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Structural Platform Solution for Seismic Arctic Environments--Sakhalin II Offshore Facilities

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Abstract

This paper describes the development of technical solutions for the offshore platforms of the Sakhalin II development project in order to cope with the special characteristics of the arctic and seismic environments. The emphasis is on describing the design challenges and how these challenges were addressed during the Front-End-Engineering-Definition leading to the concepts currently going through detailed design. Transportation and installation aspects in such a remote environment with a limited weather window form an integral part of concept development and need to be addressed early on.

The structural solutions for the two offshore platforms comprise a four-legged concrete gravity based structure (GBS), supporting an integrated deck with a seismic isolation system between topsides and GBS. In addition to providing seismic isolation, such a system also effectively isolates the topsides from wave, ice and thermal contraction loadings and hence simplifies the design considerably.
The structural concepts which are currently going through the final stages of detailed design, provide effective solutions taking due consideration of constructability, transportation, installation and operation for 30 years

Introduction

This paper provides a description of the design challenges and how they were overcome during the Front-End-Engineering-Definition (FEED) work for the Lunskoye A and Piltun-Astokhskoye B platforms for the Sakhalin II development. There is currently little track record of feasible platform solutions in the new frontier environment of the seas around Sakhalin Island. Thus, the focus of the paper will be the structural and naval architectural design issues that were faced due to the remote location of the Sakhalin II fields, in combination with a harsh arctic environment and the significant seismicity of the region.

This paper describes the conceptual solution that addressed the specific design drivers that are unique to this region of the world and these will be presented and critiqued in the paper. At the time of writing this paper the detail of the design has advanced, however the design drivers and principal solutions remain unchanged from the FEED.

The technical challenges and the engineering solutions developed are addressed with reference to the following: Sea transportation, Installation, Arctic design (material and temperature issues), Sea ice influence (Topside to Substructure interactions), seismic design (Topside to Substructure interactions including seismic isolation methods considered), wave impact, blast (highlighting differences encountered in arctic conditions).

Readers may find the design experience presented here beneficial for future developments in this region of the world. This paper initiates the population of a database of structural platform solutions. The paper highlights the unique combination of design drivers and presents structural design solutions that could be referenced for future field developments in this region of the world.

At the date of publication of this paper, there is only one existing platform facility on location in the Sakhalin region. This is the Piltun-Astokhshkoye A platform, which is a caisson based platform. Part of the Sakhalin II field development is to import oil from this existing platform to the new Piltun-Astokhskoye B platform.

Structural Platform Solution

The Structural Platform Solutions for both Lunskoye A (Lun-A) and Piltun-Astokhskoye B (PA-B) platforms for the Sakhalin II development are in principle identical and comprise a four-legged concrete gravity base structure (GBS), supporting an integrated deck, with a proprietary seismic isolation system known as Friction Pendulum Bearings (FPBs), interposed between the topsides and GBS. Four FPBs per platform are utilized that correspond with the four concrete GBS support legs. Figure 1. Isometric view of one of the

Sakhalin II platforms complete with all facilities and cladding. Figure 2 shows an isometric view of the topsides main supporting steelwork of one of the Sakhalin II platforms. Table 1 provides some high level data for both Sakhalin II platforms.

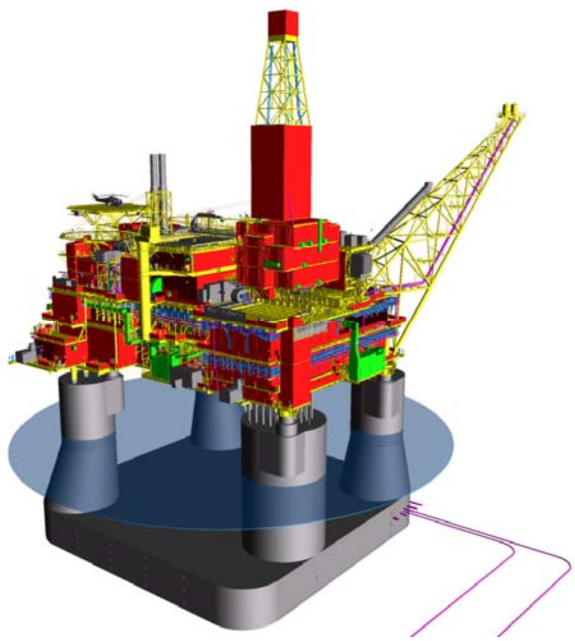


Figure 1. Isometric view of one of the Sakhalin II platforms

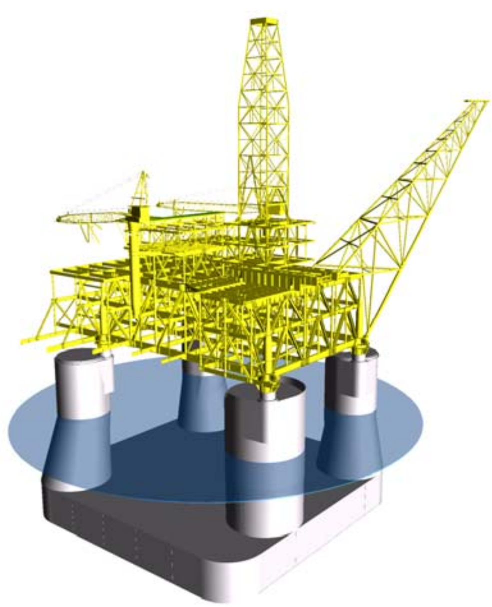


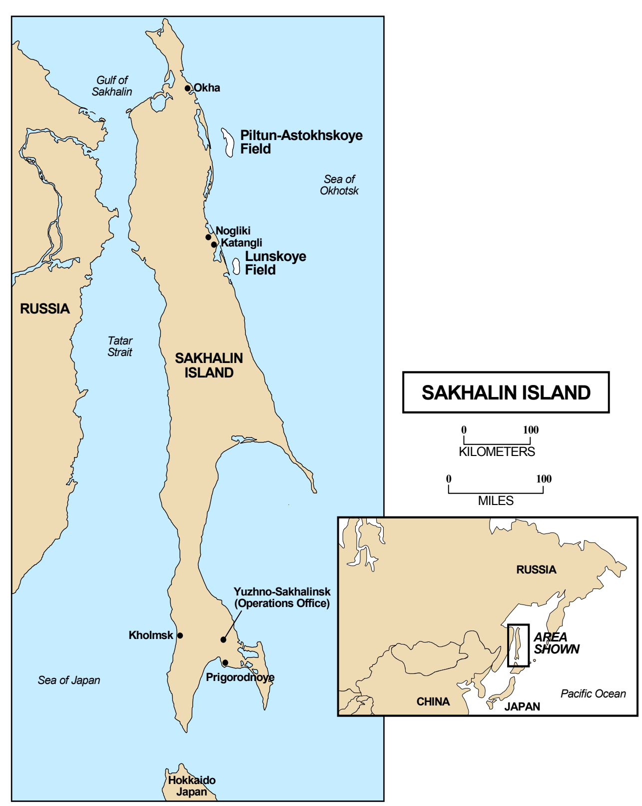
Figure 2. Isometric view of the topsides main supporting steelwork of one of the Sakhalin II platforms

Table 1. High Level Platform Data

	Lun-A	PA-B
Design Life (years)	30	30
Topsides Dry weight (tonnes)	21 000	27 500
Topsides Operating weight (tonnes)	27 000	33 500
Approximate Topsides Plan Dims (m)	100 x 50	100 x 70
Water Depth (m)	49	30
Approx % Topsides Structural weight vs. Topsides Dry weight	56	57
Number longitudinal Primary Trusses and Transverse Primary Trusses	4 and 6	4 and 7
Number of Conductors	27	45
Facilities	Drilling Production Utilities Living Quarters	Drilling Production Utilities Living Quarters
Gas production	1850 MMSCFD	100 MMSCFD
Oil/ Condensate production	50000 BPD	70000 BPD
GBS caisson size LxBxD (m)	105x88x13.5	105x88x13.5
Number of GBS columns	4	4

In addition to loadings conventionally experienced in most offshore environments, the Sakhalin location introduces additional structural design challenges due to the harsh arctic climate and significant seismicity. The location of the both Lunskoye A (Lun-A) and Piltun-Astokhskoye B (PA-B) field is shown on Figure 3.

Figure 3. Lun A and PA-B Location Map



The global structure (Topsides and GBS) has to be designed to resist effects of sea ice and extreme temperatures. With topsides rigidly connected to the GBS, (the environmental loads apply differential or opposing leg loads), this means a significant amount of additional topsides steelwork and thus increased topsides weight. The increased weight has a direct knock-on effect to topsides transportation and installation; increased topsides weight requires a bigger barge with a deeper draft, which in turn has the cyclical effect of increasing the topsides size and thus the weight.
The earthquake activity in the region also has a detrimental influence on the topsides design, with the GBS proving itself to be a good transmitter of seismic loads to the topsides. The seismic loads were such a significant influence on the economic and technical viability of the project that early in FEED, the decision was made to investigate the possibility of seismically isolating the topsides from the substructure, a decision that led ultimately to the adoption of the Friction Pendulum Bearings (FPBs).
Environmental forces transmitted from multi-leg GBSs to topsides have historically caused problems for topsides designers who may be constrained by weight limitations. The spin off of the decision to adopt FPBs was that not only were the seismic loads reduced, but the effects of sea ice, waves and thermal expansion loads were decoupled from the topsides as FPB friction forces are relatively low, effectively removing these three loadings from the topsides structural design. The down side of this decision was that services that span between Topsides and GBS have to absorb the high relative movements between the two structures.

The description of the design phases that the Sakhalin topsides will experience fall into two categories, namely 'Permanent Condition' and 'Temporary Condition'. The Permanent Condition includes the Facilities Self Weight and Operating loads plus all externally applied loads such as environmental loads, accidental loads and Seismic events. The Temporary Condition includes Construction, Loadout, Transportation, Float-over and Set-down phases.

The influence of the permanent and temporary phases on the configuration of the structural platform solution, are both significant and inter-related. To develop a structural platform solution that works most effectively or at least as a best compromise for both phases, the permanent and temporary conditions need to be studied concurrently.

The following two major sections of the paper comprise the **Permanent Condition** and the **Temporary Condition**. These sections describe the externally applied loads and the design philosophies that were used during the FEED for the Sakhalin II platforms. These sections provide explanations for the main influences on the design and where appropriate design philosophies are included to support the production of the optimum design solution.

Permanent Condition – Environmental, Accidental, Seismic Design, Structural Design and brittle Fracture

Environmental loads. The following section details the environmental loads that are a feature of this region of the world.
Sea Ice. The sea ice forms in the enclosed northern part of the Sea of Okhotsk. Sea ice emanates from here and is present in the Sakhalin II region for about six months of the year, manifesting itself as huge floes of ice drifting with the tide and wind, sometimes as vast sheets up to 2m thick, sometimes as broken floes with unpredictable directions, all constantly moving.
As sea ice sheets advance and contact the platforms, they must break over a 100m-wide path across the concrete structures, exerting high horizontal ice crushing loads on the columns. The global platform can be subjected to horizontal loads of up to 269MN operational ice (1-yr return), 324MN design ice (frequent event 100-yr return) and 405MN design ice (infrequent event 10,000-yr return), due to sea ice movement.
Conductors, Caissons, J Tubes and Risers have to be protected from the effects of sea ice loadings. This can only be practically achieved by locating all services within the legs.
The robust design needed to resist such arctic specific forces and the need to protect the services from the sea ice by locating them within the legs, are very influential on the configuration, diameter and design of the GBS legs.
The resulting GBS leg diameters are thus larger than could be expected in a non-arctic GBS in similar water depths.

Influentially for the topsides is that sea ice can also apply loads to the GBS legs differentially and multi-directionally. The exposed legs experiencing larger loads than the downstream legs. The maximum operational (1-yr return), design ice (frequent event 100-yr return) and design ice (infrequent event 10,000-yr return) differential leg load is in the order of 103MN, 124MN and 155MN respectively.
If the topside is rigidly connected to the GBS, then it is calculated that about 20% of theses sea ice loads will track through the topsides structure between the GBS legs. This splitting/crushing load is a significant additional topsides design load and causes the requirement for more robust topsides primary truss bottom chords. However, if the topsides is weakly constrained to the GBS then differential sea ice loads cannot effectively track through the topsides with relative structural movement between the topsides and GBS occurring instead.

Sea ice crushing on the legs can induce topsides vibrations that have to be assessed against habitability standards. The standards were met without the requirement for vibration isolation.

Similar configuration non-arctic GBS structures (e.g. Malampaya) have terminated the main leg elevation some distance below the underside of the topsides, exposing to the atmosphere the Conductors, Caissons, J Tubes and Risers. This has provided the benefit of allowing waves to pass through the GBS, reducing the influence of the structure on the waveform. However in early winter (Ice freeze-up) and late spring (Ice break-up) there is a joint probability of the occurrence of broken ice and waves. For arctic environments, exposed services run the risk of being impacted with ice fragment lifted by high waves. The significance of these wave induced ice momentum forces requires the termination of the shaft elevation to be raised in order to create a physical barrier protecting the services. However the consequence of this is an increase in the blockage effect to waves passing through the GBS causing irregular wave effects and wave run-up resulting in significant wave impact slam forces to the underside of the topsides that has to be catered for in the design. No damage is allowed to topsides for wave slam forces from the 100-yr wave with specific survival criteria set for wave slam forces from the 10,000-yr wave.

Wave Apart form the wave slam impact forces to the topsides described in the previous section, waves do not have a dominant effect on the design of the platform.

Temperature The atmospheric 100-yr return low is -36°C, to a high +36°C.
The temperature variation of the atmosphere relative to the sea causes thermal expansion/contraction of the topsides relative to the GBS. This generates significant internal forces or relative structural movement depending on the connectivity of the topsides to the GBS. These effects, which ever is relevant, need to be accounted for in the design.
Low temperature has an adverse affect to the toughness of structural steel. High toughness at low temperature structural steel is specified for the Sakhalin II project to support fracture mechanic calculations and avoid the occurrence of brittle fracture failures.

Snow and Ice accumulations Snow and ice accumulations are significant in the area and have some influence on the global design. Extreme 100-yr return snow and ice accumulations are predicted to be in the order of:
Lun-A 2000tonnes
PA-B 2500tonnes
A reduced snow and ice accumulation load is considered concurrent with seismic events.
Localised structures such as helideck support structures and flare booms are highly influenced by snow and ice loads respectively.

Accidental loads. The following section details accidental loads that are a feature of platforms located in this region of the world.
Blast. The environmental temperature leads to the requirement for creating a workable environment for personnel to operate the topsides, referred to as winterisation. This comprises maintaining a minimum temperature of +5°C in many frequently visited process areas. These areas need to be temperature sealed in compartments bounded by architecturally insulated cladding and serviced by large volume HVAC ducting. Even following good layout practice and with the use of blow out panels, the sealed compartments leads to confinement problems not normally encounted in open naturally ventilated modules. This in turn leads to predicted blast pressures greater than normally encountered. This increase is significant to the design of the topsides.

Seismic Design. The following sections describes the seismic design philosophy and the initial seismic results. The initial results are critiqued and as a result an alternative solution is defined. The design methodology for the alternative solution is detailed with final seismic analysis results presented.
Seismic Philosophy
The Structural seismic design of the Sakhalin structures was performed according to a two-level design check, referred to as the Strength Level Earthquake (SLE) and the Ductility level Earthquake (DLE) events. This approach stems from the high degree of randomness and uncertainty in seismic events and actions that would render design on the basis of strength alone uneconomic.

The first level was the SLE event, the primary seismic design stage. Criteria were developed to ensure that the design was not susceptible to damage during comparatively frequent seismic events and also that the criteria would lead to a design that was likely to satisfy the DLE performance criteria. Members were adequately sized for strength and stiffness to ensure that no significant structural damage occurred. Globally the platform was designed to remain elastic throughout the SLE event, although limited local damage was permitted, such as local yielding of members. The primary objectives were that there would be no loss of life and all equipment would remain functional after an SLE event, although shutdown and inspection are likely.

To meet the above objectives the SLE event was based on a 200-yr return period. Upper and lower bounds were considered for input parameters with a high degree of uncertainty such as soil stiffness and FPB friction. Three time-histories were used with the most onerous of the three used in the design.

The secondary design level was the DLE event for which criteria were developed to ensure that the structure could sustain a rare, intense earthquake with facilities to evacuate the platform and without major environmental damage. By definition global structural or system level failure would not occur. Structural damage was acceptable provided that the Temporary Refuge (TR) remained intact and means of escape to the TR were not impaired. In addition safety critical items of equipment such as lifeboats and safety evacuation equipment, Emergency Shut Down (ESD) valves, risers, major blow-down facilities, equipment with major hydrocarbon inventories and some Control room equipment must survive with full functionality.

The DLE event was based on a 3,000-yr return period but with a best estimate for all input parameters, including soil stiffness and FPB friction. However for the design of the FPBs, in accordance with industry practice for isolator design and in recognition of the criticality of the FPBs to the global structure integrity the DLE seismic input parameters were bounded, as per SLE.

Initial results

The initial analyses were based on the topsides being rigidly connected to the GBS. Maximum average horizontal topsides accelerations experienced during an SLE event were unacceptably high with the topsides supported conventionally on the concrete GBS. The maximum topsides shear force experienced during SLE was 195MN and the corresponding maximum average accelerations are shown in Table 2, below.

	No FPB (g)
Deck Overall	0.73
Deck El(+)27m	0.65
Deck El(+)38m	0.74
Deck El(+)47m	0.84
Flare	4.37
DES	1.22
N Crane	1.74
S Crane	2.27

Table 2 – Lun-A Max Average Accelerations:

Of particular note is the fact that the DES acceleration for SLE was in excess of 1g which was unacceptable from a safety perspective as performance standard required that there should no fatalities during an SLE event.

The impact of accelerations of such magnitude was:

1. Increased topsides steelwork resulting in an increase in economically expensive steel fabrication
2. Increased topsides weight such that installation feasibility is very suspect.
3. Structural steel occupies increased volume putting restrictions on routing of facilities
4. Vendor increase in costs due to high seismic accelerations
5. Unsafe for personnel during a seismic event

The conclusion was made that these initial seismic results indicate that this topsides to GBS configuration is unviable as a platform solution.

Solution. The following section fully describes the solution to mitigating the impact of the initial seismic acceleration results in order to produce a viable platform solution.

Reduction of seismic accelerations

The substructure is required to be sufficiently strong and stiff in order to support the functional loads and to resist the significant environmental, sea ice and wave loads in shallow water, whilst at the same time the topsides accelerations require to be reduced to practical levels. The most practicable way identified of reducing topsides accelerations was by uncoupling, to some extent, the movements of the topsides from the supporting GBS. For the Sakhalin platforms this could have been achieved by:

1. Introducing structural flexibility between the topsides and GBS, similar to that used for the Malampaya platform.
2. Introducing proprietary seismic isolators

Item 1 was initially investigated. However, due to the combination of high functional and environmental loads, there was limited scope to increase global structural flexibility to an extent that would be beneficial in reducing the transmittal of seismic accelerations from the seabed to the topsides.

Indeed, early investigations proved that the introduction of structural flexibility was not feasible for the Sakhalin platforms. The Malampaya type topsides-to-GBS connection solution works on the basis that a single tubular column per support point provides dynamic flexibility, and thus isolation between the topsides and GBS, whilst being robust enough to retain their integrity under operating loads and seismic events. For the Sakhalin project, these two performance criteria could not be reconciled, since the tube dimensions required for structural integrity were very large, and as a conquence were not flexible enough to provide sufficient isolation to the topsides.

Item 2 was subsequently investigated and the resulting recommendations are presented below.

Adopted seismic isolation

Proprietary seismic isolators were identified as the most practicable way forward and several products were appraised for their suitability for use for the Sakhalin topsides. Product performances were assessed against project requirements, resulting in a recommendation to adopt FPBs.

The principal reason for selecting FPBs was obviously to reduce the seismic loads experienced by the topsides and early studies clearly showed that the adoption of FPBs satisfied that objective. Some of the other performance characteristics that support the selection of FPBs include:-

1. Self centering after an earthquake
2. Limited sensitivity to extremes in temperature
3. Good functionality with high vertical loads capacity
4. Isolation becomes more effective the more severe the earthquake
5. Compact in size thus weight impact of incorporating in design minimized
6. Temperature, environmental and ice loads are decoupled from topsides.

The selection of the FPB natural period and friction values control the seismic shear forces for the topsides, and the seismic displacement in the bearings. Two FPB natural period/friction permutations were considered for the Sakhalin II project, namely:-

1. 4 second period, 5% friction
2. 5 second period, 5% friction

The 4 second bearing is smaller in plan, but taller and heavier than the 5 second bearing, but the 5 second bearing, being larger in plan, requires more weight of support steelwork. Topsides accelerations and shear forces for a 5 second bearing are lower than would be obtained from a 4 second bearing. However, the re-centring capability of a 4 second bearing is greater than that of a 5 second bearing and the seismic displacements in the bearings increase with increased FPB period. Thus adopting a 5 second bearing would increase the deflections on the facilities that span across the FPB between the topsides and GBS as well as increasing the likelihood of a permanent offset between the GBS and topsides following an earthquake. The 4 second, 5% friction FPB was adopted as it represented the best compromise between a viable and economic topsides weight and the ability to economically design the facilities that span between the topsides and GBS that are subject to relative seismic displacements.

FPB description

The Friction Pendulum Bearing (FPB) is a propriety seismic isolation device that will be interposed between the topsides and GBS, reducing the seismic accelerations and hence shear forces transmitted to the deck structure and equipment. However, it introduces an additional design consideration for facilities such as conductors and risers that span across the FPBs due to the differential displacements that occur between the topsides and GBS.

The FPB comprises three cast steel components, namely the housing plate, the concave plate and the slider, as shown below in Figure 4

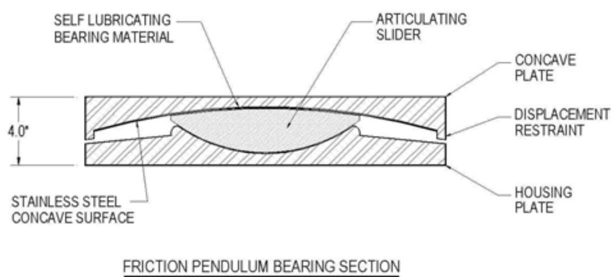


Figure 4. Schematic diagram of bearing components

The smaller radius, spherical surface of the slider is supported in the hemispherical socket of the housing plate, which is lined with a high load low friction composite bearing material or liner, creating an articulated joint. The other surface of the slider, which is also lined, slides along or across the spherical concave surface of the concave plate, which is covered by a polished stainless steel overlay, to produce the required low friction sliding.

It is this sliding motion that is in effect the pendulum motion and the radius of the concave surface that determines the natural period (T) of the bearing, as given below in Equation 1:-

$$T = 2\pi\sqrt{(R/g)} \qquad \text{Eq 1.}$$

where,
R – radius of curvature of concave plate, corresponding to pendulum of length R.
g – gravity.

It should be noted that the bearing period is independent of the mass of the supported structure, facilitating practical applications, such as selection of a desired period of oscillation, simply by varying the radius of curvature of the concave plate.

As the slider travels across the concave surface, causing the supported mass to rise, the gravitational force component parallel to the surface (i.e. the shear force) acts as the restoring force, and provides the stiffness of the FPB during sliding motion. The shear force mobilized at the bearings is given by Equation 2.

$$F_{sh} = \left(P/R\right)\delta + \mu.P\,\mathrm{sgn}(\dot{u}) \qquad \text{Eq. 2}$$

where,
μ - coefficient of friction
δ – relative horizontal displacement across the bearing
sgn(ů) – the sign of the velocity of slider
P – load normal to the bearing, comprised of the following, as given by Equation 3.

$$P = W\left(1 + \frac{\delta P}{W} + \frac{\ddot{u}}{g}\right) \qquad \text{Eq. 3}$$

where,
W – static bearing load
δP – additional bearing load due to spherical geometry of bearing surface
ü – vertical acceleration

The first term in Equation 2 is the isolator restoring force that determines the slope of the force displacement relationship during sliding. The second term is the friction force between the slider and the concave sliding surface. The coefficient of friction μ is a function of the sliding velocity ů and bearing pressure. The friction velocity relationship was determined by Constantinou et al. (1990) [1] and is given by Equation 4.

$$\mu = \mu_k + (\mu_s - \mu_k).e^{(-a.|\dot{u}|)} \qquad \text{Eq. 4}$$

where,
μ_K – kinematic coefficient of friction
μ_S – static coefficient of friction
a – a parameter that controls the variation of friction w.r.t velocity.

The friction increases swiftly from μ_S to μ_K at low velocities (approx 25mm/sec) and thereafter remains constant for higher velocities. The upper and lower bound friction values used in the Sakhalin analyses are presented in Table 3 and Figure 5, below.

	μ_S	μ_k	*a* (sec/m)
Lower Bound	.025	.040	150
Upper Bound	.075	.095	150

Table 3. Upper and Lower bound friction values

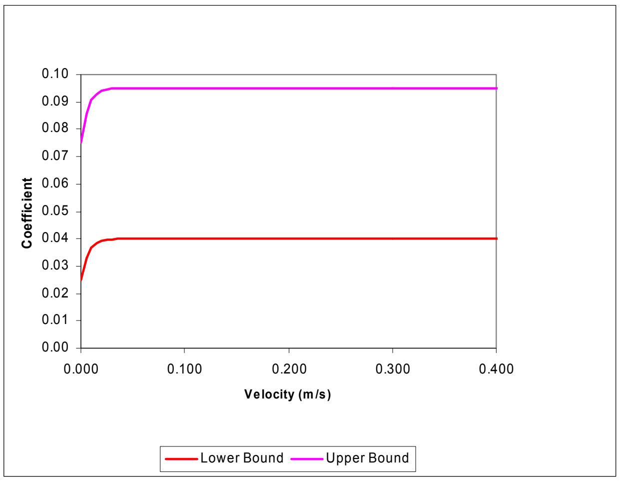


Figure 5. Upper and Lower bound friction values

These values were based on average 3-cycle friction of tested fresh bearings under normal temperature conditions in the range μ of 0.04 to 0.06 [2]. The bounding coefficients were determined by applying system property modification factors or λ-factors to the nominal values. The λ-factors take account of ageing – 30yrs, contamination, travel - 2.9km and extreme temperature –40°C. Velocity effects were captured explicitly in the seismic analyses. The factors used were as follows

- Ageing: sealed bearing in a severe environment λ_a = 1.2
- Contamination: sealed bearing installed with thestainless steel Overlay surface facing down, λ_c = 1.0
- Travel: cumulative travel estimated to be 2.9km under non-siesmic conditions, λ_{tr} = 1.1
- Temperature: extreme temperature of –40°C, λ_t = 1.4
- Adjustment factor: to take account of the likelihood of simultaneous occurrence of extreme events, λ_x = 0.75

The Sakhalin FPB centralized and at maximum capacity excursion of 700mm is shown in Figure 6 and Figure 7.

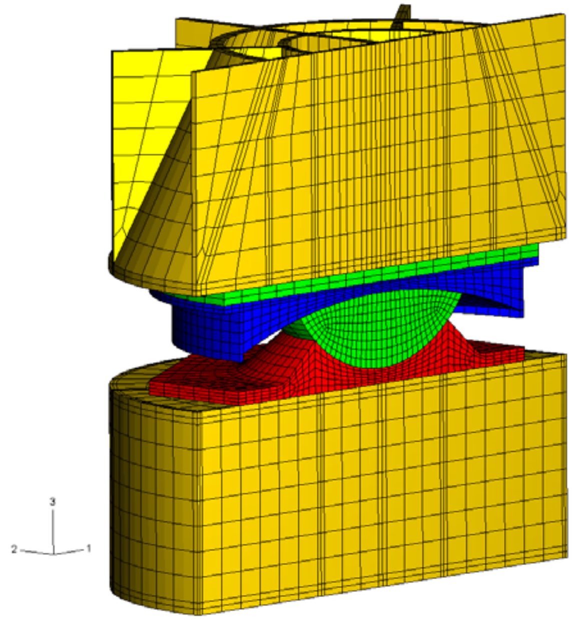


Figure 6 – **Centralised FPB**

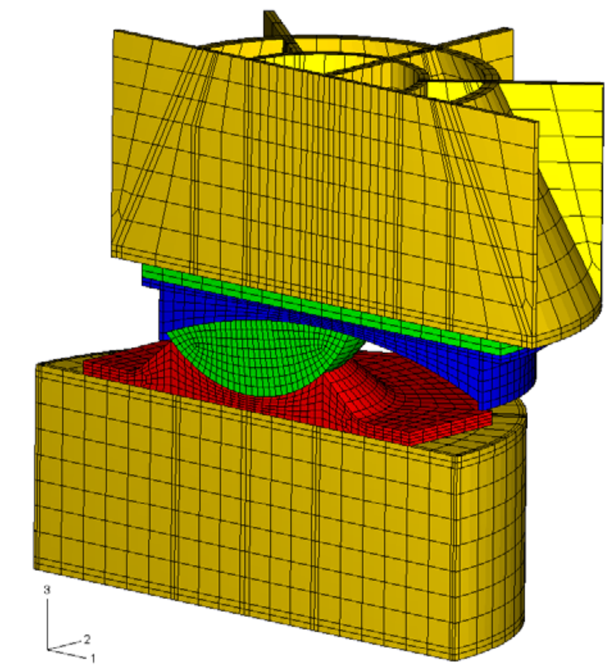


Figure 7 – **Offset FPB**

FPB Assurance

The FPBs play a key role for the Sakhalin II platforms enabling the GBS and Topsides to meet project survival criteria during seismic events. In order to provide assurance for the functionality of the FPBs during such events, a comprehensive testing schedule will be performed. Prototype full-size and reduced-size bearings and all production bearings will be tested.

Whilst the Sakhalin II FPBs will experience the largest imposed gravity loads of any seismic isolation bearing ever constructed and tested in the world, the FPB Slider Displacement, Slider Velocity, Generated Heat Flux and Average Duration of Heat Flux are within the bounds of previously produced FPBs.

The imposed gravity loads that the Sakhalin FPBs support, are too large to be tested to full-scale loads, displacements, and velocities in any test machine in the world today. To address the inability to perform full-scale dynamic testing, reduced-size prototype bearings, which are scaled versions of the production bearings, will be used. The scaling principles employed for the prototype bearings are as follows:

(a) maintain average bearing pressure - requires change in the contact area

(b) maintain the additional edge stress due to rotation of the articulated slider - requires change in the radius of the articulated slider

(c) maintain the liner thickness and manufacturing process - in order to preserve the wear characteristics

(d) maintain the thickness, manufacturing process and free length of the stainless steel overlay - in order to preserve the characteristics responsible for the overlay waviness and potential for rupture

(e) maintain the radius of curvature and displacement capacity of the bearing - in order to test under prototype conditions of amplitude of displacement

The testing protocols are based on the calculated dynamic response of the most onerous platform bearings. The duration and form of dynamic testing of the prototype bearings was selected so that the imposed motion was within the capabilities of available test machines and also was thermodynamically equivalent to the most demanding case established from the output of the advanced time history structural analyses of the global platform. The thermodynamic equivalence was based on maintaining the average temperature rise at the sliding interface.

Analysis Methodology

The introduction of seismic isolation in the form of FPBs precluded the use of response spectrum or pushover analyses for the global platform SLE and DLE, respectively and dictated that non-linear time-history analyses procedures be used for both.

The seismic analyses of the Sakhalin platforms were performed in the time domain using ABAQUS/Explicit [3]. The seismic model consisted of four key components, namely, a detailed model of the topsides, the FPBs, a beam element model of the concrete GBS and the foundation model. The FPBs and the foundations were key features in the seismic assessment and these are discussed in detail below.

Modelling and Benchmarking of FPB Analysis simulation

To ensure that the FPBs could be modelled in ABAQUS, a series of 'Benchmark' analyses were performed. The results proved that there are features available in ABAQUS that could be used to accurately model an FPB. Furthermore, the results obtained from ABAQUS were in good agreement with those obtained from hand calculations, other models and published data from physical tests.

The FPBs were modelled using a contact feature known as an analytical rigid surface that creates master/slave sliding contact between coincident nodes, as shown in Figure 8. This feature allowed each component of the FPB to be modelled. The concave sliding surface of the FPB was represented by the rigid surface itself, whilst the convex slider was defined as the slave. The articulation of the slider in the housing plate socket, in effect a pinned connection was defined implicitly, since the sliding surface feature activates only translational degrees of freedom at nodes connected to it.

The concave sliding surface is a smooth, continuous surface, which is important for contact since it avoids the inherent discontinuities that arise when using a surface defined with discrete facets and was modelled with a radius of 3.976m, giving a period of 4 seconds.

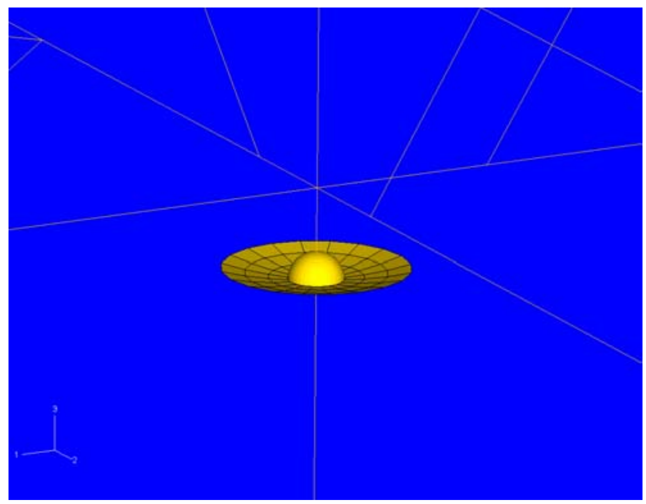


Figure 8. Diagrammatic view of FPB modeled in ABAQUS

The interaction between the slider and sliding surface was defined using the friction model described above, in which the coefficient of friction varied with respect to the velocity of the slider, in effect creating a continuously varying friction force once sliding had been initiated

The FPBs were benchmarked in two separate exercises. In the first of these, a simple model comprising 4 bearings connected by stiff beams supporting a point mass was created, as shown in Figure 9.

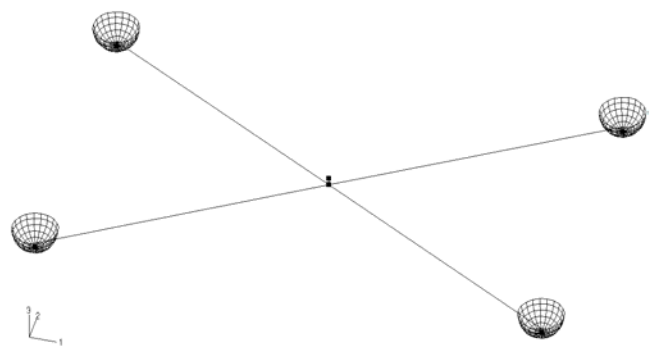


Figure 9. First FPB benchmarking model configuration

The purpose of this model was to prove that there were features available in ABAQUS that could be used to model an FPB, as ABAQUS had not previously been used to model FPBs
anywhere in the world. In this first exercise a static gravity load was applied, to the point mass to generate the initial static bearing load, which was then pushed horizontally using displacements or forces.

The contact surfaces used to model the FPBs produced output in the form of forces due to friction (for which the ABAQUS label is CFS), the restoring force (CFN), as well as the sum of the friction and restoring force (CFT). For each result type the vector components and the magnitude are calculated and identified by the label subscripts 1,2,3 and M, respectively. e.g. CFT1, CFT2, CFT3 are the vector components of the total force in the global x, y and z directions, whilst CFTM is the magnitude. CFT1 and CFT2 are in effect the shear forces across the bearing as defined in Equation 2, whilst CFTM, is the total bearing load, as defined in Equation 3. The results of these simple static analyses were compared to hand calculations, from which it was shown that for a given horizontal displacement the resulting shear force could be predicted using Equation 2, and vice-versa.

The results of this exercise demonstrated clearly that there were features in ABAQUS that could more than adequately represent the FPBs. Indeed the ABAQUS model of the FPBs is a particularly elegant solution as it replicates the physics of the FPB, unlike other packages that use a combination of springs and dampers. In the ABAQUS model the sliding surfaces of the FPB are modelled directly and the shear and bearing forces are generated by the motion of the slider moving across the sliding surface, exactly as happens in an actual bearing. In addition the ABAQUS model also allows velocity dependent friction to be defined, as described above, as well as retaining the correlation between bearing load and shear force due to the curvature of the sliding surfaces.

In the second part of the benchmarking exercise, results from shake table tests of a 7-storey seismically isolated steel frame building, performed by Al-Hussaini et al [4], at State University New York (SUNY) Buffalo and analyses of the same structure by Sheller & Constantinou, using SAP2000 [5] were compared to results obtained from ABAQUS. The results confirmed that the ABAQUS model of the FPBs works as required, when combined with the features found in a typical structural model.

The structure comprised two longitudinal frames of 3 bays by 7 storeys connected by horizontal and diagonal cross-bracing, as shown below in Figure 10.

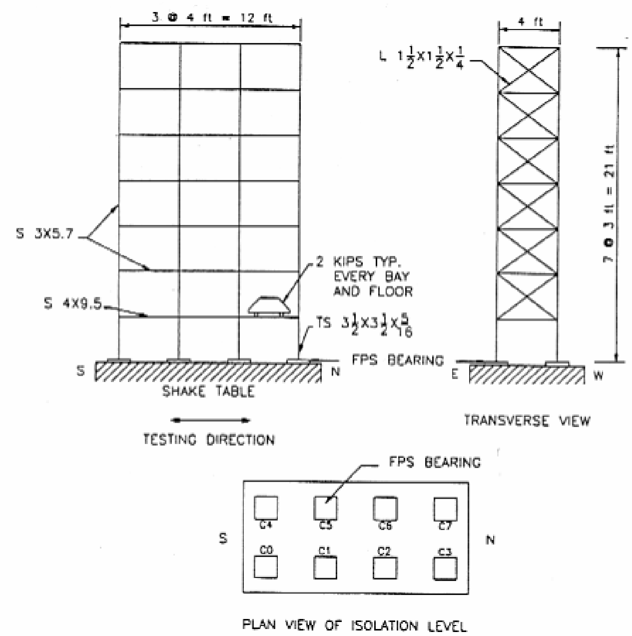


Figure 10. 7-storey seismically isolated steel frame

In the SAP2000 model, created by Sheller & Constantinou the building was modelled and analysed as a 2-D frame. This modelling approach was retained for the ABAQUS validation reported here and results are presented below. The input ground motion is shown in Figure 11.

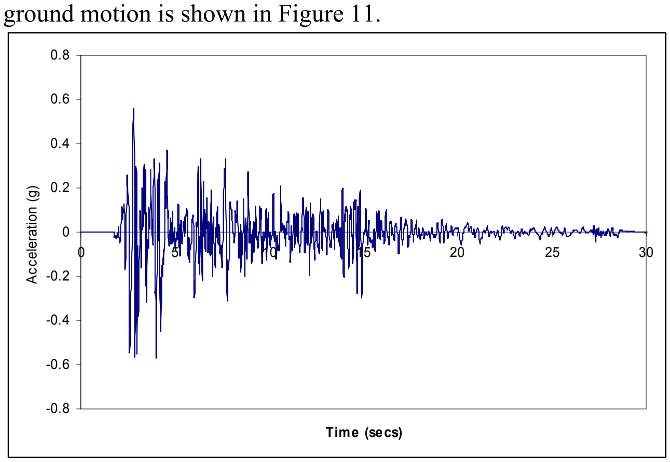


Figure 11. Input ground motion

The results obtained are summarized in Table 4, Figure 12 and

Figure 13 below.

From comparison of the FPB displacement and the shear/weight ratio it is apparent that the ABAQUS model of the FPB is capable of accurately replicating the behaviour of an FPB. From Figure 12 it can be seen that the displacement is matched not only in terms of peak response, but also throughout the simulation.

	Test	SAP	ABAQUS
Peak FPB Displacement	2.04”	1.94”	2.035”
Peak FPB Shear / Weight	0.25	0.26	0.244
Peak Roof Acceleration	0.60g	0.78g	0.57g
Peak Roof Displacement	N/A	2.61”	2.70”

Table 4 ABAQUS vs. Shake Table Comparison

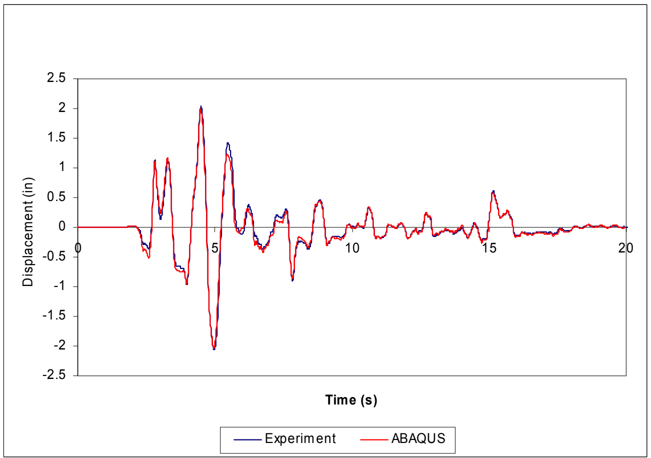


Figure 12. FPB Displacement

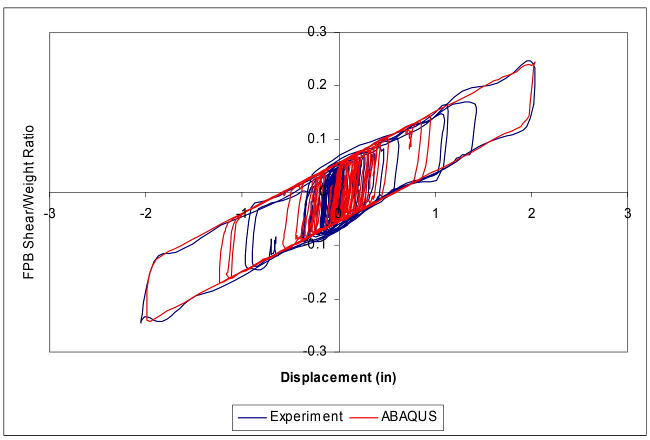


Figure 13. FPB Hysteresis

Figure 13 is typical of a FPB isolation system response plot from a shake table test and again a very good match from ABAQUS is achieved. This response plot represents the hysteretic, energy dissipating behaviour of an FPB during an earthquake. Sliding, which is analogous to yielding, occurs when the shear force exceeds the friction force. The steep part of the hysteresis curve represents the elastic, pre-yield displacement of the bearing, which for Sakhalin corresponds to the shear deformation of the composite liner. The slant part of the hysteresis loop represents the FPB stiffness during sliding. The energy dissipated by the FPBs during an earthquake is represented by the area enclosed by the hyseresis loops.

The ABAQUS model slightly underpredicts the peak roof acceleration, *with a corresponding over-prediction of the roof displacements*. These small differences were due to structural modeling differences in stiffness, initial weight distribution and damping models employed.

Overall it is apparent that good agreement was achieved, generating confidence that the ABAQUS model of the FPBs works correctly when incorporated in a ‘real’ structural model.

Foundation Model

Geotechnical analyses produced foundation soil stiffnesses that were non-linear with respect to frequency in vertical translation and rocking degrees of freedom (dofs) and non-linear with respect to displacement in horizontal translational dofs.

For the frequency dependent dofs, the soil properties varied over a frequency range 0 to 3.5Hz. Results of eigenvalue analyses showed that this range was sufficient to capture over 99% of the participating mass of the structure. To model this variation with respect to frequency in the time domain, the foundation spring was represented by a spring of stiffness K_O and a mass, m that was added to the GBS in the respective degree of freedom. The resulting effective stiffness, K, varies with frequency, ω, as a quadratic function as shown in Equation 5 below

$$K = K_0 - m\omega^2 \qquad \text{Eq. 5}$$

Typical stiffness obtained using this approach and it’s approximation to the required stiffness is shown in Figure 14.

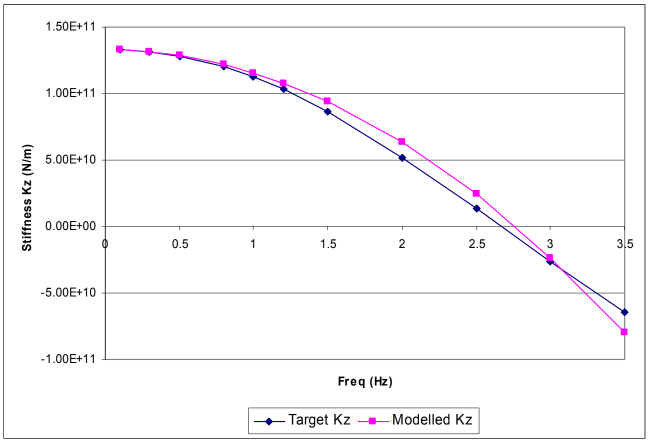


Figure 14. Foundation Stiffness Model - vertical

For the horizontal dofs the non-linear force-displacement relationship was defined as follows in Equation 6

$$F_x = \frac{K_{xx}\delta}{1 + \dfrac{\delta}{F_{\max}}\left(K_{xx} - F_{\max}/\delta_{\max}\right)} \qquad \text{Eq. 6}$$

where,

F_x - lateral force at base of GBS

δ – lateral displacement at base of GBS

K_{xx} - the initial lateral stiffness

F_{max} - the foundation sliding capacity

δ_{max} - the displacement at which foundation capacity is reached.

A typical foundation stiffness obtained is shown in Figure 15.

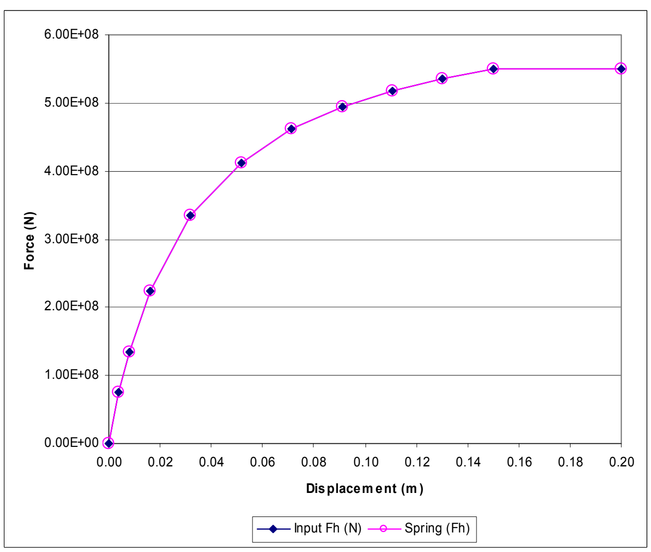


Figure 15. Sliding Foundation Stiffness

This sliding foundation was modelled using a system of 10 parallel elastic-perfectly plastic spring elements, based on a methodology originally proposed by Iwan [6]. To demonstrate the functionality of this model the saw-tooth force time-history shown in Figure 16 was applied to a simple model comprising only the sliding foundation and a dummy mass.

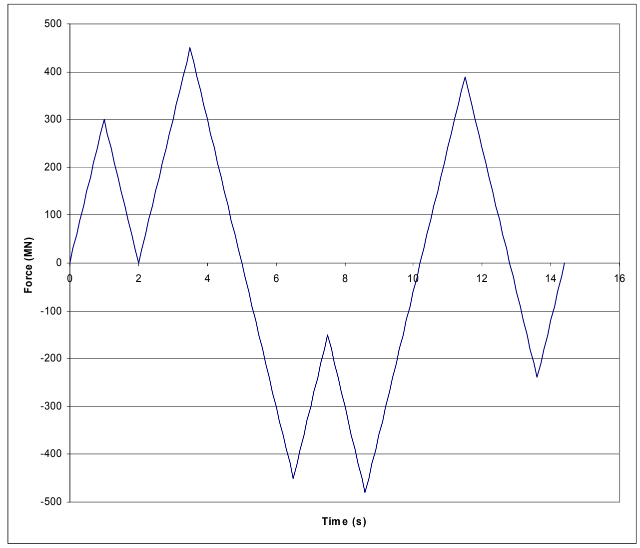


Figure 16. Saw tooth load-time history

The time-history was applied at varying angles wrt to global X and the resulting hysteresis loops shown in Figure 17 demonstrates that the foundation model performs consistently and as required in 3-D.

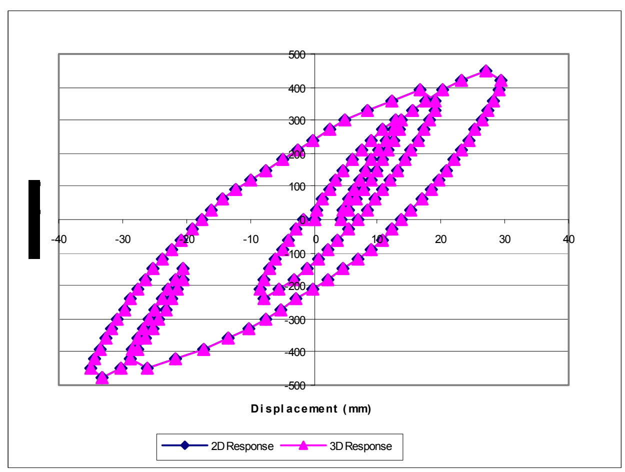


Figure 17. Foundation response Hysteresis

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Final Seismic Analysis Results

The seismic analyses were performed in two discrete steps. In the first step the functional (gravity) loads were applied, in order to generate the initial static bearing or axial load in the FPBs. In the second step, the platforms were subject to seismic excitation, applied to the foundations in the form of time-series' of ground accelerations.

The results from these analyses were used not only for design of primary steelwork, but also for seismic design of safety critical or dynamically sensitive equipment, as well as providing input to the technical basis for design and testing of both the prototype and production FPBs.

For the FPB design and testing, results of instantaneous variation in bearing pressure, slider velocity and orbit and duration of motion were extracted and used to determine liner wear, heat build up and energy dissipation, as previously described in 'FPB Assurance'. The results shown in Figure 18 to Figure 21, below are from the south-west FPB on the Lunskoye platform for SLE acceleration time-history set 3 with upper bound soil properties and an FPB coefficient of friction of 0.095; the upper bound friction used for FPB design.

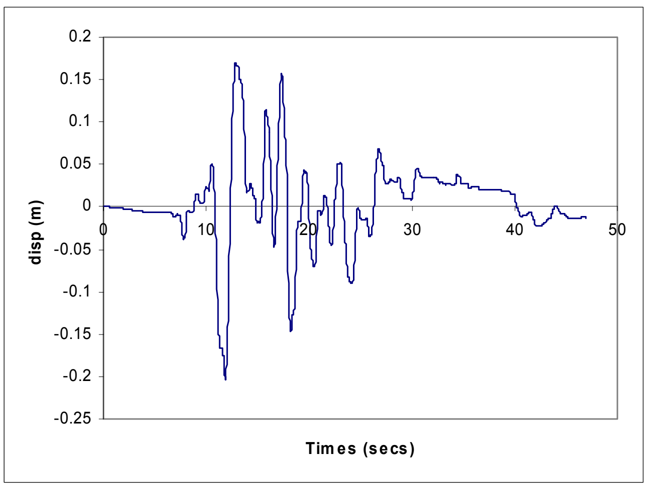

Figure 18. FPB Displacement

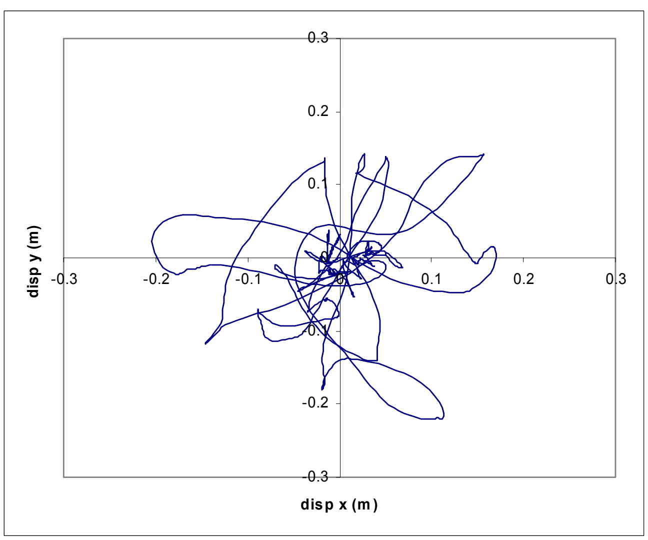

Figure 19. FPB Orbit

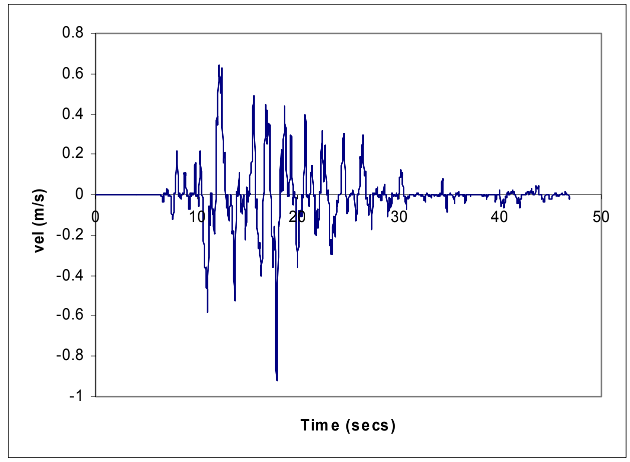

Figure 20. FPB Velocity

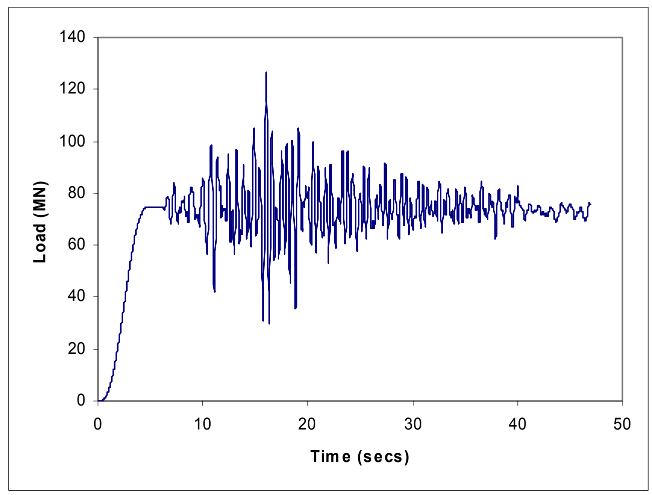

Figure 21. FPB Bearing load

The maximum FPB responses for both SLE and DLE events, extracted from the advanced non-linear dynamic analyses are presented in Table 5. The DLE UB and LB results were used for the FPB design, whilst the SLE and DLE_BE were used for structural design.

	SLE	DLE_BE	DLE_UB/LB
FPB δ (mm)	118	550	630
FPB Shear (MN)	13	22.3	24
Max FPB P (MN)	123	146	149
FPB Travel (m)	1.5	9.0	10.4
Max FPB vel (m/s)	0.45	1.24	1.48

Table 5 – FPB: Results

The maximum seismic shear forces across the FPBs are presented in Table 6 below. The maximum deck shear force during SLE drops from 195MN to 29MN, clearly demonstrating the effectiveness of the FPB isolation.

	With FPB (MN)
SLE	29
DLE_BE	65
DLE_UB	45

Table 6 – Seismic Shear Loads with FPBs

The maximum average horizontal accelerations for Lun-A SLE are presented in Table 7. Comparison with corresponding accelerations without FPBs as presented in Table 2, demonstrates the effectiveness of the FPB isolation; the maximum average deck acceleration dropping from 0.73g to 0.24g.

	With FPB
Deck Overall	.24
Deck El(+)27m	.31
Deck El(+)38m	.25
Deck El(+)47m	.31
Flare	2.0
DES	.61
N Crane	.82
S Crane	1.46

Table 7 – Lun-A SLE: Max Average Accelerations

For the SLE design of the primary structure, maximum average SLE accelerations were extracted from the advanced dynamic structural analyses. Accelerations occurring in discrete structures such as the flare and living quarters were averaged separately so that a true reflection of the accelerations experienced by those structures was obtained. These maximum average SLE accelerations were applied as quasi-static accelerations to the primary static structural design model and combined with appropriate functional load cases in accordance with Table 8. Conventional linear elastic analyses and code checks were performed using these load combinations.

For the SLE design of secondary structures, facilities and vendor equipment, the same principals were applied of extracting and applying the quasi-static maximum average SLE accelerations. However the averaging was performed over smaller zones in order to correctly identify localised effects.

Dynamically sensitive structures, such as the flare boom, helideck support structure and vent towers were designed using a response spectrum approach, as an SLE averaging method would have been inappropriate. SLE acceleration time-histories were extracted at the deck supports of each structure and transformed into the frequency domain using a Duhamel integral solution technique. Spectra obtained for the flare are shown in Figure 22.

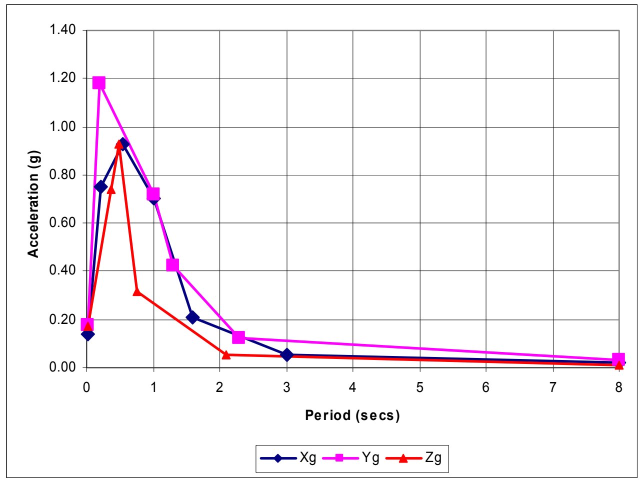

Figure 22. Lun-A Flare Design Spectra

For the DLE design, the structural integrity assessment of the primary structure, was based on checking the strains, and in particular the plastic strains that developed during the time-domain DLE structural analyses. The object was to ensure that the platform survival criteria, defined previously, were met. In reality, the plastic strains experienced were negligible, a tribute to the efficacy of the FPBs.

For the DLE design of selected secondary structures and equipment, such as lifeboats and their structural supports quasi-static DLE accelerations were extracted from the time-domain dynamic DLE structural analyses at the relevant location. Conventional linear elastic analyses and code checks were performed using these accelerations.

Dynamically sensitive secondary structures, such as external escape cantilever walkways to TR, for which a DLE averaging method would have been inappropriate or that were not accurately represented in the global dynamic analysis model were designed using a response spectrum approach, based on spectra extracted from the DLE time-domain analyses and transformed into the frequency domain as described previously.

Result of using FPBs

The accelerations and hence loads experienced by the topsides were reduced to reasonable levels such that the governing effect of SLE seismic design was brought into line with other non-seismic loadcases. This resulted in an economic and safe structural design.

Structural Design. The following section defines how the basic load cases are combined for the Sakhalin II platforms. In addition the definition of associated loads are made.

Load Combinations. Each topsides member and connection was assessed for internal forces arising from combinations as defined in Table 6, Section 6.5.7 of API RP2N [7]. However, as a product of a rigorous design assessment, several additions and a modification were made to Table 6 of API RP2N [7] and these are highlighted below in Table 8 in *italics*.

Table 8**. Load Combinations and Load Factors**

		Case					
		1	2	3	4	5	6
	Primary Load	**Operating Wave**	**Operating Ice** ***1-yr return***	**Extreme Wave Wind Current**	**Design Ice Frequent Events** ***100-yr return***	**Design Ice Infrequent Events** ***10,000-yr return***	**Earthquake** ***SLE***
Gravity Dead	D_1	1.3	1.3	1.1 0.9	1.1 0.9	1.1 0.9	1.1 0.9
Gravity Dead	D_2	1.3	1.3	1.1 1.9	1.1 1.9	1.1 0.9	1.1 0.9
Gravity Live	L_1	1.5	1.5	1.1 0.8	1.1 0.8	1.1 0.8	1.1 0.8
Gravity Live	L_2	1.5	1.5	-	1.0 [a]	1.0[a]	1.0[a]
Inertial Load	D_n	1.5	1.5	1.7	1.7	1.7	-
Operating Wind, Wave, Current	W_O	1.2[b]	-	-	-	-	-
Extreme Wind, Wave, Current	W_E	-	-	1.35[b]	-	-	-
Operating Ice	I_{OP}	-	1.2[b]	-	-	-	-
Design Ice (Frequent Events)	I_F	-	-	-	1.35[b]	-	-
Design Ice (Infrequent Events)	I_I	-	-	-	-	1.0[b]	-
Earthquake	E	-	-	-	-	-	*1.0*
Associated Wave, Current	*AWo*						*1.0 [c]*
Associated Sea Ice	*AI*						*1.0 [c]*
Associated thermal Expansion/ Contraction temperature	*AT*						*1.0*
Extreme Expansion/. Contraction temperature	*T*		*1.2*		*1.2*	*1.0*	

[a] Gravity live load L_2 for combinations 4, 5 and 6 is the load due to the superstructure icing and snow accumulation, corresponding to 1-year return period.

[b] Associated loads, where appropriate, are included with the primary loads.

[c] *The 'Associated Wave, Current' load case and the 'Associated Ice' load case are not for simultaneous application.*

Modifications

In order to satisfy Russia design code requirements the load factor for SLE seismic from Table 6 of API RP2N [7] was increased from 0.9 to 1.0.

Additions

Associated loads with seismic events and extreme thermal expansion/contraction load cases were included as additions to Table 6 of API RP2N [7].

Associated Loads

It was reasonable to include extra basic load cases that can be associated with seismic loads. Since wave, sea ice and temperature events are persistent in nature; there is a high probability that there will be a level of environmental loading that occurs simultaneously with a seismic event. Both seismic and environmental conditions may be defined in statistical terms i.e. the magnitude of forces exerted on the platform may be reported in terms of a return period. It is clear that earthquake loads and environmental loads are statistically independent. The following lists load cases, which may be defined as associated loads with the seismic event; the concurrent probabilities are discussed below.

(a) Wave, Current
(b) Sea Ice
(c) Temperature

Seismic events are by nature, not persistent, and the reverse consideration is not necessary.

Associated loads were not derived between load cases that have some statistical dependence e.g. Ice and Temperature.

The use of FPBs for the Sakhalin platforms meant that the effect of associated loads manifested themselves predominantly as associated FPB displacements in addition to seismic displacements. However, it is important that these additional displacements are included in:

The SLE design of the services (Risers, conductors, caissons) spanning the gap between the topsides and GBS.

The DLE design of safety critical services (Risers) and that the displacement capacity of the bearing was adequate.

It should be noted that for platform solutions with topside to GBS connections designed as constrained, then internal forces arising from these additional associated load cases need to be considered. For constrained configurations, the effects of associated loads on the GBS have significant influence on topsides steel internal forces.

The selection of the magnitude of associated (a) Wave/Current, (b) Sea Ice and (c) thermal expansion/ contraction loads, that were considered in conjunction with SLE and DLE earthquake events is discussed below. As stated in Note [c] above, it was not deemed appropriate to take loads (a) and (b) simultaneously.

The return period or exceedance level of the associated load (a), (b) and (c) were selected on the basis that when it is

combined with an SLE event, the combined probability does not reclassify this event as an Abnormal event.
Likewise for the selection of associated loads for combination with a DLE event, the combined probability should not reclassify this event as beyond an Abnormal event.
The method for deriving the resulting associated (a) Wave/Current, (b) Sea ice displacements with earthquake events were performed to the same principles explained below for the derivation of (c) associated temperatures for thermal expansion and contraction calculations.

Thermal Expansion and Contraction displacements
To derive the thermally induced topsides expansion/contraction values, the extreme and associated temperatures were used to obtain a temperature change against a conservative as-installed temperature of +15ºC. By inspection, the thermal contraction case (i.e. lowest temperature) will govern to maximise potential off-center movement of the FPBs.

The derivation of extreme and associated Temperature for thermal contraction with Operational, Extreme and Abnormal events are defined below. For the structural platform solution adopted for Sakhalin II, contraction of the topsides causes the FPBs to move off center and hence reduce the available displacement capacity and increase the design displacement of the GBS to topsides spanning services.

There were three relevant thermal contraction categories to consider:
1. Low temperature together with **operational** loads
2.Low temperature together with **extreme** loads (200-yr SLE earthquake)
3. Low temperature together with **Abnormal** loads (3000-year DLE earthquake).
For category 1 the temperature would typically be the lowest temperature over service life, including a factor of safety.
For category 2, for the above SLE return period, the temperature would typically be [mean temp. - 1.5*sigma] if aiming for a 6.7% exceedance level.
For category 3 the temperature would typically be the [mean temp. – 0.536*sigma] if aiming for a 30% exceedance level.

This solution is further developed:-

Case 1: Operational Case
The temperature here would be the minimum temperature with a return period of 100 years= -36ºC

Case 2: Extreme Case
For the SLE event if we use a temperature $T_{6.7\%}$, which is exceeded on the low side only about 6.7% of the time, it means that the combined event of getting SLE event and a temperature lower than $T_{6.7\%}$ has a probability of occurrence

$P\ (SLE\ \&\ T_{6.7\%}) = P(SLE) \cap P\ (T_{6.7\%}) = 1/200 * 0.067 = 1/3000$ /yr

This should not go lower than 1/3000 /yr for the SLE check, because then it can no longer be deemed to be classed as an extreme event but it falls in the category of an abnormal event. The temperature was obtained from Figure 23, a non-exceedance plot for the region area temperature.
The non-exceedance SLE temperature was $T_{6.7\%}$ = -19ºC.

Case 3: Abnormal Event
From reliability analyses on this issue we would conclude that we need to use a value somewhat below the mean, typically 0.536 sigma below the mean is sufficient. In exceedance terms this corresponds to a non-exceedance level of 30%. For the DLE event, if we use a temperature $T_{30\%}$, which is exceeded on the low side only about 30% of the time, it means that the combined event of getting DLE event and a temperature lower than $T_{30\%}$ has a probability of occurrence

$P\ (DLE\ \&\ T_{30\%}) = P(DLE) \cap P\ (T_{30\%}) = 1/3000 * 0.30 = 1/10{,}000$ /yr

With reference to Figure 23, the non-exceedance DLE temperature was $T_{30\%}$ = -9ºC.

The estimated maximum associated thermal contraction at a support, assuming one of the other FPBs does not slide is:
Operating condition = 52mm
Extreme seismic = 35mm
Abnormal seismic = 25mm

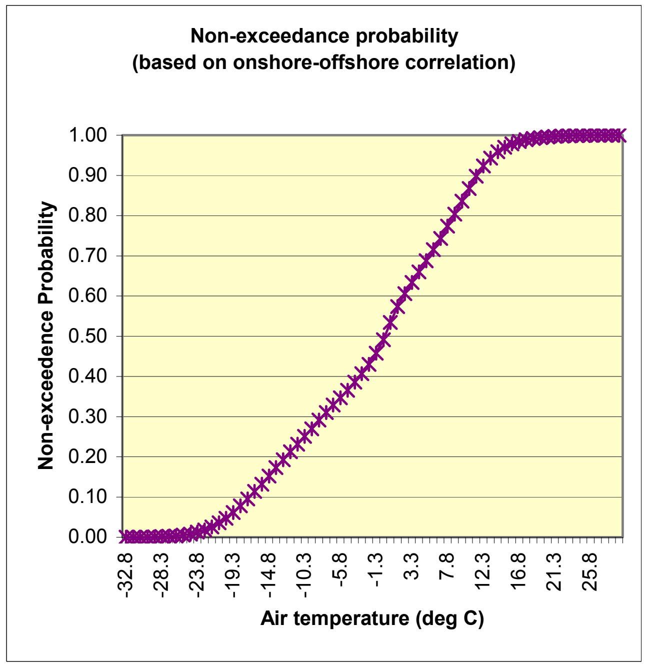

Figure 23 Air temperature non-exceedance probability curve

The same calculation method can be used for for deriving associated Wave/Current and Sea ice displacements with seismic events.

Brittle Fracture
Fracture Mechanics assessments are performed to ensure that there will not be any Brittle fractures.
For Operating conditions, the 100-yr low temperature is appropriate for performing fracture mechanics analyses. However, for seismic events, toughness values at temperatures associated with SLE and DLE are used in fracture mechanic

analyses.
To achieve a fracture resistant structure, enhanced low temperature properties are specified in the procurement of the structural steel.

Temporary Condition – Initial Evaluations, Construction and Loadout, Transportation, Float-over, Set-down, FPB Activation, Design Process Summary

Overview

The remote, exposed locations of the LUN-A and PA-B Fields, coupled with the arctic conditions, have significant influence on the installation designs and methodologies for the Topsides structures. This section will present the temporary condition difficulties recognised during the FEED phase of the Project and the solutions chosen. The impact of these design solutions on the overall Topsides designs will be shown.

Initial Studies and Selection of Installation Method

At commencement of the FEED Phase of the Project, a number of Topside Platform installation options were reviewed i.e.:

- Multi-modular Topside concept comprising a large number of modules, weighing less than 2,000tonnes each. Topsides to be installed onto a Gravity Base Structure (GBS), either in-shore at a near-shore construction site adjacent to the Topsides construction yard, or offshore at the Field location. Lift vessels to be locally (Far East) available,
- Large modular Topsides concept comprising three to four modules weighing less than 10,000tonnes each. Topsides to be installed onto a GBS, either in-shore or at Field. Semi-submersible Heavy Lift Vessel required (Thialf or Saipem 7000) with associated availability issues, particularly with the remote at-field lift option,
- Fully integrated Topsides float-over concept, with installation weights of 21,000tonnes and 27,500tonnes for LUN-A and PA-B Topsides respectively. Float-over options are:
    - In-shore mating of Topsides with floating GBS at near-shore mating site, adjacent to the Topsides construction yard,
    - Offshore mating of Topsides with fixed GBS, previously installed at Field location,

The Single Topsides float-over solution was the only option that reliably satisfied the Field delivery schedule. The hook-up and commissioning content required for the large module and especially the multi-module alternatives would have extended the key project delivery dates by unacceptable amounts for both the in-shore mated option and especially for the at-field mated option. The remote locations of the PA-B and LUN-A Fields, the lack of effective support infrastructure in the area and the constraints of working in a harsh environment for nine months of the year, including pack-ice for six months of the year, make minimising the amount of in-field completion work a Project priority.

During FEED, both in-shore and at-field integrated Topsides Float-over options were studied, the selected solution being Topsides mating at Field location over a multi-legged concrete GBS. In-shore mating of the Topsides with concrete GBS proved to be impracticable for the Sakhalin II platform solution due to stability issues associated with setting down the global platforms in the 'shallow' water at the Field locations.

Topsides Tow Route Options and Transportation Sea state

The FEED phase marine work focused on establishing a Transport and Installation solution that satisfied SEIC's commercial options. Potential Topsides construction yards were identified. A number of tow route options were addressed, including Sea of Japan versus Pacific coast. Tow durations, based on a 4-knot tow speed and no contingency, ranged from 14 days (from Korea) to 42 days (from Singapore). Safe haven studies along the tow routes were also performed. These showed the hazard associated with finding in-shore protection from offshore storms was more onerous than weathering storms in deep water. Consequently, the tow design sea state was based on a months exposure during the summer tow period of May to August, resulting in a design sea state of H_s = 6m.

The Topsides installation weather window is relatively short, the severe climate means there is the potential for pack-ice to remain at the platform locations to as late as June and stormy weather commences early September. There is the further influence of fog, which restricts operations on still summer days. To mitigate the risk of installation postponement, the weather window was prolonged by two weeks to mid-September. The design transportation sea state was increased accordingly to Hs = 7m.

Barge Size and Form Selection

A number of single and twin barge options were reviewed. The exposed Field location and long tow strongly favoured a single barge Topsides float-over option; a twin-hull tow being impractical and an offshore barge Topsides transfer at the exposed Field location considered an undesirable risk. Previous concept studies had also favoured multi-leg platform structures to a single tower option on in-place considerations, an arrangement more compatible with the single barge float-over. An offshore jacking option was reviewed; the Topsides being floated over the GBS at 'Low-level' and then the topsides jacked to permanent elevation after barge removal. The consequent facility modifications, Topsides weight increases and the associated increase in at-field structural and mechanical completion were considered unfavourable and the 'High-level' single barge float-over option selected.

For the FEED design basis described above, a suitable transport barge was selected. The principal design influences on the barge requirements are described below:

- Topside weights, which are directly affected by the span length between the support points i.e. the GBS leg locations. Heavier barge payload, requires larger barge,

- Barge to be suitable for LUN-A and PA-B Topsides, which are planned to be installed in consecutive years,
- Loadout Support and Transport Frame (LSF) height, which is determined by a compromise of balancing wave slam loads to the Topsides with the detrimental effect of raising the transport height of the Topsides, and its adverse influence on barge stability and deck accelerations. Refer to section 'Permanent Condition',
- LSF weight, increased payload,
- Field environment, tides, and ballasting capacity of the barge to ensure successful float-over and barge withdrawal,
- The Topsides is transversely located on the barge, for GBS mating requirements, resulting in a significant concentration of load at the barge mid-span,
- Shape of the GBS the barge is to engage with, a principal consideration if barge sponsons are to be considered,

The resulting barge requirements of stability, ballast capacity and motions control are achieved by the following:

- Barge width selection; constrained by the GBS leg spacing and gaps required for barge fendering systems, barge width provides the most significant contribution to stability,
- Barge depth selection; constrained by Construction yard loadout quay heights and height of the GBS base slab. Increased barge depth improves barge stiffness and reduces influence of barge flex effects on the LSF and Topsides structure. The depth has a secondary influence on stability but a fundamental influence on the barge payload capacity and ballast capacity for Topsides unloading,
- Barge length selection; second order influence on stability, though provides additional payload and ballasting capacity for mating. Inefficient way of adding capacity due to detrimental effect of barge bending on the barge structure and cargo. A significant driver where sponsons are used as the Topsides location on the barge is constrained by the sponson interference with the GBS, determining the Topsides location on the barge,

The selected barge shape that meets the requirements discussed above has further influence on the Topsides, resulting in an iterative approach to determine the optimum barge form:

- Resulting transport motions of the barge system, generating the design accelerations on the Topsides and consequent forces in the Topsides and LSF. This has a significant effect on structure and facilities design,
- Barge system inertia and mating motions which result in mating impact loads between the Topsides and GBS during set-down,

The optimum barge solution selected was a new-build 'T' shaped barge, 185m length, 43.5m breadth and depth of 12.5m. Two 50m long by 20m wide sponsons are provided to achieve adequate barge stability. Approx. lightship weight = 21,000tonnes.

The FEED Topsides transport arrangement of the PA-B platform is presented in Figure 24 below, the LUN-A Topsides transport arrangement is similar: -

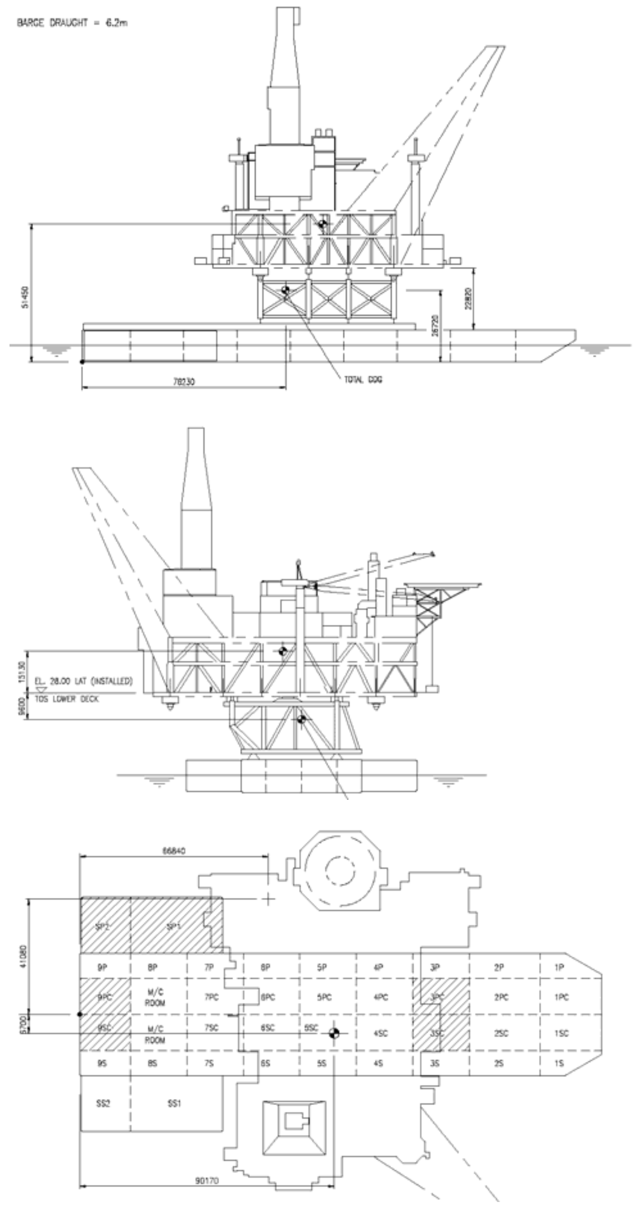

Figure 24 PA-B Topsides Transport Arrangement

Construction and Loadout

Construction and Loadout issues are not significantly different from industry norms for the Sakhalin II Field structures. For the purpose of this paper, it is sufficient to present significant consequential effects of the concept Topsides solution below: -

- The Field mating option and high LSF requires the structure to be loaded out at high level, requiring the

Topsides to be either built at high level or lifted ('jack &packed') to high level during the construction process,

- The single barge float-over concept requires the Topsides to be transversely located on the barge, with associated large cantilevers supporting the Drilling Equipment Set (DES), Flare and Living Quarters (LQ). There is significant global relative hog and sag flexure of the Topsides structure during change between the temporary and permanent conditions, which has to be catered for in the facilities design (e.g. significantly cladding, piping, HVAC),
- The transverse Topsides and LSF configuration results in a relatively short length for load distribution to the construction yard skid beams, resulting in very high load transfer to the support foundation and quay wall,
- Barge height has fundamental effect on yard skid-way height and barge skid-beam height, the combination of which need to tolerate required barge draught for load transfer, tidal variation and water depth at quay. The barge sponsons significantly assist transfer of loadout weight from the quayside to the barge, providing massive buoyancy at a critical location. This also reduces hull-bending moment during loadout when the Topside is moved across the quay to barge interface.

Transportation

After loadout, the barge is ballasted/deballasted to its transport draught, determined by stability and motions analyses, to minimise accelerations on the Topsides structures. Seafastenings are then attached between the LSF and the barge. Transportation design methodology is conventional; design transport accelerations determined from the FEED motions analysis are presented below:-

Linear accelerations at the Topsides centroids are: -

LUN-A Lateral = ±0.59g
LUN-A Vertical = ±0.19g
PA-B Lateral = ±0.47g
PA-B Vertical = ±0.20g

These are significantly amplified by roll, pitch and yaw motions at the peripheries of the Topsides. Accelerations at a remote point on the main structure are: -

LUN-A Lateral = ±0.75g
LUN-A Vertical = ±0.58g
PA-B Lateral = ±0.61g
PA-B Vertical = ±0.43g

The high transport accelerations provide a significant design consideration for both the structure and facilities, particularly as the incorporation of seismic isolation reduces the influence of earthquake accelerations in the design. In addition to the high accelerations, the concept solution selected has a more onerous impact on the Topsides and LSF than a conventional lifted installation. These issues are briefly discussed below: -

- The transverse structure centrally loads the barge, the resulting barge curvature has a significant influence on the LSF and Topsides structural designs,
- The barge sponsons provide significant buoyancy during the quartering and beam sea cases resulting in significant barge twist, again adversely influencing the LSF and Topsides designs,
- The transport support points for the Topsides are remote from the in-place supports, leading to a significantly different load distribution within the Topsides structures. Coupled with the high float-over requirement (accelerations presented above), the Transport design case has a significant effect on overall structure design,
- The long tow, coupled with the above, means that structure fatigue during transportation is not insignificant, and must be considered,
- The risk of installation postponement, due to adverse summer weather conditions, needs to be addressed, as tow design sea state may have to be increased to cater for a tow to a wintering site and return to Field the following year. The fatigue exposure is lengthened accordingly.

Float-over Considerations and Weather Window Selection

The Topsides to GBS mating process is the most sensitive part of the Transport and Installation operation because of the restrictive weather window, discussed previously, and the design considerations during the operation for both impact between the Topsides and the GBS during set-down and re-impact between the Topsides and the LSF during barge separation. There is a balance between limiting the installation forces on the Topsides and GBS to sustainable levels and accepting the pertinent installation sea state that has an acceptable probability of not being exceeded for long periods during the June to September weather window.

The following describes the critical considerations for optimising the Topsides to GBS mating operation: -

- Barge depth/ballasting capacity to provide sufficient float-over clearance with predicted motions (based on selected installation sea state) and to provide sufficient operational freeboard on withdrawal of the barge,
- Barge ballasting rate to be sufficiently rapid to minimise risk of significant LSF/Topsides re-impact forces after initial Topsides set-down on the GBS,
- Topsides/LSF nominal clearance at withdrawal to be sufficient to preclude LSF/Topsides re-impact during barge removal,
- Environmental restrictions to successful completion of the mating operation: -
  - Probability of exceedance of selected operational sea state during the mating procedure,
  - Sea ice restrictions on the weather window during early summer,
  - Operational restrictions due to fog; adversely influences suitable weather opportunities with calm conditions,
  - Tidal cycles, indicative maximums: -

    PA-B = ±1.30m
    LUN-A = ±0.90m

- o The influence of environmental conditions on the installation risk will be mitigated by targeting the design of the system as tidally independent, (feed back to barge depth).
- Provision of energy absorption, anti-shock elastomer devices to mitigate the mating impact forces on the Topsides. Leg Mating Units (LMUs) are used at the interface between Topsides and GBS and Deck Separation Units (DSUs), used at re-impact points between the Topsides and the LSF.

The duration of the critical operation, from commencement of seafastening cutting to barge withdrawal, was estimated to be 18 hours + 6 hours contingency; a total of 24 hours. Directional weather statistics were addressed, based on the probability of a 48-hour period in which the operational sea state would not be exceeded. Using iterative time domain hydrodynamic analyses to establish impact loads and structural analysis to determine the influence of these impact loads on the Topsides structures, an installation/mating solution was developed for the FEED phase of the Project that had an acceptable probability of occurrence during the weather window, without unduly impacting the Topsides designs. The limiting FEED operational sea state and impact force thresholds were selected as follows: -

Head/Stern sea	Hs = 1.0m	Tp = 8 secs
Quartering sea	Hs = 0.75m	Tp = 8 secs
Beam sea	Hs = 0.5m	Tp = 8 secs

Using DNV recommendations, the operational thresholds are converted to the following installation design sea states: -

Head/Stern sea	Hs = 1.6m	Tp = 4 to 8 secs
Quartering sea	Hs = 1.2m	Tp = 4 to 8 secs
Beam sea	Hs = 0.8m	Tp = 4 to 8 secs

The resulting installation forces are constraints for both the Topsides and the GBS: -
Maximum Docking Lateral Impact Load = 3,500tonnes.
Concurrent Maximum Docking Vertical Impact Load = 3,200tonnes,

Once the Topsides is at Field location, future weather conditions will be projected using weather forecasting and sea state monitoring, backed up by a system of wave rider buoys. When a window of at least 48 hours of acceptable weather is predicted, the seafastenings between Topsides and LSF will be cut and the barge floated between the GBS legs. Float-over is shown pictorially in Figure 25.

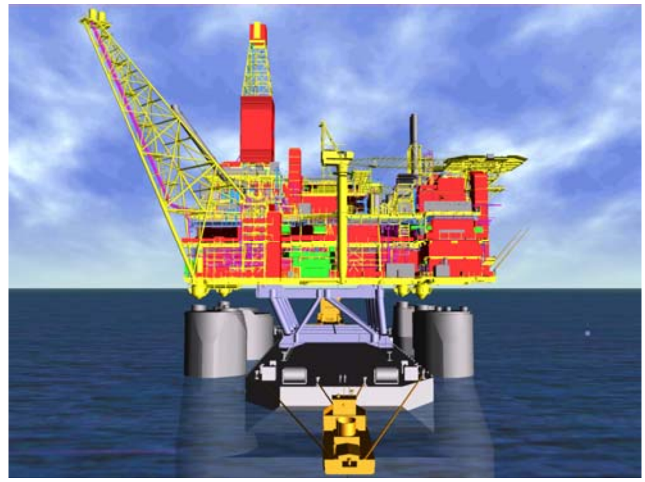

Figure 25 Representation of Float-over Condition

Set-down
The FEED design proposal for mating is to use a rapid ballasting system to minimise risk of increases in sea state during the mating operation. Each of the four Topsides support nodes has a primary docking cone which stabs into a receiver supported on the GBS leg top. The receiver is supported vertically on a series of stiff elastomers that mitigate the impact of landing the Topsides directly to the GBS structure. The receptacle structure is laterally supported by a system of lateral elastomers, designed to centralise and align the Topsides with the GBS and limit lateral impact loads between the Topsides structures and the GBS as described above. Vertical and lateral stiffnesses of the LMU interface units are consistent with stiffnesses modelled in the mating analyses.
The design requirements and responses of the LMUs are benchmarked against the mating analysis; design constraints are presented below: -

- LMU to fit within 5.0 m leg casing,
- LMU maximum length to be < 12.0m to suit GBS arrangement,
- LMU lateral stiffness at impact point = 100 MN/m, with controlling force of 3,500tonnes,
- Lateral energy absorption = 6MJ,
- Vertical static load = 10,000 tonnes, elastomer compression = 1.0m,

The FEED LMU proposed mating device is presented in Figure 26. The figure shows the LMU after completion of the set-down operation and release of the sand within the sand chamber.

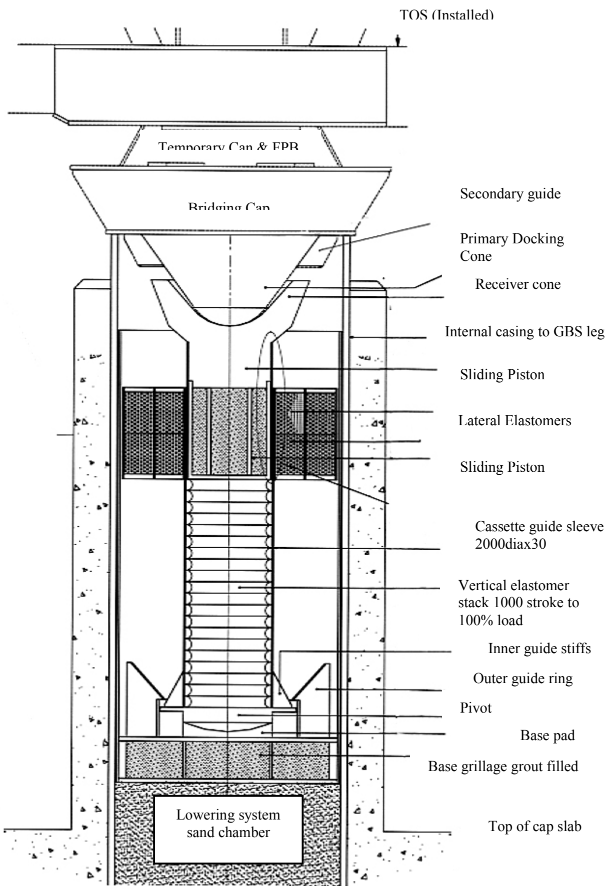

Figure 26 Representation of Float-over Condition

After initial engagement, the barge is continually ballasted until the Topsides is supported entirely on the LMU elastomer columns. During the Topsides/Barge separation operation elastomer Deck Separation Units become active to reduce re-impact loads between the Topsides and the LSF. These are located at the eight support points on the LSF.

The FEED design proposal is for the barge to be continually ballasted to provide 1.5m nominal clearance between the LSF and the Topsides structure for withdrawal. The ballasting capacity of the barge is based on the following: -

- Float-over clearance of 1.0m for Topsides motions
- Receptacle engagement length of 1.0m
- Elastomer compression length of 1.0m
- Topsides/LSF clearance of 1.5m post ballast
- Minimum barge freeboard at withdrawal 1.0m
- Minimum freeboard at float-over = 5.5m

Based on the above, the FEED float-over and set-down operation is tidally independent for LUN-A but not for PA-B.

After barge withdrawal, the FEED design proposed that the GBS leg support casings would be trimmed to match as-installed shape of the Topsides and the sand released from the sand-jacks in a controlled manner (to minimise racking of the deck) to lower the deck to final elevation. Racking loads are not significant due to the global flexibility of the decks across the GBS support points. Sand chamber depth is 3.3m, allowing: -

- +0.8m for GBS levelling for construction tolerances, seabed depth tolerance and GBS sea-bed slope allowance of 0.5°
- +1.0m for elastomer compression
- +0.5m for contingency on elastomer compression
- +1.0m sand flow facility

The riser leg has a circumferential secondary guide, providing nominal annular gap of 25mm, achieving the close tolerance necessary for riser Hook-up and integrity; directly opposite, the secondary guide provides location radially to fix the Topsides on the GBS, no secondary guidance is provided on the remaining legs. Set-down condition is presented pictorially in Figure 27.

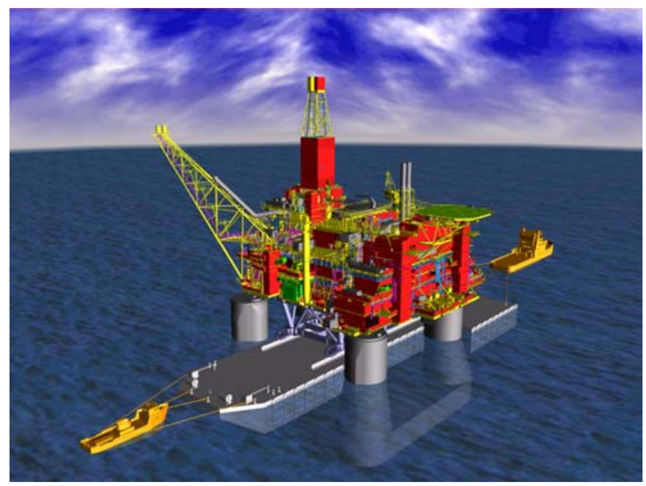

Figure 27 Representation of Set-down Condition

The GBS leg support casings are welded to the underside of the bridging caps.

FPB Activation

After set-down and weld-out of the permanent Topside to GBS support detail, the temporary conical installation can, used to transmit docking forces and protect the FPB during the temporary phases has to be removed. The FEED design proposes ten 1,030 tonne hydraulic jacks be energised between the bridging cap and Topsides to support the leg load, and locked-off. The cone is then removed piece-small and the deck lowered to rest on the FPB. Each FPB is activated in turn, the sequence managed to ensure platform integrity, minimise lateral forces to the jacks and minimise racking loads on the maximum loaded legs. Jack locations are presented in Figure 28.

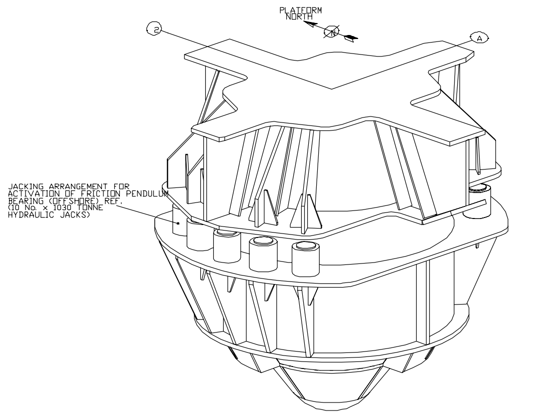

Figure 28 Topsides Support Node and Bridging Cap Showing FPB Activation Jacks

Design Process Summary
The above presentation provides an outline description of the issues that have to be considered for the Temporary conditions during Platform installation and their influence on the permanent Topsides facilities, from both structural and facilities perspectives. It is also apparent that the design process is iterative, such that influences of the in-service designs significantly affect the temporary design constraints. The following flow diagram, Figure 29, figuratively demonstrates the inter-related aspects of the marine engineering and Topsides temporary design cases on the fixed platform configuration and visa-versa.

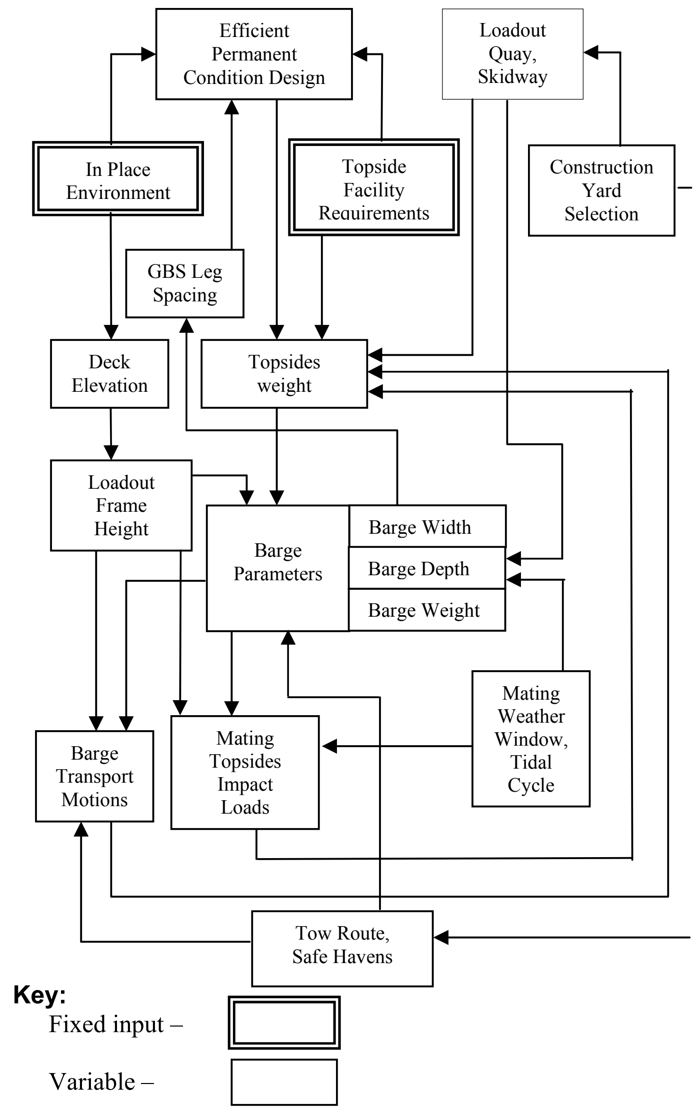

Figure 29 Flow Diagram Showing Interrelated Aspects of Permanent, Marine and Temporary Design Conditions

Conclusions

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Offshore Kinematics Inc. who developed the FEED LMU design.
A third party installation contractor independently verified the FEED work. However, the detailed-design-phase installation contractor develops the final design solutions based on the output of the FEED work.
It is recognized that the final solution will exhibit some variations to the FEED basis presented in this paper.

Nomenclature

SI units have been used throughout paper

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