Nonlinear Static Analysis of a Coker Support Structure

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SUMMARY:
In this study, the seismic performance of a cocker structure which is a complex structure commonly found in large oil refineries. Nonlinear static pushover analysis is conducted to study the inelastic behaviour of these structures. Structural over-strength resulting from redistribution of internal forces in the inelastic range, design assumptions, and behaviour of special anchor bolts connecting the pressure vessels to the supporting frames are evaluated. The results show that the reserve strength of structure is greater than that prescribed by the ASCE7-08 for ordinary-concentric braced frame systems. It is also found that the anchor bolts are the most susceptible components of reaching high-non linear demands. Special attention needs to be considered to the design of the anchor bolts to eliminate undesirable seismic response.

Keywords: Braced Frame; Structural Over strength; Ductility; Capacity Design; Nonlinear Pushover Analysis

1. INTRODUCTION

Coker structures units are commonly found in large refineries where they are used to process heavy crude oil. The unit is a complex structure that consists of massive reinforced ordinary reinforced concrete frame which supports pressure vessels and steel towers carrying maintenance platforms. The current practise is to estimate the seismic loads for these structures using parameters of similar building-type structures. The coker structures are substantially different than typical building structures where there are no floor or roof diaphragms with lumped mass. Therefore, building code design equations are not necessarily suitable to predict their performance during earthquakes. Many of these structures are constructed or planned to be constructed in an area of high seismic activities and a safe and economic design of these units is of a great value to the society.

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 are driven from the fact that most structures contains higher strength than what is accounted for in initial design (overstrength) and theses structures have a certain capacity to dissipate energy before failure (ductility). The reduction factor 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 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 coker structure. The 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 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 Ro 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. Rμ is a ductility factor which is a measure of the global nonlinear response of a structure, and Rr 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 Rμ and the overstrength factor R0 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 Ve is the maximum seismic demand for elastic response, and Vy is the base shear corresponding to the maximum inelastic displacement. Vd is the design base shear,

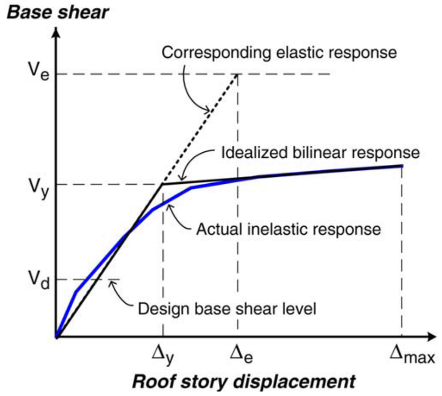


Figure 1. Lateral load-roof displacement relationship of a structure

3. COKER STRUCTURE

Figure 2 shows a schematic diagram of a Coker structure. The structure is usually composed of massive reinforced concrete frame structures (table top). The pressure vessels are connected to the table top using large size anchor bolts (Figure 3). Open frame steel structures are used to support maintenance platforms for the Coker drums. Two steel towers to carry mechanical equipments (Derrick Towers) are placed in top of the open steel frames. The lateral resisting system for the open steel frames and the steel tower are formed of ordinary concentric brace frames. Lateral stability of the platforms is provided by horizontal braced members below the grating elevation. Due to constructability consideration, all steel connections are bolted at the field using bearing type connection. The table top structure is supported on mat foundation carried by precast piles.

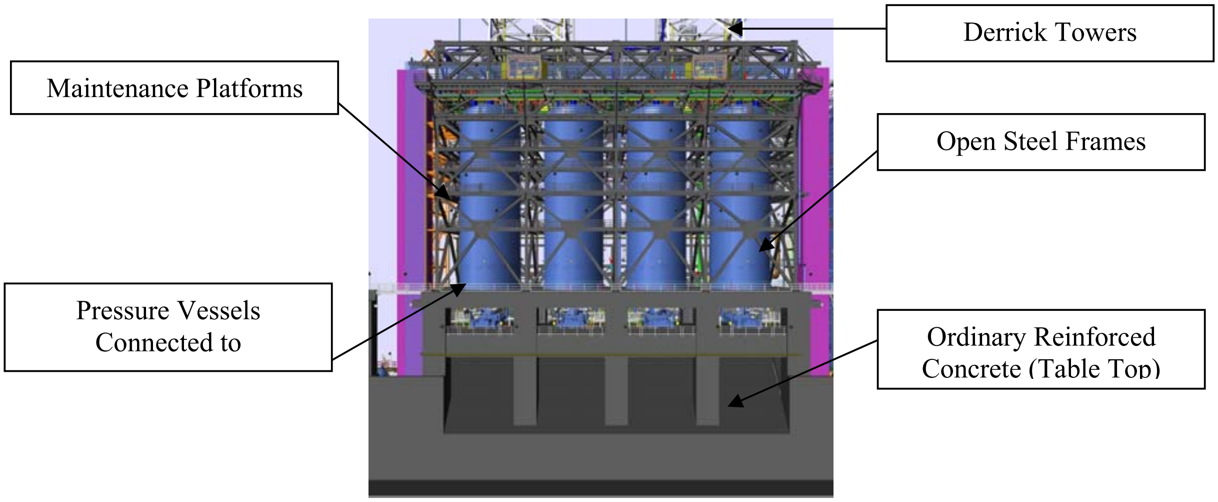
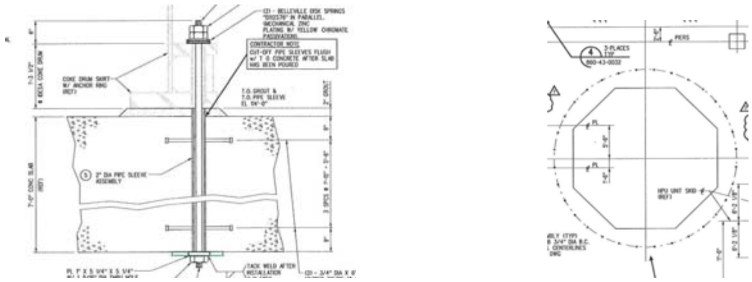


Figure 2. Schematic Diagram of a Typical Coker Structure



a) Anchor Bolt Detail b) Anchor Bolt Layout

Figure 3. Anchor Bolt Connecting Pressure Vessel to Table Top Frames

4. NONLINEAR STATIC ANALYSIS OF COCKER STRUCTURE

The analysis considers different types of nonlinear elements defining nonlinearity in braces, columns, vessel anchor bolts, each one with its own force deformation relation. The Anchor bolt was models with a force control element as shown the frame systems are modelled using the nonlinear finite element computer program SAP2000 NL. Special attention is given to the unique detailing of the anchor bolt connecting the pressure vessel to the table top. A finite element analysis using displacement control protocols were conducted using ABOQUIS program to study the behaviour of anchor bolt under cyclic loads (Figure 5). The analysis revealed that the failure mechanisms of the anchor bolt is attributed to the crushing of the concrete at the bottom washer as the bolt is pulled away from the table top.

Nonlinear static push-over analysis is conducted to determine the ultimate lateral load resistance as well as the sequence of yielding/buckling events. Eigen value analysis was conducted first to determine the elastic natural periods and mode shapes of the structure. Then pushover analysis were carried out to evaluate the global yield limit state and the structural capacity by progressively increasing the lateral story forces proportional to the fundamental mode shape. Steel columns were modeled using a lumped-plasticity model at each element end considering the interaction between the axial load and the bending moments (in the strong and weak axis) of the section. The post-yield stiffness of the beams and columns was assumed to be 2% of the initial stiffness, and that of the braces was assumed to be zero. The expected yield stress of structural members was assumed to be 1.5 times the nominal yield stress as recommended by the Seismic Provisions for Structural Steel Buildings for ASTM A992 steel. The model proposed by Jain and Goel, which was also presented in FEMA-274, was used for modeling nonlinear behavior of braces (Fig. 4). The post-buckling residual compression force is set to be 20% of the buckling load. The nonlinear behaviour of the interaction between the coker drum and the table top is shown on Figure 6, where path (1) represent compression on concrete, (2) non restrained zone,(3) elastic tension of bolts and (4) plastic tension of bolts.

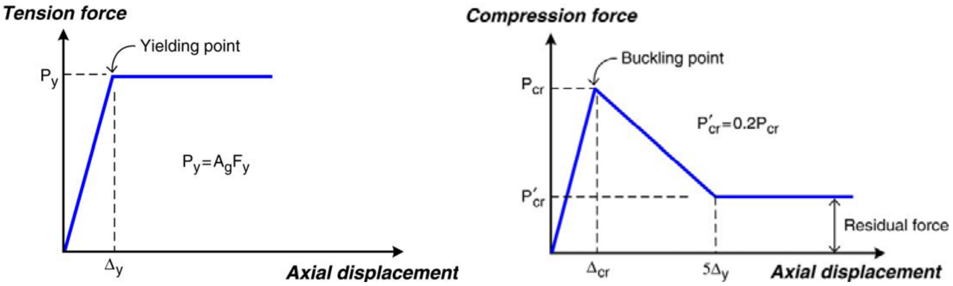


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Figure 4. Simplified analysis model for force–displacement relationship of Brace member**.**

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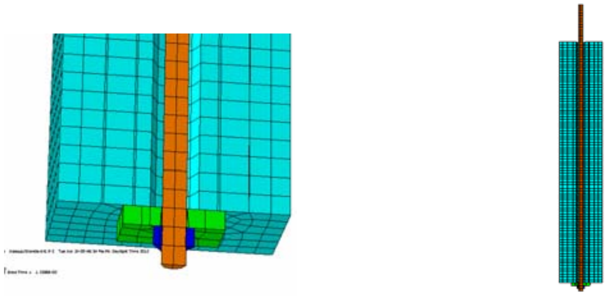


Figure 5. Finite Element Modelling of Anchor Bolt Attachment to Concrete Frame

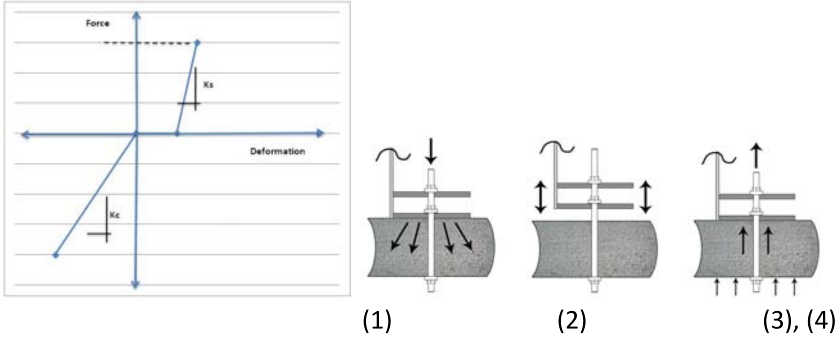
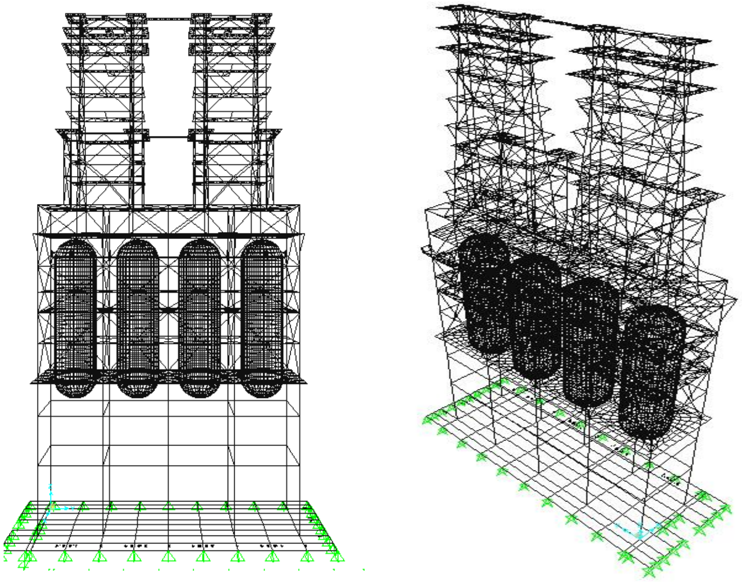


Figure 6. Simplified model for force-displacement relationship of anchor bolt



a) Front view b) 3D general view

Figure 7. Finite Element Model of the Cocker Structure

5. RESULTS AND DISCUSSION

Fig. 8 shows the pushover curves of the Coker structure for the transverse and longitudinal direction, respectively. In the base shear–roof displacement curves, the points corresponding to the design base shear, the first buckling and yielding of braces, maximum inter-story drift of 1.5% and 2.0% of story height between platforms are indicated. The Figure also shows the push over curve of the pressure vessel (Coker Drum) overlays in top of the steel frame curve.

In the figures it can be observed that the stiffness of the braced frames decreases slightly by the buckling of a compressive brace and the maximum strength is reached slightly before the first yielding of a tensile brace. The maximum strength is about two times as high as the design base shear. However in the Coker Drum, the structure behaves elastically and then the strength drops sharply at the occurrence of the crushing of the concrete at the interface between the anchor bolt and the table top structure.

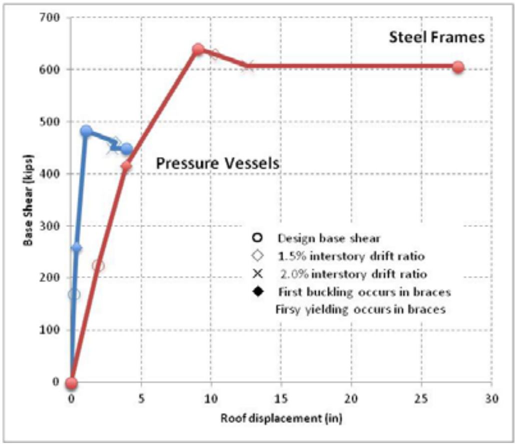


Figure 8. Pushover Curve for the Coker Structure

Figure 9 shows the inter-story drift ratio of the open frames and the braced frame tower structure, where it can be observed that large drift occurs in lower stories where buckling occurs in braces. Fig. 9 depicts the state of damage in structural members and the ductility ratio in braces of in the steel portion of the coker structure. It can be also observed that when the maximum inter-story drift reaches 1.5% at he interface between the open space frame and the tower structures. This can be explained due to the interruption of story stiffens due to the change of the structure geometry moving from the rectangular frames into the derrick towers. The interruption of the story stiffness resulted in redistribution of forces that may lead to have most of the brace members in the lower level to buckle.

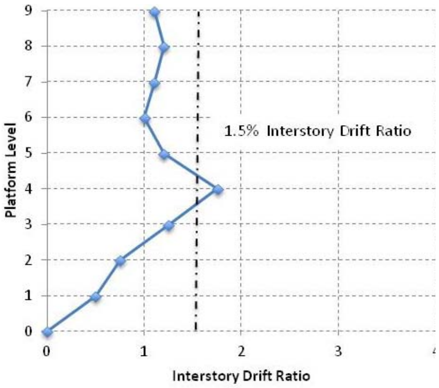


Figure 9. Inter-story Drift Ratio of the braced frames

5.1. Overstrength Factor

The capacity envelopes obtained from pushover analysis were utilized to evaluate overstrength factors. To find out the yield point, a straight line was drawn in such a way that the area under the original curve is equal to that of the idealized one as recommended in FEMA-356 for structures with negative post-yield stiffness (Fig. 10).

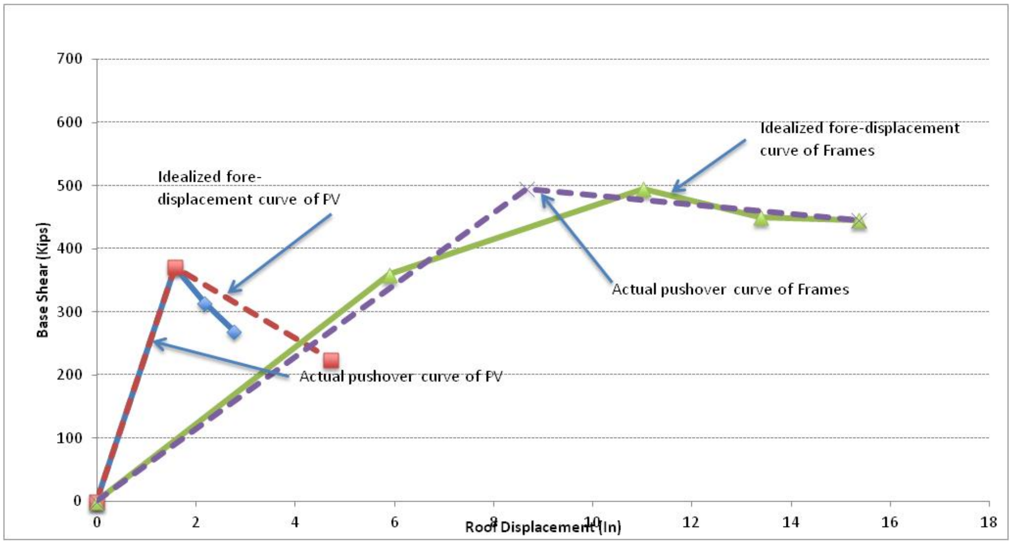


Figure 10. Idealized force–displacement curve for braced frames

5.2. Ductility Factor

The ductility factor Rμ was obtained using the system ductility factor μ by the procedure proposed by Newmark and Hall where the following equations were proposed for the system ductility factors:

$$R_\mu = 1.0 \qquad (T < 0.03 \text{ s}) \tag{5.1}$$

$$R_\mu = \sqrt{2\mu - 1} \qquad (0.12 < T < 0.03 \text{ s}) \tag{5.2}$$

$$R_\mu = \mu \qquad (T > 1.0 \text{ s}) \tag{5.3}$$

Where T is the natural period of the structure. Miranda and Bertero developed general Rμ–μ–T relationships using 124ground motions recorded on a wide range of soil conditions the following equation is for a rock site:

$$R_\mu = \frac{\mu - 1}{\phi} + 1 \tag{5.4}$$

$$\phi = 1 + \frac{1}{1oT - \mu T} - \frac{1}{2T} e^{-1.5(\ln(T) - 0.6)^2} \tag{5.5}$$

Where Φ is a coefficient reflecting a soil condition. The system ductility ratio μ is obtained by dividing the roof displacement at the limit state by the system yield displacement.

5.3. Response Modification Factor

The response modification factor, presented earlier is computed by multiplying the overstrength and the ductility factors obtained in the previous sections. In the braced frame the response modification factors are obtained when the maximum inter-story drift ratio at 2.5%. While for the coker drums it is calculated when the anchor bolt reach its failure limit state. In the spaced frame structures the response modification factors turns out to be larger than 4.5 which is prescribed in IBC2000. The response modification factors obtained for the cocker drum anchorage are less than the values currently used in practice.

6. CONCLUSION

Structural over strength factors are extracted from the observed response curves and compared with code-specified values, those reported for regular steel-braced frames and ordinary reinforced concrete frames. The results obtained show that "anchor bolts" are the elements most susceptible to reach a high non-linear behavior demand. Analysis has also indicated that vertical bracings of the steel tower have also rather large demands however the demands were within the code limit. The drift values obtained between the main platforms levels were within the allowable drift in the IBC code. The analysis also revealed that care must be taken in the ductility design of anchor bolts at the interface between the table top and the steel tower. The ductility of this particular component plays a major role in the overall ductility of the entire structure.

The results were compared with code-specified overstrength values as well as experimentally and analytically determined values for regular braced frames. The use of the code’s value for the design of braced frames is shown to be conservative.

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