Nonlinear Time-History Analysis of Modular Structures Isolated by Sliding Plates under Seismic Loads

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SUMMARY:
Modular steel structures are the common method of construction in the arctic region where they are largely used by the oil and gas industry to support and enclose process equipments. Non-linear time history dynamic analysis is performed on typical modular structure to assess the seismic performance of these structures. The analysis accounts for their unique detailing especially to elements that contribute to energy dissipation during major seismic events. Maximum inter-story drift and peak global roof drift were adopted as critical response parameters. This study revealed significant global seismic capacity as well as a satisfactory performance at design intensity levels. Special attention needs to be considered for the detailing of the interface between the super structure and the pile foundation.

Keywords: Modular Steel Building; Braced Frame; Earthquake Ground Motions; Dynamic Analysis;

1. INTRODUCTION

Modular steel structures are the common method of construction in the arctic region where they are largely used by the oil and gas industry to support and enclose process equipments. The modular structures are usually preassembled in a fabrication yard and then transported with barrages and special trucks to the final site location. To accommodate for the transportation equipment, the modules are usually built in top of platform that is made of five feet deep plate girders that can span over the transportation truck. Once it is transported to the final site location, the plate girders sit in top of series of pile foundation that project about five feet from the ground surface. Module structures are usually separated from its pile foundation using lubrite plate assembly to allow for thermal expansion between the super structure and the pile foundation. The current design practice ignores the affect of the lubrite plate in providing additional damping and base isolation to the structure during seismic events.

Most modal building codes use a reduction factor R to reduce elastic spectral seismic force to a design level. The basis of the reduction factor is driven from the fact that most structures contains higher strength than what is accounted for in initial design (overstrength) as theses structures have a certain capacity to dissipate energy before failure (ductility). The reduction factor of any of any structure is a function of the overstrength and the ductility factors. The rational of reducing the elastic seismic loads are based on the premises that a ductile structure will be able to withstand a larger lateral loads beyond its design strength when it is capable of developing and sustaining large inelastic deformation without collapse. The design of seismic resistant structures is in large is based on this concept.

The response modification factors was first introduced in the ATC 3-06 report were it was selected through a committee consensus based on observation of building performance in previous earthquakes and on the estimates of system overstrength and damping. Uang [1991], Bruneau et al. [1998] pointed to the followings parameters as common sources of overstrength: The material effects caused by higher yield stress compared with the nominal values, the use of discrete member sizes that often led to significant difference between member capacity and demands; strain hardening in steel;

redistribution of internal forces in the inelastic range; in addition to the contribution of the non-structural components in overall structural strength.

The main purpose of this study is to evaluate the overstrength and response modification factors of a typical module structure. A model structure was designed in accordance with IBC 2000 and AISC seismic provisions for structural steel buildings and ACI 318-08 seismic provision of reinforced concrete buildings. Nonlinear static pushover analysis and nonlinear time history dynamic analysis were carried out to obtain such behaviour factors; the ultimate goal is to implement the results of this study into the design guidelines for heavy industrial structures such as ASCE7.

2. RESPONSE MODIFICATION FACTOR

Mazzolani and Piluso introduced different theoretical procedures to calculate the response modification factor, such as the maximum plastic deformation approach, the energy approach, and the low-cycle fatigue approach. ATC-19 proposed a simplified procedure to calculate the response modification factors, in which the response modification factor, R, is calculated as the product of the three parameters that influence the structure response during earthquakes:

$$R = R_0 R_\mu R_r \tag{2.1}$$

Where R_0 is the overstrength factor that measures the lateral strength of a structure compared to its design strength; FEMA-369 specified three components of overstrength factors in Table C5.2.7-1: design overstrength, material overstrength, and system overstrength. R_μ is a ductility factor which is a measure of the global nonlinear response of a structure, while R_r is a redundancy factor to quantify the improved reliability of seismic framing systems constructed with multiple lines of strength. In this study it is assumed that the redundancy factor is equal to 1.0. In this case the response modification factor is determined as the product of the overstrength factor and the ductility factor. Fig.1 represents the base-shear versus roof displacement relation of a structure, which can be developed by a nonlinear static analysis. The ductility factor R_μ and the overstrength factor R_0 are defined as follows:

$$R_\mu = \frac{V_e}{V_y} \tag{2.2}$$

$$R_0 = \frac{V_y}{V_d} \tag{2.3}$$

Where V_e is the maximum seismic demand for elastic response, and V_y is the base shear corresponding to the maximum inelastic displacement. V_d is the design base shear,

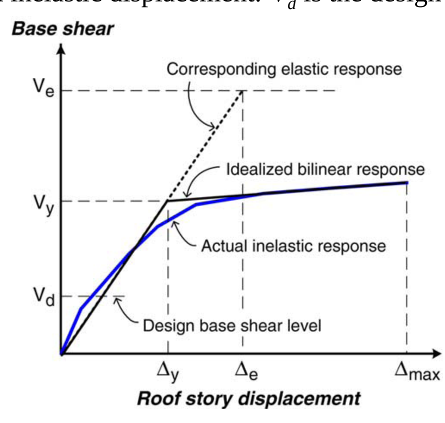


Figure 1. Lateral load-roof displacement relationship of a structure

3. MODUAL STRUCTURE

Modular structures are preassembled in a fabrication yard and then transported to the final site location where it is supported on slurried adfreeze piles. The superstructure is constructed from structural steel where gravity loads are carried by steel columns and beams while the floor assembly is made from built-up plate girders. The lateral resisting system is formed of concentric steel brace frames in the longitudinal direction. The plate girders span over the width of the module and are supported on slurried adverse pile system. To allow for thermal expansion, a sliding connection made of the librate plates are incorporated between the plate girder and the adfreeze piles (Figure 4). The main purpose of the sliding plate is to release thermal stresses due to extreme changes in the environment temperature from winter to summer season in the arctic.

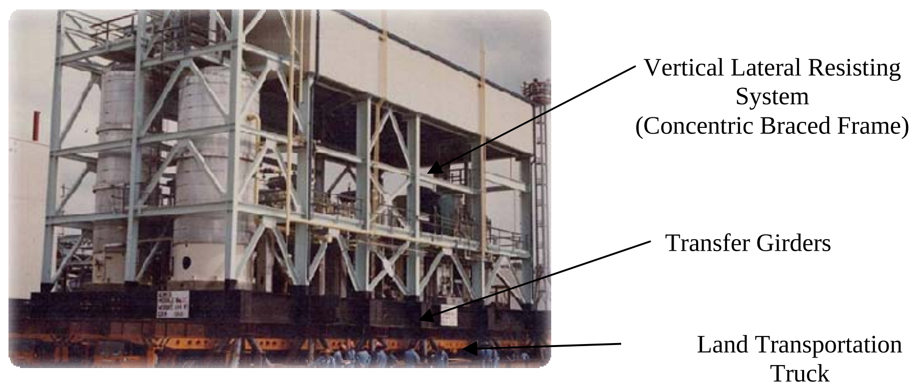


Figure 2. Modular Structure during Land Transportation

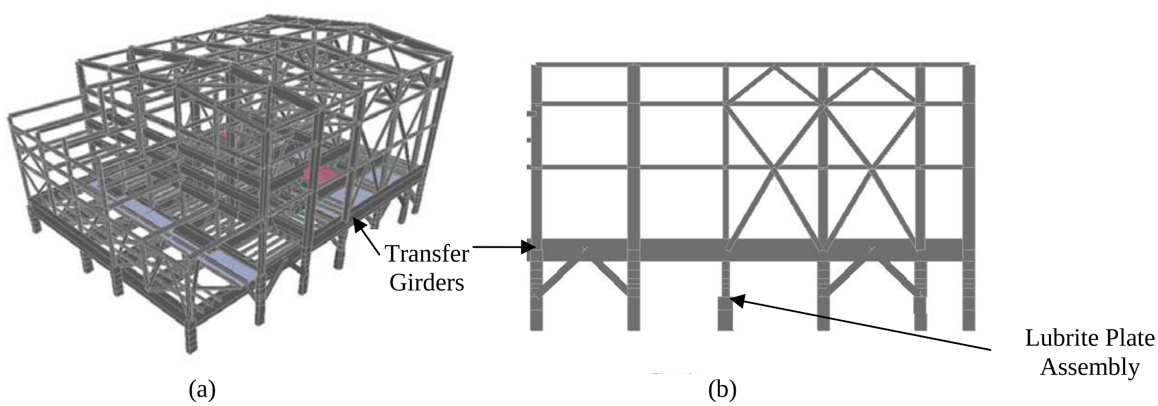


Figure 3. Schematic Diagram of a Typical Modular Structure (a) 3-D View, (b) Elevation View

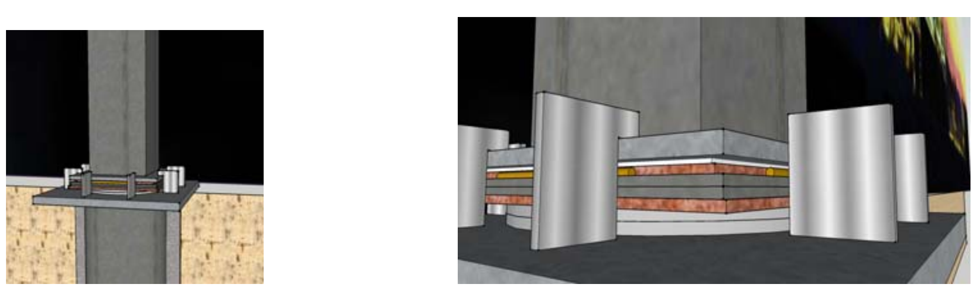


Figure 4. Lubrite Plate Assembly

4. ANLYTICAL MODEL

The analytical model of the structures was developed using the SAP2000NL© program. This program is finite element software which is capable of conducting step by step non-linear time history analysis. The following techniques were followed in modelling the different element types of the structure.
All frame members including beams, columns and braces, are considered as pin-ended. This way, the gravity loads are not carried by the brace members and are supported only by the columns. For the dynamic analysis, story masses were placed in the roof and floor platform. Due to presence of a quarter inch checker plate at the plate girder level, a rigid diaphragms action is assumed at this level. The roof lateral stability is provided by series of horizontal brace members which were included in the analytical model to represent the in-plane lateral stiffness of the roof framing.

All beams and columns were modelled using lumped-plasticity model at each element taking into account the 3D interaction between the axial load and the bending moments (in the strong and weak axis) of the section. A “smooth” yield surface was used, based in the AISC equations. Figure 5 shows the hysteresis model of a steel brace member that was used in the model and intended to captures the inelastic behaviour under alternate axial tension and compression. The member only permits this hysteresis in the axial component. The force displacement relationship include the following stages : (1) Yield plateau in tension, (2) Elastic zone,(3) Buckling plateau, (4) Buckling zone with second order effects, (5) Elastic zone with second order effects and (6) Inelastic unload zone with second order effects.

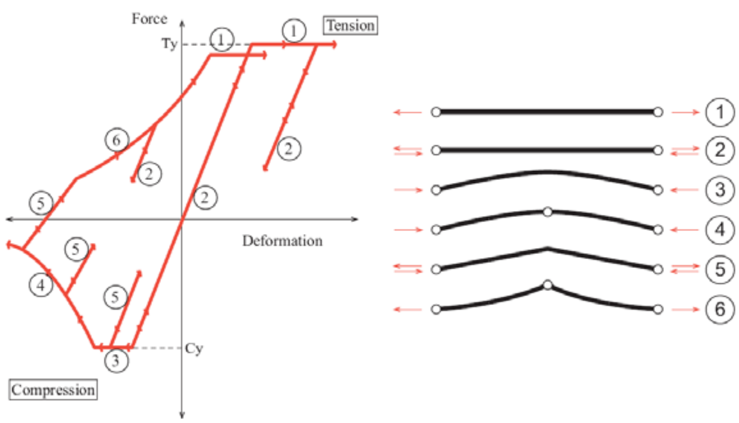
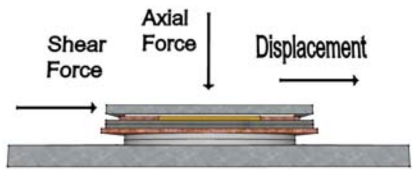


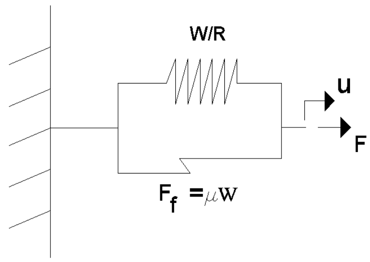
Figure 5. Force-deformation curve for bracings:

Figure 6 shows the idealization of the Lubrite plate assembly. It is assumed that the Lubrite plate will behave in a manner similar to friction pendulum system bearings. Figure 6-b shows the mathematical model that is used to simulate the behavior of a friction pendulum system bearing. The coefficient of friction on this model was obtained from the manufacture specification of Teflon plates that are used in the assembly. It is expected that a hysteresis loop similar to the one that is shown in Figure 6-C will be produced when the system is subjected to cyclic loading. These hysteresis shows the damping (energy dissipation) that it can add to the structural system and the equivalent lateral system that can be generated out of the assembly when the structure is subjected to lateral loads.

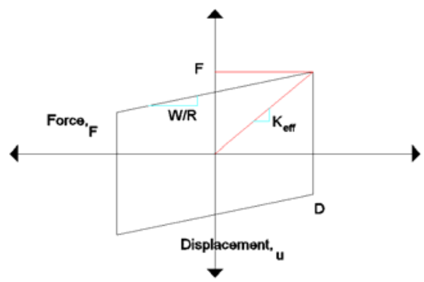
The adfereze piles are assumed to act like a cantilever beam when it is subjected to lateral loads. The point of fixity of this cantilever is assumed to be at five feet below the finish grade which is the elevation where the pile is assumed to be fully confined with the surrounding permafrost. This assumption has to be further assessed using elaborate soil structure interaction modeling. Figure 7 shows a line diagram of the module structure including the cantilever portion of the pile foundation that was used in this study.



(a)



(b)



(c)

Figure 6. Modelling of the Lubrite Plate Assembly

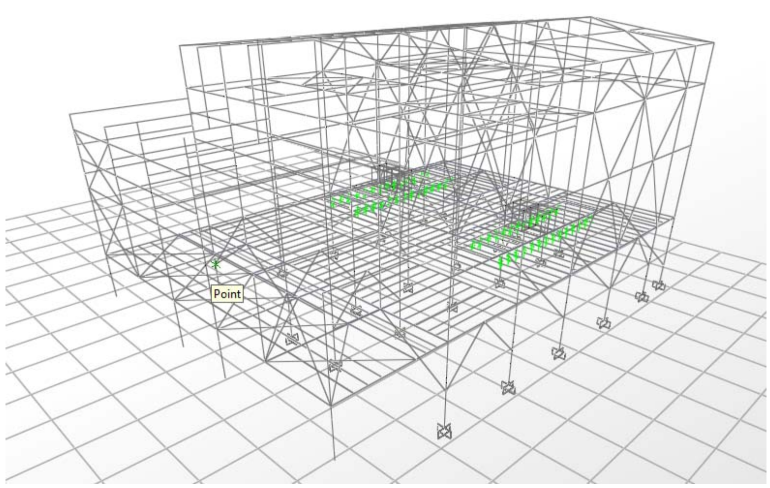


Figure 7. Finite Element Model of the Modular Structure Including Pile Supports

4.1. Selection of Ground Motions

Table 4.1. Ground Motion Summary

Earthquake	Year	PGA(g)	Duration (s)
Electro	1940	0.348	53.74
Kobe	1955	0.599	48
Northridge	1978	0.420	45

5. RESULTS AND DISCUSSION

The global performance can be observed in terms of three main response quantities: base reactions (base shears), roof displacement, and drift between platform levels and the roof. The base reaction gives an idea of total strength of the structure, the roof displacement of its level of deformation, and the drifts a notion of the expected damage level.

In Figure 9 the maximum values of drift between platform levels for the modular structure are shown. The figure shows that nonlinear analyses results do not exceed the IBC Code limit of 15%. It was also found that the maximum drift occurs in a region where the structure has a large discontinuity of stiffness and the internal parts transfer most of the "seismic lateral forces" to the structure in which it occurs at the interface between the plate girders and the pile foundation. The analysis has shown that the sequence of forming plastic hinges it starts at the cantilever piles then it moves slowly to the bracing members which confirm the previous observation of the increased ductility demands at the pile super structure interface.

The hysteresis loops of the Lubrite plate indicate that it was affective in providing both desirable damping to the system and in providing some sort of base isolation to the super structure. This can be seen by comparing the elastic base shear to the nonlinear base shear where a substantial reduction was observed. The reduction in one of the base shear in the non linear analysis can be attributed to the reduction in the system stiffness due to isolation effect of the Lubrite plate and the yielding of some of the lateral bracing members.

To calculate V, a nonlinear dynamic analysis of the model was carried out using the ground motions listed in table 4.1 after they were scaled to the design spectrum. The scaling was conducted to produce a base shear that is equivalent to the inelastic design base shear. To obtain the base shear that correspond to the first plastic hinge formation in structure Vs, the pushover analysis was carried out by progressively increasing lateral forces proportional to the fundamental mode shape. The failure criteria were defined based on the maximum relative story displacement limit was selected to be limited to 0.2h where `h' is story height. To calculate R the nonlinear dynamic analysis and linear dynamic analysis were carried out to calculate the base shear for each case. Once the base shear was calculated the Ro was calculated using equation 2.3 then the response modification factor was finally calculated using equation 2.1

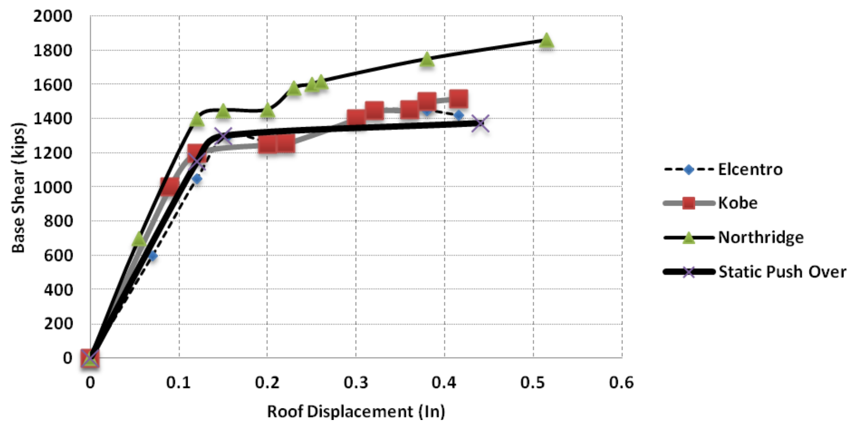


Figure 8. Roof Displacement-Base Shear Curve

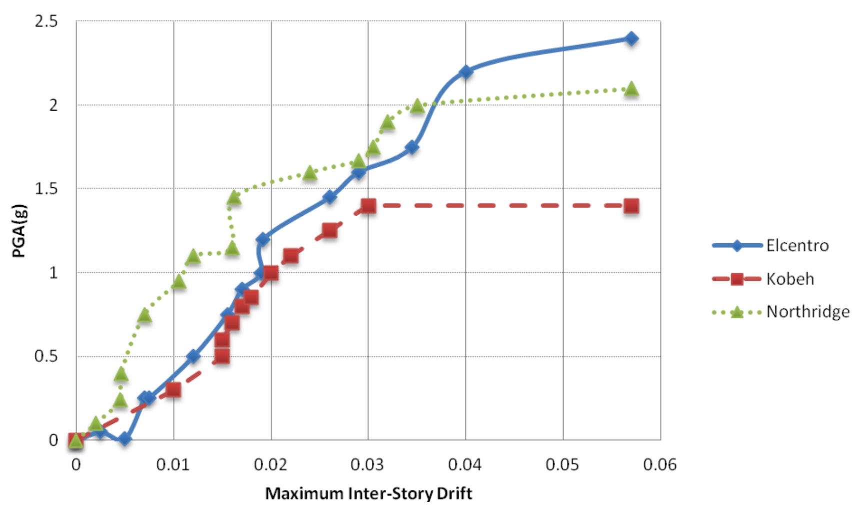


Figure 9. Non-Linear Dynamic Analysis Results

6. CONCLUSION

Modular structures posses a reserve strength and additional energy dissipation capacity that is not recognized by the current building code that is used for the design of regular buildings. The unique configuration of the modular structure especially in the presence of the Lubrite plate improves the seismic performance of the structure.

The interface between the slurried piles and the plate girder was found to be the most critical element of the structure that found to contain the formation of most plastic hinges. The cantilever slurried piles contribute to the increased ductility demands and the interface. Special detailing is recommended to increase the stiffens of the cantilever piles in order to alleviate the high ductility demands in this region

In conclusion, the overstrength, ductility, and the response modification factors of the modular structure were calculated. It was found that response modification factor for ordinary braced structure which is currently used in practice underestimate the response modification factor that would be expected from a typical modular structure.

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