is represented as pure or impure CO_2.

As a starting point, a steady state simulation was run with a pressure inlet boundary of 180 barg (no compressors are modelled) and a mass flow rate node at D-HUBN (196 km away) representing an extraction of 22 MTPA. The model included an (initially) open beach valve, which separates the onshore section from the offshore section. Figure 1 below shows the model set-up for the rupture event.

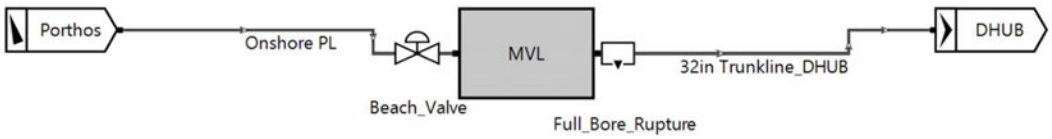


Figure 1 - Pipeline Rupture Model

The rupture event is simulated to occur after 5 hours of steady state operation at a location of 1 m upstream of the micro-tunnel shaft. The rupture point is approximately 2.2 km away from the Aramis mixing point of Porthos and CO2Next gas. To model the rupture event, the following steps were followed:

- Immediate drop off flow from 22 MTPA to 0 MTPA at D-HUBN to stop CO_2 exit and pipeline depressurisation at the outlet
- Full bore rupture of the 32" pipeline from 0 mm to full bore in 45 seconds
- Closure of the beach valve within 32 seconds. Note that the Beach valve closure is faster than the rate of pipeline rupture to prevent numerical instability
- The pressure reduction due to the rupture event is solely determined by the emptying of the pipeline

Results

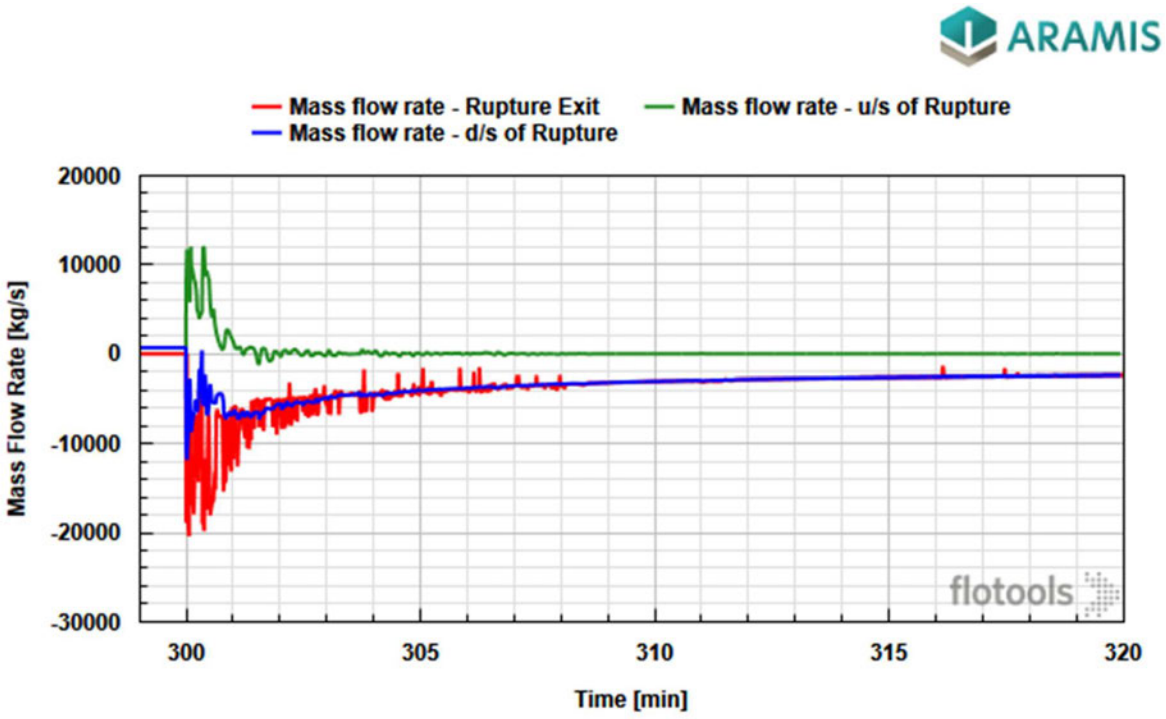


Figure 2 – Mass Flow Rate after Rupture

Figure 2 shows the mass flow rates immediately upstream (u/s rupture) and downstream (d/s rupture) of the rupture. The upstream mass rate is shown as positive as it is in the natural flow direction of the transported CO_2 towards D-HUBN. The downstream mass flow rate is shown as a negative flow as the CO_2 flows away from D-HUBN towards onshore/rupture location. The combination of the upstream and downstream flow rate is summed up as the negative flow rate at the rupture exit.

Because of the low volume upstream of rupture, this section is emptied a lot faster (reaching 0 kg/s at around 18,500 seconds) compared to the upstream section.

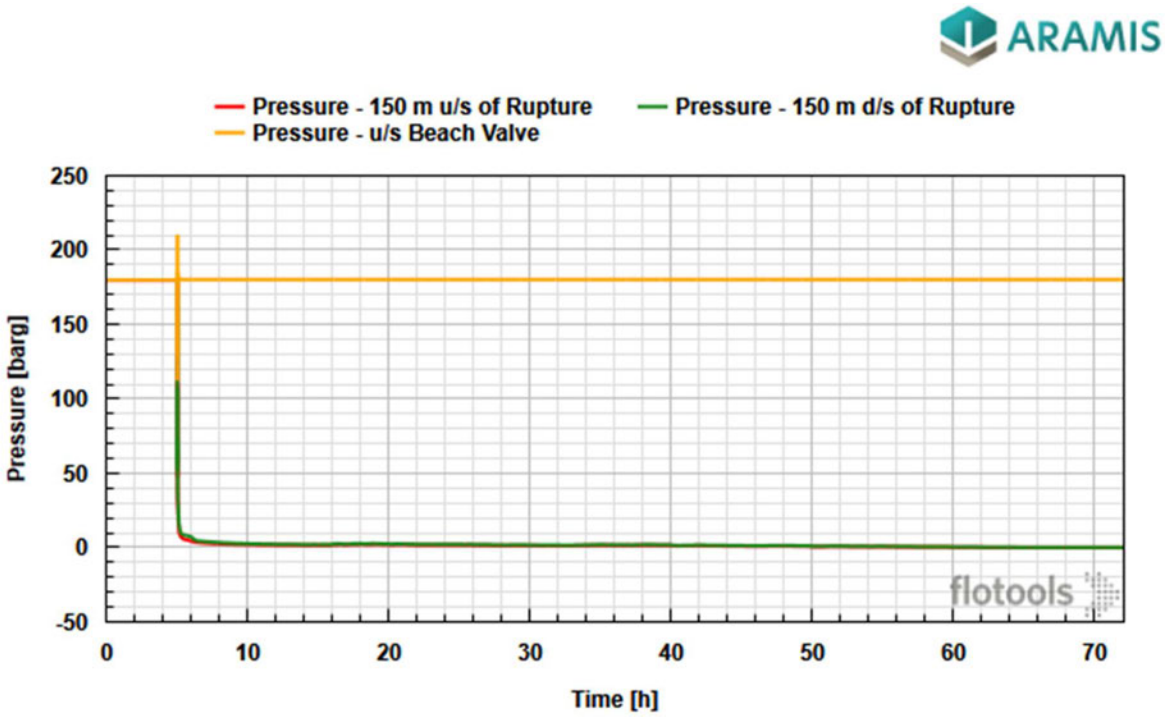


Figure 3 – Pressure after Rupture

Figure 3 shows the pressure trends as the system depressurises due to the rupture event. On rupture, the pressure upstream of the beach valve, falls suddenly but immediately stabilises and is maintained at high pressures when the beach valve is closed. For monitored locations downstream of the beach valve, the pressure falls to immediately to around 20 -30 barg within 30 minutes. Thereafter the fall in pressure towards 1 bar is very gradual, taking about 5 hours.

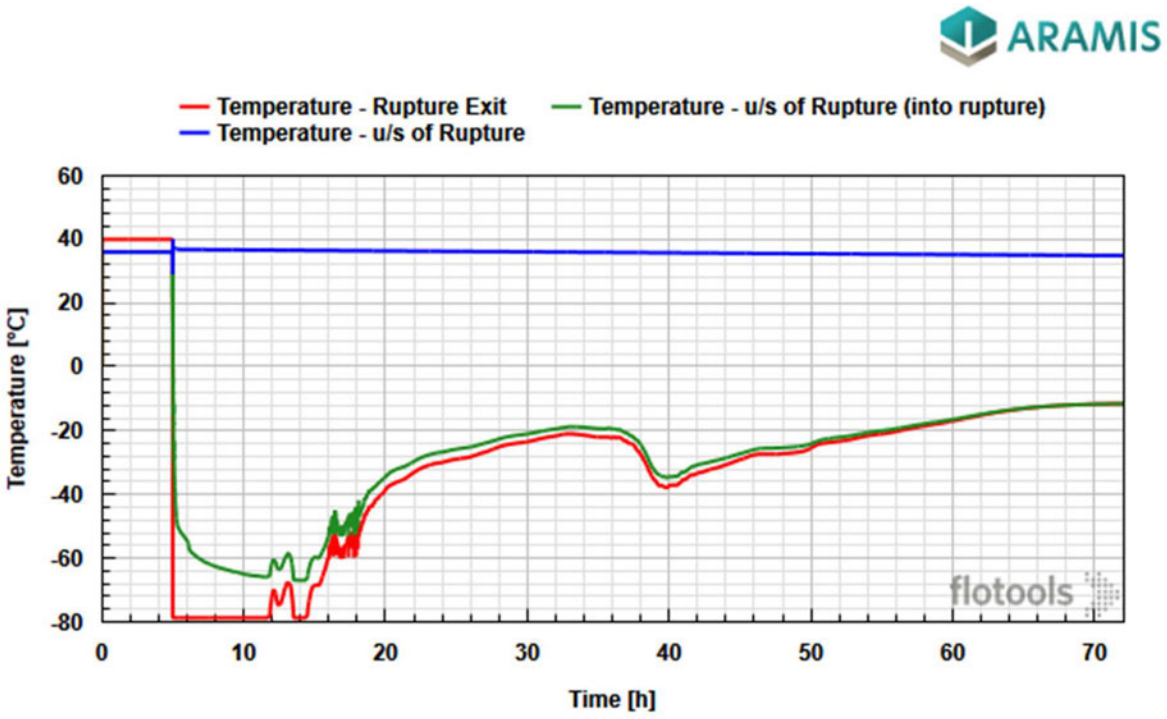


Figure 4 -Temperature after Rupture

Figure 4 shows the temperature trends as the system depressurises due to the rupture event. On rupture, the temperature upstream of the beach valve, experiences a slight sudden fall due to the JT-effect, but immediately stabilises and is maintained at high temperatures when the beach valve is closed.

For monitored locations downstream of the beach valve, into the rupture and exit of the rupture, the temperature falls to less than -60 ℃. Which reflects the expected temperature at the given pressure, a characteristic of the CO_2 phase behaviour.

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Royal HaskoningDHV is een onafhankelijk internationaal advies- en ingenieursbureau. We combineren 140 jaar engineering- en ontwerpexpertise met consultancy, software en technology diensten. We leveren hiermee toegevoegde waarde voor klanten en hebben een positieve impact op mensen en onze leefomgeving. Dat is onze drijfveer: Enhancing Society Together. Daar hoort bij dat we onszelf en anderen voortdurend uitdagen om bij te dragen aan duurzame oplossingen voor lokale en wereldwijde vraagstukken in de gebouwde omgeving en de industrie.

In onze snel veranderende wereld wordt de agenda bepaald door onder meer klimaatverandering, de digitale transformatie, een veranderende consumentenvraag en hybride werken. Met onze geïntegreerde duurzame oplossingen willen we bijdragen aan het bredere technologische en maatschappelijke plaatje.

Gesteund door de kennis en ervaring van meer dan 6.000 collega's werken we vanuit kantoren in meer dan 20 landen. We ondersteunen klanten om de transitie te maken naar een slimme en duurzame organisatie. We koppelen onze engineering- en ontwerpexpertise aan onze software- en technologische diensten om toegevoegde waarde te leveren voor onze klanten en de lifecycle van hun assets.

We zijn oprecht, handelen integer en transparant in al onze activiteiten, ook onze bedrijfsvoering Ons team is divers en inclusief. De veiligheid en het welzijn van mensen, in ons team en daarbuiten, staat onder alle omstandigheden voorop.

In projecten en initiatieven werken we actief samen met overheden en het bedrijfsleven, partners en stakeholders. We zien een belangrijke rol voor onszelf in innovatieve duurzame ontwikkeling en willen bijdragen aan een betere leefomgeving, nu en in de toekomst.

Ons hoofkantoor is gevestigd in Nederland en we hebben kantoren in Europa, Azie, Afrika, Australie en Amerika.

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BH8744_Aramis beperkingengebiedactiviteit Noordzee publiceerbaar

Uw verzoek

Ingediend bij	**Ministerie van Economische Zaken en Klimaat**
Soort	Aanvraag vergunning
Activiteit(en)	Bouwwerken, werken en objecten in de Noordzee
Doel	Aanvullen
Status	Aangevuld
Verzoeknummer(s)	20241220 01810 001 (ingediend op 11-02-2025)
	20241220 01810 000 (ingediend op 20-12-2024)

Project

Naam van dit project

BH8744_Aramis beperkingengebiedactiviteit Noordzee

Projectomschrijving

Deze aanvraag betreft de aanleg van de tunnel ten behoeve van de Aramis CO2-transportinfrastructuur in het beperkingengebiedactiviteit Noordzee

Locatie

Teken een gebied op de kaart



Algemeen

U kunt een bijlage toevoegen over het contact met anderen (participatie).

Document	Vertrouwelijk
BH8744-112-103-IB-ME-F1-Informatie voor omgevingsvergunningaanvraag beperkingengebiedactiviteit Noordzee.pdf	Nee

Document	Vertrouwelijk
BH8744-112-103-IB-ME-F2-Informatie voor omgevingsvergunningaanvraag beperkingengebiedactiviteit Noordzee.pdf	Nee

Voeg als bijlage toe: gegevens over de grens van de locatie.

Document	Vertrouwelijk
BH8744-112-103-IB-ME-F1-Informatie voor omgevingsvergunningaanvraag beperkingengebiedactiviteit Noordzee.pdf	Nee
BH8744-112-103-IB-ME-F2-Informatie voor omgevingsvergunningaanvraag beperkingengebiedactiviteit Noordzee.pdf	Nee

Participatie: anderen betrekken bij uw plannen

Heeft u contact gehad met anderen voor wie de omgeving anders wordt door uw plannen?
Ja

Hoe heeft u anderen betrokken bij uw plannen?
geen openbare informatie

Welke reacties heeft u gekregen?
geen openbare informatie

Verzoek

Geef uw verzoek een naam
BH8744_Aramis beperkingengebiedactiviteit Noordzee

Toelichting op uw verzoek
geen openbare informatie

Uw referentienummer
geen openbare informatie

Hierbij verklaar ik alle vragen naar waarheid te hebben ingevuld.
Ja

Zijn er gegevens die u later opstuurt? Denk aan bouwtekeningen, foto's, plattegronden, etc. Geef hier aan welke gegevens dat zijn en waarom u die later opstuurt.
geen openbare informatie

Zijn er gegevens die u nu niet opstuurt? Geef aan welke gegevens dat zijn en waarom u die niet opstuurt. Bijvoorbeeld omdat u die eerder heeft opgestuurd.
geen openbare informatie

Uw gegevens

E-mailadres en telefoonnummer gemachtigde

E-mailadres
geen openbare informatie

Telefoonnummer
geen openbare informatie

Gegevens gemachtigde vestiging of bedrijf

KVK-nummer
56515154
Vooraf ingevuld antwoord.

Handelsnaam
HaskoningDHV Nederland B.V.
Vooraf ingevuld antwoord.

RSIN
852164087
Vooraf ingevuld antwoord.

Adresgegevens gemachtigd bedrijf

Straatnaam
Laan 1914
Vooraf ingevuld antwoord.

Huisnummer
35
Vooraf ingevuld antwoord.

Huisletter
-

Huisnummertoevoeging
-

Postcode
3818EX
Vooraf ingevuld antwoord.

Plaats
Amersfoort
Vooraf ingevuld antwoord.

Is het postadres hetzelfde als het hoofdadres?
Nee
Vooraf ingevuld antwoord.

Postadres gemachtigd bedrijf

Wat voor adres wilt u opgeven als postadres?
afwijkend adres
Vooraf ingevuld antwoord.

Wat voor adres wilt u opgeven als afwijkend adres?
postbusnummer
Vooraf ingevuld antwoord.

Nummer
1132
Vooraf ingevuld antwoord.

Postcode
3800BC
Vooraf ingevuld antwoord.

Plaats
Amersfoort
Vooraf ingevuld antwoord.

E-mailadres en telefoonnummer initiatiefnemer

E-mailadres
geen openbare informatie

Telefoonnummer
geen openbare informatie

Gegevens vestiging of bedrijf initiatiefnemer

KVK-nummer
02029700
Vooraf ingevuld antwoord.

Handelsnaam
N.V. Nederlandse Gasunie
Vooraf ingevuld antwoord.

RSIN
001605768
Vooraf ingevuld antwoord.

Adresgegevens bedrijf initiatiefnemer

Straatnaam
Concourslaan
Vooraf ingevuld antwoord.

Huisnummer
17

Vooraf ingevuld antwoord.

Huisletter
-

Huisnummertoevoeging
-

Postcode
9727KC
Vooraf ingevuld antwoord.

Plaats
Groningen
Vooraf ingevuld antwoord.

Is het postadres hetzelfde als het hoofdadres?
Nee
Vooraf ingevuld antwoord.

Postadres bedrijf initiatiefnemer

Wat voor adres wilt u opgeven als postadres?
afwijkend adres
Vooraf ingevuld antwoord.

Wat voor adres wilt u opgeven als afwijkend adres?
postbusnummer
Vooraf ingevuld antwoord.

Nummer
19
Vooraf ingevuld antwoord.

Postcode
9700MA
Vooraf ingevuld antwoord.

Plaats
Groningen
Vooraf ingevuld antwoord.

Contactpersoon

Wilt u een contactpersoon voor deze aanvraag of melding opgeven?
Ja

Functie contactpersoon
Permitting and Stakeholdermanagement Lead Aramis

Voorletters
geen openbare informatie

Voorvoegsel
geen openbare informatie

Achternaam
geen openbare informatie

E-mailadres
geen openbare informatie

Telefoonnummer
geen openbare informatie

Wat voor adres wilt u opgeven als postadres?
binnenlands adres

Straatnaam
geen openbare informatie

Huisnummer
geen openbare informatie

Huisletter
geen openbare informatie

Huisnummertoevoeging
geen openbare informatie

Postcode
geen openbare informatie

Plaats
geen openbare informatie

Vragen en antwoorden

Bouwwerken, werken en objecten in de Noordzee

Nieuwe of bestaande activiteit

Geef aan waarover deze vergunningaanvraag gaat.
Starten van een nieuwe activiteit

Milieueffectrapportage

Wat geldt er voor de activiteiten van deze aanvraag?
Geen ‘project-mer-plicht’ en geen ‘project-mer-beoordelingsplicht’

Diverse vragen

Geef aan waar uw aanvraag betrekking op heeft. Kies alle opties die van toepassing zijn.
Beperkingengebiedactiviteit die wordt verricht in de Noordzee buiten de zone tussen de duinvoet en de laagwaterlijn

U heeft (ook) gekozen voor een beperkingengebiedactiviteit die wordt verricht in de Noordzee buiten de zone tussen de duinvoet en de laagwaterlijn. Geef daarvan aan waar uw aanvraag betrekking op heeft. Kies alle opties die van toepassing zijn.
Bouwen of in stand houden van een ander bouwwerk, het aanleggen, plaatsen, in stand houden of veranderen van een ander werk dat geen bouwwerk is en het plaatsen of in stand houden van een ander object .

Geef de contactgegevens van de partijen die direct betrokken zijn bij het verrichten van de werkzaamheden.
Zie toelichting

Op welke datum verwacht u te gaan starten met de activiteit?
01-04-2026

Op welk tijdstip verwacht u te gaan starten met de activiteit?
07.00 uur

Hoe lang verwacht u dat de activiteit gaat duren?
Zie toelichting

Heeft uw aanvraag betrekking op een locatie in de Noordzee maar buiten het provinciaal en gemeentelijk ingedeelde gebied?
Nee

Bijlagen

Bouwwerken, werken en objecten in de Noordzee

Beschrijving gevolgen activiteit

Document	Vertrouwelijk
NL-ARM-040-ARM1-000002-Toelichting aanvraag beperkingengebiedactiviteit Noordzee-Aramis tunnel-versie F1.pdf	Nee
NL-ARM-040-ARM1-000002-Toelichting aanvraag beperkingengebiedactiviteit Noordzee-Aramis tunnel-versie F2.pdf	Nee

Boorplan

Geen documenten.

Werkplan met toelichtende tekening

Geen documenten.