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Verification of a Simplified Method to Evaluate the Capacities of Template-Type Platforms

R. G. Bea, M. M. Mortazavi, K. J. Loch, and P. L. Young
Department of Civil Engineering, University of California at Berkeley

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ABSTRACT

This paper summarizes development of simplified procedures to evaluate storm loadings imposed on template-type platforms and to evaluate the ultimate limit state lateral loading capacities of such platforms. Verification of these procedures has been accomplished by comparing results from the simplified analyses with results from three-dimensional, linear and nonlinear analyses of a variety of template-type platforms. Good agreement between results from the two types of analyses has been developed for the evaluations of both loadings and capacities.

The verification platforms have included four-leg well protector and quarters structures and eight-leg drilling and production Gulf of Mexico structures that employed a variety of types of bracing patterns and joints. Several of these structures were subjected to intense hurricane storm loadings during hurricanes Andrew, Carmen, and Frederic. Within the population of verification platforms are several that failed or were very near failure. The simplified loading and capacity analyses are able to replicate the observed performance of these platforms. Realistic simulation of the brace joints and foundation capacity charcteristics are critical aspects of these analyses. There is a reasonable degree of verification of the simplified methods with the observed performance of platforms in the field during intense hurricane storm loadings.

These methods can be used to help screen platforms that are being evaluated for extended service. In addition, the results from these analyses can be used to help verify results from complex analytical models that are intended to determine the ultimate limit state loading capacities of platforms. Lastly, and perhaps most importantly this approach can be used in the preliminary design of new platforms.

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INTRODUCTION

During the past three decades, an immense amount of effort has been devoted to development of sophisticated computer programs to enable the assessment of storm wind, wave, and current loadings and the ultimate limit state capacity characteristics of conventional, pile-supported, template-type offshore platforms.[1-3] These programs require high degrees of expertise to operate properly, are expensive to purchase and maintain, and require large amounts of manpower and time to complete the analyses. Due to the sophistication of these programs, experience has shown that it is easy to make mistakes that are difficult to detect and that can have significant influences on the results.[4]

This paper summarizes the second phase of verification of simplified procedures to evaluate environmental loadings and ultimate limit state lateral loading capacities of template-type platforms. Reasonable simplifications and high degrees of "user friendliness" have been employed in development of the computer software to reduce the engineering effort, expertise, and costs associated with the analyses.

define major deficiencies and errors in either the complex analysis software or in the input to this software. Based on this experience, there is little doubt in the researchers' minds concerning the importance and utility of simplified methods.

Input Information

The geometry of the platform is defined by specifying a minimum amount of data by the user. These include the effective deck areas, the proportion and topology of jacket legs, braces, and joints, and of the foundation piles and conductors. The projected area characteristics of appurtenances such as boat landings, risers, and well conductors also must be specified. If marine fouling is present, the variation of the fouling thickness with depth may be specified by the user.

Specialized elements may be designated including grouted or ungrouted joints, braces, and legs. In addition, damaged (corrosion, holes, dents, bent, cracked) or defective elements (misalignments, under-driven piles) can be included. Dent depth and initial out-of-straightness are specified by user for braces with dents and global bending defects. User-defined element capacity reduction factors are introduced to account for other types of damage to joints, braces, and foundation elements.

Steel elastic modulus, yield strength, and effective buckling length factor for vertical diagonal braces are specified by the user. Soil characteristics are specified as the depth variation of "effective" undrained shear strength (for cohesive soils) or the "effective" internal angle of friction (for cohesionless soils). The effective soil characteristics are intended to recognize bias introduced by soil sampling, laboratory testing, and static analysis methods. A scour depth can be specified by the user.

Storm wind speed at the deck elevation, wave height and period, current velocity profile, and storm water depth are defined by the user. These values are assumed to be collinear and to be the values that occur at the same time. Generally, the load combination is chosen to be wind speed component and current component that occur at the same time and in the same principal direction as the expected maximum wave height. The wave period is generally taken to be expected period associated with the expected maximum wave height.

To calculate wind loadings acting on the exposed decks the user must specify the effective drag coefficient. Similarly, the user must specify the hydrodynamic drag coefficients for smooth and marine fouled members. User specified coefficients can also be introduced to recognize the effects of wave directional spreading and current blockage.

Environmental Loadings

Wave, current, wind, and storm tide are considered. Aerodynamic and hydrodynamic loadings are calculated according to API RP 2A guidelines.[9,10]

Wave horizontal velocities are based on Stokes 5th order theory. The specified variation of current velocities with depth is stretched to the wave crest and modified to recognize the effects of structure blockage on the currents. The total horizontal water velocities are taken as the sum of the wave horizontal velocities and the current velocities.

The maximum hydrodynamic force acting on the portions of structure below the wave crest are based on the fluid velocity pressure or drag component of the Morison Equation.

All of the structure elements are modeled as equivalent vertical cylinders that are located at the wave crest. Appurtenances (conductors, boat landings, risers) are modeled in a similar manner. For inclined members, the effective vertical projected area is determined by multiplying the product of member length and diameter by the cube of the cosine of its angle with the horizontal (to resolve horizontal velocities to normal to the member axis).

For wave crest elevations that reach the lower decks, the horizontal hydrodynamic forces acting on the lower decks are computed based on the projected area of the portions of the structure that would be able to withstand the high pressures.[11,12] The fluid velocities and pressures are calculated in the same manner as for the other submerged portions of the structure with the exception of the definition of the drag coefficient, Cd. In recognition of rectangular shapes of the structural members in the decks a higher Cd is taken. This value is assumed to be developed at a depth equal to two velocity heads (U^2/g) below the wave crest. In recognition of the near wave surface flow distortion effects, Cd is assumed to vary linearly from its value at two velocity heads below the wave crest to zero at the wave crest.[11]

Deck Leg Shear Capacity

The ultimate shear that can be resisted by an unbraced deck portal is estimated based on bending moment capacities of the tubular deck legs that support the upper decks.

A collapse mechanism in the deck bay would form by plastic yielding of the leg sections at the top and bottom of all of the deck legs. The interaction of bending moment and axial force is taken into account. The maximum bending moment and axial force that can be developed in a tubular deck leg is limited by local buckling of leg cross-sections.

action in the deck portal. Given the geometry of the deck portal and the load acting on deck areas, the moment distribution along the deck legs is estimated. Thinking of a jacket leg as a continuous beam which is supported by horizontal framing, the applied moment at the top of the leg rapidly decreases towards the bottom. Based on geometry of the structure, in particular jacket bay heights and the cross-sectional properties of the jacket leg (if non-prismatic), and in the limiting case of rigid supports, an upper-bound for the desired moment distribution is estimated.

The braces are treated as though there are no net hydrostatic pressures (e.g. flooded members). Based on a three-hinge failure mode, the exact solution of the second order differential equation for the bending moment of a beam-column is implemented to formulate the equilibrium at collapse.

Elasto-perfectly plastic material behavior is assumed. The ultimate compression capacity is reached when full plastification of the cross-sections at the member ends and mid-span occurs. It is further assumed that plastic hinges at member ends form first followed by plastic hinge formation at mid-span.

The results have been verified with results from the nonlinear finite element program USFOS.[13,14] Using the same initial out-of-straightnes for both simplified and complex analyses, the axial compression capacity of several critical diagonal members of different structures has been estimated. The simplified method slightly over-predicts the axial capacity of compression members (less than 10%).

Given the conservative formulation of buckling capacities when compared with test data (refer to Commentary D in API RP 2A-LRFD guidelines)[9], this over-prediction may in fact be closer to the expected or best estimate capacity.

In case of dent damaged braces or braces with global bending damage, the axial capacity is reduced according to the equations given by Loh[15] which were developed for evaluating the residual strength of dented tubular members. The unity check equations have been calibrated to the lower bound of all existing test data. The equations cover axial compression and tension loading, in combination with multi-directional bending with respect to dent orientation.

Tubular Joint Capacity

The stress analysis of the circular tubular joints and the theoretical prediction of their ultimate strength has proven to be difficult. Hence, empirical capacity equations based on test results have often been used to predict the joint ultimate strength. For simple tubular joints with no gussets, diaphragms, or stiffeners, the capacity equations given in the API RP 2A LRFD guidelines are used (1993).

It is generally recognized that the equations for joint capacity are conservative. Bias factors (true capacity / nominal or guideline capacity) are provided in ULSLEA so that the user can utilize the expected or best estimate capacities of the elements to determine the capacity of the platform components (deck legs, jacket, foundation).

Pile Capacity

The pile shear capacity is based on an analysis similar to that of deck legs with the exception that the lateral support provided by the foundation soils and the batter shear component of the piles are included. Virtual work based limit equilibrium equations have been developed to characteize the ultimate limit state lateral loading capacity of piles embedded in cohesive and cohesionless soils.

The horizontal batter component of the pile top axial loading is added to estimate the total lateral shear capacity of the piles. This component is computed based on axial loads carried by the piles due to storm force overturning moment.

The axial resistance capacity of a pile is based on the combined effects of a shear yield force acting on the lateral surface of the pile and a normal yield force acting over the entire base end of the pile.

It is assumed that the pile is rigid and that shaft friction and end bearing forces are activated simultaneously. Correction factors can be introduced to recognize the effects of the pile shaft flexibility.

It is further assumed that the spacing of the piles is sufficiently great so that there is no interaction between the piles (spacing to diameter ratios exceed approximately 3). In the case of compressive loading, the weight of the pile and the soil plug (for open-end piles) is deducted from the ultimate compressive loading capacity of the pile. For open-end piles, the end bearing capacity is assumed to be fully activated only when the shaft frictional capacity of the internal soil plug exceeds the full end bearing.

PLATFORM VERIFICATIONS

In this paper we summarize results from five second generation analysis and verification studies of Gulf of Mexico template-type platforms. The verification cases include two eight-leg and one four-leg drilling and production platforms, and two, four-leg well protectors. These structures are identified as platforms 2A through 2E.

The simplified estimates of total forces acting on the platforms during intense storms and predictions of ultimate

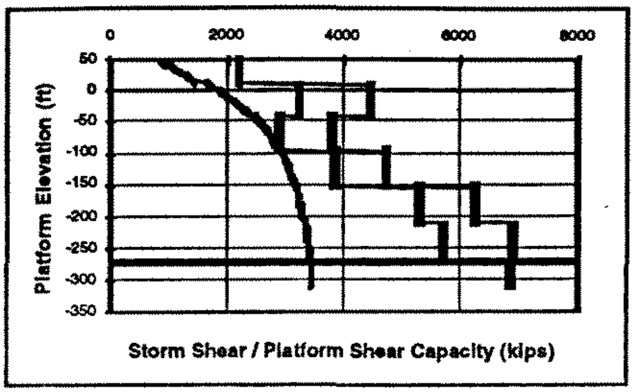


FIGURE 2: PLATFORM 2A BROADSIDE STORM SHEARS AND PLATFORM SHEAR CAPACITIES

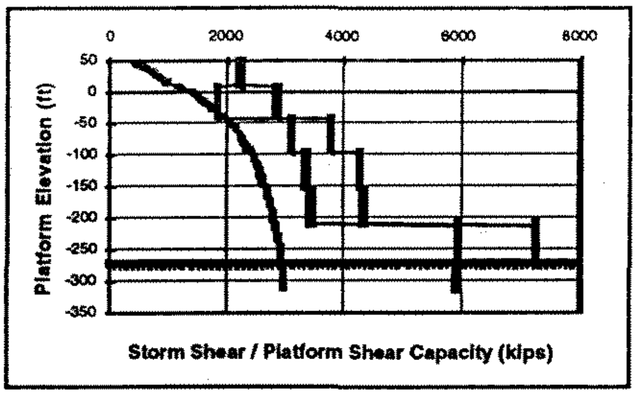


FIGURE 3: PLATFORM 2A END-ON STORM SHEARS AND PLATFORM SHEAR CAPACITIES

These results are 10 to 15% higher than those gained from detailed nonlinear analyses.[4] The principal difference lies in the nonlinear modeling of vertical diagonal braces which results in different buckling loads.[13]

Both the ULSLEA and detailed nonlinear analysis results are in conformance with the observed performance of the platform during hurricane Frederic. The platform survived this storm without significant damage and the results of the analyses indicate that it should have.

PLATFORM 2B

Platform 2B is an eight-leg structure located in a water depth of 118 ft.[18] The platform was designed using a design wave height of 55 ft. The cellar and main decks are located at +34 ft. and +47 ft., respectively. The 39 in. diameter jacket legs are battered in two directions and have no joint cans. The 36 in. diameter piles are grouted inside the jacket legs.

This platform sustained severe loadings from hurricanes Carmen (1974) and Andrew (1992).[18] The maximum wave height at the platform during hurricane Andrew was estimated to be 59 ft.[18, 19] The estimated maximum total lateral loading on the platform during hurricane Andrew was estimated to be approximately 3,700 kips. Damage sustained during Andrew indicated that the platform was loaded so that the upper bay of K-brace joints were loaded into the nonlinear range with two of the joints reaching their ultimate capacity.[18]

Nonlinear push-over analysis results summarized in Figure 4 indicated that the platform is capable of resisting approximately 3,900 kips in broadside loading.[4] The failure mechanism occurs in the uppermost jacket bay due to buckling of the compression braces and the associated joints. The analysis indicates a brittle strength behavior and little effective redundancy.

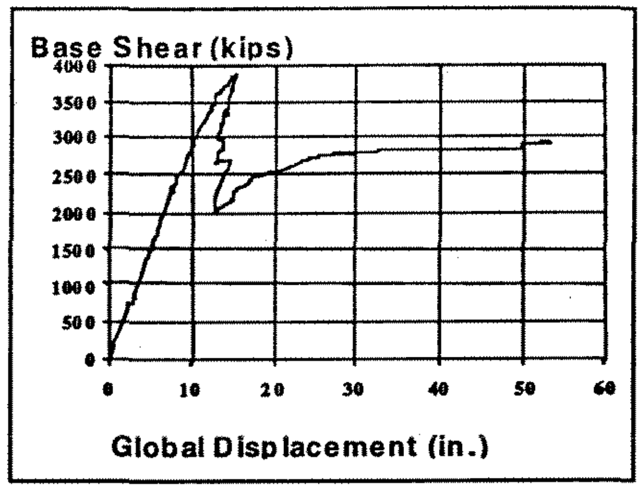


FIGURE 4: PLATFORM 2B BROADSIDE FORCE - DISPLACEMENT RELATIONSHIP

These results can be compared with those published by Imm, et al.[18] Their broadside static-push over analysis was based on an Andrew loading pattern that did not involve deck loadings. The static push-over analyses reported here did involve deck loadings.[4] The results reported by Imm et al.[18] indicated a total lateral loading capacity of approximately 4,900 kips. As noted by Imm, et al., the loading pattern used to perform the static push-over analyses can have a marked influence on the ultimate limit state performance of the structure. In this case, the lateral loading capacity involving deck loadings is 80 % of the lateral loading capacity without deck loadings.

The predicted lateral loading capacity and failure mode is in agreement with the observed platform performance in hurricane Andrew.

PLATFORM 2C

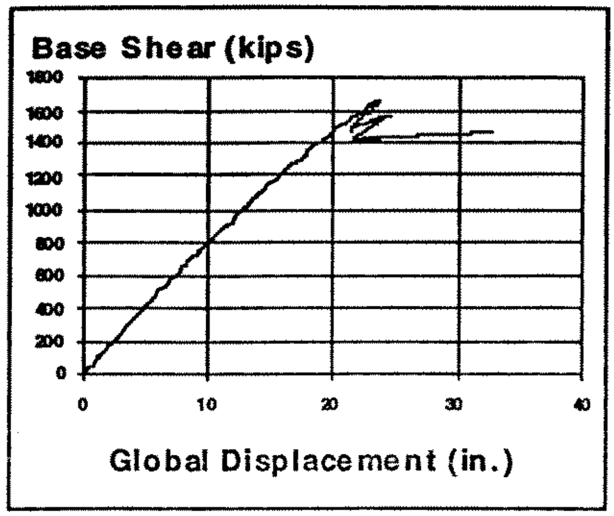
Platform 2C is a four pile drilling and production platform. It was installed in the Gulf of Mexico Ship Shoal region in a water depth of 157 ft. in 1971. The platform has four decks at elevations +33 ft., +43 ft., +56 ft., and +71 ft. The deck legs form a 30 ft. By 30 ft. Plan and the jacket legs are battered in two directions (1:11) and have joint cans. The leg-pile annulus is ungrouted and the 36-in. Diameter piles are attached to the jacket with welded shimmed connections at the top of the jacket. The vertical bracing is comprised of horizontal K-braces.[4]

The piles extend 355 ft. Below 28 ft. of soft to stiff gray clay and 27 ft. of fine dense sand. The sand layer starts at 197 ft. Below the mudline. The clay above the sand is generally soft and silty, while the clay below the sand is stiff to very stiff.

This platform was located close to the track of hurricane Andrew. The estimated wave height at the platform location was estimated to be approximately 60 ft. The platform survived the storm without significant damage.

This platform has been the subject of extensive structural analyses.[20] As part of an industry wide effort to assess the variability in predicted performance of offshore platforms in extreme storms, the storm loadings and ultimate capacity of this "benchmark" platform has been assessed by 13 qualified investigators using a variety of nonlinear analysis software packages. All of the analysts were given the same platform drawings, soil conditions, and oceanographic conditions. It was specified that the storm loadings should be computed according to API guidelines.[9,10] It is noteworthy that the range of broadside lateral loading capacities was from 1,600 kips to 3,400 kips; a range in excess of 2 (mean value of 2,400 kips with Coefficient of Variation of 22 %).

Platform 2C was analyzed using USFOS.[14] As for all of the nonlinear analyses, an attempt was made to use "unbiased" characterizations for all loading and capacity factors to develop best estimate lateral loadings and capacities. The results from the USFOS static push-over analyses of platform 2C are summarized in Figures 8 and 9. These results indicated a maximum total lateral loading of 2,900 kips and a lateral capacity of 1,700 kips to 3,400 kips. This range brackets the range developed in the "benchmark" study.[20]



DISPLACEMENT RELATIONSHIP BASED ON STATIC PILE CHARACTERISTICS

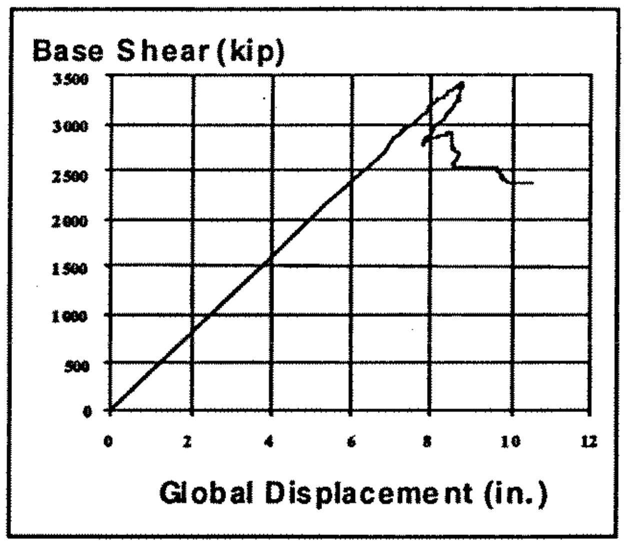


FIGURE 9: PLATFORM 2C FORCE - DISPLACEMENT RELATIONSHIP FOR DYNAMIC PILE CHARACTERIZATION

The range in lateral capacity was a function of how the foundation piles were modeled. If "static" capacities were utilized (based on the sampled soil strength test results and static pile capacity methods)[21], the initiating failure mode was in the foundation and the lower lateral loading capacity resulted. If "dynamic" capacities (based on corrected soil strength results to reflect the sampling disturbance and cyclic - dynamic loading effects) were utilized[21-23], the initiating failure mode was in the jacket and the upper lateral loading capacity resulted. As found in previous analyses[7,8], the methods used to evaluate and model the performance during hurricane Andrew. This motivated a detailed study of the platform construction and installation records. During this study, it was discovered that the piling on the south side of Platform 2D had been under-driven by 5 to 10 ft. This finding was integrated into the analyses reported here. This experience pointed out the importance of having very detailed information on platforms that are loaded close to their ultimate limit states. Without such information, observations of failures and non-failures might be attributed to "probabilistic reasons"[27] when the real reasons are founded in deterministic characteristics.

The two structures were loaded only along their principal axes to provide consistency between the various approaches employed to analyze structural response. Wave loads for USFOS were generated by the program WAJAC.[16] The global base shears developed on Platform 2D and Platform 2E during the passage of Andrew were based on hindcast study results.[19] The results indicated Platform 2D experienced peak lateral loadings that were about 20 % larger than those on Platform 2E. During hurricane Andrew, the hindcast peak lateral loading on Platform 2D was 1,100 kips and on Platform 2E was 850 kips.[4]

The static push-over results for Platform 2D and Platform 2E based on the USFOS results are summarized in Figure 11. The "double humps" in the load - displacement results are due to the increased stiffness of the structures when contact between the jacket and caissons occur. The negative stiffness found at the end of all analyses represents pile pullout. The large lateral deformations produce plastic hinges in the piles which produce a near mechanism. It is the additional strength and rigidity of the caissons which prevents the structures from soft story collapse. This added stiffness allows the full axial capacity of the soils to be exceeded to produce pile pullout.

The USFOS results indicated that the maximum lateral load capacity of Platform 2D (end-on and broadside loadings) is 910 kips and Platform E 880 kips.

The USFOS results indicated that the ratio of the peak lateral loading during hurricane Andrew to the maximum lateral loading capacity is 1.2 and 0.95 for Platform 2D and Platform 2E, respectively. The analyses indicate that Platform D should have failed due to pile pullout and Platform E should have survived.

The paradox of why these two seemingly identical structures behaved differently was due to the differences in the appurtenances (well conductors), the manner in which the wells were tied into the structures, and the under-driven piles in Platform 2D. The effects of these differences only became evident when these "details" were determined and their implications integrated into the analyses. The results from the analyses were in conformance with the observed behavior of the platforms.

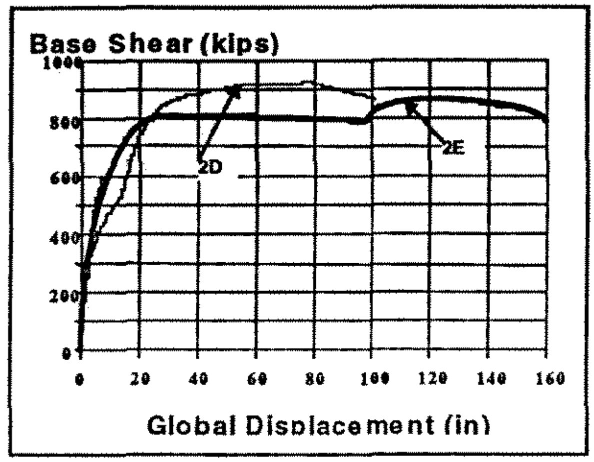


FIGURE 11: PLATFORMS 2D & 2E LOAD - DISPLACEMENT CHARACTERISTICS

Figures 12 and 13 summarize the ULSLEA analysis results (end-on results shown, broad-side results were comparable). The results indicate that the lateral loading capacity of Platforms 2D and 2E would both be about 1,100 kips. The ULSLEA results indicated that the maximum lateral load capacity of the two platforms was about 1,100 kips, resulting in an overestimated capacity of 21 % and 25 %, respectively.

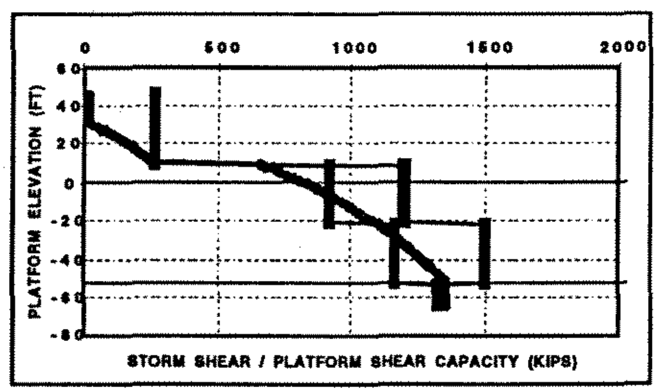


FIGURE 12: PLATFORM 2D STORM SHEARS AND PLATFORM SHEAR CAPACITIES

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TABLE 1: COMPARISON OF USFOS AND ULSLEA RESULTS

Platform	Configuration	Wave Direction	ULSLEA Collapse Base Shear (kips)	USFOS Collapse Base Shear (kips)	Ratio of USFOS / ULSLEA Base Shears
2A	8 leg double battered K-braced	End-on Broadside	2,900 3,400	2,600 2,900	0.90 0.85
2B	8 leg double battered K-braced	End-on Broadside	3,100 3,700	3,900 3,900	1.22 1.05
2C	4 leg double battered horizontal K-braced	End-on (dynamic) End-on (static)	3,200 2,000 (1,700)*	3,400 1,700	1.06 0.85 (1.00)*
2D	4-leg double battered vertical K-braced	End-on	1,100	910	0.83
2E	4-leg double battered K-braced	End-on	1,100	880	0.80

* includes platform deadweight in pile axial loading

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