Assessment of Response Modification Factor of Reinforced Concrete Table Top Frames Structures Subjected to Seismic Loads

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Abstract. In this study, the seismic performance of table-top reinforced concrete frame structure which is commonly found in large oil refineries. Nonlinear static pushover analysis is conducted to study the inelastic behaviour of these frames. The analysis accounts for their unique detailing especially to elements that contribute to energy dissipation during major seismic events. The results show that the reserve strength of structure is greater than that prescribed by the ASCE7-10 for Special reinforced concert moment frame.

1 Introduction

Table-top structures are unique type of structures and 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 pressure vessels are connected to the frames using a circular pattern of relatively deep anchor bolts. To provide sufficient embedment of the anchor bolts and the required strength for the anchor bolts to support uplift force due to wind loads, the beams of the frames are very deep and wide in cross section. For Frame action computability, the columns cross sections are seized based on the sizes of the beams rather than the size that are required to resist gravity and lateral loads. In other words, the sizes and layout of the pressure vessels is what determine the cross section of the frame beams and columns.

The current practice is to estimate the seismic loads for these structures using parameters of similar building-type structures. The structures are substantially different from typical building structures where there are no 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. The dilemma exists in active seismic sites which dictate all reinforced concrete moment frames to be designed as a Special Moment Frame (SMF) with its restricted requirements in ties spacing, lap splice, support of vertical bars, and the continuation of ties into the beam-column connection. This requirement complicates the construction of these frames in active seismic zones. Due the massive nature of these frames, it is believed that these structures will most likely remain elastic during major earthquake.

This main purpose of this research is to assess the vulnerability of these unique structure during major seismic event by evaluating the Response Modification Factor (R factor) for these structures through numerical simulations in order to determine the proper classification of theses frames from the building code stand point.

2 Table Top Structure

Figure 1, 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 Fig. 2. Open frame steel structures are used to support maintenance platforms for the Coker drums. The structure was designed in accordance with IBC 2000 and ACI 318-08 seismic provision of reinforced concrete buildings. Nonlinear static pushover analysis and nonlinear dynamic analysis were carried out to obtain such behavior factors, the ultimate goal is to implement the results of this study into the design guidelines for Table Top structures such as ASCE7.

Fig. 1. Schematic diagram of a typical structure

3 Response Modification Factor

Conventional seismic design in most modal codes is force-based, with a final check on structural displacements. The force-based design is suited to design for actions that are permanently applied. The seismic design follows the same procedure, except for the fact that inelastic deformations may be utilized to absorb certain levels of energy leading to a reduction in the forces for which structures are designed. This leads to the creation of the Response Modification Factor (R factor); the all-important parameter that accounts for over-strength, energy absorption, and dissipation as well as the structural capacity to redistribute forces from inelastic highly stressed regions to other less stressed locations in the structure. This factor is unique and different for different type of structures and materials used. Hence, classification of Response Modification

Fig. 2. Anchor bolt connecting pressure vessel to table top frames response modification factor

Factor for various structural systems is extremely important in order to do an evaluation based on demand and capacity of the structure.

The Response Modification Factor (R factor) reduces the seismic force to a design level as it is assumed that structures contain reserve strength, an extra energy dissipating capability as observed from previous earthquakes (ATC-19). This extra capability is based on a structure's ductility, an ability to absorb energy by deforming. This relationship is illustrated by an inelastic force-deformation curve as seen in Fig. 3. The diagonally rising line represents the linearly increasing deformation as the force increases. The linearly elastic response goes up until the point of V_e , elastic seismic force - which is the base shear if the structure were to maintain an elastic behavior. In reality, structures lose their ability to deform proportionally to the applied force. After point V_d , design base shear, the structure may have larger deformations yet dissipate lesser force. At this given stage, plastic hinges begin to form, which identify the points where the energy dissipating ability of the structure drops step by step until the structure's maximum seismic force, V_y , is reached (ATC-19).

An elastic design would assume the structure to have a linear force-deformation relationship, and thus be able to deform less and dissipate less energy. On the other hand, inelastic design acknowledges the ability to continuously deform and keep on dissipating energy. The use of right ductility and Response Modification Factors can result in a safer and more cost effective structure than one designed using the elastic design force, which does not acknowledge the inelastic energy dissipating ability.

ATC-19 was introduced 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:

$$R = R_0 R_\mu R_r$$

Where:

R_0 = Over strength factor

R_μ = Ductility factor

R_r = Redundancy

Fig. 3. Seismic performance factors as illustrated by commentary to the NEHRP recommended provisions (FEMA-450)

The response modification factor is determined as the product of the overstrength factor and the ductility factor and redundancy factor, these factors can be idealized by Base shear versus Displacement, it can be seen in Fig. 3, which can be developed by a nonlinear static pushover analysis.

Overstrength Factor (R_0)

The maximum lateral strength, V_y , generally exceeds the design lateral strength, V_d . This extra strength depends on many parameters that are not easily quantified. For instance, possible sources of this strength may result from the material strength if it is actually larger than the calculated design capacity. Other sources may be from non-structural elements providing extra strength or from redistribution of internal forces in the inelastic range (ATC-19; Balendra T & Huang). The strength factor, R_0 is period dependant and is the ratio of the lateral strength at the maximum considered drift, V_y , to the required strength, V_d or design base shear. Design Base shear equation so it was used to determine the design base shear corresponding to first plastic hinge (V_d).

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

Where:

V_y = available yielding strength.

V_d = design base shear determined from ASCE 7-10.

Ductility Factor (R_μ)

Structures lose their elastic behavior once a major yielding point occurs. The relationship between the system's elastic response and actual inelastic response can be seen in Fig. 3. The ductility reduction factor, R_μ , is a factor which reduces the elastic force demand to the level of idealized yield strength of the structure and, hence, it may be represented as the following equation:

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

Where:

V_e = Linear elastic force.

V_y = available yielding strength.

Redundancy Factor (R_r)

The Redundancy factor, R_r , measures the reliability of multiple vertical lines to transfer seismic-induced inertial force to the foundation (ATC-19). The following limitations apply to Table Top structures, the value of R_r is permitted to equal 1.0 for the following conditions:

- Seismic Design Category B or C.
 - There are four or more columns and three or more bays at each level.
- Otherwise redundancy factor shall equal to 1.3.

For this study, the redundancy factor is equal to 1.0 due to many moment frames.

4 Numerical Simulations

4.1 Design of Model Structure

Table top frames usually consists of two parts; first the bottom part, the structure is usually composed of reinforce concrete frame i.e. it is usually a Moment frame therefore in this thesis the bottom part of the frame is special reinforced concrete moment frame (SMRF's). Which is used to support maintenance platforms and secondly, it is consisting of steel cage system at the top of the reinforced concrete frame to support the crude oil distillation unit. The ordinary braced frames used to resist lateral force. It is a conventionally reinforced concrete table-top vessel support structure, massive concrete frame that is pile supported.

In order to represent the table top reinforced concrete frame, two existing prototype table top reinforced concrete moment frames. The structure has 4×1 bays and three story see Fig. 4, below for longitudinal and side view of model. The structure configuration of the two prototype table top reinforced concrete frame is shown in Table 1. One of them represent a frame with cross sections that meet the ductility requirement of special moment (D-frame) and the other prototype represent a frame with cross sections that does not meet the ductility requirements and commonly found in frames to carry gravity load (ND-Frames). Table 2. Shows the cross section geometry and reinforcement configuration for the beams and columns for all the cases that was considered in this study. A parametric study for different cross sections as shown in Table 1, was also carried out to envelope to study the effect of the frames geometry in the R-factor.

The base of the structure is fixed, and the loading applied into the structure is; Dead load 100 k/ft at the top and 10 k/ft at the bottom stories and Live load 50psf.

Fig. 4. Longitudinal and side view of the model

Table 1. Structure configuration

Group		Structure model	Compressive strength of concrete (fc')	Size of column (ft)	Size of beam (ft)
Group I	Group IA	G1-3ksi-D	3ksi	8' × 8'	6' × 9'
		G1-4ksi-D	4ksi	8' × 8'	6' × 9'
		G1-6ksi-D	6ksi	8' × 8'	6' × 9'
	Group IB	G1-3ksi-ND	3ksi	8' × 8'	6' × 9'
		G1-4ksi-ND	4ksi	8' × 8'	6' × 9'
		G1-6ksi-ND	6ksi	8' × 8'	6' × 9'
Group II	Group IIA	G2-3ksi-D	3ksi	6' × 6'	4' × 7'
		G2-4ksi-D	4ksi	6' × 6'	4' × 7'
		G2-6ksi-D	6ksi	6' × 6'	4' × 7'
	Group IIB	G2-3ksi-ND	3ksi	6' × 6'	4' × 7'
		G2-4ksi-ND	4ksi	6' × 6'	4' × 7'
		G2-6ksi-ND	6ksi	6' × 6'	4' × 7'

4.2 Material Properties

The mixed design of concrete used for this thesis is aimed at design cylinder strength is 3ksi, 4ksi, 6ksi. Typically design for this strength has a slump test is about 1–2 in., the maximum size of course aggregate is ¾ in. The mix proportion is about 1:3:5 as refer to cement, sand and course aggregate.

The steel reinforcement used in reinforced concrete frames is grade 60 (yielding strength is $f_y = 60$ ksi).

4.3 Nonlinear Static Analysis (Push over Analysis)

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.

Table 2. Detailing of cross sections

Group		Column	Beam
Group I	Group IA	8x8D	6x9D
	Group IB	8x8ND	6x9ND
Group II	Group IIA	6x6D	4x7D
	Group IIB	6x6ND	4x7ND

Non-linear pushover analysis serves the basis for determining the capacity of the structure in terms of base shear and roof displacement (Δ), is a method for determining the ultimate load and deflection capability of a structure. In SAP2000, moment curvature curves with post yield behavior are predetermined and can be used to determine hinge properties. The hinges are placed to predict possible hinge formation locations, which are usually near the joints between members. Incremental lateral load applied, the model is then run to view the conceptual force capacity. Local nonlinear effect, such

as flexural hinges at the member joints, are modelled and the structure is deformed until it reaches to enough hinges form to develop a collapse mechanism or until it reaches to the plastic deformation limit of a hinges is reached (Fig. 5).

The numerical parameters of the hinges are obtained from moment curvature analysis that were carried out for the different cross sections used in this study. Figures 6 and 7, shows the moment curvature relationship for the different cross sections used in the study.

Fig. 5. Hinge location at table structure

4.4 Nonlinear Dynamic Time–History Analysis

Direct-integration time-history analysis is a nonlinear, dynamic analysis method in which the equilibrium equations of motion are fully integrated as a structure is subjected to dynamic loading. Analysis involves the integration of structural properties and behaviors at a series of time steps which are small relative to loading duration. The equation of motion under evaluation is given as follows:

$$m\ddot{u}(t) + Cu'(t) + ku(t) = -p(t)$$

The basic Newmark Constant acceleration method can be extended to nonlinear dynamic analysis. This requires that iteration must be performed at each time step in order to satisfy equilibrium. Also, the incremental stiffness matrix must be formed and triangularized at each iteration or at selective points in time. Many different numerical tricks, including element by element methods, have been developed in order to minimize the computational requirements. Also, the triangularization of the effective incremental stiffness matrix may be avoided by the introduction of iterative solution methods.

For each ground motion, the maximum base shear corresponding displacement is obtained (Fig. 8).

Fig. 6. Moment curvature curves for ductile and non-ductile column–cross sections (columns)

5 Results and Discussions

For this study, the result shown below the demand capacity curve.

The pushover analysis and the dynamic analysis are conducted in the transverse direction, which is along the moment frame direction (X-axis). The base shear versus roof displacement capacities for (Group IA through Group IIB) were obtained from pushover analysis and represent capacity of the structure under incremental lateral loading, presented in appendix B, as result shown that:

Fig. 7. Moment curvature curves for ductile and non-ductile column-cross sections (beams)

- If compressive strength (f'_c) increases the capacity of the structure also increases and it is also reaches to the demand of the structure (ATC-40). From Fig. 10, the design base shear, yielding shear and the maximum seismic demand for the elastic response could be calculated for each case; resulting in the calculated R factors for this study.
- As the size of the cross section increase the capacity of the structure also increases.

From Fig. 11, contain maximum base shear versus roof displacement demands from dynamic analysis. The dynamic analysis utilized 22 strong ground motions. Thus,

Fig. 8. Design spectrum and response acceleration of 22 selected ground motions

Fig. 9. Comparison of overstrength, ductility, and response modification factor

the Figures show that the pushover envelopes form an upper bound to the dynamic results. As results indicates that,

Fig. 10. Demand and capacity curves

- The maximum base shear does not reach to the maximum base shear from pushover curve,
- It is roughly coincide to the yielding range of the pushover, which indicate that the structure itself a huge capacity to reach the failure mechanism of the structure during earthquake,

Fig. 10. (continued)

- These 22 earthquakes does not reach to the pushover and these structure remain elastic and did not reach to plastic b/c of the massive size of beams and columns.
- And it is also indicating that SMRF's behaved elastically when subjected to the earthquake obtained from the dynamic analysis.

There is no added value for the special ductiling requirement and ductility that usually exhibit in the special reinforced concrete moment frames. Therefore, based on this study the ordinary moment frame was a non-ductile cross-section are suitable for this types of structure in high seismic zone thus the structure does not require the additional ductility. Furthermore, the R-factor equal to 9, for these types of frames.

Fig. 11. Dynamic curve

The above histograms discuss a detailed comparison of the Overstrength, Ductility, and Response Modification factor of Group I and Group II.

In this case study, the over strength factor for the Table-top Reinforced Concrete Moment Frames had calculated value is 4, which is above the current code value of 3 in ASCE 7-10.

The ductility factor for table-top reinforced concrete moment frames was almost equal in all cases of Group I, while it is increased as the cross section sizes decreased of Group II.

Fig. 11. (continued)

In typical table-top reinforced concrete frames, in group II the Response Modification Factor was very close to the code value of 7, while the value prescribed in ASCE 7-10 is 8 seem to greatly underestimate it. Figure 9 is an illustration of the different R factors due to the various parameters applied in the different cases. The ASCE 7-10 value show with Black Dotted line.

As you can see that in Group IB and IIB (6ksi) the response modification factor is decreases as compared to the 3ksi and 4ksi. It is because of the less ductility into the system as you can see the Pushover curve the strength is increases as compressive

Table 3. Response modification calculated values

Group		Structure model	Ductility	Over strength factor	Response modification coefficient R
Group I	Group IA	G1-3ksi-D	2	5	10
		G1-4ksi-D	2	5	11
		G1-6ksi-D	2	5	11
	Group IB	G1-3ksi-ND	2	5	11
		G1-4ksi-ND	2	5	11
		G1-6ksi-ND	2	5	10
Group II	Group IIA	G2-3ksi-D	3	3	8
		G2-4ksi-D	3	3	8
		G2-6ksi-D	3	3	9
	Group IIB	G2-3ksi-ND	3	3	7
		G2-4ksi-ND	3	3	7
		G2-6ksi-ND	2	2	6
Average	3	4	9		

strength increases but the displacement also decreases that is why the response modification factor in Group IB and IIB (6ksi) decreases. So, I can say that the increase in compressive strength is not beneficial and it seem that the structure is fail in concrete.

6 Conclusions

As the SMF's were designed based on preliminary response modification factor and their tentative values were evaluated. According to the mentioned procedure all models were analyzed and their final seismic response modification factors were calculated.

The Special Moment Frame were evaluated using nonlinear static pushover analysis and (time history) analysis. The response modification factor was discussed in previously. However, the response modification factor turns out to be 9 for table top reinforced concrete frames as shown in Table 3, which is larger value as prescribed in ASCE i.e. 8 for Special reinforced concrete moment frame.

- In (Group I) the R factor is greater than Group II, which means that larger the cross section higher the R factor.
- The system has a considerably high overstrength value which is around (3–5),
- Response modification factor is between (7–10), see Table 3, below for response modification factor calculated for prototypes frames compared to that value which is prescribed in ASCE 7–10 values i.e. 8,
- And ductility factor comes out to be (2–3), for these types of frames.
- As we noticed that the increase in the cross section increased the overstrength and response modification factors.

We can see, there is no added value imposes this ductile requirement. to the structures and it is not beneficial the structure remains elastic so, easy reinforced

configuration will probably suffice the requirement and make these structure easy to build and more cost effective.

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