ABSTRACT
An insight is provided into the design of a large diameter high pressure gas pipeline shore approach for an LNG project off the coast of North Western Australia. The shore approach is located in a challenging and congested area, within one of Australia’s busiest ports. In close proximity to the pipeline route are two existing high pressure gas pipelines, several major shipping channels, a large spoil ground and environmentally sensitive coral reefs. Throughout the design process, careful consideration has been given to environmental and heritage constraints. In the case of the pipeline shore approach, prevention of coral mortality due to dredging induced turbidity has been a key priority. Ensuring the preservation of the heritage site adjacent to the landfall site has also been a major consideration.
This paper describes the numerous challenges faced in designing the pipeline shore approach. A description of the design processes used to develop the secondary stabilisation and accidental external impact designs is provided. The unique methods employed to install the trunkline system across an operating shipping channel are also described. The information provided in this paper can be applied to the design and development of numerous pipeline shore approaches in the future. Individuals interested in pipeline secondary stabilisation, impact protection, shore crossings and pipelay techniques will also find this paper of value. This paper contributes to the technical knowledge base of the offshore industry by displaying the benefits of effectively combining relevant theoretical approaches with state of the art physical model testing methods and best practice installation techniques. This work demonstrates how innovative design practices can be successfully applied to produce a safe, cost effective and robust pipeline shore approach design in a challenging area.

1 INTRODUCTION
The Northwest Shelf of Western Australia is home to Australia’s LNG industry. The LNG hub lies within the sheltered waters of Mermaid Sound. Mermaid Sound is also home to export terminals for industrial salts, ammonia and iron ore. In addition to this, it is situated within a marine park consisting of protected flora and fauna, and its adjacent coastline is home to several culturally significant indigenous heritage sites. With the rapid industrialisation of the region, it has become increasingly difficult to route offshore pipelines through the nearshore zone to the LNG processing terminals. This design case study examines a gas field located off the North Western coast of Western Australia. The natural gas is being conveyed from an offshore platform through a 180 km long, 36-inch diameter pipeline to an onshore processing facility. A 6-inch diameter pipeline has also been installed to supply the platform with MEG for use in the natural gas processing.
In the context of this project, the shore approach is defined as the pipeline route section requiring secondary stabilisation measures over and above pipeline self weight. This paper describes the various aspects of the pipeline shore approach design including the pipeline route selection, shore crossing design, shipping channel crossing design, secondary stabilisation design and accidental external impact protection design.
It also highlights the technical challenges faced during the design process including a tightly restricted landfall site, the crossing of an operating shipping channel, challenging seabed geotechnical conditions and exposure to extreme tropical cyclonic conditions. These challenges have been overcome in a timely and cost effective manner using robust design processes, leading to an optimised pipeline shore approach design.

2 DESIGN DATA
2.1. Pipeline Data:
The trunkline system was designed with the MEG pipeline to be installed piggybacked on the natural gas trunkline, except where the trunkline crosses the shipping channel. In this location the trunkline and MEG pipeline were installed side by side. A typical trunkline system section is shown in Figure 1.
The mechanical properties of the trunkline system are listed in Table 1.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Description</th>
<th>Property</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trunkline Outside Diameter</td>
<td></td>
<td>914 mm</td>
</tr>
<tr>
<td>Trunkline Wall Thickness</td>
<td>(KP0.75 – KP2.1)</td>
<td>26.7 mm</td>
</tr>
<tr>
<td>Trunkline External Corrosion Coating</td>
<td></td>
<td>Asphalt Enamel</td>
</tr>
<tr>
<td>Trunkline External Corrosion Coating Thickness</td>
<td></td>
<td>5 mm</td>
</tr>
<tr>
<td>Saturated Concrete Weight Coating Thickness</td>
<td></td>
<td>From 0 - 90 mm</td>
</tr>
<tr>
<td>MEG Pipeline Outside Diameter</td>
<td></td>
<td>168.3 mm</td>
</tr>
<tr>
<td>MEG Pipeline External Corrosion Coating</td>
<td></td>
<td>3LPP</td>
</tr>
<tr>
<td>MEG Pipeline External Corrosion Coating Thickness</td>
<td></td>
<td>From 3-10 mm</td>
</tr>
</tbody>
</table>

2.2. Metocean Conditions:
The hostile metocean conditions along the North Western Australian coast posed a significant challenge to the stabilisation design for the trunkline system. Table 2 summarises the significant wave heights and peak wave periods for the 100 year and 1000 year return periods from KP 4 to KP 24.

### Table 2. Nearshore Pipeline Route Metocean Data

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Description</th>
<th>Property</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth (m)</td>
<td></td>
<td>11.6</td>
</tr>
<tr>
<td>Hs (m)</td>
<td></td>
<td>6.90</td>
</tr>
<tr>
<td>Tp (m)</td>
<td></td>
<td>29.20</td>
</tr>
</tbody>
</table>

### Table 3. Shore Crossing Metocean Data

2.3. Geotechnical Data:
Geotechnical conditions are a key factor in determining the most suitable trunkline shore approach design. The seabed along the shore approach section of the trunkline route primarily comprises of a combination of fine to medium sized carbonate sediments and calcarenite/limestone rock. Table 4 summarises the geotechnical conditions for the trunkline route between KP 0 and KP 50.

### Table 4. Geotechnical Conditions

<table>
<thead>
<tr>
<th>Zone</th>
<th>KP Range</th>
<th>Geotechnical Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shore Crossing</td>
<td>0 - 0.15</td>
<td>Granite base with some pockets of overlying carbonate sand, possibly cobbles and boulders throughout</td>
</tr>
<tr>
<td></td>
<td>0.15 - 0.8</td>
<td>Up to 2m of sediment with layers of gravel and calcarenite</td>
</tr>
<tr>
<td></td>
<td>0.8 – 5.8</td>
<td>Primarily deep carbonate granular sand</td>
</tr>
<tr>
<td></td>
<td>5.8 - 10.7</td>
<td>Overlying variably cemented carbonate rock</td>
</tr>
<tr>
<td></td>
<td>10.7 - 18.1</td>
<td>Calcite and calcarenite rock outcrop</td>
</tr>
<tr>
<td></td>
<td>18.1 - 19.4</td>
<td>With overlying outcrops of medium to coarse sand and gravel</td>
</tr>
<tr>
<td>Nearshore</td>
<td>19.4 - 21.3</td>
<td>Predominantly coarse/gravelly calcareous sand overlying low to medium strength calcarenite below 4 m</td>
</tr>
<tr>
<td></td>
<td>21.2 - 24.5</td>
<td>Limestone/calcarenite rock outcrop with pockets of calcareous sand</td>
</tr>
<tr>
<td></td>
<td>24.5 - 50</td>
<td>Predominantly coarse/gravelly sand with gravel layers and limestone/calcarenite below 3.5 m</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Predominantly coarse/gravelly sand with layers of gravel and limestone / calcarenite below 2 – 3 m</td>
</tr>
</tbody>
</table>

2.4. Data Collection Program:
A data collection program was developed at the outset of the project to obtain the required information for the shore approach design. Pipeline design engineers collaborated closely with surveyors and geotechnical engineers in the planning and execution of this program. The inclusion of pipeline design engineers was critical to the success of this process, especially given the fast-tracked nature of the project. They ensured that the data obtained over the course of the program was of optimal use to the shore approach design. The data collection program saw a number of surveys performed within Mermaid Sound and the adjacent coastline. These included:
- Seismic reflection surveys
- Seismic refraction surveys
- Detailed bathymetric surveys
- Borehole testing
- Cone Penetrometer Testing
- Vibrocore Testing

The information gathered during this data collection program set the foundations for the shore approach design.

3 PIPELINE ROUTE SELECTION

3.1. Shore Approach:
Mermaid Sound is a congested area, in close proximity to two existing high pressure pipelines, several major shipping channels, and a large existing material spoil ground. It is
situated within a marine park consisting of protected flora and fauna including whales, dolphins, sea turtles, dugong and sensitive coral reefs. In addition to this, several indigenous heritage sites of cultural significance to the local community are located along the coastline. These factors, as well as the difficult seabed geotechnical conditions, have made the definition of a suitable route for the high pressure gas pipeline extremely challenging.

Key drivers in the pipeline route selection process are listed below:

- Seabed geotechnical conditions
- Seabed roughness
- Landfall site characteristics
- Existing high pressure gas pipeline routes
- Spoil ground and borrow ground locations
- Existing anchorage and pilot boarding areas

A design pipeline route was selected following a detailed evaluation of all possible routing options. This design pipeline route and its surrounding features are presented in Figure 2.

3.2. Pipeline Shore Crossing:
The pipeline shore crossing site stands out as one of the most difficult features of the shore approach design. The shore crossing design’s functional requirement is to adequately guide, stabilise and protect the trunkline system where it transits from offshore to onshore.

A number of sites were considered during the conceptual engineering phase of the project. However, constraints including heritage, environmental, geotechnical and other existing and proposed infrastructure led to the shore crossing being located behind an igneous outcropping headland at the proposed LNG processing plant site.

The congested pipeline landfall site and the presence of high strength igneous rock presented a significant challenge for this design. An image of the pipeline shore crossing site prior to work commencing is shown in Figure 3. The southern boundary of the shore crossing site abuts the project’s LNG export jetty. The proposed jetty entrance road has high side slopes creating a physical barrier to the shore crossing area.

A design pipeline route was selected following a detailed evaluation of all possible routing options. This design pipeline route and its surrounding features are presented in Figure 2.

Figure 3. Shore Crossing Location

An indigenous heritage site was also identified adjacent to the northern boundary of the shore crossing site. This area was marked as a ‘no disturbance’ zone.

The site layout provides sufficient room to allow for site entry requirements, a winch pad, shore pull equipment, and a stockpile area for rock backfill material. A plan view of the site layout is presented in Figure 4.

Figure 4. Shore Crossing Site Layout

3.3. Pipeline Shipping Channel Crossing:

With the landfall site selected the next challenge was to locate a suitable location to cross the existing shipping channel. As shown in Figure 1, the crossing of an existing shipping channel was required for the trunkline route to access the specified landfall site. The location for the shipping channel crossing was determined using the data from an intensive geotechnical survey campaign. Pipeline installation constraints such as horizontal curves and required length of straights for above water tie-ins (AWTIs) also contributed to the decision making process. The eventual alignment was selected based on the area with the most favourable dredging conditions whilst also allowing for the pipeline installation constraints.

The alignment of the shipping channel crossing is straight with start and end points positioned 900 m from the channel entrance.
centreline on the southern side and 1100 m on the northern side respectively. A minimum vertical radius of curvature of 1500 m was specified in the trunkline profile of the shipping channel crossing to minimise pipeline stresses and spanning during installation.

4 PIPELINE STABILISATION AND PROTECTION DESIGN

Secondary stabilisation and mechanical protection can be addressed by seabed intervention techniques such as trenching and sand backfill or rock dumping. A robust process was developed to evaluate the cost, schedule and risk of each stabilisation and protection strategy.

4.1 Secondary Stabilisation:

Due to the tropical cyclone induced extreme hydrodynamic loading experienced by pipelines on the Northwest Shelf, secondary stabilisation systems are required to ensure the structural integrity of the trunkline system.

A number of secondary stabilisation methodologies were identified to stabilise the trunkline system including:
- Gravity anchors
- Ploughing
- Pre-trenching and backfill
- Rock dumping
- Rock trenching.

Each methodology was investigated to examine feasibility and cost effectiveness.

4.2 Accidental External Impact Protection:

The pipeline shore approach design must account for the risk of accidental external impacts. A thorough quantitative risk assessment (QRA) has identified anchor drag, anchor drop, ship sinkage and ship grounding from LNG tankers and tug boats, as credible impact scenarios to the trunkline system. The risks are particularly relevant between KP 0.55 and KP 4.55, where there is the possibility of an LNG tanker drifting from its shipping channel and crossing the trunkline route. This scenario is depicted in Figure 5.

A review of possible external impact scenarios identified anchor drag from a 177,000 m³ LNG carrier as the governing design case for pipeline impact protection between KP 0.55 and KP 4.55. These vessels use 19.5 Tonne AC-14 high holding power anchors. Figure 6 depicts an LNG carrier and the anchors that they are typically equipped with.

Similarly to secondary stabilisation, accidental impact protection can be addressed using a number of methods including trenching, sand backfill and rock dumping.

A summary of protection requirements along the trunkline route is presented in Table 5.

Table 5. Protection Requirement Summary

<table>
<thead>
<tr>
<th>KP Range</th>
<th>Protection Requirements</th>
</tr>
</thead>
<tbody>
<tr>
<td>KP 0.0 – KP 0.10</td>
<td>Protection from onshore motor vehicle impact</td>
</tr>
<tr>
<td>KP 0.07 – KP 0.55</td>
<td>Protection from sinking/grounding tugs and tug anchor drag</td>
</tr>
<tr>
<td>KP 0.55 – KP 4.55</td>
<td>Protection from sinking/grounding LNG tankers and LNG tanker anchor drag</td>
</tr>
<tr>
<td>KP 4.55 – KP 50.3</td>
<td>No protection required</td>
</tr>
</tbody>
</table>

4.3 Method Selection:

A screening process for selecting the most suitable trunkline system stabilisation methodology was developed by combining a multi-criteria analysis (MCA) to assess risks with a comparison of expected cost and schedule requirements. The MCA assessed the following types of risk:
- Technical
- Cost
- Schedule
- Health, Safety and Environment

Through this process it became evident that pre-trenching using a TSHD in soft sediments was the optimal methodology based on cost, schedule and risk. Where the required trench depth could not be achieved cost effectively, it was found that stabilisation through rock dumping was favourable over the use of gravity anchors, or rock trenching using a CSD or rock trencher. Rock backfill also provided synergies with the trunkline system’s accidental external impact protection requirements.

5 TRENCH AND BACKFILL DESIGN

5.1 Trenchability Assessment:

The critical step for determining the trenchability of the pipeline route was to prepare detailed Geo-Data alignment sheets. These sheets combined all of the geotechnical, geophysical and bathymetrical data for the route alignment and consolidated them on single drawings. These sheets provided
the basis for route optimisation based on trenchability. An example Geo-Data sheet is displayed in Figure 7.

![Figure 7. Geo-Data Sheet](Image)

Using the Geo-Data sheets, the ability to achieve a trench depth of up to 3.5 m using a TSHD in water depths of up to 50 m was evaluated. Where a greater trench depth than the minimum required design depth was expected to be achieved cost-effectively, this was identified and implemented to provide additional mechanical protection and remain consistent with the project’s ALARP philosophy.

### 5.2. Trench Design:

The pipeline shore approach trench and sand backfill design scope involved defining the trench depth, trench bottom width, trench side slopes and sand backfill material specification. The drivers for trench depth in areas requiring only stabilisation were seabed scour and the depth of liquefaction in the sand backfill. Where accidental external impact protection was required, additional trench depth was needed to prevent damage from ship’s anchor penetration.

### 5.3. Seabed Liquefaction and Scour:

One of the functional requirements for the sand backfilled trench design is to keep the trunkline system stable for the 1000 year RP (Return Period) hydrodynamic loading condition and the 500 year RP seismic loading condition. An adequate trench depth and backfill material is required to prevent soil liquefaction. Soil liquefaction is caused by soil pore pressure build-up due to seismic or repetitive hydrodynamic loading. This phenomenon can lead to pipeline flotation when the pipeline’s specific gravity is lower than that of the liquefied sand. Should the trunkline float to the surface, it would become exposed to hydrodynamic loading which, in turn, may lead to trunkline instability. The pipeline can also be pushed up before the soil is fully liquefied due to excess soil pore pressure differences at the top and bottom of the trunkline.

A liquefaction assessment was performed to determine the depth of sand cover required to prevent pipeline flotation during a 1000 year RP hydrodynamic event or a 500 year RP seismic event. These minimum trench depths are based on liquefaction assessments made using a backfill material with a minimum D$_{10}$ of 0.1 mm. This particle size limit has been specified as a requirement for the trench backfill material.

The liquefaction assessment was performed for the trunkline alone, given that the SG for the MEG pipeline will exceed that of liquefied sand and will not contribute negatively to the flotation of the pipeline. To prevent seabed scour, the backfill material needed to be similar to or coarser than the in-situ seabed surface material. A minimum backfill material D$_{50}$ of 0.3 mm was specified to meet this requirement. Based on the requirements for sand backfill, suitable sand was required to be sourced. A previous project in the region had identified a potential borrow ground to source the coarse sand that was suitable for the trench backfill. Further investigation of the material confirmed the suitability of the material for use as sand backfill for the pipeline trenches.

### 5.4. Design Optimisation:

The minimum allowable trench depths along the sections of the trunkline route requiring sand backfill have been defined by summing the fully coated trunkline diameter, the depth of cover to prevent pipeline flotation and the scour allowance. The original impact protection design between KP 0.8 and KP 3.03 was a 3.5 m deep trench backfilled with quarry rock. Upon proving favourable geotechnical data along this section, it was deemed possible to cost-effectively increase the trench depth. This provided an opportunity to lower the trunkline system deeper into the seabed and cover it with sand backfill, thereby providing the required accidental external impact protection. This solution significantly reduced the volume of quarried rock to be produced and dumped.

Whilst it was also possible to achieve a suitable trench depth up to KP 0.55, a sand backfill solution was not feasible inshore of KP 0.8 due to the inability of the TSHD (Trailer Suction Hopper Dredger) selected for this project to operate in shallower water depths. As a result, a rock berm impact protection solution was adopted inshore of KP 0.8. Between KP 0.8 and KP 3.03, where impact protection requirements are present, the required trench depth was governed by the depth to which ships’ anchors can penetrate into the seabed whilst being dragged. In these cases, anchor penetration depth was used to calculate the required minimum trench depth, as opposed to using depth of cover to prevent pipeline flotation.

The trench side slopes were required to remain stable over the period between trench dredging and backfilling. The tendency for slope slumping to occur is dependent on the seabed material characteristics and hydrodynamic loading. Previous testing had demonstrated that a trench, in sand, can remain stable for the 10 year RP hydrodynamic condition with a side slope of 1v:2h. Based on this result, a trench side slope of 1v:2h was defined for the trench design. An example of a typical trench and sand backfill design is presented in Figure 8.
6 ROCK BERM DESIGN

Rock berms were required to stabilise and protect the trunkline system during extreme hydrodynamic and accidental external impact events. They were used where the seabed comprises of rock or it has been assessed that adequate trench depth could not be achieved to stabilise and protect the trunkline route. In addition to this, a rock berm solution was also adopted for the shipping channel crossing. An assessment of propeller wash caused by passing LNG carriers operating within the shipping channel concluded that a material with a D₅₀ of less than 160 mm would be subject to scour. This eliminated sand as a backfill material solution.

The scope of the rock berm design work included defining the rock berm configuration, dimensions and rock particle size gradings.

6.1. Design Criteria:
The rock berms were designed to remain intact for the 100 year return period hydrodynamic loading conditions. This design criterion allows for some minor reshaping of the rock berms, provided the crest level does not degrade to such a level that repair is required.

It is expected that when the rock berms are subjected to 1,000 year return period hydrodynamic loading conditions, crest levels will reduce with some exposure of the crown of the pipeline. However, pipeline break out from the berms should not occur. Repair of the rock berms is likely to be required when hydrodynamic loads exceed the 1,000 year return period hydrodynamic loading conditions. When subject to up to the 10,000 year hydrodynamic loading conditions, it is expected that large scale degradation of the rock berms will occur; however, the pipeline should remain embedded within the quarried rock.

6.2. Design Configurations:
Rock berms consist of a filter layer to prevent upward migration of sand particles, and an outer armour layer to provide the necessary stability and accidental impact protection under the design loading conditions. Two rock berm configurations were used in the shore approach design; the full cover rock berm configuration and the no-cover rock berm configuration. These rock berm configurations are displayed in Figure 9 and Figure 10 respectively.

Full cover rock berms are typically used where external impact protection is required. The rock layer thickness, crest width and armour rock particle size grading are driven by the most onerous accidental impact load.

No-cover rock berms were designed to provide secondary stabilisation along sections of the pipeline route where impact protection is not required and trenching could not be achieved cost effectively due to the nature of the seabed. No-cover rock berms act to stabilise the pipeline and reduce seabed scour whilst using the minimum amount of rock around the sides of the pipeline. The design leaves the top of the pipeline exposed to dropped objects such as anchors and is only suitable where external impact protection is not required.

Where a no-cover rock berm was proposed for secondary stabilisation, the piggybacked MEG pipeline was disconnected from the top of the trunkline and lowered to the seabed using divers, prior to rock dumping. This allowed the MEG pipeline to be covered by a layer of filter rock prior to armour rock placement, protecting the MEG pipeline from potential damage due to armour rock particle impact.

The minimum filter rock layer thickness was defined based on the maximum expected penetration depth of an armour rock particle into the filter rock layer. The impact energy of the largest rock that could be produced within the specified armour rock grading was used and compared with the expected energy absorption from the filter rock to determine the maximum expected penetration depth.

A minimum armour rock layer thickness of twice the rock particle D₅₀ was applied to the no-cover rock berm armour rock layer thickness. This was done to ensure sufficient interlocking between rock particles.

Filter layer and armour layer side slopes of 1v:3h were specified for all of the rock berm designs. This slope corresponds to the natural angle of repose of the rock particles.

6.3. Shore Crossing Rock Berm:
Full cover rock berms were designed for the shore crossing and areas where impact protection is required. The shore crossing requires a larger rock berm to withstand the relatively large
hydrodynamic loads caused by tropical cyclone induced waves breaking onto shore. The shore crossing rock berm was designed as a three-layer full cover rock berm. The armour rock particle size calculations suggested that an armour rock particle \( D_{50} \) size greater than 400 mm would be required to ensure rock berm stability. Typically, rock berms with armour rock particle \( D_{50} \) sizes greater than 400 mm are designed with three rock layers. This ensures stability between the interfacing rock layers.

6.4. Rock Particle Sizing:
The minimum filter rock \( D_{50} \) particle size was selected based on stability requirements during the rock berm installation period, where the filter layer may be exposed to hydrodynamic loads prior to armour rock dumping. \( D_{50} \) is defined as the diameter exceeded by 50% (measured by weight) of the rock particles in a sample or stockpile. The minimum armour rock particle \( D_{50} \) size required for hydrodynamic stability was initially calculated using empirical equations. The empirical equations used were selected based on the location of the rock berms. Equations suited to shallower water where breaking wave conditions are prevalent were used to obtain the shore crossing rock berm armour rock particle sizes. The minimum armour rock particle \( D_{50} \) sizes for rock berms further offshore were calculated using equations based on sediment transport theory. These \( D_{50} \) sizes were then used to design suitable rock gradings based on yield curves from various local quarries. These rock gradings were then verified through physical model testing.

With the exception of the shore crossing rock berm, full cover rock berm rock particle \( D_{50} \) sizes were driven by impact protection requirements and not hydrodynamic stability.

6.5. Physical Model Testing:
An extensive flume model testing program was undertaken to verify the stability of the no-cover rock berm designs during extreme hydrodynamic events at critical locations along the trunkline route. This testing was used to confirm the minimum rock particle \( D_{50} \) size calculated using empirical methods. An image of the no-cover rock berm physical model testing in progress is shown in Figure 11.

Three dimensional physical model testing in a wave tank was undertaken for the shore crossing rock berm, to verify the design. An image of the shore crossing physical model testing in progress is shown in Figure 12.

The results of the physical model testing suggested that the calculated rock berm particle sizes would not withstand the design storm conditions. Failure of the rock berm was observed to be caused by waves breaking directly onto the structure as well as funneling of wave energy through a second (open) trench.

Following the results of the initial physical model testing, several changes were made to the rock berm design. These changes included increasing the armour rock particle size, increasing the density of the armour rock and backfilling the pipeline shore crossing to match the original coastline.

Additional physical model testing was performed following these changes. The results of these additional tests indicated that the shore crossing rock berm design would be stable for the specified design conditions, thus confirming the amended design.

Anchor drag model testing was performed to determine the expected penetration depth of a dragged 16.1 Tonne AC-14 HHP (High Holding Power) anchor in rock berms. The model testing consisted of dragging a model scale AC-14 HHP anchor across a model scale berm. Figure 13 displays the outcome after one particular anchor drag model test was performed.

Three dimensional physical model testing in a wave tank was undertaken for the shore crossing rock berm, to verify the design. An image of the shore crossing physical model testing in progress is shown in Figure 12.
clearance was used to determine the required armour layer thickness and sand backfill thickness along these areas. The armour rock particle size grading curves used for the physical model testing was also applied to full cover rock berm designs.

6.6. Design Summary:
A summary of the various rock berm designs along the pipeline shore approach is presented in Table 6.

<table>
<thead>
<tr>
<th>KP Range</th>
<th>Rock Berm Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>-0.02 – 0.02</td>
<td>Two layer full-cover rock berm in a 1.7 m - 3.5 m deep trench (1.9 m rock cover on top of pipe)</td>
</tr>
<tr>
<td>0.0235 – 0.061</td>
<td>Three layer full-cover rock berm in a 1.5 m – 3.6 m trench (3.5 m rock cover on top of pipe)</td>
</tr>
<tr>
<td>0.072 – 0.35</td>
<td>Two layer full-cover rock berm in 3.5 m deep trench (1.2 m rock cover on top of pipe)</td>
</tr>
<tr>
<td>0.55 – 0.80</td>
<td>Two layer full-cover rock berm in 4.3 m deep trench (1.2 m rock cover on top of pipe)</td>
</tr>
<tr>
<td>3.080 – 3.630</td>
<td>Two layer full-cover rock berm in 3.5 m deep trench (2 m rock cover on top of pipe)</td>
</tr>
<tr>
<td>3.630 – 4.550</td>
<td>Two layer full-cover rock berm in shipping channel trench (trench depth varies) (2 m cover on top of pipe)</td>
</tr>
<tr>
<td>4.842 – 4.996</td>
<td>Single layer rock berm in 3.5 m deep trench (0.3 m rock cover on top of pipe)</td>
</tr>
<tr>
<td>6.115 - 10.965</td>
<td>Two layer no-cover rock berm on seabed</td>
</tr>
</tbody>
</table>

7 CONSTRUCTABILITY

7.1. Pre-Trenching:
The nearshore section of the pipeline route from KP 0.35 to KP 6.24 was pre-trenched using a large BHD (Backhoe Dredge), with the exception of the shipping channel crossing trench. A TSHD (Trailer Suction Hopper Dredger) and a CSD (Cutter Suction Dredge) were employed to construct the shipping channel crossing trench. The CSD was used to crush the harder calcarenite material that could not be dredged directly using the TSHD. Basic diagrams of a BHD and a TSHD are displayed in Figure 14 and Figure 15 respectively.

A TSHD was also used to pre-trench the pipeline route in the deeper section of the shore approach between KP 6.24 and KP 50. In addition to this, a TSHD was employed to perform the sand backfill work along the pipeline shore approach route. Due to the size and draft of the TSHD, sand backfill could not be undertaken inshore of KP 0.8. The trunkline system was flooded prior to the backfill work to ensure pipeline flotation did not occur.

Due to the strength of the underlying granite base inshore of KP 0.120, drilling and blasting was required to pre-fragment the rock prior to excavation of the trench. Some of the drilling and blasting was performed onshore, however a large proportion could not be achieved without the use of a jack up barge. The blast patterns were carefully planned and prepared to ensure that blasting would not impact the heritage area adjacent to the shore crossing site.

The digital terrain modelling software, 12D, was used to verify the as constructed trenches and ensure that they met the specified design requirements. Bottom roughness assessments were also performed to ensure that the as constructed trenches could accommodate the trunkline system without affecting its structural integrity. The bottom roughness assessments were performed using Sage Profile 3D. Sage Profile 3D is a finite element software package capable of accurately simulating pipelay and pipe behaviour along a 3D seabed under a variety of loading conditions. Advanced soil springs were used to accurately model the pipeline-soil interaction according to the latest design codes.

The trench verification and bottom roughness assessments were performed efficiently in real time such that delays were minimised and the dredging contractor could respond rapidly to areas requiring additional dredging or rectification.

7.2. Pipeline Installation:
The trunkline system was installed using a standard S-lay pipeline installation procedure, with the exception of the shore pull and the shipping channel crossing.

Frequent LNG tanker movements prevented the pipelay barge from remaining in the channel for the amount of time required to perform a standard S-lay pipeline installation operation.

To work around this problem, pre-laid trunkline and MEG pipeline strings, approximately 1000 m in length each, were bottom towed separately into the pre-dugedged shipping channel crossing trench.
A procedure known as an above water tie-in (AWTI) was then performed on either side of the channel. This procedure was used to connect the separately laid sections of the trunkline and MEG pipeline in the shipping channel with the trunkline and MEG pipeline laid in piggyback configuration either side of the shipping channel. A photograph of an AWTI being performed is presented in Figure 16.

![Figure 16. Above Water Tie-In](image16)

Figure 16. Above Water Tie-In

A summary of the steps involved in the pipeline installation is provided below:

1. Sections of trunkline and MEG pipeline were laid separately, north of the shipping channel, prior to being pulled across the shipping channel.
2. The section of trunkline was pulled into the shipping channel from its initial position in the pre-dredged trench north of the shipping channel.
3. The section of MEG pipeline was pulled through the pre-dredged trench across the shipping channel from its initial position north of the shipping channel.
4. The pipe lay vessel was relocated to the shore crossing location and the shore pull was performed over a length of 450 m.
5. The trunkline system was laid in piggy back configuration, south to the northern edge of the shipping channel.
6. Four AWTIs were undertaken to tie in the pipelines on the north and south side of the shipping channel.

Effective communication and interfacing between the shipping channel operator, the local port authority and the shore approach construction management was critical in ensuring that the trenching, pipeline installation and subsequent rock dumping was performed efficiently and without incident.

7.3. Post-Lay Rock Dumping:

The rock dumping was performed using a side stone dumping vessel (SSDV). The SSDV pushes rock out of its bays at a steady rate using hydraulic rams. This is performed while the SSDV is travelling at a predefined speed along the pipeline route using a dynamic position (DP) system. The thickness and width of the rock layer can be varied by decreasing the speed of the vessel or changing the angle of orientation of the vessel as it travels along the pipeline. An image of a loaded SSDV is shown in Figure 18.

![Figure 18. Side Stone Dumping Vessel](image18)

Figure 18. Side Stone Dumping Vessel

Vertical construction tolerances were applied to all rock berm layers. The tolerances ensure that the minimum design level can be realistically achieved during construction, whilst restricting the levels of overdump and minimising rock wastage. The tolerances were specified based on the dumping method, water depth and rock particle size.

The digital terrain modelling software, 12D, was used to verify the as-constructed rock berms and ensure that they met the specified design requirements. The software was particularly useful in ensuring that the MEG pipeline had sufficient filter rock cover prior to armour rock dumping in the no-cover rock berm areas.

Ensuring that the specified rock could be produced, transported and loaded out cost effectively in the necessary volumes was a key concern for the shore approach construction. The total quantity of rock required for the project was driven by the rock berm profiles, the rock berm lengths, overdump allowances, survey tolerances and expected losses during the rock supply and rock dumping process.

Two local rock supply contractors were employed to produce and supply the rock. The rock was stockpiled at a load out facility, prior to its loading onto the rock dump vessel.
8 CONCLUSION
A number of lessons have been learned over the course of this design. These lessons are summarised below:

- Ensuring design engineers are involved from the outset of the project to the end of the construction phase provides several advantages and ensures the timely delivery of a robust design.
- Comprehensive geotechnical data collection at the beginning of a project is a fundamental requirement for a successful shore approach design. This also applies to the collection of metocean data.
- The use of physical model testing to validate and optimise rock berm designs is invaluable. This is especially the case for areas of complex bathymetry and areas exposed to breaking waves.
- A process driven design approach as opposed to a results driven design approach led to the development of a robust and cost effective design. All options were screened based on numerous factors including risk, cost and constructability, allowing the most suitable option to be selected based on a logical criterion.
- Effective use of appropriate software for design and assessment of as-constructed trenches and rock berms was essential on this fast-tracked project. It allowed quick information turnaround and ensured that the integrity of trunkline system was not compromised.
- Regular communication and discussions with all interfacing parties from a very early stage in the project allowed all operations to be performed without interference and with a minimum of delay. This was particularly important given the fast-tracked nature of the project.

Despite the numerous technical difficulties faced during this pipeline shore approach design, it was possible to produce a cost-effective and robust design within budget and within the allowable time frame. The final result is a trunkline system that is well protected from exposure to accidental external impacts and extreme hydrodynamic loading. This work demonstrates how innovative design practises can be successfully applied to produce a safe, cost effective and robust pipeline shore approach design in a challenging location.

9 REFERENCES