15 Mar 1991, 8:00 am - 9:00 am

Earthquake Geotechnology in Offshore Structures

R. G. Bea
University of California, Berkeley, California

Proceedings: Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, March 11-15, 1991 St. Louis, Missouri, Paper No. SOA13

Earthquake Geotechnology in Offshore Structures

R. G. Bea
Professor, Department of Civil Engineering and Department of Naval Architecture and Offshore Engineering, University of California, Berkeley, California

SYNOPSIS Current developments in earthquake geotechnology in offshore structures are summarized. These developments include measurements of sea floor and platform motions induced by earthquakes and analyses of platform structure and foundation systems. Results from these recent developments are illustrated.

INTRODUCTION

Earthquake geotechnology in offshore structures involves many of the same problems and issues as their onshore counterparts. However, there can be some very important differences; the most important is the presence of a water column over the soil surface. This water column has important effects on the ability locate and characterize earthquake sources, identify important travel path geology characteristics, and determine site soil properties. The water column influences the response of the sea floor soils, the types of foundations and structures that comprise offshore platforms, and the mass, stiffness, damping and strength characteristics of the platforms.

In this paper, one particular type of offshore platform will be discussed; steel, pile supported, tubular member space framed drilling and production platforms used in development of marine hydrocarbon resources. Approximately 70 such major platforms have been installed in potentially intense earthquake zones such as offshore California, Alaska, New Zealand, Japan, China, and Indonesia. In general, fixed, bottom supported offshore platforms tend to be fundamentally long period systems having natural periods (lateral, flexure, torsional) in the range of 1 to 5 seconds.

During the past 20 years, advanced earthquake design guidelines have been developed for this type of offshore platform (API, 1991). These guidelines involve detailed seismic exposure evaluations for the platform locations, analyses of local geologic and soil effects on potential earthquake ground motions, and fully coupled (soil-foundation-structure-superstructure), three dimensional dynamic response analyses. The response analyses are generally of two types:

1) elastic response spectra based analyses to provide basic design forces, and

2) nonlinear, inelastic time history analyses to permit evaluation of the capacity and ductility characteristics of the platform system.

Background for the earthquake design guidelines have been provided by extensive laboratory and field testing of key components that comprise the platform (e.g. tubular joints and braces), including the local soils and foundation elements. Some of the testing has involved assemblies of components (e.g. groups of piles) to determine their interactive characteristics. This testing has provided the basis for formulation of the analytical models and for verification of the results from the analytical models (API, 1991).

These design criteria have also been substantiated by extensive experience with design of the platforms for intense environmental loadings developed by storm winds, waves, and currents, and from ice in arctic (e.g. Alaska) and subarctic (e.g. Bohi Bay offshore China) areas. Approximately 6,000 structures of this type have been installed on the Continental Shelves of the world in water depths exceeding 400 m.

These structures are inherently designed for very large lateral loadings in addition to very large vertical operating loadings. The loadings developed by storms and intense ice conditions can equal or exceed those associated with even very intense earthquakes. These other sources of loadings have important influences on the strength, stiffness, and mass characteristics of the platform systems. Storms and ice can also have some very important effects on the foundation soils.

RECENT DEVELOPMENTS

Recent developments (1981-1991) in earthquake geotechnology in offshore structures have been concentrated primarily in two areas:

1) development and deployment of instrumentation to measure the responses of sea floor soils and platforms, and

2) development of advanced analytical models and capabilities to assist evaluations of the performance characteristics of platform systems when subjected to very intense earthquakes (ultimate limit state conditions).

The following sections of this paper will summarize the recent developments in each of these two areas.

INSTRUMENTATION

Figure 1 illustrates one of the advanced instrumentation systems that has been developed during the past 10 years to permit measurements of sea floor responses to earthquakes. The SEMS (Seafloor Earthquake Measurement System) is a sophisticated accelerometer system that can be installed remotely in the sea floor or hard-wired to a nearby platform (Smith, 1991).

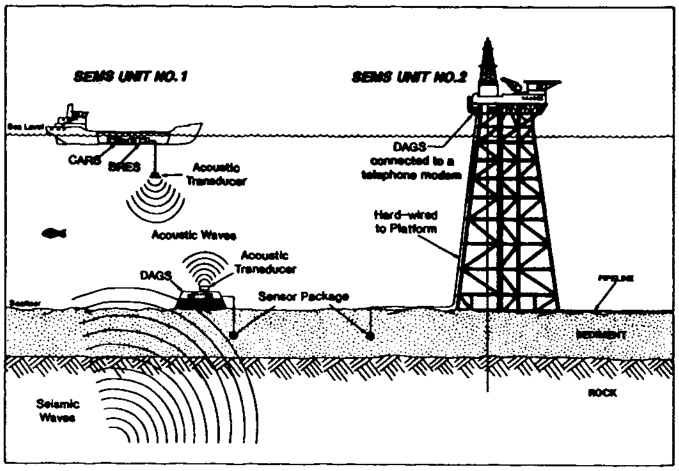


Fig. 1. Remote and Platform Supported Seafloor Earthquake Measurement System (SEMS)

In its remote operation, the triaxial accelerometer package is embedded in the sea floor and connected to a nearby data storage system. This data storage system can be periodically interrogated with an acoustic transducer to determine if the system is working properly and to retrieve any data that might have been recorded. In the case of the hard-wired version, the data is stored onboard a platform and periodically interrogated via a microwave telephone system.

Figure 2 shows earthquake epicenter and magnitude data that has been gathered at two SEMS locations; one offshore Long Beach, California and the other offshore Point Arguello, California (Sleefe, 1990). The system has measured several hundred small magnitude (M = 3 to 4) earthquakes, providing important insights into the potential locations of earthquake generating faults and into the recurrence rates associated with small magnitude earthquakes on these faults.

The SEMS has recorded strong ground motions from several earthquakes having magnitudes in the range of 5.0 to 5.5. Figure 3 shows the SEMS recording the the Upland California of February 28, 1990 (Magnitude = 5.5). The instrument was located about 80 km from the epicenter (Sleefe, 1990). The early body wave arrivals and later surface wave arrivals are apparent in these recordings.

What was particularly remarkable about this and the other similar records was the magnitude of the vertical component of the motions in the immediate vicinity of the sea floor; they were about a factor of 10 smaller than had been expected based on onshore recordings (Smith, 1991).

Subsequent evaluations and analyses have indicated that the low ratios of vertical to horizontal peak ground accelerations (≤ 5 %) are due to the lack of any significant change in impedance to vertical compressional body waves in the vicinity of the sea floor. The sediments are water saturated; the vertical compression wave components experience no significant change as they propagate from the water saturated sediments into the overlying column of water. Only at the water - air interface is there a significant change in impedance, and vertical compression wave component energy is reflected at this interface back to the sea floor.

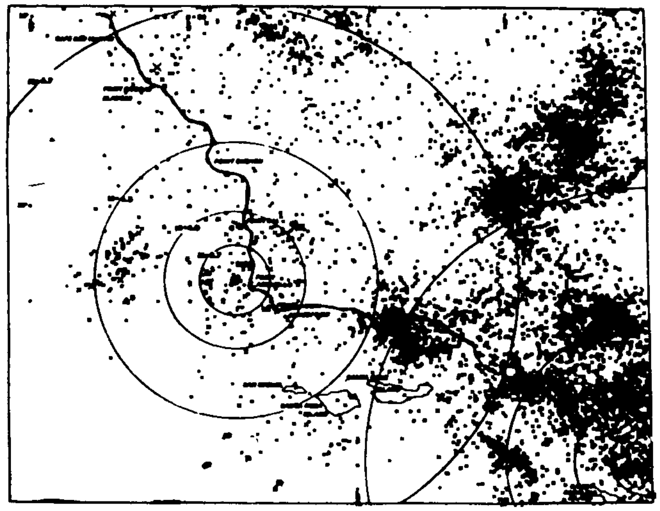


Fig. 2 SEMS Recorded Earthquake Epicenters

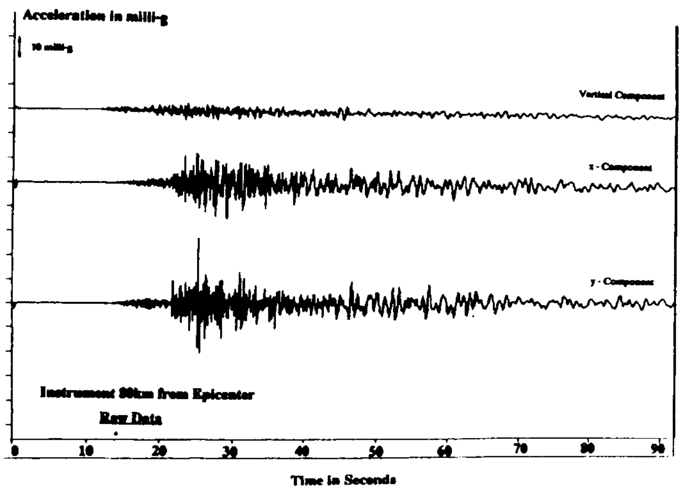


Fig. 3 SEMS Recording of Upland, California Earthquake

This insight has some important ramifications for foundation systems that are sensitive to vertical motions in the vicinity of the sea floor; systems such as mat supported platforms and pipelines. Given seismic exposure and vertical ground motion characterizations based on onshore data, the inferred vertical motions of the soils generally would be overestimated.

As shown in Figure 4, accelerometer systems have also been integrated into the platform structure and foundation components (Husid, et al, 1985). These systems have involved accelerometers mounted on the decks above water and in instrument chutes mounted on the underwater portions of the legs and braces of the platforms. The data are recorded by an onboard data acquisition system, and the data periodically dumped for later analysis onshore. The instrument chutes permit periodic maintenance and reinstrument chutes permit periodic maintenance and replacement of the underwater accelerometer packages; an inevitable requirement for instrumentation located underwater.

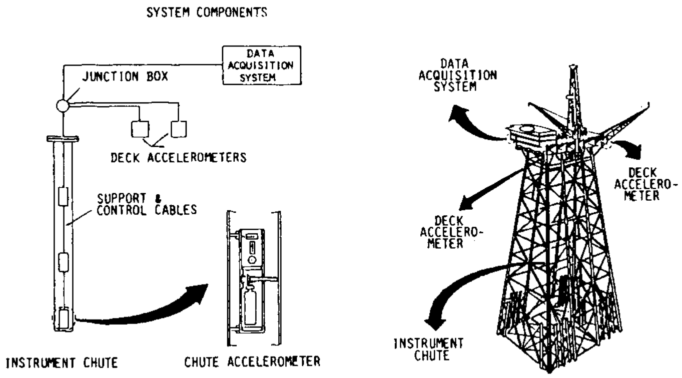


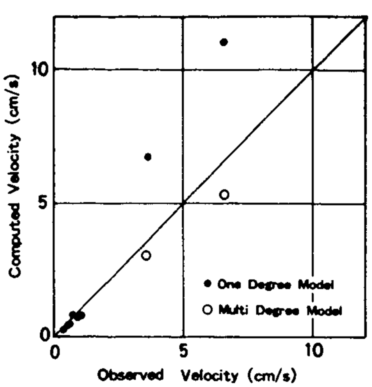
Fig. 4. **Platform Earthquake Response Instrumentation System**

Recordings from accelerometer systems (Figure 5) have provided data to verify platform response analytical models (Ueda and Shiraishi, 1982). In addition, the instrumentation serves the purpose of permitting post earthquake evaluations of platform integrity and providing guides to assist diver underwater inspections of critical platform components.

Much of the data that have been gathered by these earthquake instrumentation systems remains proprietary. Only recently has some of the data began to become publicly available (Smith, 1990). In general, the recordings have tended to confirm the platform system analytical models. The data has indicated that the hydrodynamic mass (or "added mass") effects are somewhat smaller than normally assumed and that the platform system damping is somewhat less than normally assumed (at least for intensities of motions that have been measured). Computed forces and displacements tend to exceed the recorded forces and displacements.

ANALYTICAL MODELS

Developments in the analytical models will be illustrated with an example from recent experience with a platform located in Cook Inlet, Alaska (Figure 6). This platform is typical of those in that region, being comprised of very large (5 m diameter) steel and concrete filled legs interconnected with large vertical and horizontal X-bracing components. The X-bracing is stopped below the water level to facilitate passage of the thick accumulations of ice that confront this structure each winter.



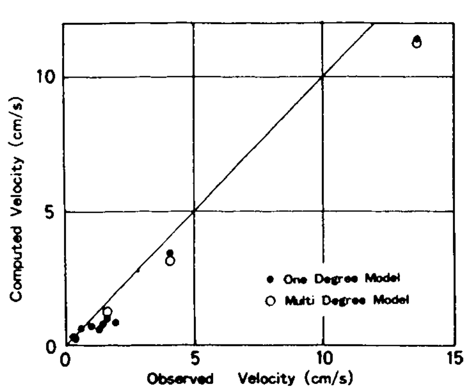


Fig. 5 **Computed and Recorded Maximum Horizontal Velocities at Platform Deck**

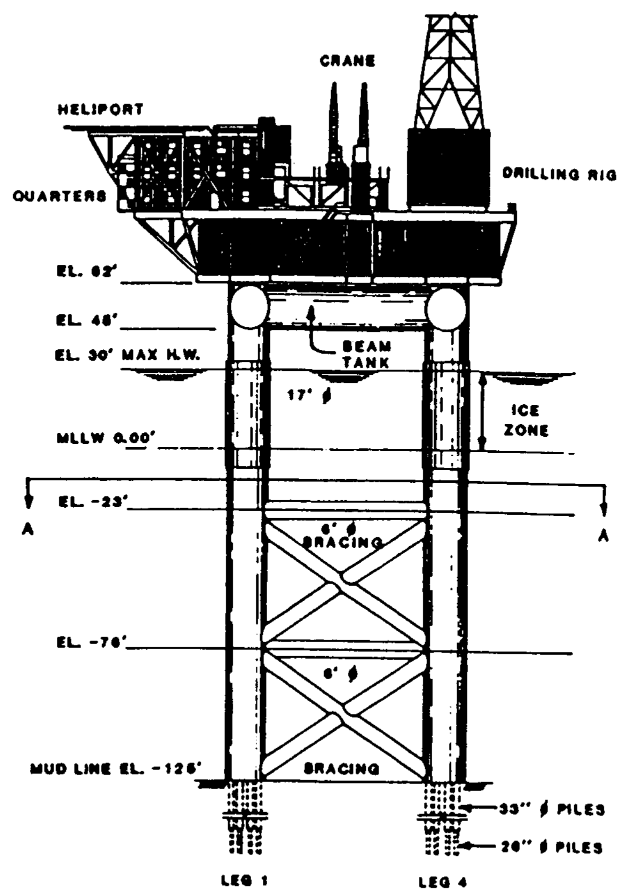


Fig. 6 **Cook Inlet, Alaska Drilling and Production Platform (Elevation)**

Above the main tower is the multi-level deck system that supports the production, drilling, and quarters packages. These packages are supported on very large steel "beam tanks" (5 m diameter) that interconnect the four vertical legs.

Below the main tower is the foundation system. Multi-stage, steel piles pass through the interiors of the legs and penetrate below the sea floor to depths of 250 feet (Figure 7). The piles are laid out on a closely spaced circular pattern of 12 piles.

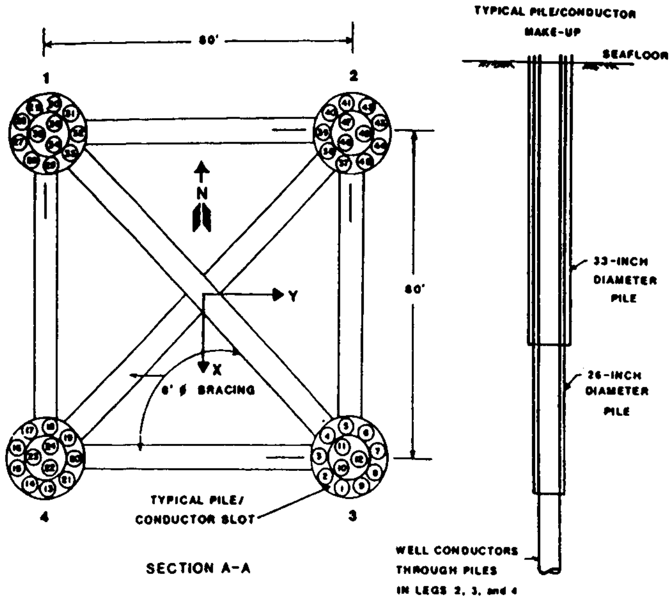


Fig. 7 **Tower Cross Section (Fig. 6) and Elevation of Foundation Pile and Well Conductor System**

Inside the piles are well conductors. The well conductors are comprised of multiple, smaller diameter strings of high strength steel pipe through which drilling is accomplished and the hydrocarbons produced. The well conductors are cemented from the producing level several thousand feet below the sea floor back to the surface. The foundation piles are an extremely strong and ductile composite steel-cemenent-soil/formation component whose axial strength is limited primarily by the strength of the mechanical connectors that connect the various segments of pipe and laterally by the composite strength of the multiple steel pipes as they are supported by the soils.

The soils at this location consist of layered silty clays, sands, gravels, and finally shales at a penetration of 250 feet (Figure 8). The soils increase in strength and stiffness with depth. Soil strengths along the lower one-third of the piles ranges from about 8,000 pounds per square foot (psf) to 10,000 psf. Shear wave velocities (determined using down hole seismic instrumentation) of the soils range from 600 feet per second (fps) near the sea floor to over 1,800 fps in the shales.

Laboratory tests (direct simple shear and triaxial) performed on soil samples obtained at the platform site were used to characterize soil shear modulus, degradation, and damping as a function of shear strain (Figures 9 and 10). The stress-strain behavior of the free-field soils was based on a Ramberg-Osgood characterization with cyclic degradation (Bea, 1984, 1990).

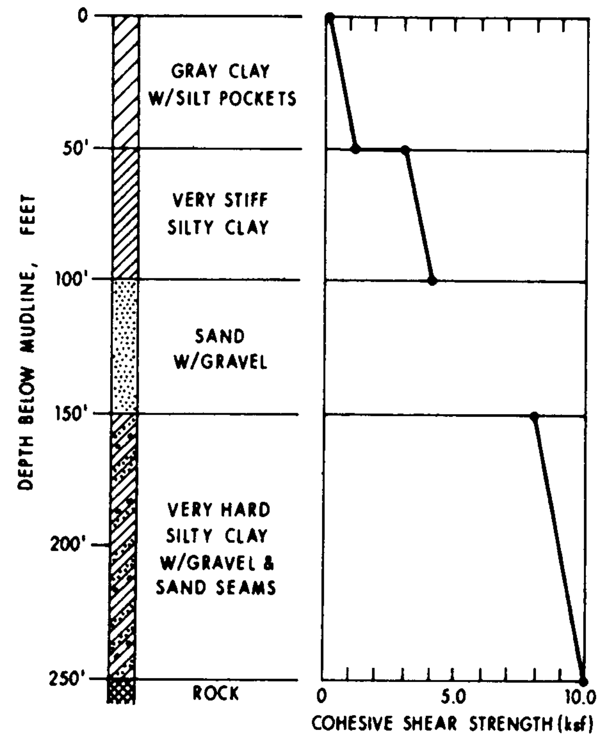


Fig. 8 **Platform Location Soil Characteristics and Undrained Shear Strengths (Triaxial Tests)**

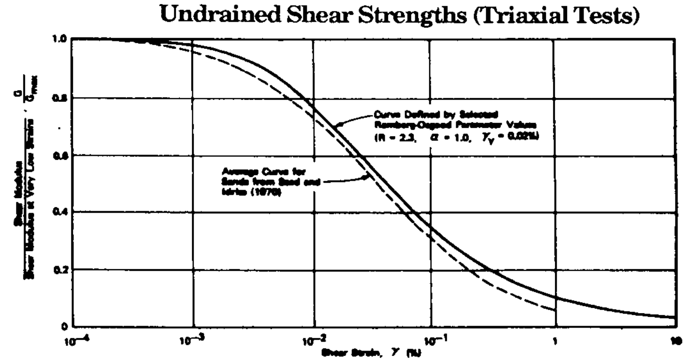


Fig. 9. **Soil Shear Modulus As Function of Cyclic Shear Strain Amplitude**

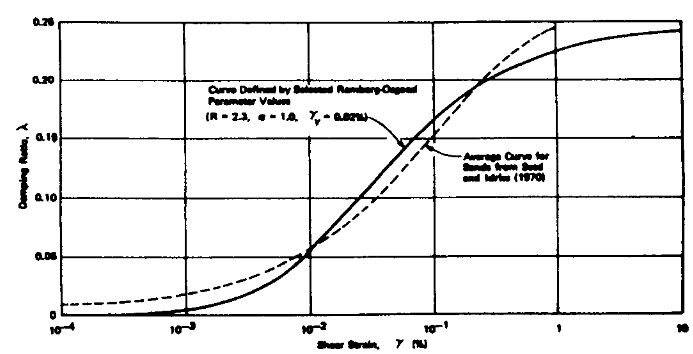


Fig. 10. **Soil Damping Ratio As Function of Cyclic Shear Strain Amplitude**

An overview of the platform analytical model is shown in Figure 11. The model included the free-field soils, the near-field soils, piles, legs, and braces. The platform legs were modeled with nonlinear, inelastic beam column elements. The braces were modeled with nonlinear, inelastic strut elements whose characteristics were based on results from large-scale laboratory tests.

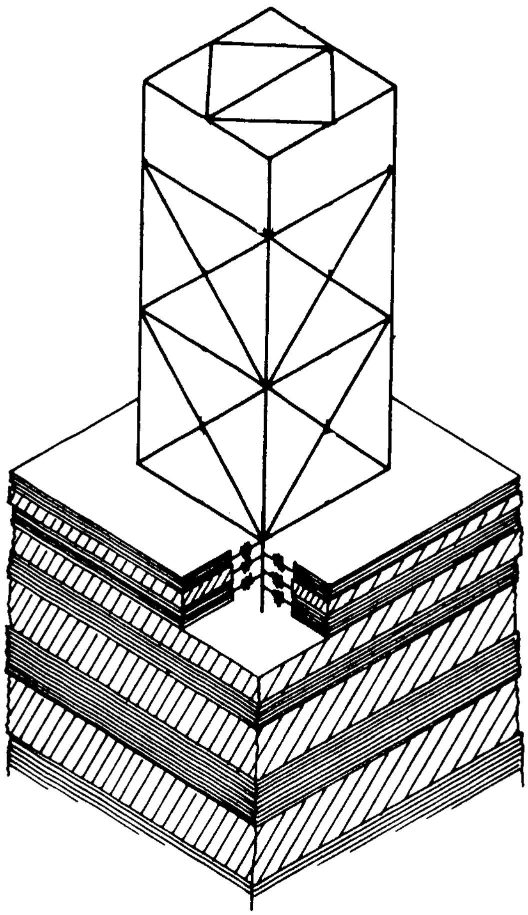


Fig. 11. Coupled (Structure - Foundation - Free-Field Soils) Three-Dimensional Platform Analytical Model

The piles were modeled with nonlinear beam column elements (Figure 12). The near-field soils were characterized with a series of discrete nonlinear, inelastic elements and viscous dampers distributed along the pile length. These elements were connected to companion free-field soil elements and pile elements to allow transmission of forces and displacements along three translational axes. The properties of the near-field soil-pile interactions elements were based on results from model and prototype field dynamic pile load tests on single piles and groups of piles. Detailed guidelines have been developed for definition of these pile - soil interaction elements (API, 1991).

The first two modes of the platform system dynamic response are illustrated in Figure 13. The first two periods of the platform (orthogonal lateral) were approximately 2 seconds. The importance of the flexibility and interactions of the pile - soil elements are evident in the exaggerated displacements exhibited in these first two modes.

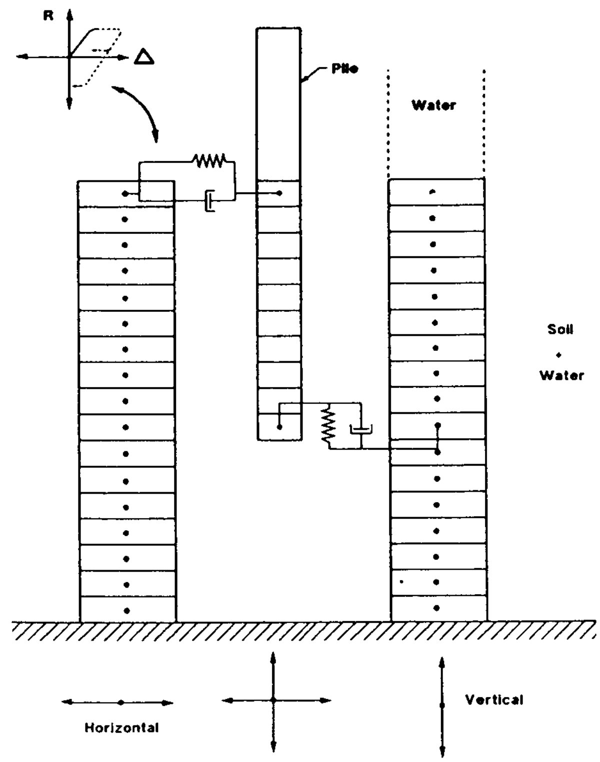


Fig. 12. Foundation Pile, Near-Field, Free-Field Soils Elements (Two Dimensional Shown for Clarity)

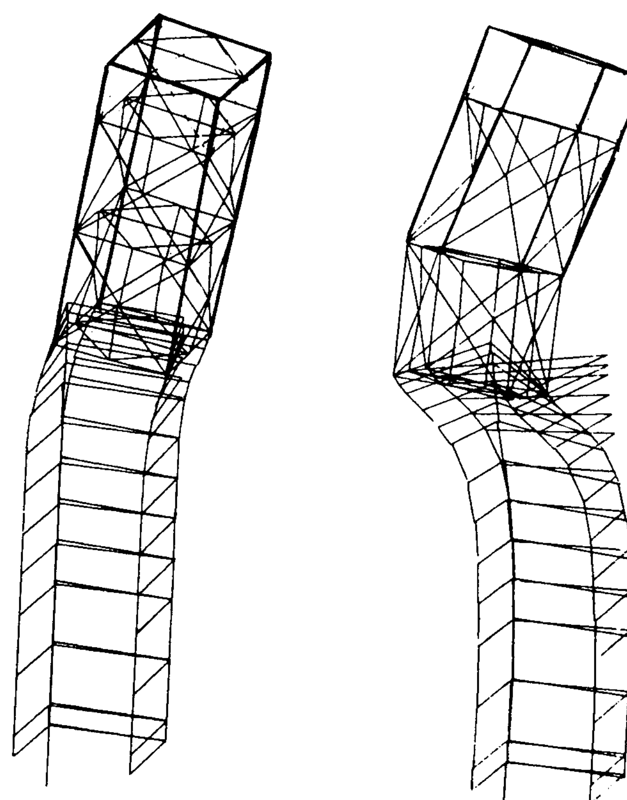


Fig. 13. First Two Platform Dynamic Response Modes

A series of "static, push-over" analyses were performed on the platform system to determine at what loadings and displacements and where the major nonlinear developments might be expected in this system (Figure 14). In these analyses, a series of lateral and vertical loading vectors are established based on the results from elastic response spectra analyses. The lateral loading pattern is uniformly increased until the platform collapses (not able to support the vertical loadings).

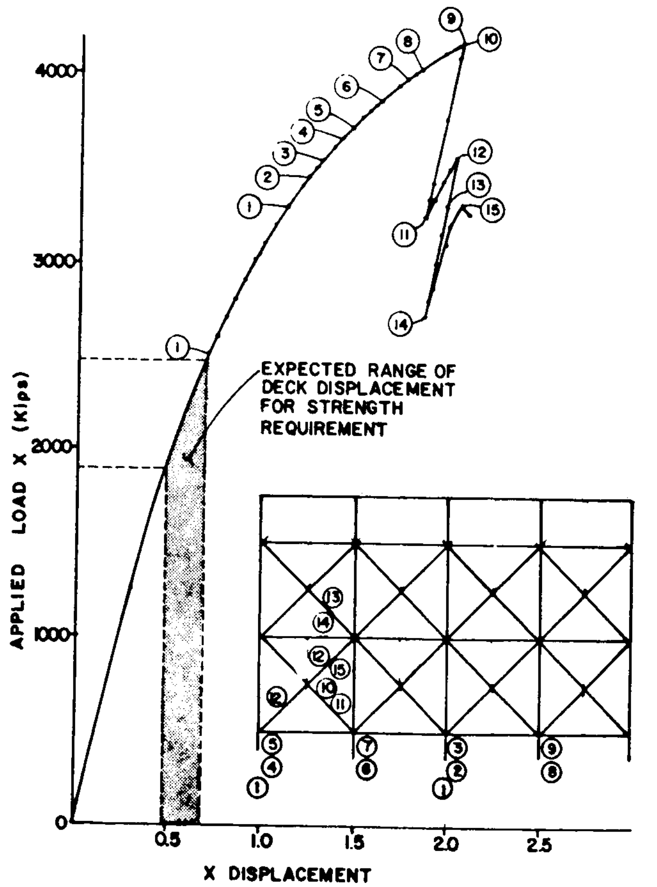


Fig. 14. **Static Push-Over Nonlinear Analysis Results (X - Horizontal Displacement Measured at Top Deck)**

While this type of analysis fails to capture dynamic, transient, and cyclic loading effects, it can provide important insights into the performance characteristics of the platform system at ultimate limit states leading to collapse of the system.

For this platform, the static, push-over analyses indicated that the first nonlinear action would develop in the piles supporting the platform. The first 9 nonlinear events were all concentrated in the foundation piles. The tenth nonlinear event involved the buckling of a primary diagonal brace, and after this point, the nonlinearity was rapidly (and with decreasing load resistance) distributed into the diagonal bracing system.

Three seismic exposure studies were performed for the platform location. This involved definition of seismic sources, characterization of intensity - recurrence relationships for each of the sources, and evaluation of limiting source intensities. Empirical attenuation relationships were adopted for each of the major classes of sources: shallow sources, and deep sources concentrated on the subduction zone that underlies the platform location. The attenuation relationships were evaluated for soil conditions similar to those at the platform site.

The results are summarized in Figure 15. The expected annual maximum horizontal elastic spectral velocity (PSV) for a period of T = 2 sec. and damping ratio of D = 5 % is a shown as a function of the return period in years. This PSV was chosen as an index to reflect the intensity of the earthquakes because it reflects the force and displacement producing potential of the earthquake ground motions on the platform. Current studies indicate that the PSV for a specified elastic period, damping, and ductility (ratio of maximum allowable plastic strain/displacement to elastic strain/displacement) can provide an even better index for the effects of very intense earthquakes.

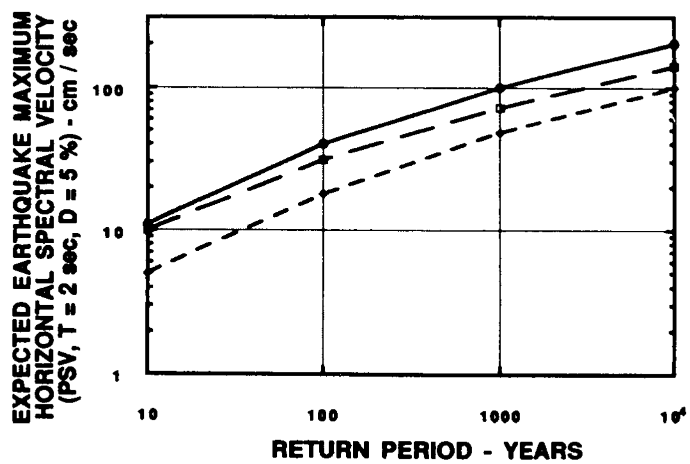


Fig. 15. **Seismic Exposure Characterizations for Platform Location**

The PSV range at a given return period is a factor of about two for the range of return periods shown. This range is due to differences in the evaluations of source, transmission, and local site - geology effects in the three different seismic exposure assessments. The major sources of these differences are founded in the attenuation or transmission relationships used for the different sources and the limiting or maximum magnitudes attributed to the major nearby sources. These are chiefly modeling and parametric uncertainties and are not reflective of inherent or natural variabilities associated with intense earthquake ground motions. The differences can only be reduced by gathering additional information to better define the source, transmission, and local site effects.

The seismic exposure evaluations developed equal probability elastic response spectra (Figure 16) that were used to scale different recorded three component ground motions to have intensities comparable with those indicated by the equal probability elastic response spectra. The recorded ground motions were chosen to replicate effects of intense earthquake sources, travel paths, and local geology that could affect the platform.

The scaled recorded ground motions were introduced at the base of the analytical model, and the forces (Figure 17) and displacements determined in the platform system as a function of time through the earthquakes.

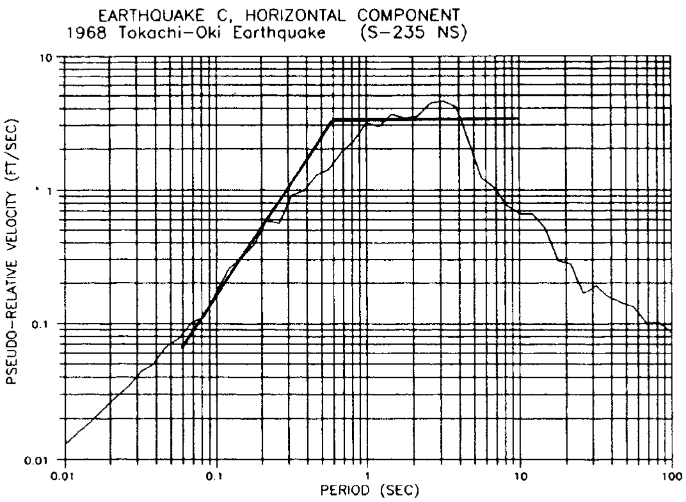


Fig. 16. Equal Probability Response Spectrum (1,000 Year) and Example Scaled Ground Motion Time History Spectrum

The results for the example ground motion (1940 El Centro scaled to peak ground acceleration of 0.5 g) indicates that the platform acts as a high frequency filter. The platform system force-time histories primarily reflect the effects of the first two lateral response modes (recall both have periods of approximately 2 sec.). At this ground motion intensity, nonlinearity in the foundation has resulted in increasing the effective period of the system. The overturning moment (at the sea floor) time histories reflect the close coupling of the first two lateral response modes.

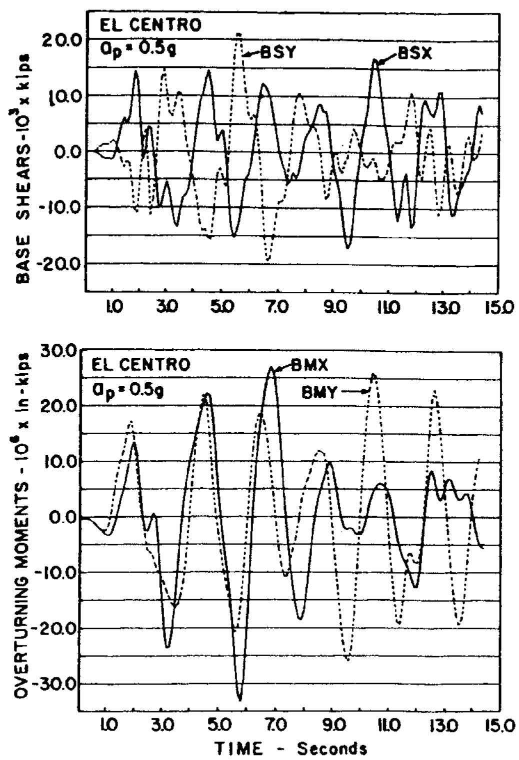


Fig. 17. Base Shear and Overturning Moment at the Sea Floor as a Function of Time Through the Earthquake

Figure 18 shows the peak horizontal displacement profiles of the piles and soil column during the El Centro ground motion scaled to PGA of 0.33 g and 0.5 g. The pile horizontal displacements exceed those of the soil primarily in the upper 100 feet of the soil column. Below this depth, the pile essentially follows the movements of the soil column.

Figure 19 shows the maximum combined stresses (axial tension - compression, and bending) in the piles. The stresses are shown for three different earthquake ground motions (all scaled to produce the same PSV). The high stresses are concentrated at three points along the pile; near the top, near the mid point of the pile, and near the bottom of the pile.

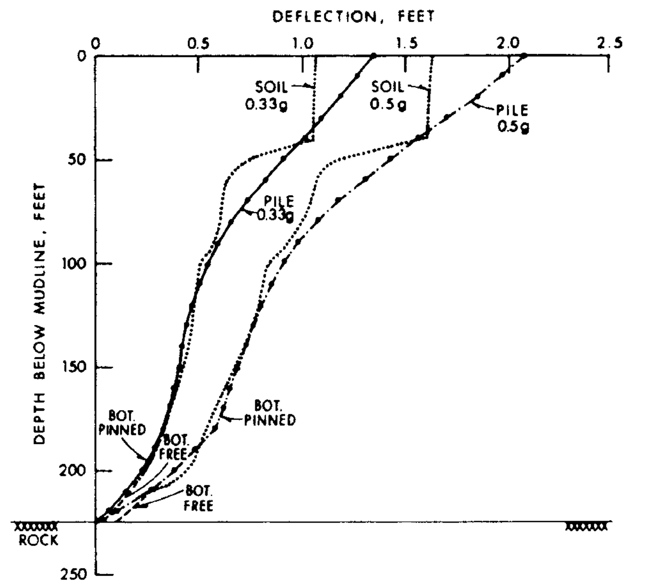


Fig. 18. Maximum Pile - Soil Horizontal Displacements

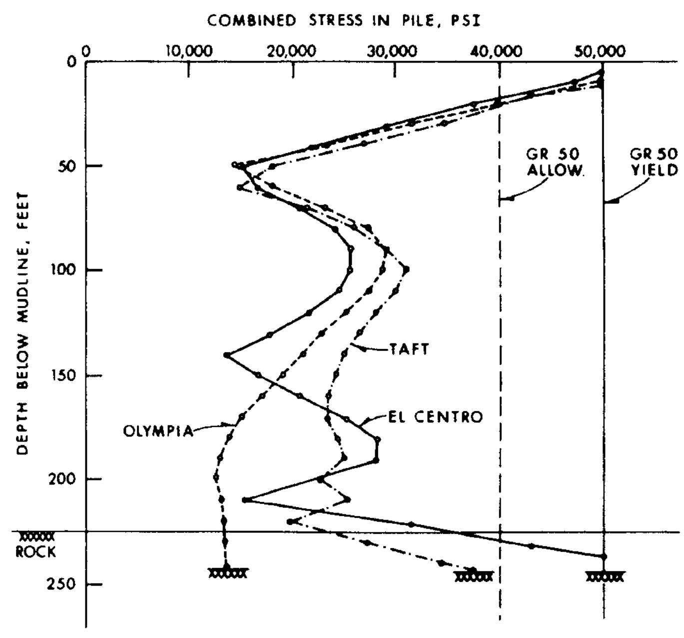


Fig. 19. Combined Stresses in Pile For Three Recorded Earthquake Time Histories

The high stresses at the pile top are due to the flexural restraint provided by the structure as the piles enter the bottom of the platform legs. The high stresses in the pile mid section are due to the soil restraint provided by the dense sands below 100 feet. The high stresses in the pile bottom are due to the restraint provided by the rock in which the piles are tipped.

For these ground motions, at the pile top and bottom, the pile combined stresses equal the nominal yield strength of the steel. The pile wall thickness must be varied to provide adequate strength to resist the stresses developed in these different zones of the soil column.

Figure 20 shows a typical lateral force - deformation and lateral force - time plot of the near field soil-pile interaction elements near the top of the pile (for the El Centro event scaled to 0.5 g). Note the "gapping" hysteretic behavior of the soil-pile elements that is developed as the pile wallows a hole between itself and the soils. Note the relatively few pulses of maximum forces that are developed to degrade the resistance and stiffness of the soils.

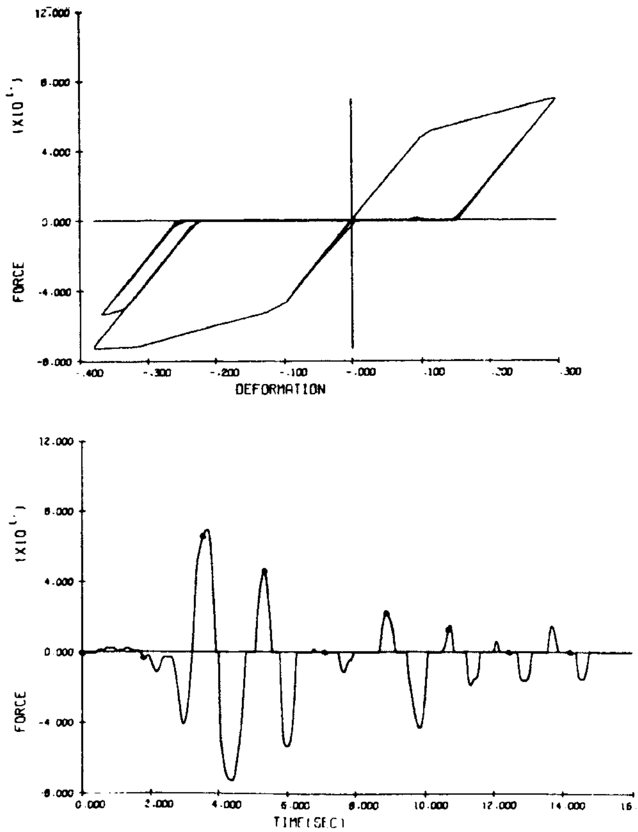


Fig. 20. Pile - Soil Lateral Interaction Force - Time and Force - Deformation Plots For Elements Near Sea Floor

Figure 21 shows a typical axial force - deformation and axial force - time plot of the near field soil -pile interaction elements at the bottom of the piles (for the El Centro event scaled to 0.5 g). The pile-soil elements are repeatedly cycled from tension to compression. There is limited degradation in strength and stiffness of the elements due to this repeated tension - compression cycling (Bea, 1990). At the end of the earthquake time history, the pile has settled and there is a residual force locked into the pile.

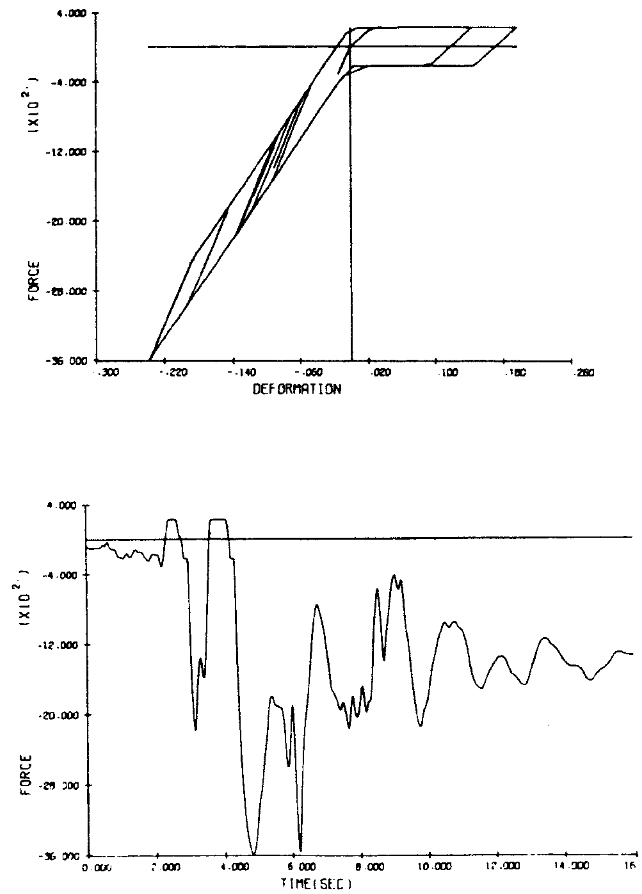


Fig. 21. Pile - Soil Axial Interaction Force - Time and Force - Deformation Plots For Elements Near Pile Tip

Evaluation of the response of the platform system to a variety of earthquake time histories indicated that the critical model of behavior was focused in the lateral displacements of the piles at the sea floor. Figure 22 summarizes the peak and residual lateral displacements of the piles during four of the earthquake time histories studied.

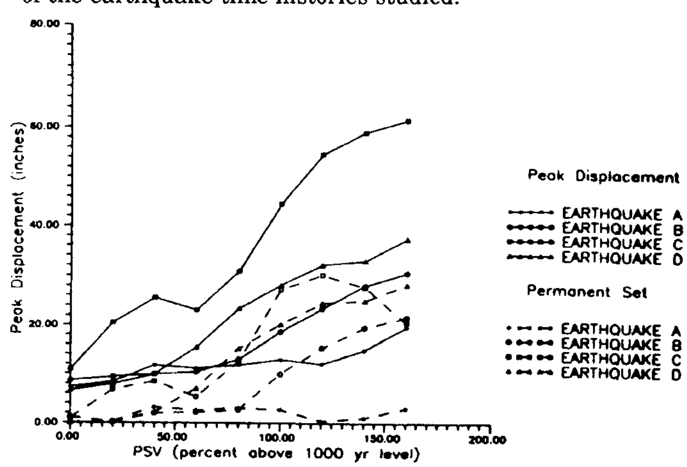


Fig. 22. Peak Horizontal Displacements Induced In Piles For Four Earthquake Time Histories As a Function of Earthquake Intensity

The piles and the well tubulars contained inside the piles were capable of developing a peak horizontal displacement at "failure" of 40 to 50 inches. At such displacements, the internal well strings would be ruptured (failure of the mechanical connectors). The results indicate that a peak horizontal displacement of 40 inches would be developed "on the average" by earthquakes having a return period in excess of 10,000 years.

Note the wide variation between the peak displacements produced by the four earthquakes. Even though all of these earthquakes have been scaled to produce identical PSV at the elastic natural period of the platform, they produce very different peak displacements as the intensity of the earthquakes increases. Fundamentally, this is due to the change in the effective stiffness of the platform system as additional nonlinearity is developed in the system. In the case of some time histories, the increase in intensity is partially offset by decreased effective stiffness; the increased period results in the system "sliding into a valley in the response spectra." However, in the case of some time histories, the increase in intensity and decrease in stiffness result in the system "climbing a hill in the response spectra."

Note that at the most intense level studied, there is more variation between the peak displacements produced by the different time histories than there is variation in the average peak displacement developed for 170 % increase in the PSV. The variation in peak displacements produced by the different time histories swamps out the variation attributed to the increase in intensity. This is a natural variability caused by the inherent variability in earthquake time histories, and can not be easily reduced.

CONCLUSIONS

Recent developments in earthquake geotechnology in offshore structures have been focused in improvements in analytical capabilities and instrumentation. Data from field instrumentation programs is becoming available to further refine and verify analytical capabilities.

The state-of-art and state-of-practice in earthquake geotechnology in offshore structures are very close to each other; recent research advances have been rapidly incorporated into practice. The guidelines presently used to design and evaluate the type of offshore platforms discussed in this paper represent some of the most advanced approaches incorporated into earthquake engineering practice. Specific guidelines have been developed for the conduct of seismic exposure evaluations including seismotectonic and site characterizations, seismic exposure assessment, ground motion characterization, and design ground motion specifications. Guidelines for response spectra based methods have been defined to assist engineers in incorporating adequate strength and ductility into the platform. In addition, guidelines for the performance of nonlinear time history analyses of structure and foundation elements and systems have been developed; these analyses are intended to confirm that the platform system has adequate deformation and load capacity.

Although limited, experience with platforms subjected to earthquakes indicates that these engineering guidelines and processes are producing platforms having sufficient load resistance and ductility. Available data indicates that the analytical methods tend to produce conservative results. Perhaps most importantly, the performance of platforms that have been subjected to comparably and even more intense forces from storm waves and ice indicates that the design and construction processes are producing structures that have adequate strength and ductility.

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