

Energy Based Design of Rc Staging in Elevated Service Reservoir

G. Darwin¹, Ratnesh Kumar^{2,*}, Onkar Kumbhar³, Vijay N. Khose⁴

¹ Department of Applied Mechanics, PG Student, Visvesvaraya National Institute of Technology, Nagpur, 440010, India

² Department of Applied Mechanics, Associate Professor, Visvesvaraya National Institute of Technology, Nagpur, 440010, India

³ Department of Applied Mechanics, Research Scholar, Visvesvaraya National Institute of Technology, Nagpur, 440010, India

⁴ Senior Project Engineer, Thornton Tomasetti, Mumbai, 400013, India

Paper ID - 060204

Abstract

Elevated service reservoirs (ESRs) are inverted pendulum structures and are often categorized under lifeline structures. They are expected to remain functional after the earthquake to provide essential drinking water and water for post-earthquake firefighting. The current code-based design procedure of ESR in India is similar to the design procedure of other structures and does not explicitly provide insight into its post-earthquake behavior. Energy-Based Design (EBD) can provide a better and more reliable alternative to overcome the deficiencies of code-based design. EBD considers a coherent approach to estimate the input energy, the elastic energy dissipation, and the plastic energy dissipation, taking into account the inelastic behavior of structural members. In the present research, various issues of the EBD for frame staging of the ESR have been identified and assessed using linear and nonlinear analysis. A comparison of the seismic behavior of the ESR according to the Indian code and the EBD has been made. Besides, the structural design procedure based on the column tree method (CTM) has been modified to suit the specific behavior of the ESR.

Keywords: Energy-Based Design, Elevated service reservoir, Performance Assessment

1. Introduction

Generally, elevated service reservoirs (ESRs) are used in a public water distribution system and are considered as structures of high importance due to its emergency functional requirement during and after the earthquake. Moreover, failure of the ESR may lead to a local disaster which is another reason to consider ESR as an important structure. To mitigate the failure of the ESR it is essential to ensure its safety against earthquakes. Though the design and detailing procedure in the current force-based design method [1] has improved, still the design method lags in providing desirable performance with sufficient confidence. Many ESRs failed during the Chile earthquake from which many researchers showed their interest in studying the seismic performance and behavior of the ESRs. From the past studies [2] – [8], researchers reported that many tanks damaged during past earthquakes which highlight the importance of developing a robust design technique considering various factors such as acceptable damage limit, displacement, and energy dissipation. Rai [7], [8] has studied the performance of the ESRs in the Bhuj earthquake. From the study, the author found that many ESRs seem to have severe damage in their supporting structure, and three ESRs were collapsed. Rai [8] also pointed out that the brace and columns did not meet the required ductility, poor detailing of brace column joints, the longitudinal bars were

terminated in joints, stirrups bent up were 90° instead of 135°, the insufficient number of stirrups, poor concrete quality and not following the design and detailing properly as per codes IS 13920 (1993) and IS 11682 (1985).

To overcome the challenges faced in the code-based the energy-based design method can be used which considers the inelastic property of the structure. Many researchers have worked on the energy-based design for buildings and the concluded the performance is better than the code-based method. Since the code-based method is not providing the required performance, in this study the reinforced concrete (RC) frame staging ESRs has been designed using the energy-based approach and their performance is compared. Housner [9] was among the first researchers to use the energy-based design approach for seismic design from the energy balance concept. In seismic design, the dissipation of energy imparted to the structure in desirable modes will allow the structure to successfully sustain the earthquake. Thus, Housner [9] indicated that the structures should be designed to allow plastic deformation through which the earthquake energy can be dissipated. After the evolution of the energy-based approach, many researchers [10] – [12] worked on the same for the design of portal frames and buildings, and they concluded that their results hold good with the nonlinear analysis.

*Corresponding author. Tel: +919372548232; E-mail address: ratnesh.eq@gmail.com

2. Energy-Based Design

The energy-based design evolved from the energy balance concept which is obtained from the equation of motion of a single degree of freedom system. The energy balance concept is explained by Housner [9]. The same energy balance concept is used by Meter and Ucar [13] and obtained the design base shear equation for a building frame with different combinations of input energy and plastic energy modification factor. Out of the formulated 16 combinations, the combination with factors given by Benavent-Climent *et al.* [14] and Gulkan and Sozen [15] were concluded to show better results in comparison with nonlinear time history analysis. The design base shear equation given by Meter and Ucar [13] is given below

$$\frac{1}{2}M\left(\frac{T_e}{2\pi}\cdot\frac{g}{W}\right)^2V_y^2 + \eta_p\left(\sum_{i=1}^n F_i H_i + \Delta F_N H_N\right)\theta_p = \lambda \cdot \frac{1}{8}M\frac{T_n^2}{\pi^2}S_{a_n}^2 \quad (1)$$

Where M is the seismic mass of the structure, T_e is the elastic vibrational period, W is the seismic weight of the structure and g is the acceleration due to gravity, V_y is the design base shear, F_i and H_i are design lateral force and height of i^{th} Storey, η_p is the plastic energy modification factor, ΔF_N is the additional equivalent seismic force acting on the top of the structure, H_N is the height of N^{th} storey, λ is the input energy modification factor, T_n is the natural period, S_{a_n} is pseudo acceleration.

The building has diaphragm at different levels therefore, it can be idealized as a multi-degree of freedom system, whereas the ESRs are idealized as a single degree of freedom (SDOF) system. Thus, equation (1) is rewritten in the form of the SDOF system as given below.

$$\frac{1}{2}M\left(\frac{T_e}{2\pi}\cdot\frac{g}{W}\right)^2V_y^2 + (\eta_p\theta_p H)V_y = \lambda \cdot \frac{1}{8}M\frac{T_n^2}{\pi^2}S_{a_n}^2 \quad (2)$$

The plastic energy modification factor given by Gulkan and Sozen [15] is shown below.

$$\eta_p = \frac{0.2\pi\mu(1+r\mu-r)}{2(\mu-1)(1-r)}\left(1 - \frac{1}{\sqrt{\mu}}\right) \quad (3)$$

The input energy modification given by Benavent-Climent *et al.* [14] is shown below.

$$\lambda = \left(\frac{1.15\eta}{(0.75 + \eta)(1 + 3\xi + 1.2\sqrt{\xi})}\right)^2 \quad (4)$$

The above equations (3) and (4) are substituted in equation (2) and the design base shear of the tank is obtained. The obtained design base shear is then used for designing the ESR.

3. Specification and Modelling

The initial size of the members and the geometry of the ESR is obtained by comparing the Indian Standards [1], [16] – [22]. Even though the initial sizing of the members was done as per the codes discussed above but the final sizing of the members was governed by the gravity loads and the lateral load combinations for beams and columns. Regarding the geometry of the ESR, based on the capacities of the water tank, IS 11682 [16] categorizes the water tank into 5 categories. Classification is shown in Table 1.

Table 1. Classification of Tank Size Based on Capacity

Description	Capacity in Million liters	Capacity in cubic meters
Very small	0.01 - 0.08	10 - 80
Small	0.08 - 0.25	80 - 250
Medium	0.25 - 0.8	250 - 800
Large	0.8 - 2.3	800 - 2300
Very large	>2.3	>2300

From the above-given classification very small water tank is not considered for the study as the small and very small tank is having the same geometry. In this study, the 20m frame staging is considered. The plan and elevation of the water tank are shown in Fig 1.

The four types are tanks that are shown above are modeled and designed using SAP2000. Generally, the ESR containers are designed by the working stress method and the possibility of failure of the container is less. From past studies, many researchers have identified that the ESR failure can primarily be attributed to the failure of staging. Thus, in this study, the primary focus of research is on the staging frame design rather than the container. In modeling, the container is not modeled explicitly, however, to ensure the correctness of modelling, the container loads and water loads are directly applied on the bottom beams. And, the lateral force has been applied on the CG of the container. Thus, a special joint is modeled at the calculated CG point where the lateral force is applied and the same diaphragm is assigned to that joint and the bottom beam to ensure the container action. One of the tanks modeled in SAP2000 is shown in Fig 2.

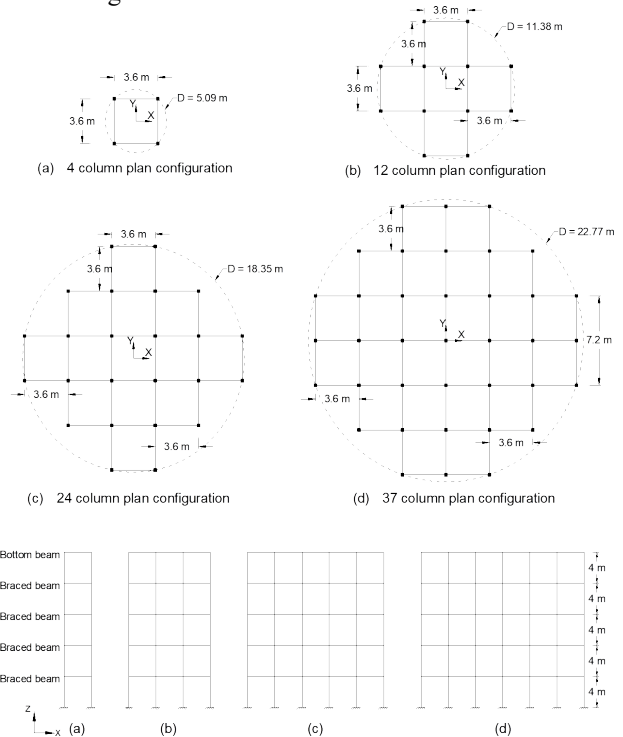


Fig 1. Plan and Elevation of (a) Small, (b) Medium, (c) Large and (d) Very Large Tank

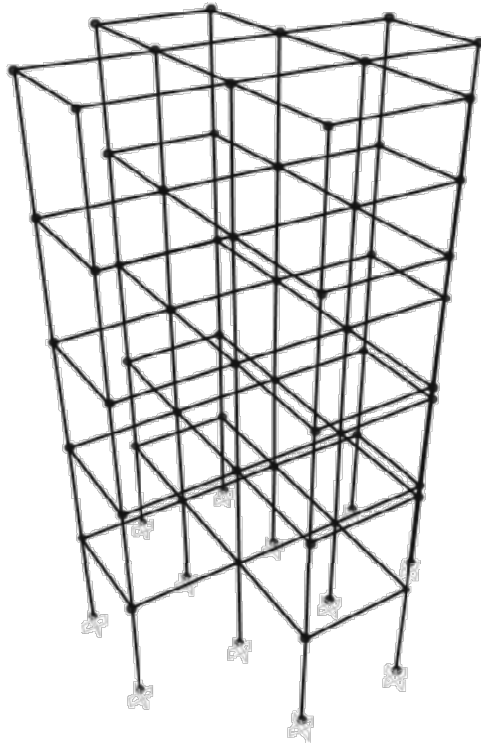


Fig 2. Frame staging of the medium capacity tank
(Modeled in SAP2000 software)

4. Design Approach

Various Indian standards were compared which are explained in the specification and the minimum grade of steel, concrete, and the cover depth based on the exposure condition were identified. For the design of all four types of tanks M30 grade of concrete, Fe415 grade of steel, and 45mm cover depth is used. Since the EBD approach accounts for the inelastic behavior in design, instead of 'characteristic strength' the 'expected strength' is used in the design. The Indian standards also specify the minimum dimension of column and brace beams to be used, but this study is concentrated on the comparison of the performance by each design approaches those guidelines were not applicable. For comparing the design approaches the geometry and the members sized should be the same because a small change in geometry or member size will change the period and its corresponding design forces like base shear. The sizing of the members was done on two approaches, one based on the gravity load and the other based on the lateral load. Generally, the FBD approach considers the load factor. Due to enhanced gravity load, the beam sizing is governed by the FBD approach. The column sizing is governed by the lateral load computed from the EBD approach. The final member sizes used for the design are shown in Table 2. After fixing the sizing, analysis and design have been carried out using SAP2000.

Table 2. Beams and Columns sizes

Tank	Column Size (mm)	Bottom Beam Size (mm)	Brace Beam Size (mm)
Small	450 x 450	300 x 500	300 x 350
Medium	525 x 525	450 x 600	300 x 400
Large	550 x 550	500 x 700	300 x 450
Very Large	550 x 550	450 x 700	300 x 450

The design base shear calculation by FBD is done based on [1] considering the tank is resting over a hard stratum. For calculating the design base shear by EBD, the same spectra considered in FBD is used, but in FBD the base shear will again be enhanced by the load factor. Thus, the spectral acceleration value which is obtained from the response spectra is multiplied by 1.5 (*load factor used in FBD*) and then divided by 2 consider design level response value instead of maximum response (*In FBD it is used in form of Z/2*). The design base shear in EBD is calculated with an assumed target drift of 1.4%. In this design, the yield rotation is assumed to be 0.25%, in order to have a reasonable amount of energy to be dissipated by plastic deformation. Selecting the appropriate post-yield stiffness (*r*) is important [23]. Sufficient post-yield stiffness will reduce the maximum displacement response for a short period, for intermediate and long period structures, increasing of post-yield stiffness slightly increases maximum displacement response. The design base shear obtained from the calculation is shown in Table 3. The ESRs are designed using IS 456 [24] and ductility is ensured as per IS 13920 [18]. Generally, the columns are designed by the column tree method in the EBD approach, however, it has been found there are some limitations to use this approach for ESR. It is observed from the literature that the ESR designed by direct-displacement-based design approach [25] using the capacity design approach given by Priestley *et al.* [26] shows a good performance. Therefore, in the present study, the capacity design approach is used to design the ESRs for the EBD approach.

From Table 3, it is observed that the base shear obtained by the EBD approach considering the inelastic properties is lesser than the base shear calculated from the FBD approach without applying the response reduction factor in the load. Thus, the elastic analysis estimates a higher design force than inelastic analysis.

Table 3. Design Base Shear Comparison

Tank	Base Shear by FBD (kN)	Base Shear by FBD x R (kN)	Base Shear by EBD (kN)
Small	99*	396	261
Medium	514*	2056	1446
Large	1300*	5200	3877
Very Large	1982*	7928	6267

* design base shear calculated using a load factor as 1.5

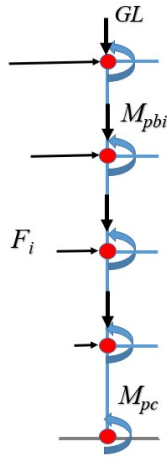


Fig 3. Free Body Diagram of Column Tree Method

4.1 Limitations of Column Tree Method

The column tree method is a design method for the column given by Leelataviwat et al. [27]. In this method, it is assumed that all the beams are yielded and plastic rotation started without any hinge formation in the column. At that point, the beam will only transfer the gravity load, yield moment capacity of the beam to column with the lateral load at each diaphragm level. As stated above the column is idealized form the complete structure as shown in Fig 3 which will behave as a determinate member. The lateral forces shown at each level has to be calculated based on the participation of the storey mass. But it is known that the ESR is an inverted pendulum structure with lumped mass at its top. The mass is not distributed at each level like building. Thus, almost the complete base shear has to be applied at the top level due to which the moment in the column will be very high. From the design, it is observed the column shows reinforcement percentage much higher than the prescribed limit. In case of increasing the size of the column, the base shear will increase, and again the same repeats. Thus, from this study, it is observed that the column tree method has certain limitations and therefore, not suitable for ESRs.

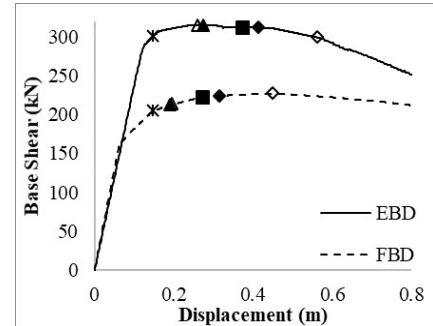
5. Nonlinear Analysis

To assess the performance and behavior of the ESRs designed by each approach nonlinear static and dynamic analysis were done. For nonlinear analysis, the designed ESRs are used directly without grouping the designed reinforcement. As the performance of models with grouped reinforcement overestimates the performance.

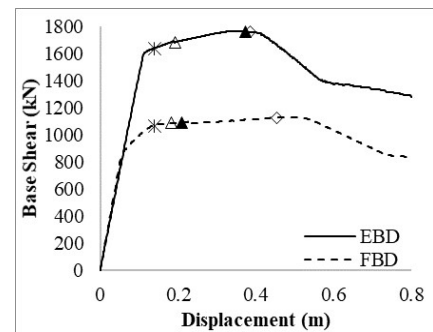
5.1 Nonlinear Static Pushover Analysis

Pushover analysis is a method where the structure is subjected to gravity loading and monotonic displacement controlled lateral load in a specified pattern. Through this method, the capacity of the structure is obtained in the form of base shear versus displacement. To capture the actual nonlinear properties of the beam and column nonlinear property should be assigned. The nonlinear property is assigned in the form of lumped plasticity like user-defined

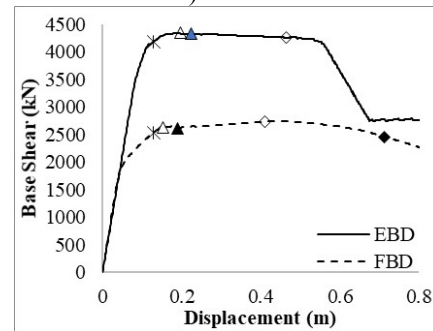
or auto hinges. In this study, the auto hinges defined from tables of ASCE 41-13 is assigned to column and beams. After assigning the hinges the analysis is performed and the results are extracted. The capacity curve obtained from pushover analysis for all 4 types of tank designed by each approach is shown in Fig 4. The capacity curves are then bi-



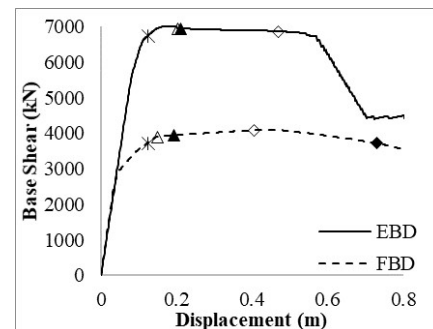
a) Small



b) Medium



c) Large



d) Very Large

* Target Dsplacement	△ IO_Beam	▲ IO_Column
■ LS_Column	◇ C_Beam	◆ C_Column

Fig 4. Capacity Curve

Table 4. Over Strength Ratio from Capacity Curve

Tank	Design Approach	Yield Base Shear (kN)	Design Base Shear (kN)	Over Strength
Small	FBD	221	99*	2.23
	EBD	298	261	1.14
Medium	FBD	1035	514*	2.01
	EBD	1632	1446	1.13
Large	FBD	2456	1300*	1.89
	EBD	4056	3877	1.05
Very large	FBD	3638	1982*	1.84
	EBD	6550	6267	1.05

linearized as per the procedure prescribed by ATC-40 [28], after which the yield base shear and the overstrength are summarised in Table 4.

5.2 Nonlinear Dynamic Time History Analysis

Standards like ASCE/SEI 41-17 [29] recommends Nonlinear dynamic analysis for all type of structures. Since nonlinear time history analysis is the most accurate method to predict the force-deformation demands and inter-storey drift of the structure. The inelastic demands of the structure are obtained by nonlinear dynamic analysis, where the real ground motions are selected to capture the performance and the behavior of the structure. The nonlinear time history analysis is very sensitive to the modeling and ground motion characteristics. The proper modeling of cyclic load-deformation characteristics and deterioration properties of structural elements should be considered. The nonlinear time history analysis exhibits cyclic loading and load reversal, the results may get affected by hysteretic behavior type. The energy dissipation of the structure depends on the type of hysteresis model and different hysteresis models are available. Takeda Hysteresis model shown in Fig 5 has been used here for stimulating degrading hysteretic behavior.

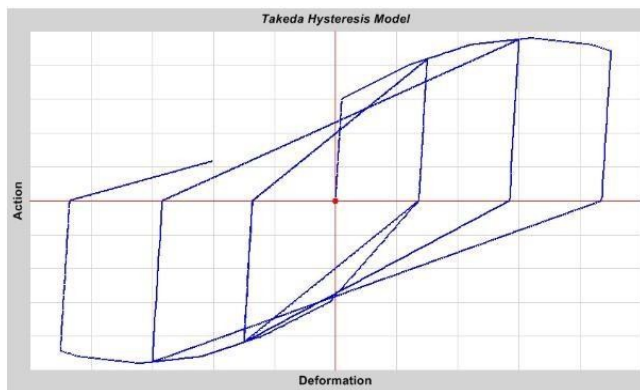


Fig 5 Takeda Hysteresis Model under Increasing Cyclic Load [30]

5.2.1 Selection of Ground Motion

The criteria for the selection of ground motion had been given by different codes. According to Eurocode-8 [31] the response spectra which gives the maximum value has to be considered for the set of three ground motion records, whereas for the set of seven ground motion records the mean response spectra has to be considered for the analysis. The Eurocode also says that the value of the response spectra obtained for the 5% damping from the ground motion records should not be less than 90% of the corresponding value obtained from 5% damping of elastic response spectra. As per ASCE 7-16 [32], a suite of not less than 11 ground motions has to be selected from the same general tectonic regime having similar magnitudes and fault distances since these parameters control the target spectrum. ASCE 7-16 [32] also says that the 5% damping mean spectra should lie above the target spectra in the range of $0.2T$ to $2T$, where T is the fundamental period of the structures for which the nonlinear time history analysis is to be done. The selection of ground motion as per FEMA P695 [33] discusses various factors affecting the selection of ground motion in which some of them are discussed below. The magnitude of the ground motion plays an important role since the large magnitude ground motions travel larger distances with longer duration and it also releases larger energy. The source type and the soil type have to be the same to eliminate the difference in the intensity of the ground motion. Selection of the number of records per event is also an important criterion, for an earthquake if more than two records are there, the two records with the highest peak ground velocity are considered to be on the conservative side. The scaling and spectral matching of selected ground motion with the target spectra is also necessary for the nonlinear analysis. The other criteria given by the researchers Araujo et al. [34] is that spectral mismatching has to be considered in the selection of ground motion. When the response spectra of the selected ground motion fall beyond $\pm 50\%$ of the target spectra, those ground motions have to be removed from the suite of selected ground motion and some other records have to be selected which lies within the range. The selected ground motion and its parameters are shown in Table 5 and the response spectra of scaled time histories and its mean is shown in Fig 6.

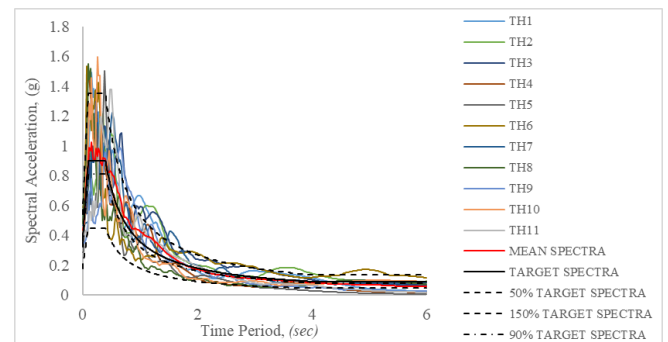


Fig 6. Response Spectra of Selected Ground Motion

Table 5 Parameters of Selected Ground Motion Records

ID	Earthquake	Year	Station Name	Magnitude	Mechanism	Vs30 (m/sec)
TH1	San Fernando	1971	Cedar Springs Allen Ranch	6.61	Reverse	813.48
TH2	Loma Prieta	1989	Piedmont Jr High School Grounds	6.93	Reverse Oblique	895.36
TH3	Loma Prieta	1989	So San Francisco Sierra Pt.	6.93	Reverse Oblique	1020.62
TH4	Northridge-01	1994	LA - Wonderland Ave	6.69	Reverse	1222.52
TH5	Northridge-01	1994	Vasquez Rocks Park	6.69	Reverse	996.43
TH6	Chi-Chi_ Taiwan	1999	ILA063	7.62	Reverse Oblique	996.51
TH7	Chi-Chi_ Taiwan	1999	TTN042	7.62	Reverse Oblique	845.34
TH8	El Mayor-Cucapah_ Mexico	2010	San Diego Road Dept	7.20	strike slip	827.00
TH9	Parkfield-02_ CA	2004	Diablo Canyon Power Plant	6.00	strike slip	1100.00
TH10	El Mayor-Cucapah_ Mexico	2010	Palm Desert	7.20	strike slip	786.00
TH11	40204628	2007	Columbia College Columbia CA USA	5.45	strike slip	760.00

5.2.2 Results and Discussion

The nonlinear dynamic analysis is done with the help of the selected time histories. The time history analysis is done by direct integration method by which more appropriate inelastic behaviour can be obtained. From the analysis, it is observed that the hinge formation starts from the mid brace level and proceeds above. It is also observed that only B-type hinges were formed in staging designed by EBD and other than B-type hinges some IO type hinges were observed in the staging designed by FBD. The hinges

formed at beams and bottom of the column for tanks designed by EBD, whereas, for tanks designed by FBD the hinges are formed at beams and also at different column levels. To study the behaviour of the ESR the displacement profile and brace level drift were extracted from the nonlinear dynamic analysis results and shown in Fig 7a and Fig 7b. Standards like FEMA 356 [35] and ASCE 41-17 [29] describe that for immediate occupancy 1% is the maximum drift. Thus, the drift results were checked for the same limit.

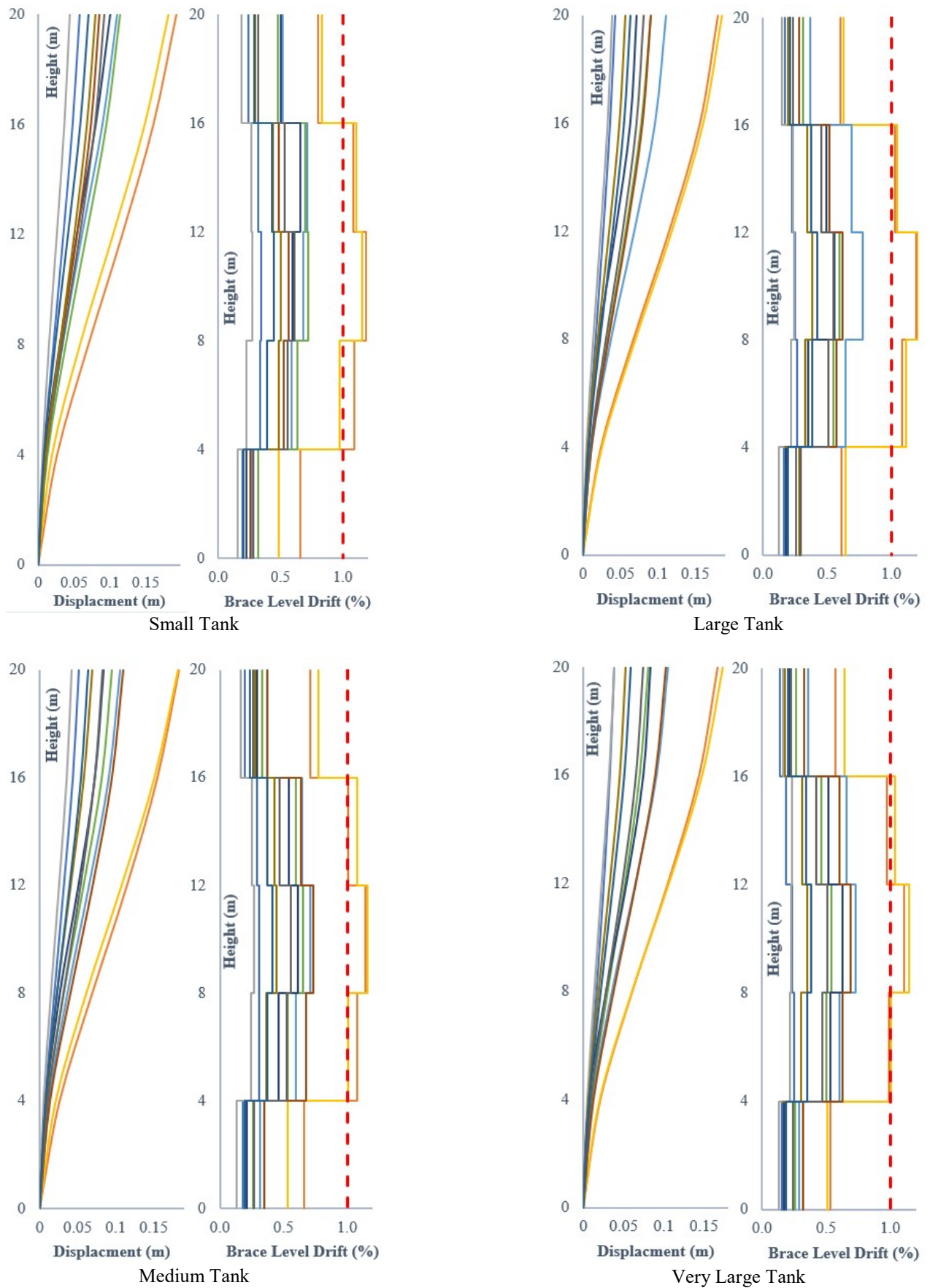


Fig 7a Displacement and Brace Level Drift Profile of ESRs Designed by FBD Approach

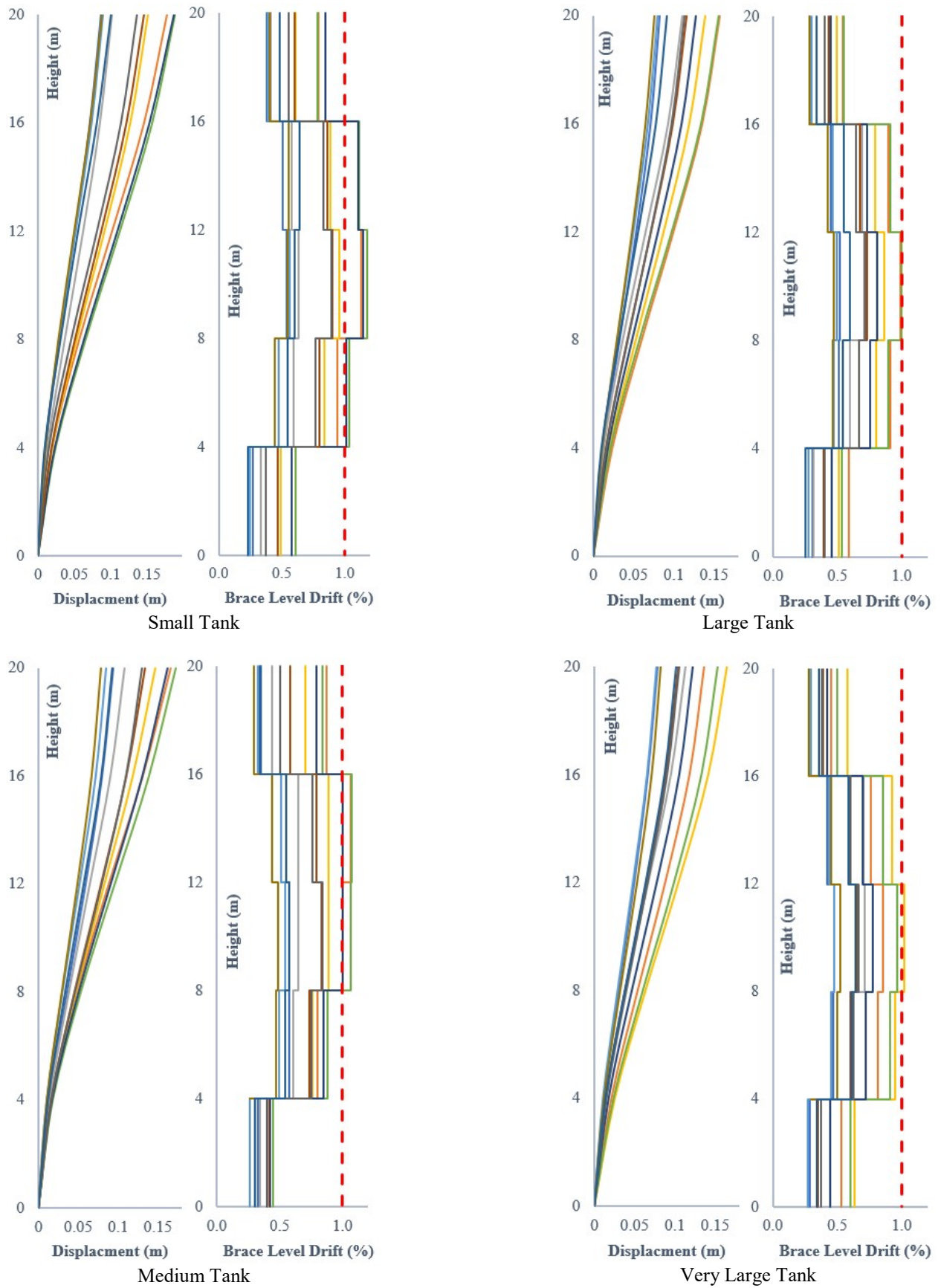


Fig 7b Displacement and Brace Level Drift Profile of ESRs Designed by EBD Approach

6. Conclusions

The ESRs with RC staging designed by FBD and EBD were analyzed by nonlinear analysis. From the capacity curve, it is clear that in both design approaches the IO-type hinges are developing after the target drift only. So, it can be concluded that the ESRs can remain operational after the earthquake with minor damage. From the hinge formation observed in the nonlinear static pushover analysis and nonlinear dynamic time history analysis, it is clear that the tanks designed by FBD will lead to local failure mechanisms (which is undesirable) whereas, the tanks designed by the EBD approach will lead to global failure mechanism. The overstrength obtained from pushover analysis shows that those values are almost equal to the overstrength factor given in ASCE 7-16 [32]. The ESRs designed by FBD has a large variation in the displacement profile but ESRs designed by EBD show displacement profile with less variation. From the drift profile, it is clear the large and very large tanks designed by EBD seems to be within the limits. Though the small and medium ESRs drift profile exceeds the limits given by standards, the limits are still within the assumed target drift.

Disclosures

Free Access to this article is sponsored by SARL ALPHA CRISTO INDUSTRIAL.

Reference

1. Bureau of Indian Standards. Criteria for Earthquake Resistant Design of Structures – Part 2: Liquid retaining tanks. IS 1893-2, 2014; New Delhi, India
2. Steinbrugge KV, Flores AR. The Chilean earthquakes of May, 1960: A structural engineering viewpoint. Bulletin of the Seismological Society of America, 1963; 53(2), 225-307.
3. Mehra M. Reconnaissance report on the Northern Iran earthquake of June 21, 1990. National Center for Earthquake Engineering Research, 1990; State University of New York at Buffalo, New York, USA.
4. Astanek A, Ashtiany MG. The Manjil, Iran, earthquake of June 1990", Report No. 24(12); Earthquake Engineering Research Institute, 1990; California, USA.
5. Jain SK, Murty CVR, Chandak N, Seeber L, Jain NK. The September 29, 1993, M6. 4 Killari, Maharashtra Earthquake in Central India. EERI Special Earthquake Report, EERI Newsletter, 1994; 28(1), 8.
6. Saffarini HS. Ground motion characteristics of the November 1995 Aqaba earthquake. Engineering structures, 2000; 22(4), 343-351.
7. Rai DC. Elevated tanks. Earthquake spectra, 2002; 18(S1), 279-295.
8. Rai DC. Performance of elevated tanks in Mw 7.7 Bhuj earthquake of January 26th, 2001. Journal of Earth System Science, 2003; 112(3), 421-429.
9. Housner GW. Limit design of structures to resist earthquakes. In Proceedings of 1st WCEE, EERI, Oakland, California, USA, 1956; Volume 5, 1-13.
10. Leelataviwat S, Goel SC, Stojadinovic B. Energy-based seismic design of structures using yield mechanism and target drift. Journal of Structural Engineering, 2002; 128(8), 1046-1054.
11. Choi H, Kim J. Energy-based seismic design of buckling-restrained braced frames using hysteretic energy spectrum. Engineering Structures, 2006; 28(2), 304-311.
12. Liao WC, Goel SC. Performance-based plastic design and energy-based evaluation of seismic resistant RC moment frame. Journal of Marine Science and technology, 2012; 20(3), 304-310.
13. Merter O, Ucar T. Energy-based design base shear for RC frames considering global failure mechanism and reduced hysteretic behavior. Structural Engineering and Mechanics, 2017; 63(1), 23-35.
14. Benavent-Climent, A, Pujades L, Lopez-Almansa F. Design energy input spectra for moderate-seismicity regions. Earthquake engineering and structural dynamics, 2002; 31(5), 1151-1172
15. Gulkan P, Sozen MA. Inelastic responses of reinforced concrete structure to earthquake motions. ACI Journal Proceedings, 71(12), 1974; 604-610
16. Bureau of Indian Standards. Criteria for design of RCC staging for overhead water tanks. IS 11682 (Draft Code), 2011; New Delhi, India
17. Bureau of Indian Standards. Criteria for design of RCC staging for overhead water tanks. IS 11682, 1985; New Delhi, India
18. Bureau of Indian Standards. Ductile design and detailing of reinforced concrete structures subjected to seismic forces – code of practice (first revision). IS 13920, 2016; New Delhi, India
19. Bureau of Indian Standards. Concrete structures for storage of liquids – code of practice – Part 1: General requirements. IS 3370-1, 2009; New Delhi, India
20. Bureau of Indian Standards. Concrete structures for storage of liquids – code of practice – Part 2: Reinforced concrete structures. IS 3370-2, 2009; New Delhi, India
21. Bureau of Indian Standards. Concrete structures for storage of liquids – code of practice – Part 3: Prestressed concrete structures. IS 3370-3, 1967; New Delhi, India
22. Bureau of Indian Standards. Concrete structures for storage of liquids – code of practice – Part 4: Design tables. IS 3370-4, 1967; New Delhi, India
23. Ye LP, Lu XZ, Ma QL, Cheng GY, Song SY, Miao ZW, Pan P. Study on the influence of post-yielding stiffness to the seismic response of building structures. 14th World Conference on Earthquake Engineering, 2008; Beijing, China, October.
24. Bureau of Indian Standards. Plain reinforced concrete – code of practice (fourth revision). IS 456, 2000; New Delhi, India
25. Kumbhar O, Kumar R, Panaiyappan PL, Norooznejad Farsangi E. Direct Displacement Based Design of Reinforced Concrete Elevated Water Tanks Frame Staging. International Journal of Engineering, 2019; 32(10), 1395-1406

26. Priestley MJN, Calvi MC, Kowalsky MJ. Displacement Based Seismic Design of Structures. In New Zealand Conference on Earthquake Engineering, 2017; Pavia
27. Leelataviwat S, Goel SC, Stojadinovic B. Toward performance-based seismic design of structures. Earthquake spectra, 1999; 15(3), 435-461.
28. Applied Technology Council. Seismic Evaluation and Retrofit of Concrete Buildings. ATC-40, 1996; Volume 1, USA.
29. American Society of Civil Engineering. Seismic Evaluation and Retrofit of Existing Building. ASCE/SEI 41-17, 2017; Reston, Virginia.
30. SAP2000. (V16). Integrated Software for Structural Analysis and Design. Computers and Structures Inc., Berkeley, California.
31. Eurocode 8: Design of structures for earthquake resistance— Part 1: General rules, seismic actions and rules for buildings. European Committee for Standardization, Brussels, EN 1998 -1, 2004; Belgium.
32. American Society of Civil Engineering. Minimum Design Loads and Associated Criteria for Buildings and Other Structures. ASCE/SEI 7-16, 2016; Reston, Virginia.
33. Federal Emergency Management Agency. (2009). Quantification of Building Seismic Performance Factors. FEMA P695, 2009; Washington, D.C.
34. Araújo M, Macedo L, Marques M, Castro JM. Code-based record selection methods for seismic performance assessment of buildings. Earthquake Engineering & Structural Dynamics, 2016; 45(1), 129-148.
35. Federal Emergency Management Agency. Prestandard and Commentary for the Seismic Rehabilitation of Buildings. FEMA 356, 2000; Washington DC.
36. Bureau of Indian Standards. Criteria for Earthquake Resistant Design of Structures – Part 1: General provisions and buildings. IS 1893-1, 2016; New Delhi, India