

OTC 16158

Bayu-Undan Substructure Foundations: Conception, Design & Installation Aspects

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This paper was prepared for presentation at the Offshore Technology Conference held in Houston, Texas, U.S.A., 3–6 May 2004.

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Abstract

The substructures for the two Bayu-Undan processing platforms were successfully installed in the mid 2002, and the floatover decks in the third quarter of 2003.

This paper describes the shallow foundation scheme and design and installation aspects adopted for the substructures for the platforms, the circumstances which led to the conception of the novel foundation system and the challenges that were overcome during design to ensure feasibility of the installation of the substructures.

Introduction

The Bayu-Undan field is located approximately 500 km northwest of Darwin, Australia and 250 km east of the Democratic Republic of Timor-Leste (formerly East Timor) in an area known currently as the Joint Petroleum Development Area (JPDA – formerly the Zone of Cooperation between East Timor and Australia). The field contains 2.7 TCF of recoverable gas reserves and roughly 375 million barrels of condensate and LPG. The development lies in 80m of water and is operated by ConocoPhillips with Eni, Santos, INPEX, Tokyo Electric Power Company, and Tokyo Gas Company as co-venturers.

The first phase of development, approved in August 2000, focuses on gas recycling with condensate and LPG recovery. The field facilities consist of:

- A Drilling, Production, and Processing (DPP) platform (with a dry topsides weight of 14,000t)
- A Compression, Utilities, and Quarters (CUQ) platform (with a dry topsides weight of 11,500t)
- A flare bridge and cantilevered flare tower connected to the DPP
- A single wellhead platform (WP1) located 7.4 km east of the two main central processing platforms (CPP)
- A 172,500 dwt FSO that stores and offloads condensate, propane, and butane as separate products

The field is set to come onstream early in 2004 with full production of 1.1 BCFD of rich gas and 115,000 BPD of liquids anticipated mid year.

Phase II of the project, which was approved in June 2003, involves building a 26" gas export pipeline to Darwin to feed a 3.52 mtpa LNG plant scheduled for start-up in early 2006.

The infield development was designed by TIGA, a joint venture between Worley Pty Ltd and Fluor Daniel Pty Ltd.

The CPP Platforms – DPP Platform Floatover Just Completed



Design of the Bayu-Undan foundations proved to be a very challenging aspect of the project due to the remote location and the geotechnical properties of the calcareous soils in this region of the world.

The conventional method of providing foundation fixity to substructures installed on calcareous material in Australian waters is by means of drilled and grouted piles. This is time consuming for the offshore installation spread and consequently very costly. It is also fraught with the risk of additional cost due to grout loss and hole collapse due to the variable nature of the calcarenite. Two major platforms off the North West shelf of Western Australia have required extensive foundation remedial works.

The two Bayu-Undan Central Processing Platforms are relatively unique in that they have heavy topsides, 'squat' 8 legged steel framed jackets configured for deck floatover (see Figure 1), low environmental loading, medium water depth of 80m and are located on a substrata consisting of 2m of silty sand overlying a hard caprock. With the deck in-place, the substructure foundations do not experience any uplift during the 100 year maximum wave height of 12.6m. This allowed the use of shallow, steel plated spread footings, founded on the

calcarene caprock surface, in lieu of the traditional drilled and grouted piles to resist the topside loads. The shallow foundation concept provided a means of reducing the installation schedule, the cost and the risk.

A small pin-pile was drilled and grouted at each corner of the jackets, designed to resist tensions anticipated to occur during floatover installation of the decks.

The novel foundation scheme was conceived as a result of the unique combination of configuration and environment described above and due to the encouragement to reduce risk and cost for the project. Adoption of the concept brought about challenges, which had to be overcome by the design team.

Background – Options Studied

Feasibility studies were undertaken by Worley in 1996/1997 for BHPP/PPCo. It was determined that the total topsides would weigh about 25,000t.

In addition to piled steel substructures, Worley evaluated a concrete gravity base structure (CGBS) in the Feasibility Study. Ove Arup was sub-contractor for this study. The concrete GBS was eliminated on economic grounds. There are not many locations in the region suitable for constructing a CGBS, and very few suitable deepwater sites for the deck mating.

Floatover decks were investigated in order to minimise installation and offshore hook-up and installation costs at this remote location. There was concern about constructing and installing a single 25,000t deck. The largest floatover deck that had been installed at that time was less than 10,000t. It was decided to split the topsides into two bridge linked decks. The option selected to be carried forward to preliminary design was two platforms with piled steel jackets and floatover decks of approximately 11,000t and 14,000t.

Preliminary engineering started at end of 1997 on the two platform concept, with piled jackets as the base case. The geotechnical site investigation was performed during 1998, and the results were becoming available towards the end of preliminary engineering phase.

The field reports indicated two significant features.

- i. At both the CPP and WP1 sites there was a 'caprock' material (1 to 2 metres thick) consisting of very well cemented calcarenite material. Overlaying the caprock was a superficial layer of uncemented silty sand approximately 2m thick at the CPP site. Under the caprock were varying layers of weakly to very strongly cemented calcarenite.
- ii. Commencing at about 48m below ML there were significant bands of uncemented gravelly material. At the CPP site this band was 7m thick, on top of which the calcarenite was very weakly cemented and vughy, with some thinner layers of gravel. The design penetration of the drilled and grouted piles at the completion of preliminary engineering was 60m.

The latter point gave rise to serious concerns about open hole stability and grout loss to the formation during piling

operations, while the former point indicated that shallow foundations were a definite alternative.

The other key driver was that, due to the relatively benign environment, gravity loads not environmental loads, governed the design of the CPP platform foundations. The CPP jacket foundations only experience up-lift (tension) during the float-over deck mating operation. At all other times the foundations are in compression, even under the 100-year wave.

At the end of the preliminary engineering phase, TIGA undertook a brief study into shallow foundation options for the Bayu-Undan steel substructures to determine if an alternative foundation system would reduce project capex costs and exposure to construction risks. The piled foundations would be replaced by spread footings.

As noted above the CPP jackets only experience uplift during the deck installation and it is a transient condition. It was concluded that stability could be achieved by filling the jacket legs with ballast. This would require a minimum of modification to the existing jacket configuration and the ballast could be placed easily offshore using relatively simple equipment.

The study identified that the shallow foundation option could provide a direct cost saving over piled structures due mainly to the significant reduction in offshore construction duration.

There was, however, concern whether the site would be sufficiently level for the shallow foundations. The bathymetry surveys indicated 'pinnacles' of calcarenite in the region penetrating the surficial layer, although not at the platform site. The 2m thick sand layer could cover smaller pinnacles which would pose a significant risk for the shallow foundation option.

Further studies into the shallow foundation option were recommended to be carried forward into detailed design, as well as surveys to determine the levelness of the caprock surface. The site investigation programme was modified to include some additional shallow CPTs and sampling to gather data that could be used in any subsequent shallow foundation study.

Detailed engineering commenced in the last quarter of 1999. Economics of the development were marginal. Also, at that time East Timor had just gained independence from Indonesia and rights to hydrocarbon deposits in the Timor Sea were being negotiated with Australia. There was encouragement to reduce project costs.

Invitations to bid for the platform fabrication and installation were prepared at the beginning of detailed design phase. The jackets were designed with piled foundations as the base case. The project could not afford delays to the schedule due to significant changes in the steel substructure configuration.

Due to the results of the geotechnical surveys, there was concern about construction risks associated with drilling and grouting the piles. The risk of hole instability and grout loss could not be reasonably assessed without open hole and grout leak off tests. If undertaken, the results of these tests would not be available until July 2000, which was too late in the project programme to influence the design of the jackets.

Based on the recommendations of the preliminary engineering study, investigations into shallow foundation options were performed early in the detailed design phase.

Shallow Foundation Alternatives Studied

The CUQ and DPP substructure foundations only experience uplift during pre-mating of the floatover decks with the jackets. The maximum corner leg tension during the deck mating is approximately 17 MN. TIGA commenced studying the ballasted option addressed in preliminary engineering. Iron ore ballast was assumed to be tremied into all the jacket legs. To accommodate sufficient iron ore to give an adequate factor of safety against uplift, the corner legs would need to be increased from 2.75m diameter to 3.9m. The spread footings would be sized to allow for the additional loading due to the ballast. The steel watertight diaphragms for the flotation compartments in the legs would need to be replaced by rubber rip-out diaphragms. Added to this was the cost of iron ore procurement, transport and placement.

The above disadvantages prompted the suggestion of an alternative configuration involving the installation of a drilled and grouted pin pile in each corner leg to resist tension, thereby eliminating the need for ballast and reducing the bearing loads. The pile would be 1.8m in diameter and the penetration was estimated to be 20m, which located the tip of the pile well above the gravelly layers that gave concerns about open hole stability and grout loss. The jacket leg diameters would not need to be modified.

The following shallow foundation options were investigated:

- Bearing on top of the 2 metre thick silty sand layer
- Bearing on the caprock layer

The main advantage of bearing on top of the surficial layer was that the time for installation could be significantly reduced, as the silt/sand did not need to be removed. Mudmats with nominal 1.5 metre skirts were sized to sit on top of the silt layer with minimal settlement. Large mudmats greater than 20 metres square would be required at the corners of the jackets. The steel weight estimated for the mudmats and associated support steel was approximately 3000 tonnes, the cost of which alone precluded this option. Significant additional costs would also be required for additional buoyancy and anodes. This option was therefore dropped in favour of options that achieved bearing directly onto the caprock.

By bearing onto the caprock, the footing sizes could be significantly reduced. A spread footing of approximately 100 sq m was estimated to be required at each corner leg. Underbase grouting would be required to cater for caprock undulations and ensure that bearing forces were transferred directly to the caprock.

For this option, the surficial silty sand layer would need to be removed at each footing location, prior to the installation of the jacket.

The tension pile gravity base option was recommended in place of the base case drilled and grouted piles. The foundation would consist of a spread footing at each corner leg supported on the caprock layer to resist the compression loads,

and a single pile installed through a sleeve in each footing to resist tension loads during floatover deck installation.

The tension piled gravity based foundation was seen to be technically feasible from both a geotechnical and a structural engineering point of view. Risk assessment results showed that the shallow foundations, while providing significant cost savings over piled foundations, did not pose any additional risk. The estimated cost saving was primarily in the area of reduced installation costs with an estimated two month reduction in schedule, but also significant steel savings could be made due to the elimination of the mudmats, pile sleeves and 1300t of piles.

The tension pile gravity base concept was a novel concept, which required acceptance of PPCo and the Certifying Authority, Lloyds.

It was recommended to perform further laboratory testing of the calcarenite surface layers to assess the properties in order to provide early input to the analyses and make potential savings on jacket and foundation steel, and to perform a grid of shallow cone penetrometer surveys to confirm that the caprock, hidden under the surficial silt layer, was level.

Footing Configuration

The ideal footing shape under the corner legs would be circular, approximately 11m in diameter to give the required area of about 100 square metres. Initial conceptual designs were octagonal to suit plate panel construction. The octagonal design however caused problems with fabrication in the yard, where either elevated skid rails would be required or excavated channels either side of the skid rails to fit the footings. Elevated skid rails would be required for loadout over the concrete quay. Notches in the footings would also be required to fit the footing over the side of the launch barge.

The octagonal footing shape was therefore dropped in favour of rectangular footings, which spanned from the inner legs and cantilevered beyond the corner legs, as shown on Figure 2. The width of the footings were limited to about 5 to 6 meters to allow sufficient clearance to the launch barge deck.

Jacket Footings



The footings were provided with 500mm high stiffened skirts around the perimeter to contain grout. The skirts had to be sufficiently high to allow free flow of the grout under the

baseplate and to ensure clearance to any undulations or 'pinnacles' on the caprock surface.

The design of the footings had to assume that when the jackets were initially installed and were supported on the perimeter skirts, undulations in the caprock could induce torsion into the footing. A box girder was therefore provided on top of the footing to provide bending strength and torsion resistance.

The footings were pre-fabricated separately in the workshop from steel plates, then offered up to the jacket legs. It was necessary to optimise the footing weight to minimise auxiliary buoyancy requirements.

To ensure that the grout displaced the water under the footing, the plates were sloped up towards the centre to allow the water and excess grout to be discharged via pipes either side of the box beam. A number of larger pipes with cones were provided along the footings for stabbing the grouting stinger, and to allow inspection if required.

It was necessary to notch the footings over the launch rails, so that the shape of the launch face footings resembled a cricket bat.

Close-Up of 'Cricket Bat' Footing



Foundation Loads and Ultimate Bearing Pressure

A number of load conditions were analysed to determine foundation loads. While all of these analyses indicated that there were no tensile forces on the foundations for in-place conditions, the survivability seismic and storm cases gave a small tension reaction on one leg. Since the mat foundations were assumed to not have any tensile capacity (disregarding the effect of the tension pile), any such theoretical tensile forces would be released and redistributed to the other foundations. This would lead to marginally increased compression loads on at least one of the other legs.

The ultimate bearing pressure of the foundations under combined horizontal and vertical static and cyclic loads were estimated using parameters derived from the cone resistance profiles and the available soil strength data from laboratory testing of the Bayu-Undan materials and other similar soils. The foundations were conservatively sized using extreme lower bound soil parameters. Further laboratory testing was performed to enable optimisation of the foundation design.

Aspects concerning the geotechnical design are covered in the OTC paper 16157 by Steve Neubecker and Carl Erbrich of Advanced Geomechanics in Perth, Western Australia.

Differential Support Settlement

Different levels of settlement at different corners of the jacket can have a large impact on the structural performance of the jacket. Analyses were performed to bound the expected settlement of the shallow foundations under static and cyclic loads, using spring supports in the most unfavourable combination (two stiff springs on opposite corners and soft springs on the other two corners) assuming the worst credible combination of soil profiles and stiffness parameters. These analyses are covered in more detail below.

Jacket and Footing Analyses

The substructure and the footing were analysed in two parts:

- A global analysis of the substructure was performed using a footing model designed to represent the rotational and translational response of the interaction between the jacket and the caprock.
- A local analysis of the footing itself was performed using a separate model reflecting the vertical and horizontal loads imposed on the footing and predicted by the global in-place analysis.

Substructure Global Analysis

The foundation system for the substructure analysis was represented as four rectangular spread footings spanning between each corner and adjacent leg. In order to accurately analyse the substructure in the in-place condition, the footings were modelled taking consideration of:

- The non-linear settlement response of the caprock.
- The rotations and displacements of the connection between the jacket and the footing.
- The possible variation in stiffness between the foundation response at each of the four legs.

A 3D finite element model of the caprock and soil conditions, described by S Neubecker and C Erbrich in OTC paper 16157, was used to develop the response of the jacket and foundation to a specific set of storm loads. This model was used in an iterative process along with the SACS substructure model to predict jacket and footing responses.

In order to reproduce the displacements and rotations of the connection points of the jacket legs with the footing, an equivalent beam element represented by a stiffness value was modelled for each of the four footings. Table 1 shows the variation in beam stiffness used depending on the stiffness of the foundation and the shape of the footing.

A set of equivalent linear springs was also developed to represent the response of the underlying foundation. They were applied to the equivalent beam element representing the footing, as shown in Figure 3. The values were developed through an iterative process between the F.E model and the substructure SACS analysis with consideration given to variation in caprock and soil stiffness as described below.

Three combinations of caprock/soil stiffness were considered:

- Response A – The lower bound degraded option representing the weakest and softest response. The caprock was considered cracked and the underlying soil is considered to be degraded.
- Response B – The upper bound degraded option representing intact caprock with degraded underlying soil conditions. This condition was used in combination with response A to maximise jacket racking options.
- Response C – This was the stiffest response representing intact caprock and non degraded soil.

Table 1 shows the variation in spring stiffness. The in-place analysis considered the following four combinations of soil stiffness:

- 4 foundations supported on weak soil - response A
- 4 foundations supported on strong soil - response C
- 2 diagonally opposite foundations on weak soil (response A) with the other two on 'strong' soil - response B.
- As above but with the stiffness on the diagonally opposite footing reversed.

In order to properly simulate the gravity base footings the spring elements described above and shown in Figure 3 were modeled as gap elements capable of only resisting compression and shear. The shear capacity was set with a coefficient of friction of 0.5 based on the axial load in the spring element.

After set down of the jacket on the seabed, and prior to grouting of the footings, the jacket could rest on three corners due to the possibility of the seabed being uneven. This would introduce stresses that would be locked into the structure as long term pre-stressing of the jacket members. In the case where the COG of the jacket is at the geometrical centre, the jacket would be balancing on two legs only. To account for these stresses a set of vertical opposing loads were applied over the opposite diagonals, in a way that no further overturning moment was introduced. The loads used to account for the locked in stresses were applied together with waves in a diagonal direction and were applied to maximise the stresses in the most compressed legs. The locked in stresses were based on an on-bottom jacket weight prior to grouting of 52MN.

Footing Analysis

To facilitate the design of the footings a separate SACS model was developed in order to replicate the pressure distribution across the foundation and consequently accurately predict the shears and moments within the steelwork. The footing model consisted of a beam element representing the actual footing bending and shear stiffness and supported on springs along its length at 1m spacing. Forces and moments taken from the global in-place analysis were applied at the leg locations. A schematic diagram of the footing model for both the north and south footings is shown as Figures 4 and 5.

The value of the foundation spring stiffness was determined by an iterative procedure until a reasonable match between the geo-technical ABAQUS model (refer to OTC

Paper 16157 by S Neubecker et al), and the elastic spring pressure distribution was obtained. The comparison showed that spring stiffnesses in the range of 44 KPa/mm to 61 KPa/mm bounded the response predicted by the ABAQUS model averaged across the footing. The springs were therefore modeled using the upper bound stiffness, which related to 350,000 kg/cm for a one metre spacing. Some variation in spring stiffness for the north footings were used as the area varied due to the 'cricket bat' shape described earlier.

The following three design load conditions were considered:

1. Installation condition: In this case the jacket is supported on the footing skirt plate prior to grouting underneath the base plate. It was assumed that due to the possible uneven nature of the caprock, the on bottom weight would be supported at a discrete length along the skirt. The length of skirt was determined accordance with a maximum local bearing stress of 100MPa and an on bottom reaction of 20MN at the footing produced by the on-bottom stability analysis. Various locations along the skirts were considered to maximize loads in the footing elements.
2. Operational condition: This load case represented the in-place, seismic and mating conditions following grouting between the base plate of the footing and the caprock.
Inplace strength analysis: The leg loads were taken from the analysis representing the upper bound (stiff) soil conditions, as the maximum footing loads always occurred under a stiff corner for the strong / weak support combination. The design axial load for the north footing was 109MN for the operating (10yr storm) case. For the south footing, the design axial load was 102MN for the operating case. The extreme storm loads (125yr storm) were 129MN and 123MN respectively but did not govern as a 1/3rd increase in allowable stress was permitted. These loads produced an average pressure of 1MPa on the footing base plate.
Seismic analysis: The design loads for the seismic condition were 113MN for the north side and 105MN for the south side. Although these loads were greater than the operating condition, they did not govern as a 70% increase in allowable stresses was allowed.
Deck mating: These loads were significantly less than the operating condition and therefore did not govern the design.
3. Operational condition combined with residual installation loads: As the footing was to be grouted prior to deck installation there could be, depending upon the unevenness of the caprock layer, a locked in residual load resulting from the jacket being supported on the skirt plates. These loads were combined with the operational loads.

The skirt load was taken as 13.5MN per footing. This load was based on the jacket legs being fully flooded and the buoyancy tanks removed. It did not include installation equipment and environmental loads considered in the on bottom stability analysis. The skirt loads were applied as uniformly distributed loads of 3MN/m over 4.5m applied at various locations along the skirts to maximize loads in the footing elements. This load case required that the leg forces be factored downwards to ensure that the extra skirt load (representing a redistribution of footing reaction) would still produce the same total compressive leg forces. This load case proved to be significant, as it produced additional torsional shear forces on the footing web and web stiffening plates together with the more concentrically applied bearing loads predicted by the straight operating conditions.

Fatigue Analysis

An area of concern for the inplace fatigue situation was the connection between the legs and the footing. Consequently, a finite element model was developed to determine stress concentration factors at the leg/footing interface. They were developed for axial forces, inplane and out of plane moments.

The maximum SCFs were determined to be 7.4 for a vertical load applied to the inner 2500 diameter leg. All other SCFs were shown to be less than 4. Consequently fatigue lives for the 8 leg connections were found to be all in excess of 200 years.

Tension Pile To Spread Footing Interaction

The role of the tension pile was to provide resistance to uplift and sliding during docking of the floatover deck vessel inside the jacket. The pile was not considered to contribute to the foundation strength for the compression loads during the inplace conditions. The pile was assumed to have a tension only connection to the footing sleeve, because the effect of the pile on the response of the combined spread footing-tension pile foundation to compression loads was difficult to predict. If compression loads applied by the deck were transmitted to the pile, its response would be stiffer than the spread footing, and may initially 'fail'. After this, the foundation would have a different response to the compression loads. It may be possible that the pile would fail at some legs and not others, leading to undesirable differential support deflections.

To 'de-link' the pile and provide a tension only connection to the pile sleeve, a collar was welded on the pile head, and a thick undercoat on the pile in the sleeve interface to prevent bonding of the grout to the pile.

Conductor Drilling and Jackup Interaction

The effect of jackup spudcans on nearby foundations was assessed and the impact on the foundations found to be negligible, as due to the stiffness of the formation, the penetration of the spudcans into the caprock was minimal.

The stability of the conductor holes, which were drilled relatively closely to the foundations, was also verified for the overburden pressure caused by the platform footings and found not to be affected to any significant extent.

Scour

The effect of scour on the foundations was found to be negligible given the cemented properties of the supporting caprock.

Jacket Launch

Initial analyses found launch problems due to the low aspect configuration of the jackets. With a conventional top first launch, the jacket re-impacted on the barge tilt beams. There were also problems finding a suitable configuration of the buoyancy tanks to give a stable launch. The solution was to launch base first, with the jacket self upending.

Launch Model tests revealed high impact loads during transfer between the primary rocker arms and secondary rocker arms. The impact loads were not predicted by the Moses launch analyses. The magnitude of impact loads was not readily defined by tests, and not significantly reduced by modifying variables (i.e. pre-launch trim, jacket pre-launch position, dampening under primary rocker arms, etc). The simple yet effective solution was to use only the secondary rocker arms and tie down the primary rocker arms.

Due to the 12 day tow from the fabrication site in Batam, the installation barge would undergo standby waiting for the second jacket to be delivered following launch of the first. The installation contractor McDermott innovatively deployed the large launchbarge I-650 to transport the two jackets at the same time. After the first jacket was launched, the second was jacked along the barge to the launch position, and then launched. This saved both the installation spread standby and towing costs had a second trip been required.

Two Jackets on One Barge



Surficial Layer Removal

The footings for the GBS option were sized assuming that platform loads were supported by bearing on the caprock. Underbase grouting was required to cater for caprock undulations and ensure that bearing forces were transferred directly and evenly to the caprock. The silty sand between the footing and the caprock had to be removed to ensure foundation integrity.

The surficial layer was removed locally at each footing location, prior to the installation of the jacket. This was achieved by using a blowing tool deployed from a supply vessel and a specially constructed jetting and airlifting tool, deployed by the derrick vessel. The size of the area cleaned at each footing location allowed for possible locational tolerances for placement of the jackets.

It was not possible to totally remove all particles of sand and gravel from the footing locations and an acceptance criteria was defined. A unique for purpose penetrometer tool was designed and built in order to provide a means for measuring the residue silt in the excavation. A calibrated small diameter rod with a weight on top was supported by a small tripod frame. The weight was sized to apply 1 MPa of pressure at the tip of the rod. This was sufficient to penetrate the sand but not the caprock. The tool was deployed by an ROV and provided a quick and easy means for accepting the cleanliness of the excavation.

Levelling

The facility to level the jackets using a hydraulic levelling tool was proposed initially, using the tension pile to support the jacket. Levelling would be carried out after the tension pile was grouted to the formation.

Detailed CPT surveys showed that the caprock was level to within acceptable tolerances and the requirement for levelling the jacket was then regarded to be a contingency only. The hydraulic levelling tool option was dropped. The contingency option was to survey the footing locations after silt removal, and then if required, grout would be pumped into the low footing location, prior to jacket installation. This contingency was never required to be exercised.

Grouting

Following jacket placement, the footings were grouted in stages. The tension pile hole was drilled and the pile installed. It was grouted to just below mudline. The footing was then grouted to just above the toe of the skirt, in order to seal the skirt. The surrounding bunds provided by the surficial silt layer retained any grout flowing below the skirt (the silt was removed only locally at the footings prior to setting the jacket for this purpose). The minor penetration of the skirts into the caprock served to provide an efficient grout seal. The remainder of the footing and the tension pile sleeve was then grouted in the next operation.

Grout was pumped via a stinger stabbed into receptacle sleeve pipes installed in the footings.

Heat of Hydration Cracking

It was assumed that heat of hydration cracking would most probably occur in the footing grout due to the large volume. This would be mitigated by the large surface area of the footing, and by the staged grouting procedure outlined above. The grout would be maintained in compression by the platform loads and contained inside the footing skirt, so that heat of hydration cracking of the grout was not considered to be of concern.

Conclusions

Western Australia has an infamous history of multi-million dollar remedial works to piled foundations in calcareous material. The Bayu-Undan jacket foundations provide an innovative solution which reduced the risk and cost of installation. These are the first foundations of their type, and were successfully installed using conventional installation equipment.

The tension pile gravity base foundation solution was a novel and unique foundation concept for an offshore platform. It is made possible by the combination of the particular circumstances pertaining to the CPP platforms and the site.

These are:

- the benign environment at the site (100yr wave height of 12.6m)
- the 'squat' configuration of the jackets driven by the deck floatover method
- the high topsides loads.
- the stratigraphy of the soil formation.

Acknowledgements

The authors wish to thank TIGA, ConocoPhillips, and the other Bayu-Undan co-venturers for their permission to publish this paper.

Nomenclature

COG- Centre of Gravity

CPP- Central Processing Platforms

CPT- Cone Penetrometer Test

CUQ- Compression, Utilities and Quarters (Platform)

DPP- Drilling and Production Platform

GBS- Gravity Based Structure

ROV- Remotely Operated (Underwater) Vehicle

SCF- Stress Concentration Factor

References

1. S.R. Neubecker and C.T. Erbrich, Bayu-Undan Substructure Foundations: Geotechnical Design and Analysis, OTC 16157 Offshore Technology Conference, May 2004, Houston.

Appendices

Tables

Table 1: Foundation Spring Stiffness

	INPLACE		
	Response A	Response B	Response C
K _{xx} , [MN/m]	3892	5695	23101
K _{yy} , [MN/m]	7651	4193	15673
K _{θxx} , [MN/rad]	3004	72967	8800
K _{θzz} , [MN/rad]	6512	3413	247435
DPP North			
K _{zz-inner} , [MN/m]	393	1107	1284
K _{zz-outer} , [MN/m]	326	1101	1941
K _{zz-centre} , [MN/m]	602	2288	6745
E _{Iequiv} , [MN/m ²]	6.21E+04	7.33E+04	8.73E+04
DPP South			
K _{zz-inner} , [MN/m]	248	697	809
K _{zz-outer} , [MN/m]	437	1475	2601
K _{zz-centre} , [MN/m]	638	2426	7149
E _{Iequiv} , [MN/m ²]	6.45E+04	7.62E+04	9.08E+04

Figures

Figure 1 - Jacket Elevations

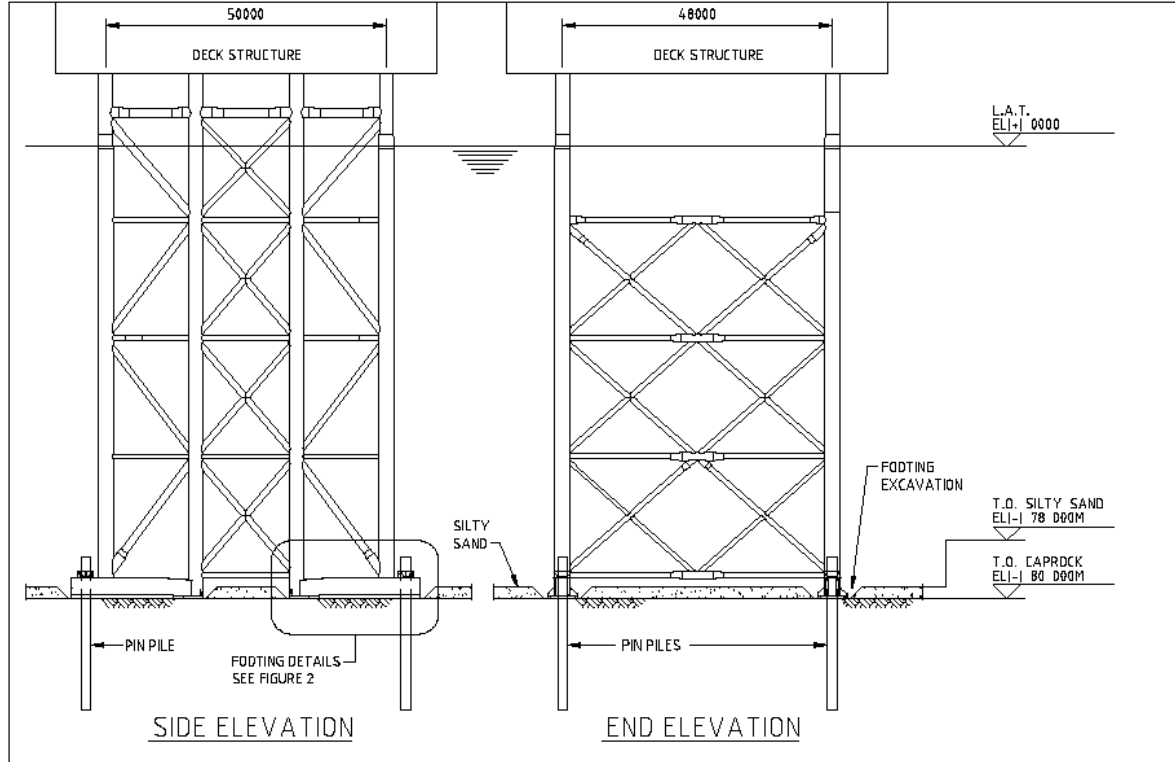


Figure 2 - Footing Details

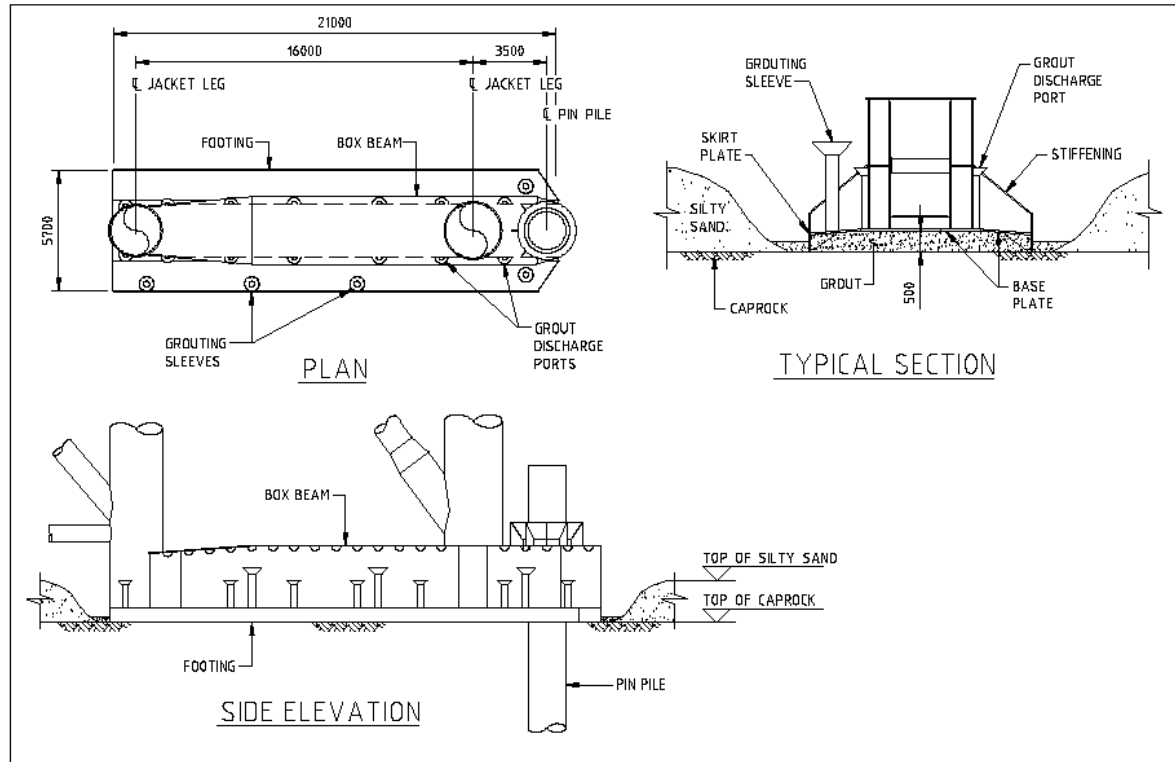


Figure 3 - SACS Foundation Spring Modelling

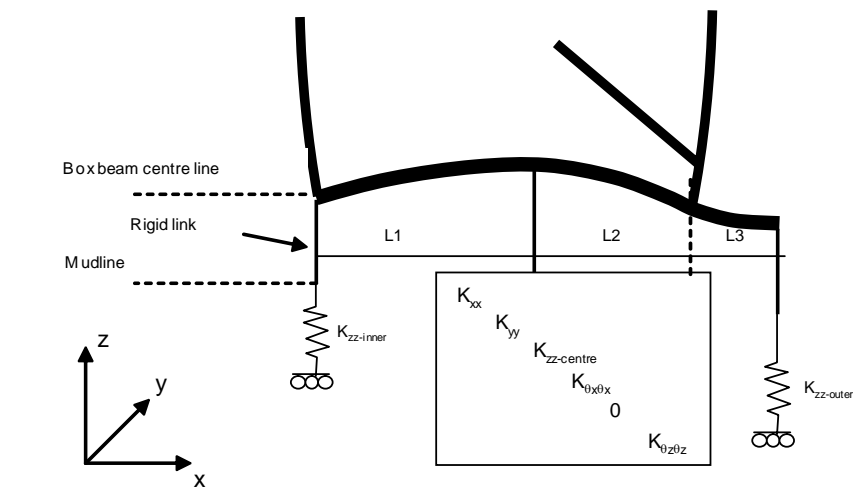


Figure 4 - Footing SACS model for DPP South (Launch Face)

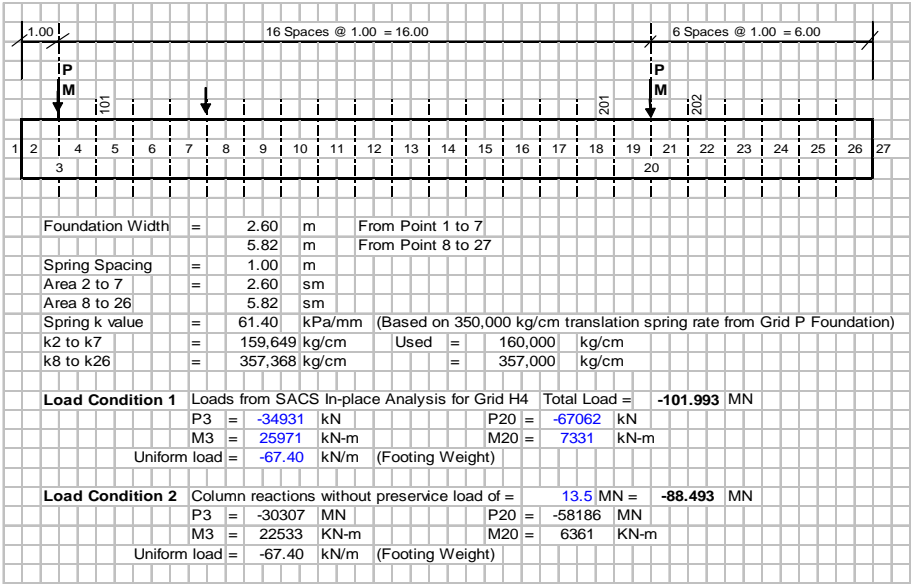


Figure 5 - Footing SACS model for DPP North (Opposite Launch Face)

