TIME HISTORY ANALYSIS OF SEISMICALLY ISOLATED FIXED BASE OFFSHORE PLATFORM

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ABSTRACT

This paper presents an overview of the application of seismic isolation to a fixed-base offshore platform. A series of nonlinear time history analyses has been undertaken for the Platform A to assess structural integrity under Ductility Level Earthquake (DLE) design events. A full nonlinear time history analysis (including soil and foundation modelling) has been undertaken using finite element software SAFJAC for an extreme earthquake records scaled for the DLE event.

KEYWORDS: *Ductility Level Earthquake (DLE), Seismic isolation, time history analysis.*

INTRODUCTION

Various sophisticated computational tools are being used for carrying out time history analysis of structures[1,2,3,4,5]. This paper highlights a three-dimensional model for time history analysis of fixed offshore platform.

The platform is located in a seismically active area and the jackets are supported on a total of twenty through-leg piles driven in the soil to depths ranging from 102 to 128 meters. The platform is founded on soft normally consolidated clay, which differs from a typical normally consolidated clay found in the North Sea or Gulf of Mexico soil because of the presence of fissures and polished cracks[6].

The Platform A is designed for a water depth of 120 metres and is located in the Caspian Sea. It consists of two similar template-type jackets that are installed back-to-back and linked by a stiff Module Support Frame (MSF). The Drilling Jacket supports 24 No. conductors and the other is the Production Jacket[7,8,9,10].

The seismicity of the central Caspian Sea region can generally be compared to an (American Petroleum Institute/ Uniform Building Code) API/UBC Seismic Zone 3 classification. Although the magnitude of expected earthquakes in the area is not particularly large, the original design of the platform made it very susceptible to damage – analysis at (Early Oil Project) EOP showed that the weak-link was the pile-soil foundation system[11,12,13,14,15].

The Platform A was accepted at Early Oil Project (EOP) as fit for purpose following the installation of specially engineered seismic isolators and further non-linear time history analysis by (Industry Security and Exchange Commission) ISEC[16]. Since then, there have been a number of topsides weight additions, and over 2,000 tonnes of flooded members have been identified. Therefore it was decided to perform seismic analyses to demonstrate the structure will sustain the changes and continue to hold its fit-for purpose status[10,12,13,17,18].

The current study involves seismic time history analysis for DLE events for the Platform A (Figure 4). The integrity of the structure will then be assessed using recommendations and strength criteria in API RP2A-WSD and guidance in the forthcoming ISO standard for offshore structures.

EARTHQUAKE GROUND ACCELERATION TIME HISTORIES

The Uniform Hazard Spectra for the area was derived by Hugh Banon and the Exxon led TSA in 1996[11]. Site-specific response analyses were undertaken for the site by (Earthquake Engineering) EQE team[12] to account for local soil conditions. This is based upon using input time histories representing 'controlling' 500-year and 3,000-year earthquake events to assess the impact of soil amplification on earthquake motions. The controlling events, i.e. those earthquakes that contribute most to the total seismic hazard, were identified using knowledge of

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the local site conditions, and their likely effect on the structure - this process is known as deaggregation. For Platform A, the deaggregation process used a logic-tree Monte Carlo analysis[9].

The controlling earthquake events (defined in terms of their magnitude and distance from the site) were selected for a structure with a natural period of 2 seconds. This value was assumed because, at the time, the natural frequency of Platform A was unknown.

Based on the controlling events derived from the deaggregation process (using a period of 2 seconds), horizontal and vertical spectra were derived by EQE in 1996[7] using geotechnical data available at the time.

The response acceleration spectra have been derived for a depth of 75 feet below the seabed, which was judged to best represent an equivalent depth where the ground motions effectively induce lateral shaking in the piles of the jacket structure. The spectra are for 5% of critical damping.

Time histories scaled to the 1996 EQE seismic design criteria were used by ISEC in the original non-linear time history analysis of Platform A at EOP.

Subsequently, in 1999 further seismic analysis of the area has been undertaken, specifically for the Azeri development. This has led to slightly higher design spectral acceleration values for periods greater than 1 second. The design spectra for the DLE event is tabulated in the Phase 1 design brief[13]. Hugh Banon[14] has recommended that these criteria be applied to all ACG sites, including Platform A.

During an earthquake, pressure waves propagate through the bedrock, pass through the overlying soil, and finally reach the foundation of the structure, causing it to move and the superstructure to vibrate. The movement of the structure and foundation in turn develops a force that acts on the soil as an inertial force.

From the viewpoint of dynamics, this dynamic soil-structure interaction can be divided into three: the kinematics, the stiffness of the soil and the structure, and the inertia of the structure. As the dynamic force builds up in the structure, the stiffness of the soil-structure interface changes, and the kinematics of the structure are modified. Inertia forces are also developed in the structure as it vibrates, and these forces are fed back to the ground causing it to deform.

In this work, this last effect is ignored, i.e. no transmission of energy from the structure to the surrounding soil is assumed. This is an assumption that is commonly used and it is clearly conservative. Accounting for soil damping would lead to reduced response or behaviour than predicted here.

The earthquake is input into the analysis as an acceleration time history for the surrounding soil. The input represents the free field motion of the soil, and accounts for the attenuation/amplification of the earthquake as it passes through the overlying soil.

The strong ground motions were obtained from Hugh Banon. The time histories have been scaled from actual earthquakes recorded in Turkey and California. For each earthquake the tri-axial records (X, Y and vertical) were scaled by a single factor so that the spectral response acceleration approximately matched the design response spectrum for the site between a period of 2 and 4 seconds.

The response acceleration spectra for the scaled time histories are shown in Figures 1, 2 and 3 for the two horizontal and vertical directions. Also shown are the arithmetic and geometric means of the time histories, and the design spectrum.

The structural analysis was undertaken with significantly smaller time steps. It should be noted that the first fundamental period of the structure is around 3.0 second; the period increases when the seismic isolators are activated, and as the soil characteristics change with increasing load levels. A time step interval of 0.02 seconds is significantly less than 1/10th of the natural period, and is sufficient to capture the actual behaviour in an implicit transient analysis.

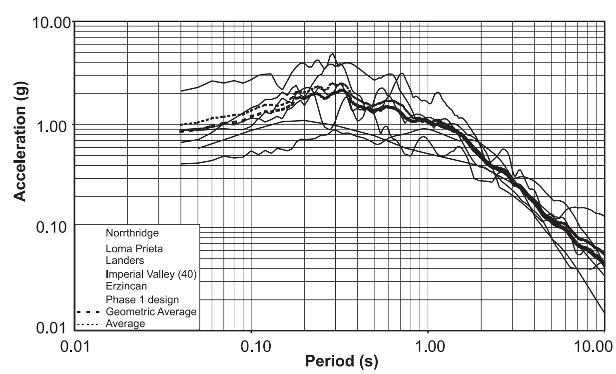

Figure 1: Comparison of acceleration response spectra for scaled earthquake time histories and design response spectra for DLE-horizontal X direction - damping ratio = 5%. Log-log plot

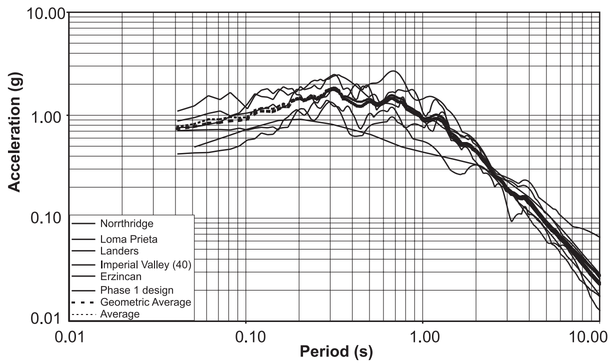

Figure 2: Comparison of acceleration response spectra for scaled earthquake time histories and design response spectra for DLE-horizontal Y direction - damping ratio = 5%. Log-log plot

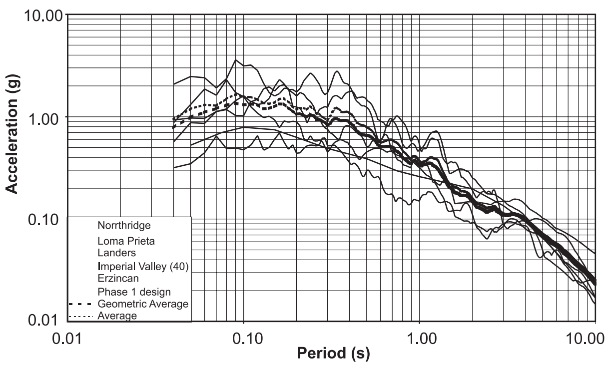

Figure 3: Comparison of acceleration response spectra for scaled earthquake time histories and design response spectra for DLE-Vertical direction - damping ratio = 5%. Log-log plot

SAFJAC NONLINEAR ANALYSIS MODEL

The SAFJAC dynamic model is based on the model used for the Monte Carlo nonlinear pushover analysis studies[9]. However, the model has been extended to include the following:

- Conductors and conductor framing members
- Topsides members and deck plating
- Seismic isolators using nonlinear spring elements.
- Concentrated mass elements
- Damping
- Soil behaviour using piecewise linear cyclic T-z and P-y springs.

Structural Modelling

The SAFJAC nonlinear dynamic model includes the following:

- the legs and all primary jacket braces
- the piles, insert piles (modelled as coexistent members above the mudline and equivalent tubular members below the mudline) and soil springs
- conductor framing members and conductors
- the MSF and seismic isolators
- the main topsides modules and DSM, including all primary longitudinal and lateral beams, columns and diagonal bracing, and members simulating the in-plane shear stiffness of the deck plating.

Although the boat landing, riser guard, risers, caissons, appurtenance supports, installation pontoons, launch truss and buoyancy tanks have not been modelled, the gravity loading and mass associated with these components have been taken into account.

Details of the SAFJAC model are as follows:

number of nodes (including piles).............	3592
Total number of structural elements (including piles).......	5071
Total number of spring elements...............	1007
Total number of mass elements.................	2631
Total number of damping elements...........	2631
Total number of degrees of freedoms.......	21552

Plot of Platform A is presented in Figure 4.

All members are initially modelled as quartic elastic beam-column elements. This type of element allows for the spread of plasticity across the cross-section and along the length of the element. The stresses are monitored over the cross-section at positions along the

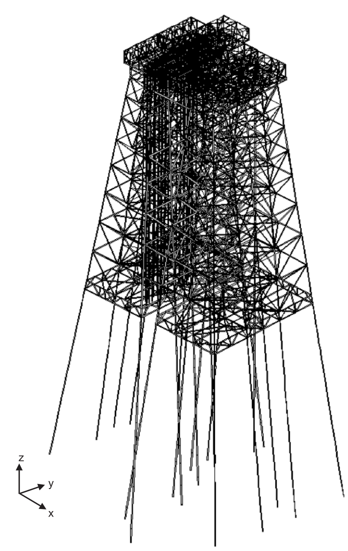

Figure 4: Model plot of Platform A

beam corresponding to a second order Gauss quadrature rule. The beam elements subdivide automatically when yield is detected; cubic elasto-plastic elements with strain hardening are introduced where yielding occurs.

The topsides members and plating members were treated as elastic, and assumed not to fail or become plastic under earthquake loading.

The conductors were guided laterally at each guide position, including the –2m level, and were supported vertically at the mudline. However, no lateral restraint was assumed at the mudline (or below). In addition, the conductors were restrained by weak springs at the Cellar Deck level so that their stiffness would not interfere with the behaviour of the isolators. In reality, there are generous annular gaps between the conductors and guides.

Material

The following design or 'characteristic' material yield strengths were used:

Piles, 720mm diameter insert piles, legs	392.2 Mpa
820mm and 630mm diameter insert piles	323.7 Mpa
All jacket braces with 720mm diameter or greater.......	392.2 MPa
Legs above pile cut-offs	344.7 MPa
All other jacket braces	245.1 MPa

Other material properties were assumed as follows:

Young's Modulus.......................	2.068810^8 MPa
Strain hardening..........................	0.02%
Poisson's ratio............................	0.3
Density of Steel..........................	7.85 tonnes/m^3
Density of Grout/Concrete...........	2.40 tonnes/m^3

Seismic isolator modelling

Platform A was the first (and is believed to be the only) offshore structure in the world with seismic isolators. Each of the 20 jacket legs has an isolator installed just below the MSF at around +12m. Figure 5 shows a photograph of the isolators installed at the top of the jacket.

Figure 5: Photograph showing seismic isolators at the top of the jacket

The isolators have four functions to:

- transmit vertical load
- provide lateral flexibility
- provide a restoring force
- provide energy dissipation.

In the vertical direction the isolators are very stiff, and are designed to transmit vertical load even when the topsides support stubs are eccentric to the jacket legs. The vertical load is transmitted through a PTFE coated spherical sliding bearing on each isolator. Thus, there is some rotational freedom or tolerance in the operation of the isolators – when the isolators were tested at manufacture, they were tested at an effective rotation of up to 0.01 radians. This is sufficient to relieve moment in this stiff area of the structure. Thus, for normal operation and storm analyses, the elements modelling the isolators have been modelled with freedom releases for all three rotational freedoms, i.e. pinned.

The isolators act as bearings supporting compressive load, and are unable to transmit tensile load. Thus it is important to check the seismic analyses for tension at the isolators, as significant uplift may be considered to be a failure condition.

For normal operating and storm loads, pairs of shear pins at four of the main legs 'lock' the isolators and transmit topsides lateral loads to the jacket. In the event of a strong earthquake, the shear pins are intended to fail. The failure load of each shear pin is nominally 1,500kN, thus for the 8 shear pins the total resistance is 12,000kN, which is significantly above extreme wind loading on the topsides.

Once operational, the isolators act in a lateral direction, and are PTFE-coated multi-directional sliding bearings equipped with specially designed steel hysteretic dampers. The ‘damper’ or ‘dissipator’ components are 50mm thick semi-circular shaped elements arranged in pairs around the top and bottom sliding plates of the isolators; there are a total of 32 dissipators per isolator. Up to a total design lateral force of 1,200kN per isolator the dissipators act elastically. With increasing load above the effective yield point, the lateral behaviour of the isolators is ‘plastic’; during unloading the behaviour reverts to elastic. It is this hysteretic behaviour that is intended to damp the motion of the topsides, and thus reduce the inertia that is transmitted to the jacket and foundations.

With 20 isolators the total effective yield or break-out force is 24,000kN.

To prevent the topsides sliding off of the jacket legs during a very severe earthquake, the isolators are fitted with an additional system (the so-called ultimate system) which is activated if the lateral displacement were to exceed 300mm. The relative displacement of the isolators is not intended to exceed 300mm under the DLE design condition, and the maximum displacements from the seismic analyses will be checked to confirm this.

Each isolator was modelled using spring elements with equal stiffness in the two lateral global axis directions and very high stiffness in the vertical direction. A 3-part piecewise linear curve was used to represent the lateral stiffness of the isolators, as illustrated in Figure 6.

Foundation modelling

The foundation was modelled using cubic elastic-plastic beam elements for the piles and nonlinear spring elements to represent interaction with the soil. Three-part piecewise linear springs (a simplified form of the 5 part piecewise linear springs used in the pushover analysis) were used to model cyclic behaviour of the soil. The soil properties are modelled for the resultant direction.

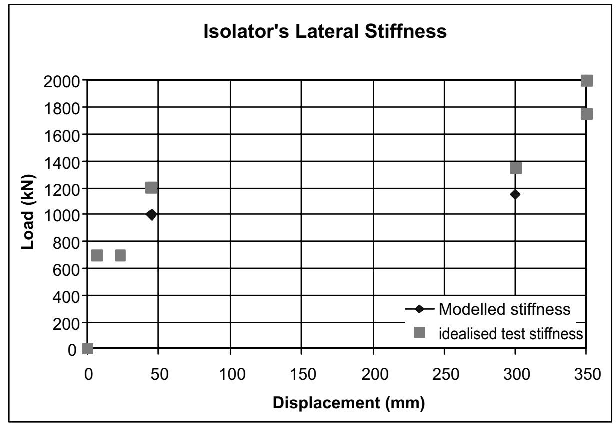

Figure 6: Isolator stiffness used in SAFJAC

Each nonlinear spring element represents the axial T-z (skin friction) and the lateral P-y soil stiffnesses associated with a particular pile element. Nonlinear springs were also introduced at the tip of each pile to represent the Q-z end-bearing stiffnesses.

The T-z soil springs were defined such that the initial stiffness was the same as that for the original five-part piecewise curve used in the pushover analysis; the original capacity was also retained, as illustrated in Figure 7.

The P-y soil springs were also defined such that the initial stiffness was the same as that for the original five-part piecewise curves. However the capacity was defined as the mean of the maximum and residual capacities, as illustrated in Figure 8. A series of test runs was performed to ensure that the overall behaviour of the structure would not be affected by the modifications to the soil curve.

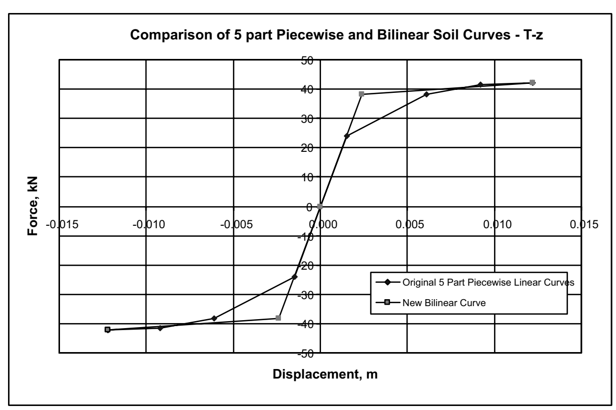

Figure 7: Comparison of 5-part piecewise linear and bilinear T-z soil springs

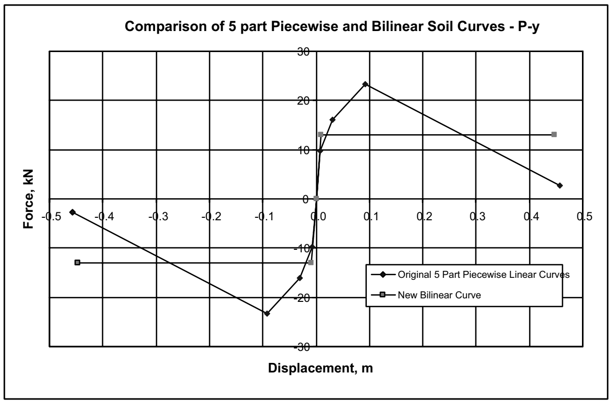

Figure 8: Comparison of 5-part piecewise linear and bilinear P-y soil springs.

Boundary conditions

Each analysis was carried out in two stages, the first stage being a static analysis for the still water condition and the second stage being the dynamic time history analysis.

For the first stage, the soil end of each soil spring was restrained rotationally (θx=θy=θz=0) and prescribed zero displacements in all three translational directions (u=v=w=0). In the second stage, earthquake acceleration time histories were applied in the global X, Y and Z directions to all nodes on the piles below mudline (Figure 9).

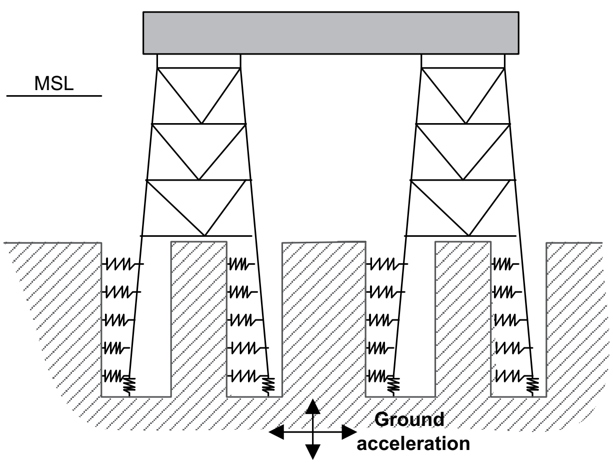

Figure 9: Schematic of soil boundary conditions (T-z springs not shown for clarity)

Summary of platform mass

A summary of all the masses and its associate centre of gravity (COG) positions are tabulated in Table 1.

Table 1: Summary of mass distribution and centre of gravity (COG).

Description	Mass (tonnes)			COG (m)		
	X	Y	Z	X	Y	Z
Generated structural mass	12209	12209	12209	-4.260	-0.713	-46.25
Non-generated mass - topsides	10752	10752	10752	-1.029	1.249	18.35
Hydrodynamic mass	11584	9868	7033	-4.777	-0.468	-75.968
Marine Growth	957	957	957	-2.13	-0.019	-45.574
Entrapped fluid mass	6852	6852	6852	-6.132	-1.865	-82.27
Non-generated mass - jacket	2965	2965	2965	-2.072	0.002	-61.612
Total (SAFJAC) (without pile & soil)	45317	43601	40767	-3.721	-0.291	-41.5
Total (ASAS) (without pile & soil)	45329	43613	40778	-3.188	-0.155	-41.450

Natural Frequency Results

Modal analysis has been performed for the Platform A using SAFJAC The results are summarised in Table 2. The first three mode shapes are shown in Figures 10, 11 and 12 respectively.

Table 2: First three modes of vibrations

	Natural Periods (s)
Mode	
1	3.01
2	2.88
3	2.20

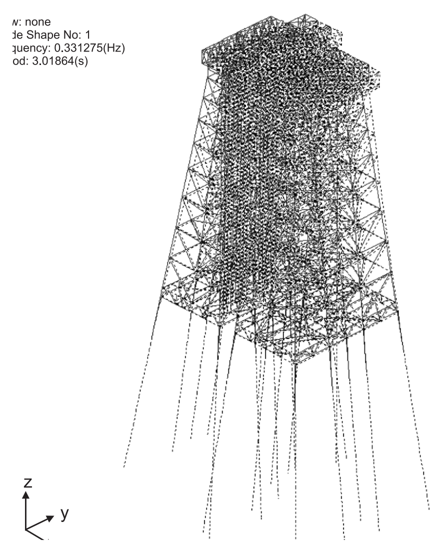

Figure 10: First mode of vibration (Sway in Y direction)

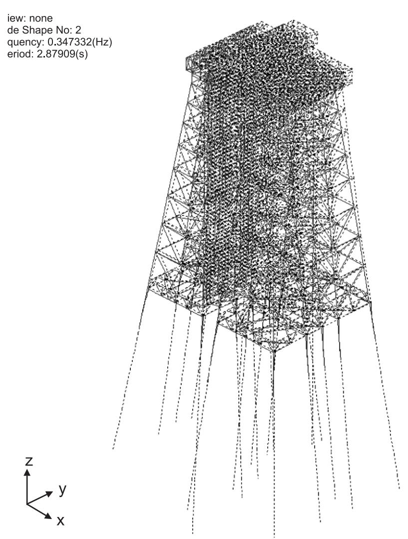

Figure 11: Second mode of vibration (Sway in X direction)

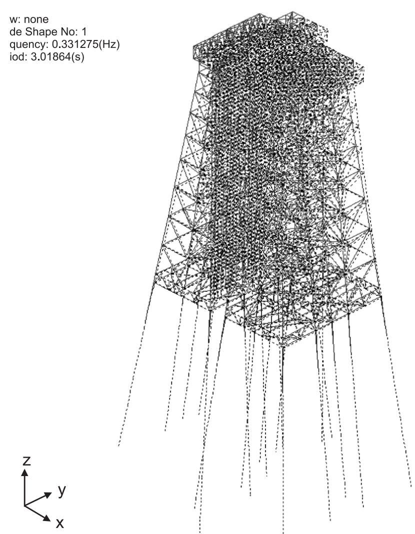

Figure 12: Third mode of vibration (Torsion)

Ductility level earthquake analysis results

DLE analyses were performed for a total of six ground time histories scaled to the design event, namely: Newhall - X & Y Inputs Switched, Loma Prieta, Erzincan, Landers, Imperial Valley, and Newhall. The results are summarised in the following sub-sections. The platform's orientation and monitoring points are shown in Figures 13 and 14.

Base Shear

Base shear force was calculated for all of the DLE analyses and it was found that DLE Landers event produces the highest base shear, a value of 52,686 kN in the global X (East-West) direction (Table 3). The

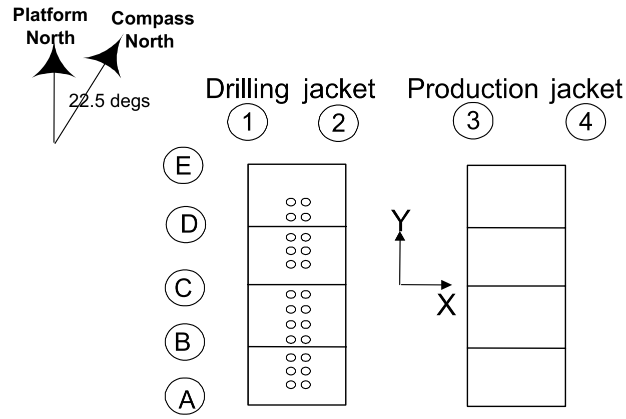

Figure 13: Platform A Orientation and Global Axis System

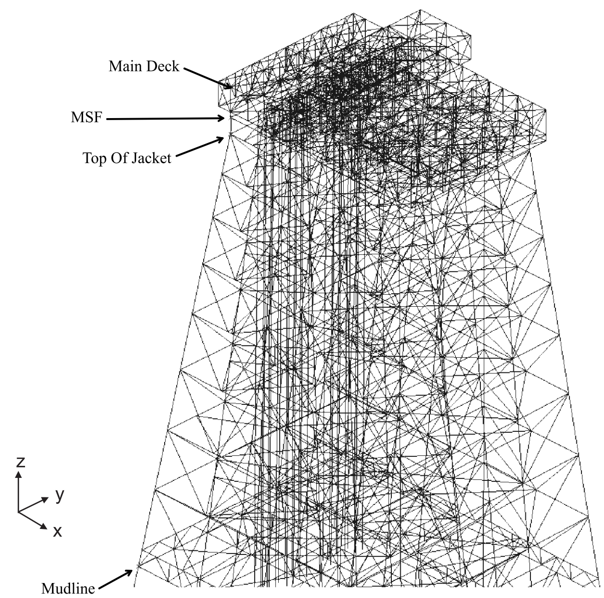

Figure 14: Monitoring points in the Analysis for Platform A

Imperial Valley event produces a value of 45,700 kN, in the global Y (North-South) direction. The Landers event also produced the highest vertical base shear of 416,597 kN. The average base shear for all DLE analyses is about 46,190 kN and 33,738 kN for the X and Y directions respectively and 327,730 kN for the vertical direction.

For all of the DLE time histories analysed, several plastic hinges form in the piles. For the Imperial Valley event the structure 'collapsed' due to extensive yielding in the piles under combined axial load and bending, (Figure 15). The maximum base shear calculated in the analyses is similar to the static capacity of the P-y soil springs which is estimated to be 45,000 kN. This is evaluated from the Fugro report[6] and assumes that the insert piles are present and effective. Although the Landers Event records a higher base shear of 52,686kN,

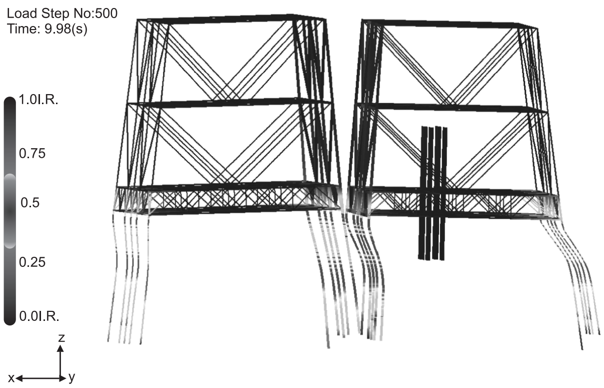

Figure 15: Deformed Shape and Member Utilisations for DLE Imperial Valley Event at Failure (Foundation and lower part of both jackets)

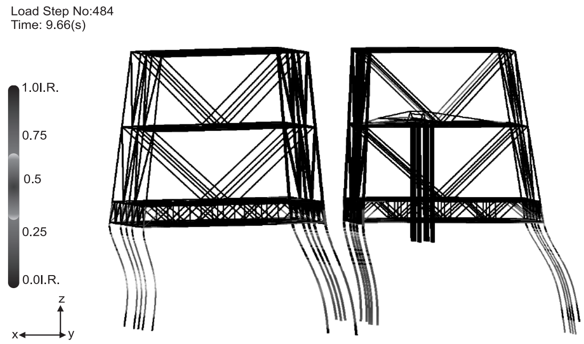

Figure 16: Deformed Shape and Member Utilisations for DLE Landers Event at Maximum Base Shear (Foundation and lower part of both jackets)

which is greater than the static capacity of the piles, it occurred only for a very short period of time. For the remaining time, the static capacity was not exceeded. Pile member utilisations and the deformed shape of the foundation for the Landers event for the maximum recorded base shear are presented in Figure 15. The extent of plasticity in the piles is significantly less than that shown in Figure 14 for the Imperial Valley event.

Table 3: DLE Analysis – Result Summary

DESCRIPTION	DIR	DLE-NH	DLE-LAN	DLE-IV	DLE-ERZ	DLE-LOM	DLE-NH-S	AVERAGE
Muldline Displacement at Leg Al (mm)	X	609	599	4642	357	823	759	**605**
	Y	775	412	5627	353	664	515	**521**
	Z	157	169	2533	108	206	195	**163**
TOJ Displacement at Leg A1 (mm)	X	722	639	1153	621	967	801	**740**
	Y	815	415	4358	410	550	610	**541**
	Z	169	210	2202	146	217	192	**185**
MSF Displacement at Leg A1 (mm)	X	760	641	1172	684	1037	880	**788**
	Y	914	404	4346	429	608	633	**572**
	Z	169	210	2021	146	217	192	**185**
Maindeck Displacement at Leg A1 (mm)	X	768	646	1127	700	1049	888	**798**
	Y	929	407	4272	431	618	342	**578**
	Z	170	212	2206	147	218	193	**186**
Isolator Displacement at Leg C3 (mm)	X	53	71	44	147	103	108	**91**
	Y	219	62	152	97	74	63	**91**
Isolaor Force at Leg C3 (kN)	X	1004	1015	977	1060	1034	1037	**1030**
	Y	1102	1010	1063	1030	1017	1010	**1033**
Base Shears (kN)	X	36,050	52,686	48,400	41,770	51,170	51,794	**46,190**
	Y	44,877	32,523	45,700	39,460	28,390	26,735	**33,738**
	Z	327,260	416,597	334,500	258,722	327,100	327,664	**327,726**

Note:

(1) Time history analysis for DLE Emperial Valley event did not complete due to pile strength failure (plastic hinge formation in all piles)

(2) Geometric average does not include results from DLE Imperial Valley event.

Isolator Forces and Displacements

The maximum seismic isolator relative displacement of 219 mm and the maximum force of 1,102 kN were recorded for the Northridge Newhall event (see Table 3). However, in each time history analysis all isolators exceeded their effective yield or break-out capacity of 45mm. On average, for all DLE analyses, the isolators moved only 91 mm in the global X (East-West) and global Y (North-South) directions. In some DLE time history analyses a small number of isolators go momentarily into tension. For the Landers Event, a tensile force of 3,500 kN was recorded for a short period of time. As this force is small compared to the topside weight of 13,000 tonnes, it is not considered significant. Further, this may be due to numerical noise in the analysis. If the topsides loading were to be less at the time of the event, the seismic uplift force (based on mass times acceleration) would also be less.

Member Plasticity

For all six DLE analyses there is very little member plasticity in the structure at any time during the analysed time periods. Some plasticity occurs at the +10m and –2m plan framing and at the base of the jackets; it is not extensive and only occurs for a very short period of time (less than 0.1 seconds). Furthermore, if the maximum utilisation ratios are 'averaged' for the six DLE analyses (as recommended in the forthcoming ISO standard), very few members are over-utilised. Figure 17 presents the member utilisations for all DLE analyses and Figure 18 presents the averaged maximum member utilisations over the six analyses. It should be noted that only structural member utilisations (not piles) are presented in Figures 17 and 18.

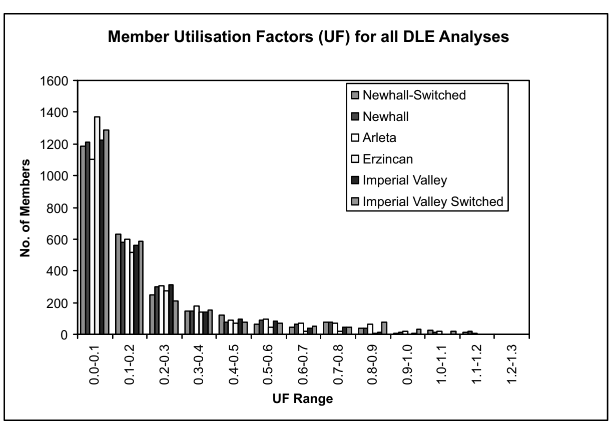

Figure 17: Plastic Member Utilisation Check for DLE Events

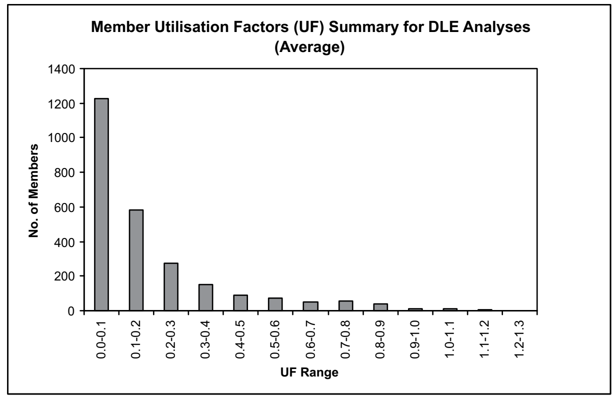

Figure 18: Plastic Member Utilisation - Geometric Average for DLE Events

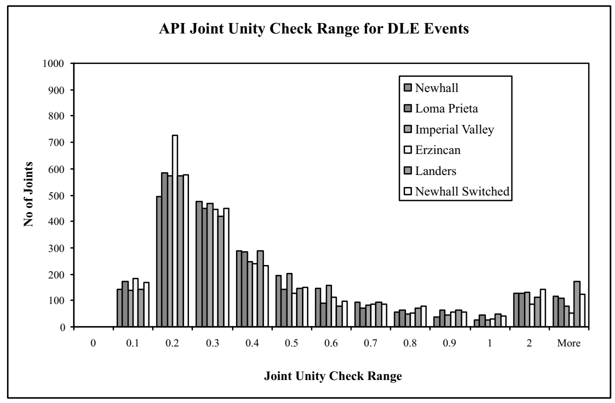

Figure 19: API Joint Unity Check for DLE Events

API RP 2A – WSD Joint Checks

Joint utilisations were evaluated for each time history using API RP2A-WSD formulations. No account was taken for poor joint fabrication or weld defects, and a 70% increase in allowable stresses was included. 284 tubular joints are over-utilised at the +10m and -2m levels and at the base of the jackets for small intervals of time during the Landers Event (Figure 19). When the maximum utilisations in each time history are averaged over the six analyses 239 joints are over-utilised (see Figure 20); this is about 10.8% of the 2204 joints for which joint utilisations were evaluated.

CONCLUSIONS

- The dynamic behaviour of the structure, based on the properties analysed, is very encouraging. Out of six DLE earthquake time histories analysed, the structure survived five with little damage to the structural members.
- The response of the structure has been shown to be sensitive to the lateral stiffness and T-z stiffness assumed for the foundation.
- The response of the structure is less sensitive to the capacity and stiffness of the seismic isolators.

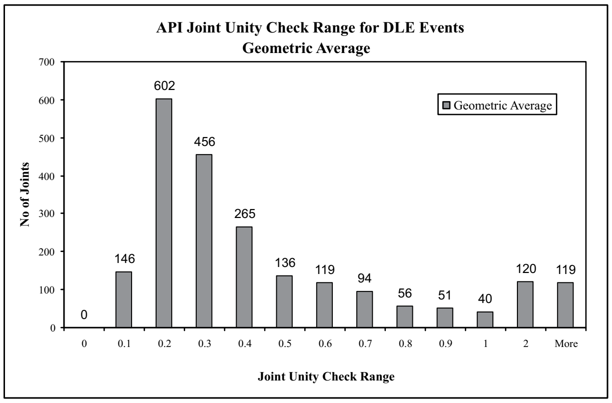

Figure 20: API Joint Unity Check - Geometric Average for DLE Events

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