BI-LEVEL SEISMIC DESIGN OF OFFSHORE PLATFORM IN 670 FEET OF WATER

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SUMMARY

This paper describes the bi-level structural design of an offshore platform. Results for plasitic and ductility analyses in relation to design guidelines are presented along with a discussion of the seismic exposure for the design levels. It is shown that a well engineered structure can readily meet present design guidelines.

INTRODUCTION

In the past several years, there has been a marked increase in the exploration and production of hydrocarbons in seismically active areas of the world. In the United States, much of the effort has been focused on the area off the coast of southern California. Fixed based platforms have been designed for this area in water depths up to 1200 feet (365 M). In addition to the traditional environmental forces of wind, wave and current, the designer is confronted with the seismic forces as a major design condition.

Recent industry practice, which has been somewhat parallel to and extended by the developments in this area, is set forth in the American Petroleum Institute recommended practice 2A, API RP 2A (Ref 1). The seismic design recommendations differ from all others in that inelastic behavior is explicitly considered. The recommendations include a bi-level design condition for the seismic loading; 1. A strength requirement that states that the structure should not sustain any permanent damage for earthquake intensities which may be reasonably expected during the life of the structure (10-30 percent probability of occurence) and 2. A ductility requirement stating that the structural system can permanently deform during a rare event (1-3 percent probability of occurence), but must absorb sufficient energy through elastic and inelastic displacement to preclude collapse or catastrophic failure. The initial cost of a structure designed to remain elastic under both levels of excitation exceeds that in which inelastic behavior is allowed by as much as tens of millions of dollars (Ref 2).

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Structure Description

Platform Harvest is a typical steel template type platform in 670 feet (204 m) of water. It consists of topside drilling and production equipment modules supported on a space frame composed of fully welded tubular members. This space frame is a "jacket" through which 50 well conductors, 8 main piles and 20 skirt piles are driven into the seafloor. The 60-inch (152 cm) diameter main piles and 72-inch (183 cm) diameter skirt piles will be driven 255 feet (70 m) into the firm to dense silty-clays and clayey silts at the site to secure the platform to the seafloor. The conductors are left free to slide vertically in the jacket, but rely on the jacket for lateral support.

Major braces in the jacket have been sized to have diameter/wall thickness ratios of 48 or less, and slenderness ratios of 100 or less. The former assures that each brace can achieve its plastic moment capacity prior to local wall buckling (Ref. 4), and the latter avoids excessively pinched hysteresis loops during post yield and post buckling response (Ref. 6, 7).

The estimated reaction at the mudline due to dead, drilling and production loads is 65,000 kips (24,000 metric tons) gross less 13000 kips (4900 metric tons) buoyancy.

SEISMIC EXPOSURE

The overall project was placed on a fasttrack schedule. This schedule required that the design criteria be obtained quickly to commence design. A site specific deterministic geological, seismological assessment of the platform site was conducted (Ref. 3). The study yielded design criteria for the strength level event in the form of a response spectrum. A second elastic response spectrum was generated for the rare event in an effort to correlate the difference in magnitude of the postulated events to help establish the level of ductility for design at this location. The correlation was desired since the ductility requirement in RP 2A was generated for a regional reference in southern California and not site or structure dependent.

The study indicated a complex series of individual and regional fault systems. Since the study was based only on a review of available data in the public domain, the faults and their associated seismicity as reported in the literature were considered. The public domain data base lacked data from deep seismic profiles. The complex faulting in the area has led to several postulations as to the source of the fault/fault for major historical events. These theories display a wide range in the seismicity level at the site. North of the platform site, several well documented faults exist. However, there has been no general consensus on whether these faults are individual or whether they are part of a larger single fault system. By the nature of the deterministic study, the more conservative approach, that being all are part of a single fault system, was chosen since it was a possibility. As several of these type of decisions had to be faced, a ripple effect of conservatism started to produce a pronounced effect on the results. Likewise, several individual faults near the site, within 12-15 kilometers, had not been well defined in the literature and seismicity related data has been extrapolated from onshore faults as much as 20-30 kilometers away.

Upon review of the deterministic study and recognizing the effect on design due to the above, it was decided to conduct a probabilistic based seismic exposure study including the review of high resolution, deep seismic profiles of the nearly postulated and known faults in an effort to better quantify those nearby faults and their past movement (Ref. 4). This study provided two important items: 1. A sensitivity and rational study of the individual versus multiple fault postulations thus eliminating the ripple effect and 2. Better defined parameters on nearby faults that had contained significant energy in the high frequency portion of the design spectrum.

Since this was a fasttrack project, the design had to proceed using the original study while the second was ongoing. The design schedule called for the elastic, strength level event analysis to be performed early while the ductility check was performed much later, after the conclusion on the second seismic exposure study. It was felt that if the D/T ratios of members were kept below 48 when sizing due to the various loadings, then the ductility check would probably indicate no major resizing would be necessary. This was based on experience that structures having redundant portal bracing patterns, i.e., X-braced with horizontals, with members capable of forming plastic hinges would absorb the required amount of energy without collapse.

The second seismic study yielded interesting results, most notably that the new rare event spectrum was very similar to the original strength spectrum.

The new results were primarily due to the downgrading in magnitude of the nearby faults. Review of the deep seismic reflections showed much less historical fault movement than indicated in literature. It was felt that this review was superior to the literature data for two reasons: 1. It was site specific and 2. The public domain information was not generated on data of this quality since much of it was postulated from trends occurring onshore in excess of 20-50 kilometers away. A second significant factor was the qualitative evaluation of the various faults as to being individual or part of a larger single fault.

The actual design was based on the results of the probabilistic study. However, a study was conducted to check the capacity of the structure relative to the design guidelines. For this, the elastic analysis referred to in this study was based on the rare event spectrum from the probabilistic study.

ANALYSIS

Elastic Analysis

Elastic analysis was based on the modal response spectrum method. The analytical model included simulation of all major framing components and an elastic spring foundation simulation. The framing is depicted in Figure 1, along with the mass distribution. The mass distribution consists of the masses due to structural framing, process equipment, drilling rig equipment and supplies, production fluids, "added virtual water mass" and contained water mass of submerged members, lateral mass of conductors, and other miscellaneous items present on the platform.

Masses were discretized to the nodal locations to produce a lumped mass model. The structural model with 1013 nodes and 6078 degrees of freedom was reduced through a kinematic condensation to 84 nodes with 242 degrees of freedom to reduce the computation cost. The master nodes consisted of from 8 to 15 nodes at each major horizontal elevation.

The foundation system is an important component of a fixed base offshore platform, often contributing a significant portion of the total system flexibility. Additionally, there can be significant soil-structure interaction during earthquake ground shaking. Consequently, reduction of the actual nonlinear soil-pile foundation system into a linear spring simulation must be handled with care. In this project, elastic mudline springs (axial, lateral, rotational, and lateral-rotational coupling) at each pile top were matched to the tangent stiffness of the actual nonlinear soil-pile system at the estimated total deflection due to the applied seismic loads. The pile resistance at this deflection was determined from detailed single pile analysis using non-linear P-Y and T-Z curves to represent the lateral and axial soil resistance along the pile shaft.

Modal analysis indicated the fundamental period in the transverse lateral direction to be 4.21 seconds, 3.52 seconds in the longitudinal direction, 1.9 seconds in torsion and 0.65 seconds vertically. Using the site specific probabilistic rare event response spectra shown in Figure 2 and SRSS combination of the lowest 25 modes, the following results were calculated:

System strain energy	=	12,240 ft.-kips	(16612 KN-m)
Longitudinal mudline shear	=	20,200 kips	(89890 -KN)
Transverse mudline shear	=	19,200 kips	(85,440 -KN)
Vertical mudline reaction	=	19,800 kips	(188110 -KN)
Overturning about longitudinal axis	=	6,724,000 kip-ft.	(9126 MN-m)
Overturning about transverse axis	=	8,214,000 kip-ft.	(11148 MN-m)
Torque	=	277,000 kip-ft.	(375 MN-m)

Member stress levels were designed to remain below 1.7 times basic API allowables, that is, to remain esentially below yield or buckling. The elastic analysis required 250 tons (227 kkg) of steel to be added to the structure beyond that required for other operating or environmental loadings. This included steel required to extend the piling penetration 10 feet (3.05 m).

Ductility Assessment

The linear elastic space frame model was updated to include finite elements capable of nonlinear inelastic response and a more explicit representation of the soil-pile system. These updated elements were based on phenomenological formulations of inelastic response, (ref. 7, 8, 9, 10)

The platform ductility (inelastic energy absorption capacity) was assessed by the initial kinetic energy method (Ref. 1). In this method, a pattern of initial velocities is applied to the nodes of the finite element model, and then the structural model is allowed to freely vibrate, demonstrating the energy absorption capacity of the structure.

The initial velocity pattern was chosen to be representative of the peak strain energy in the structure-foundation system during the elastic analysis, but scaled so that kinetic energy would reach a target value that the structure should be able to resist without collapse. For Southern California, the target energy is approximately four times the baseline energy from the elastic strength analysis. Figure 3 shows the imposed velocity profiles, which were developed by superposing the velocities of the first and second modes in each lateral direction, the modes which had contributed the bulk of the respone in the response spectrum analysis, and then scaling by 1.65. The imposed kinetic energy was 4.5×10^4 ft-kip (6 MN-m). Additionally, a vertical load pattern was imposed to represent the maximum downward seismic loading that was expected to act concurrently. Due to the limiting transmissibility of the soil-pile system and the improbability of maximum vertical and lateral responses occurring at the same time, the vertical load was limited to 112.5 percent of the maximum calculated in the elastic analysis. The vertical seismic loading added an additional 6900 ft-kips (9.4 MN-m) of strain energy to the system, bringing the total imposed energy to 51,900 ft-kips (70 MN-m). Note that other initial velocity profiles are plausible.

The structure was able to resist the imposed kinetic energy without collapse, although inelastic deformations did occur. Figures 4 through 13 summarize key results of the analysis. Figure 4 compares the system strain energy history to the energies dissipated by soil dashpots and structure-water hydrodynamic interaction. Clearly, most of the energy was dissipated by inelastic straining during the first response cycle. Response appeared to be dominated by the fundamental lateral sway modes, as evidenced by strain energy peaks occurring at twice the natural frequency of the sway modes and by deck displacements (Figure 5) occurring at the frequency of the sway modes. Base shears and overturning moments (Figures 6 and 7) reached approximately 2.5 and 1.5 the elastic values, respectively. The early portions of the base shear history displayed a jaggedness thought to be indicative of structural "ringing" and numerical sensitivity induced by the sudden application of impulsive loading. It is believed that numerical sensitivity could have been minimized by applying a consistent set of initial nodal velocities and accelerations. Figures 8 and 9 indicate element ductilities $\mu = (\delta_T - \delta_Y)/\delta_Y$, where δ_T = total deformation, and δ_Y = first yield or buckling deformation typical to the main trusslines. Inelasticity was evident in much of the vertical bracing, at the pile tops and in the soil providing axial support along the pile shafts. Individual element ductility demands were as high as 4.4 in the steel and 9.6 in the soil; several piles approached their axial load-carrying capacity. Typical member behavior of yielded strut member is shown in Figure 10 while Figure 11 shows typical foundation element Force vs. Time plots.

CONCLUSIONS

The seismic exposure studies indicate an approximate twofold increase in seismic intensity between the strength and rare intense events which corresponds to a fourfold elastic energy increase. This ratio is the same as the target original value used in the development of the industry design guidelines (Ref. 1). The ductility assessment confirmed that a) the energy capacity of the structure exceeded the guidelines, and b) that structures having judiciously chosen redundant geometries, ductile materials, and well proportioned members have the ductility to withstand a possible, but improbable event during the structure's life.

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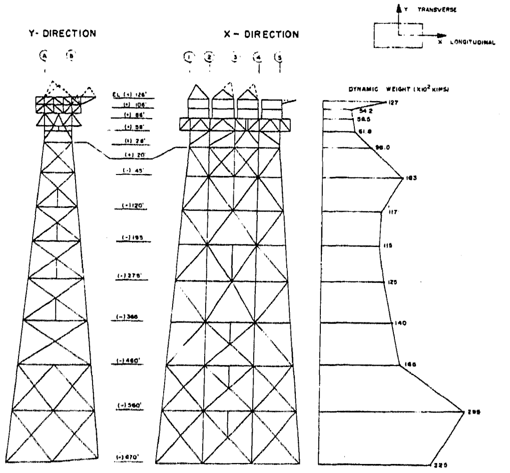


Fig. 1

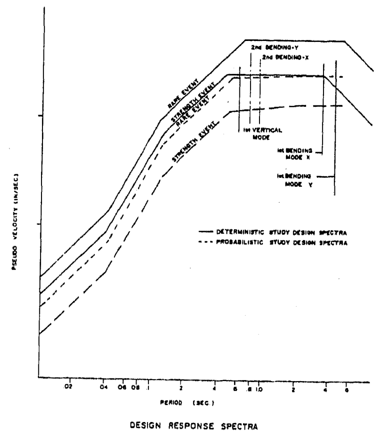
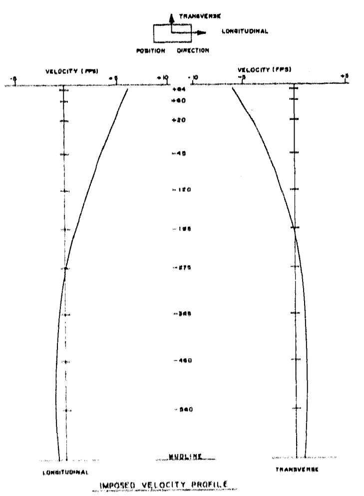


Fig. 2



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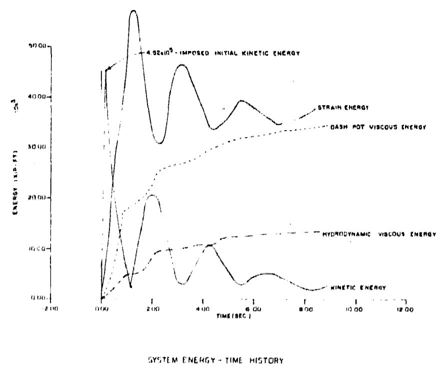


Fig. 4

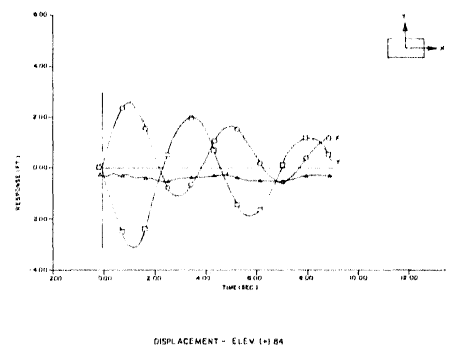
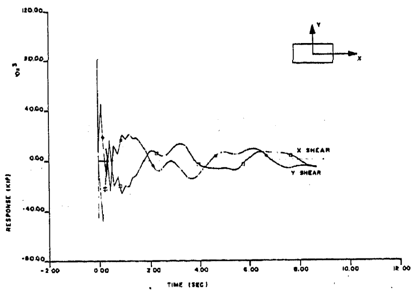
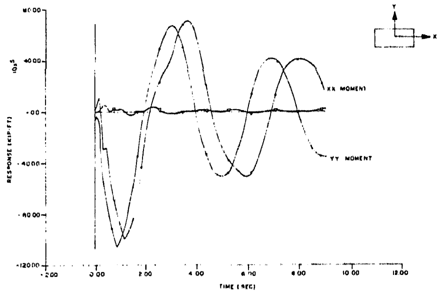


Fig. 5



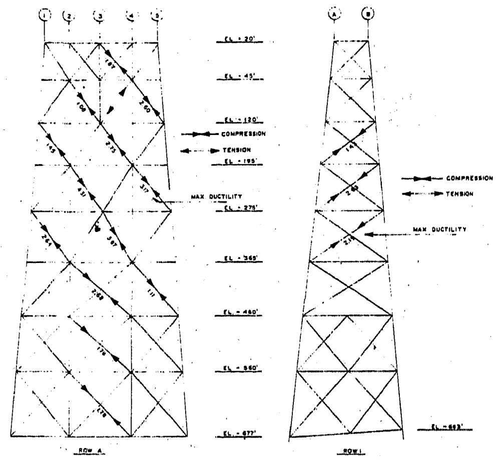
BASE SHEAR FORCE-TIME HISTORY

Fig. 6



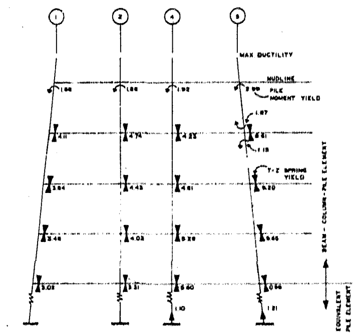
BASE OVERTURNING MOMENT
TIME HISTORY

Fig. 7



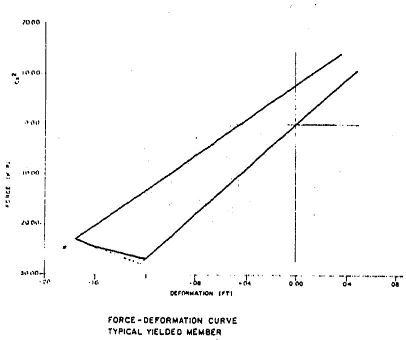
INELASTICITY OF FRAMING

Fig. 8



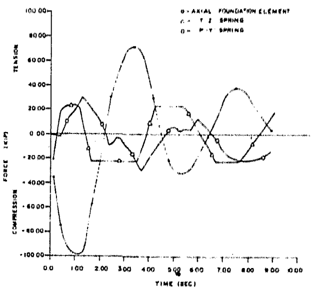
FOUNDATION
INELASTIC RESPONSE AT ROW A
MAIN PILES

Fig. 9



FORCE-DEFORMATION CURVE
TYPICAL YIELDED MEMBER

Fig. 10



TYPICAL FOUNDATION ELEMENT RESPONSE
FORCE vs TIME

Fig. 11