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Effects of numerical modeling simplification on seismic design of buildings

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Abstract. The recent seismic events have led to concerns on safety and vulnerability of Reinforced Concrete Moment Resisting Frame "RC-MRF" buildings. The seismic design demands are greatly dependent on the computational tools, the inherent assumptions and approximations introduced in the modeling process. Thus, it is essential to assess the relative importance of implementing different modeling approaches and investigate the computed response sensitivity to the corresponding modeling assumptions. Many parameters and assumptions are to be justified for generation effective and accurate structural models of RC-MRF buildings to simulate the lateral response and evaluate seismic design demands. So, the present study aims to develop reliable finite element model through many refinements in modeling the various structural components. The effect of finite element modeling assumptions, analysis methods and code provisions on seismic response demands for the structural design of RC-MRF buildings are investigated. where, a series of three-dimensional finite element models were created to study various approaches to quantitatively improve the accuracy of FE models of symmetric buildings located in active seismic zones. It is shown from results of the comparative analyses that the use of a calibrated frame model which was made up of line elements featuring rigid offsets manages to provide estimates that match best with estimates obtained from a much more rigorous modeling approach involving the use of shell elements.

Keywords: MRF buildings; codes provisions; seismic design demands; finite element modeling; modeling assumptions

1. Introduction

Through the recorded history, many earthquakes had occurred; damaged buildings and led to

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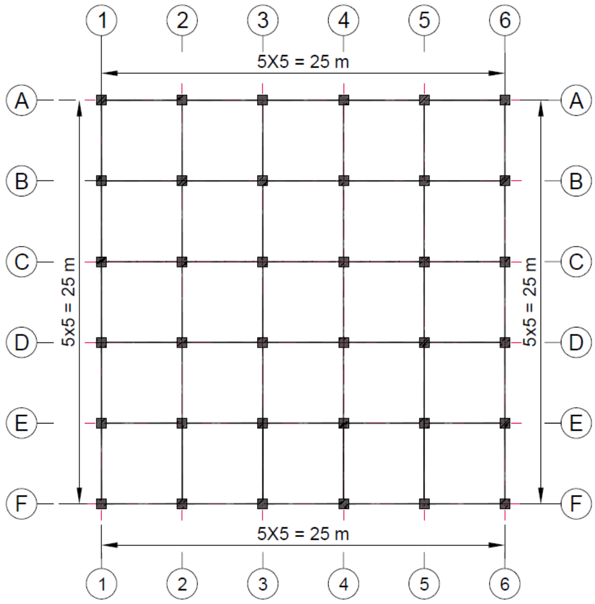
injuries and loss of lives. There is evidently potential for damaging earthquakes in the future, the seismic risk to human life and infrastructure increases. Recent awareness of potential seismic events in low to moderate seismicity regions has led to concerns about safety and vulnerability of buildings. Although extensive research has been carried out to develop robust modeling techniques for the seismic response of RC-MRF buildings (ASCE 2006), there is still a dire need for applications of relatively simple elastic modeling approach for the practical structural design of tall buildings (Wallace 2007. Practical elastic models normally reduce computational complication and design work (Shin *et al.* 2010, Celebi *et al.* 2012, Mazzotta *et al.* 2017). Modeling assumptions, boundary, and loading conditions have a significant effect on the analytical assessment of ductility supply and response demand measures for buildings. This is of particular significance when viewed in the light of the large capital investment and problems associated with the satisfaction of dynamic similitude encountered in physical testing (Elnashap and McClure 1996, Liang *et al.* 2017). During the past twenty years, extensive strong motion instrumentation projects have been developed to monitor seismic events and structural performances. Many earthquake events have been recorded and analyzed to improve our understanding of the seismic behavior of civil structures (Uang *et al.* 1997, Celebi 2003). Moreover, these recorded data are being utilized to assess and improve various modeling techniques, e.g., Finite Element modeling (Boroschek and Mahin 1991, Snaebjornsson *et al.* 1992, Liu *et al.* 2004) so that they can be applied in new structure design or existing structure retrofitting. Evaluation of seismic response of buildings is subjected to a considerable degree of approximation and simplification of the real behavior. Generating effective and accurate structural models of RC buildings to simulate the actual seismic lateral response depends on the parameters and assumptions adopted in such structural models, which have significant effects on the drift and strength seismic demands. Three-dimensional finite element structural modeling is the most applicable method for analysis and design of buildings (Abdel Raheem *et al.* 2018a, b).

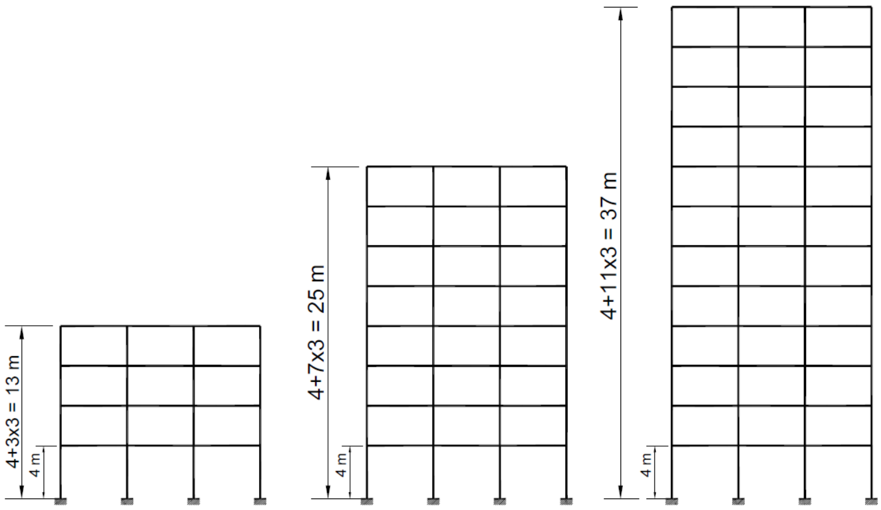
Numerical analysis of structures relies on the designer's understanding of structural behavior, choice of appropriate software and method of analysis. The simple structural analysis is obviously more cost effective and requires less work effort if it proves that the building structure is in a satisfactory condition. Otherwise, more advanced structural analysis should be used in order to prevent unnecessary remedial actions and large economic losses. For the analysis of building structure, one-dimensional finite elements using two-node Euler-Bernoulli beam-column element based on centerline dimensions and two-dimensional finite elements using four-node rectangular shell elements for plate bending together with appropriate finite element meshing schemes are routinely incorporated to represent the material, structural, and inertial properties of the buildings structural members and to define their topology and connectivity in typical numerical finite element models (Avramidis *et al.* 2016). The commercial software program, such as ETABS (CSI 2015), has frequently applied the rigid diaphragm assumption for the simplicity of the analysis procedure. In this case, the flexural stiffness of the floor slabs is usually overlooked in the analysis. Moreover, even though beams are positioned under the floor slabs in the building, the analytical model was developed assuming that the axes of beams and floor slabs are located in a common plane. Therefore, the flexural stiffness of the floor slabs and the T-beam effect are disregarded, hence substantial analytical errors could occur (Lee *et al.* 2003, Zeris *et al.* 2007). In spite of that Krawinkler (2000) has shown that a linear elastic model using centerline dimensions is acceptable for design of special moment frames. Even though this model produces sufficient results for the design, it will not always lead to good assessments of the distribution of shears, moments and axial forces throughout the building under seismic loads. In another approach, scaling up the beam inertia is used to simulate the realistic case of a projected beam (Soliman *et al.* 2012, Mehanny *et al.* 2012). The inertia of T-beams is modified to simulate the realistic behavior of beams when subjected to gravitational and lateral loads. The results show that the modification of T-beam inertia in modeling the RC frames modeling improves the overall lateral stiffness of buildings, hence could effectively simulate the realistic behavior of RC-MRF buildings under lateral loads. Moreover, buildings with high beam inertia have a smaller fundamental period of vibration, and consequently attract higher base shear values compared to the buildings without beam inertia modification and often satisfy the code requirement for drift limitations. But, the magnification approach of the beam stiffness relative to the column stiffness could violate the preferable weak beam-strong column design concept.

Toward a more refined modeling approach, a finite dimension of a beam-column joint is modeled by including rigid eccentricities at the ends of the beam-column element to account for the effect of the joint geometry; the joints are usually assumed to be rigid (Shin *et al.* 2010, Lima *et al.* 2017). Consideration of rigid offsets as an alternative to the centerline dimensions of elements led to important changes in global structural strength and stiffness as well as the change of relative story shear strengths. Consequently, there was a significant change in the distribution of inter-story drift demands along the building’s height. However, using rigid connections may not accurately represent the strength and stiffness of the structural frame as well as the story drift and the lateral displacement of the building. So, results from such models may overestimate the deformationcapacity of the buildings. Test results show that beam-column joints can experience significant shear deformations even prior to yielding of the longitudinal reinforcement within the joint (Walker *et al.* 2002). The shear deformations effects can be approximated by extending the beam or column flexibility into the joint in the analytical model. Although some structural analysis software programs allow for the modeling of panel zone shear deformation explicitly, most of them have implemented an end zone offset factor. This factor adjusts the length of beams and columns in the beam-column joint to account for the contribution of panel zone implicitly.

This paper investigates the effects of T-beam and beam-column joint rigid offset in modeling structural elements on the seismic design demands of RC-MRF buildings to more precisely predict the behavior of buildings under seismic lateral loads. The study aims to perform a thorough evaluation of the different modeling techniques ranging from simple to refined techniques in order to recommend a robust FE model that could generate structural analysis results that can match the real structural behavior of RC-MRF buildings. A series of three-dimensional FE models were created to study various approaches to quantitatively improve the accuracy of FE models of symmetric buildings located in active seismic zones. Five different approaches to model moment resisting frames with different degrees of complexity are introduced. A quantitative measure of the importance of modeling assumptions and FE Modeling refinement level on the predicted response is formulated through a comparison among simplified and refined models. A reference refined model based on shell modeling approach is formulated for the quantification of the accuracy of the simplified models and determination of modification factor for the calibration. The results obtained from each modeling approach are presented are compared in terms of the displacement profile, inter-story drift, and story shear forces. The comparative analyses results show that using the calibrated frame model, which was made up of line elements featuring rigid offsets, led to estimates that match best with the estimates obtained from the more rigorous shell-element modeling approach.



(a) Plan configuration



(b) Elevation configuration

Fig. 1 Model configuration of the studied buildings

2. Structural design of the buildings

Buildings with three different numbers of story with the typical floor height of 3 m and ground floor height of 4 m have been considered. In this study, 4-story, 8-story, and 12-story buildings are selected, where building's layout is bi-symmetric square in plan with 5 equal bays of 5 m width in both directions, as shown in Fig. 1. The columns' bases are assumed to be fixed to the foundation and have constant cross-section along the building's height. ElAssaly (2013) investigated the effects of varying stiffness coefficients of columns and steel ratios of connecting beams on seismic behavior of different configurations and structural systems, of MRF buildings. It was concluded that reducing the column stiffness along the building's height would generally lead to an increase in the fundamental periods of the structure; consequently, it becomes more flexible than structures that have constant columns along the height. Meanwhile, this has minor effects on the maximum total displacement and on the total base shear of the building. On the other hand, increasing the relative stiffness of the connecting beams, compared to that of columns in the upper floors of a building, must be avoided since it may result in substantial high values of drift ratios coupled with high damage indices for columns located at those floors. The capacity design rules are adopted, where the brittle failure or other harmful failure mechanisms (plastic hinges in columns, shear failure of structural elements, failure of beam-column joints, yielding of foundations) shall be prohibited, through the definition of the design actions in selected regions from equilibrium conditions. For the MRF structural systems, the capacity design condition should be fulfilled at beam-column joints

$$\sum M_{RC} \geq 1.3 \sum M_{Rb} \tag{1}$$

The gravitational forces are low in the higher floors and their behavior is usually controlled by the moments while, on the other hand, the gravitational forces are higher in the lower floors and their design is controlled by axial forces. Thus, both the gravitational and the lateral forces have a great effect on the column designs. As a result, the reinforcement required in the higher floors columns is more that is required for the columns in the lower floors. In this study, a design concept for the columns with the same cross-section over the complete height of the buildings is adopted. Maintaining constant cross-section of columns over the full height of the building simplifies the framework and limits the impact of modeling refinement level on seismic design demands of buildings to the modeling of the floor system. The building models are analyzed and designed using ETABS (CSI 2015) and based on the seismic design provisions from ASCE7-10 (ASCE 2010). A solid slab of 0.2 m thickness is used on all floors. The size and the reinforcement of structural members of the MRF building models are given in Table 1.

The structural design of the MRF building is done based on the centerline modeling approach. The beam-to-column connections were assumed to be fully rigid. A semi-rigid diaphragm constraint is imposed to simulate actual in-plane stiffness properties and behavior of the slab. For gravity load design, dead loads include the structure self-weight; a floor cover of 1.5 kN/m^2; an equivalent load of 1.0 kN/m^2 for plastering and partition walls. A live load of 2.0 kN/m^2 is considered. According to section 19.2.1.3 of ACI 318-14 (ACI 2014), the specified compressive strength shall be based on the 28-day test results from concrete cylinders unless otherwise specified in the construction documents. Concrete strength measured using concrete cubes produce a results different than concrete cylinders. Conservative estimates put concrete cylinders at 80% as the lower limit of concrete cubes. The concrete has a compressive strength of 30 MPa and the steel rebar have a yield stress of 460 MPa.

The seismic design has been carried out based on design response spectrum from ASCE 7-10 code provisions for lateral seismic loads as shown in Fig. 2, with assumption of soil type B; response modification factor R = 5; system over-strength factor Ω = 3; deflection amplification factor C_d = 4.5. A total seismic mass including self-weight and floor cover plus 25% of live load is considered (ASCE 2010, Abdel Raheem *et al.* 2015). The algorithm for determining seismic loads is based on ASCE 7-10 Section 12.8, where the seismic base shear, *V*, in a given direction could be

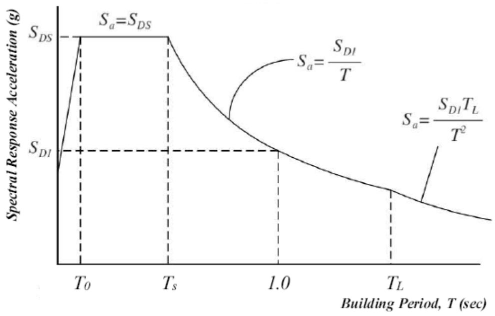


Fig. 2 Design response spectrum

Table 1 Design for Sizes and reinforcement of structural members of MRF building models

Building	Beams			Columns	
	Cross section b × h (cm × cm)	Steel bars		Cross section a × a (cm × cm)	Steel bars
		Top layer	Top layer		
4-Story Building	20 × 60	4 ϕ16	4 ϕ16	40 × 40	12 ϕ16
8-Story Building	20 × 60	4 ϕ16	4 ϕ16	45 × 45	12 ϕ20
12-Story Building	20 × 60	4 ϕ16	4 ϕ16	55 × 55	16 ϕ20

determined as follows

$$V = C_s W \tag{2}$$

where C_s is the seismic response coefficient, W is the effective seismic weight. The seismic response coefficient is determined as

$$C_s = \frac{S_{DS}}{R/I_e} \tag{3}$$

where S_{DS} the design spectral response acceleration at short periods of 5% damping, R is the response modification factor, I_e is the importance factor. The value of computed seismic response coefficient shall not exceed

$$C_s = \frac{S_{D1}}{T(R/I_e)} \quad \text{for} \quad T \le T_L \quad \text{and} \quad C_s = \frac{S_{D1}T_L}{T^2(R/I_e)} \quad \text{for} \quad T > T_L \tag{4}$$

where S_{D1} is the design spectral response acceleration at a period of 1.0 s, T is the fundamental period of the building, T_L is the long-period transition period. The design spectral response acceleration parameters S_{DS} and S_{D1} could be determined from the mapped values of S_s and S_1. The mapped risk-targeted maximum considered earthquake spectral response acceleration parametersare provided in seismic design maps at short period and at 1 s period.

$$S_{DS} = \frac{2}{3}F_a S_s \quad \text{and} \quad S_{D1} = \frac{2}{3}F_v S_1 \tag{5}$$

Where F_a , F_v are spectral response acceleration parameter at short period at 1 s period (ASCE 2010).

3. Numerical modeling assumptions

Various three-dimensional (3D) modeling techniques have been used by many researchers to idealize structure geometries that range from using detailed 3D solid modeling of all structural components to using the grillage approach (Chan and Chan 1999, Lee *et al.* 2003, Sousa *et al.* 2012, Kwon and Ghannoum 2016, Zendaoui *et al.* 2016, Abdel Raheem *et al.* 2017). To obtain an accurate seismic response of a building, it is required to develop a FE model that is capable of incorporating all the structural elements as well as knowledge of their true behavior. The currently available approaches are either too simplified or highly advanced. The simplified models neglect the appropriate inter-component interaction, and hence the resulting models are just crude approximations lacking general applicability. On the other hand, advanced simulations are needed in most cases to advanced reflect the real structural behavior. However, these advanced simulations usually require more work efforts. The outcome of advanced simulations analysis has generally been more favorable than the outcomes of analysis performed using the simplified models. Analytical methods and models of various complexities for RC-MRF buildings are developed to evaluate the different models' ability to predict the global and local performance of multistory buildings; and the effects of analysis assumptions on the seismic design demand predictions.

Three modeling approaches are investigated: centerline dimensions of elements versus rigid offsets versus shell elements. Analytical models of such frames are often developed using line elements based on centerline dimensions of beams and columns. However, it is usually required to account for the finite dimensions of the beam-column joints and the eccentric T-section beam by considering rigid offsets. Since beams and floor slabs are not located in a common plane, rigid bodies are introduced to represent the T-beam effects as the first step of model refinement. In the second step in model refinement, the dimension of beam-column joint is considered through rigid eccentricities at the ends of beam-column element while the joints are assumed to be rigid. It should be noted that using rigid connections may not properly represent the strength and stiffness of the structural frame as well as the story drift and the overall deflection of the structure. Results from such models may overestimate the ductile capacity of the buildings (Le-Trung*et al.* 2010). A simplified procedure for estimating the global and the local seismic demands is needed to facilitate decision making in the conceptual design process. The global responses in terms of base shear, lateral displacement and inter-story drift are compared. The developed calibrated frame model is made up of line elements featuring rigid offsets and leads to estimates that match best with estimates obtained from a much more rigorous modeling approach using the shell elements.

Three-dimensional finite element models are constructed using ETABS software for analysis and design. In addition, SAFE software is used to design and check the long-term deflection and the punching of the floor slab. A 3D numerical model of the physical structure is used to represent the spatial distribution of the mass and stiffness of the building for the determination of the substantial features of the building’s dynamic behavior. The serviceability limit state (SLS) aims to minimize any future structural damage due to relatively low, and not unexpected, forces. SLS criteria should be attained during elastic structural behavior, when damage indicated by, cracking or deformation, is still relatively low (Dymiotis-Wellington and Vlachaki 2004). Bertero *et al.* (1991), by considering various seismic codes, arrived at the conclusion that recommended limiting inter-story drifts vary between 0.06 and 0.6%. The SLS aims to limit damage to buildings due to earthquakes, whereas the ULS aims to provide integrity and residual capacity in case an extraordinary earthquake strikes.

The finite element model and nonlinear analysis are formulated considering geometric nonlinearity due to large geometric deformations and P-delta effect, while elastic material behavior is considered for serviceability limit state, SLS. For the seismic analysis, the response spectrum method is used to determine the lateral response demands. The modal response spectrum method provides a more realistic profile of the lateral forces and satisfies the dynamic requirement (ASCE 2010). The modal base shear force should not be less than 85% of the base shear force from the equivalent static force method (NBCC 2005, ECP 2007, 2008, ASCE 2010). For response spectrum method, the square root of the summation of the squares (SSRS) is used for the directional combination, while complete quadratic combination (CQC) is adopted for the modal combination.

4. Finite element meshing and geometry

There is an essential need to find a trade-off solution between obtaining accurate results from seismic analysis and low computational costs. There is a hierarchy of different levels of structural modeling suitable for the structural analysis of RC-MRF buildings. The geometry can be categorized as 1D, 2D and 3D based on the dominant dimensions and then the type of element is selected accordingly. When the time and computation cost are constraints, the appropriate selection of the elements and meshing are so essential. Finite element analysis (FEA) aims to build predictive computational models of real-world scenarios. The use of FEA begins with a computer-aided design (CAD) model that represents the physical parts being simulated as well as knowledge of the material properties and the applied loads and constraints. This information enables the prediction of real-world behavior, often with very high levels of accuracy. As these elements are made smaller and smaller, as the mesh is refined, the computed solution will approach the true solution. This process of mesh refinement is a key step in validating any finite element model and gaining confidence in the software, model, and the results.

The considered structural models represent the progressive steps in modeling complexity that might be adopted in a design office environment. A higher refinement modeling level approach provides a more precise simulation of the actual performance of a building under earthquake lateral loads but necessitates greater work in terms of data preparation time and computational effort. While in the common practice for the analysis and design of buildings, commercial computer programs such as ETABS is frequently used. In ETABS, the rigid diaphragm is assumed in order to simplify the analysis procedure, consequently, the flexural stiffness of the floor slabs is neglected. Furthermore, even though beams are positioned under the floor slabs in the building structure, the analytical model was established with the assumption that the axes of beams and floor slabs are located in a common plane. Therefore, the analytical model disregards the flexural stiffness of the floor slabs and the T-beam effect that would induce substantial analytical errors. Simple modeling methods widely used as well as those with more detailed modeling representations are investigated and compared. Five analytical models are used to evaluate the effect of different modeling assumptions on deformation and force demands. The numerical accuracy of the finite element discretization is analyzed in details. The characteristics of the different models are as follows:

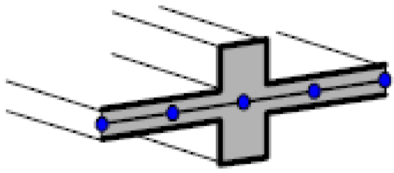


Fig. 3 Centerline based model, Model1

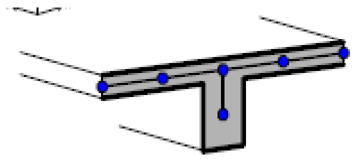


Fig. 4 Beam rigid offset for T-beam effect, Model2

4.1 Centerline based model, Model1

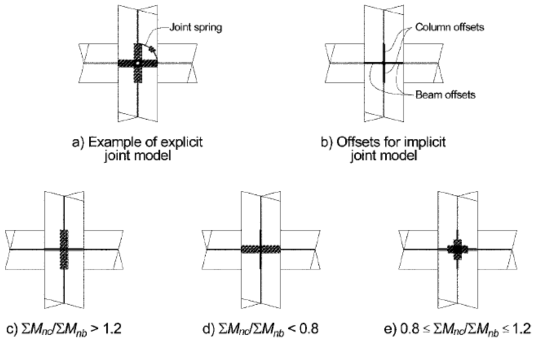
A number of modeling simplification and assumptions are made for building seismic analysis: In the Centerline based modelling approach; Model1, the floor slabs are assumed to act as semi-rigid diaphragms, which ensure integral action of all the vertical lateral load-resisting elements. The axes of beams and floor slabs are assumed to be located in a common plane. Therefore, ignoring the offsets in centerline between beams and floor slab and hence can only account for the flexural stiffness of the floor slab. All columns are typically modeled as being co-linear along their centerline and the small offsets of columns from grids are ignored in the design. Beams and columns were modeled as frame elements with the centerlines joined at nodes (CSI 2015). The strength, stiffness, dimensions, and shear distortions of panel zones are neglected. Moments in beams and column are computed at the connection centerline as against at the faces of columns and beams, which results in high estimates of design demands, Fig. 3.

4.2 Beam rigid offset for T-beam effect, Model2

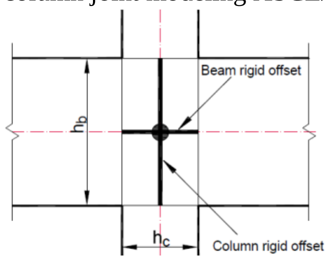
The conventional placement of the finite element discretization nodes is at the slab mid-thickness and the beam longitudinal axis. But beams and floor slabs are not located in a common plane, so an analytical model is developed to assemble the stiffness equation to a common location for the efficient seismic analysis of buildings considering the flexural stiffness of the floor beam. Rigid links are introduced to represent the T-beam effects in the structural modeling of RC-MRF buildings; Model2 can take the flexural stiffness of the floor slab and the T-beam effect into consideration, Fig. 4. T-beam effect contribution to the stiffness of beams could be important for the seismic assessment of building, as it will affect the relative beams/columns stiffness. The rigid links stiffen the structure, hence could help in satisfying the drift design criteria, and give better estimates of seismic design demands.

4.3 Beam-column joint rigid offset, Model3

The seismic response of RC buildings can be influenced by the behavior of beam-column joints



(a) Beam-column joint modeling ASCE/ACI 41-13



(b) Beam-column joint rigid offset effect: Model3

Fig. 5 Beam-column joint model

involved in the failure mechanism. Conventional modeling approaches consider only beam and column flexibility despite the fact that joints can provide a significant contribution to the overall frame deformability. So, for structural modeling of RC-MRF buildings; it is required to take into account the finite dimensions of beam-column joints through rigid offsets of the interconnected beam and column elements. All in framing members join through rigid offsets with dimensions equal to the in framing member depth, Model3. Elwood *et al.* (2007) stated that it is rationally precise to model the joint using effective rigid end offsets. Some engineering analysis software programs explicitly model the panel zone shear deformation, but most of them implicitly account for the contribution of panel zone through the implementation of an end zone rigid offset, Fig. 5. Model3 is formulated to consider the beam-column joint rigid offset only.

4.4 T-beam and Beam-column joint rigid offset, Model4

Model4-1 is formulated for both T-beam and beam-column rigid offset effects; that accounts for the flexural stiffness of dropped beam and beam-column joint dimensions and stiffness of the panel zone, so it could relatively better estimate distribution of seismic design demands in comparison that of models 1-3.

While, FEMA-356 overestimates the stiffness of reinforced concrete moment frames by recommending that beam-column joints as a stiff or rigid zone (FEMA 2000). Tests reveal that beam-column joints can experience significant shear deformations even before the yielding of the longitudinal reinforcement within the joint (Walker *et al.* 2002, ASCE 2014). The modeling the beam-column joint model as a rigid zone could overestimate the frame lateral stiffness. The contribution of panel zone deformation to the story drift of moment frames is usually significant

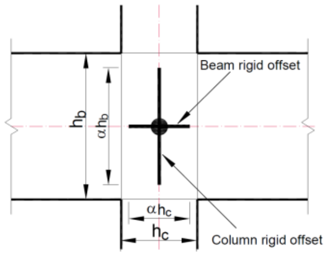


Fig. 6 Correction for beam-column joint rigid offset: Model4-2

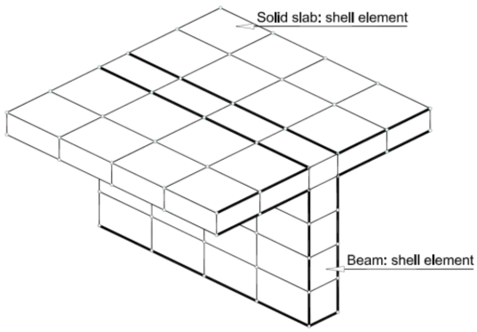


Fig. 7 Shell element based model, Model5

and should be considered using suitable mechanical models. The effect of shear zone deformations could be approximated by extending the beam or column flexibility into the joint in the analytical model. So model4-2 is introduced to consider the beam-column joint shear deformation, through an adjustment of rigid offset by a correction factor; model4-2 as shown in Fig. 6 The correction factor; α is determined from the calibration with the most refined model based on shell element approach.

4.5 Shell element based refined model: Model5

The structural configuration in terms of stiffness and strength distributions has a key role in the seismic response of buildings. A detailed finite element model based on shell elements is used for the simulation of MRF Building, Fig. 7. The beam behavior is governed predominantly by flexure, which is best modeled using shell elements. In this model, both beam and floor-slab are modeled by quadrilateral shell elements, Model5. The beam shell modeling has its simplicity of leaving the intermediate rigid links of T-beam and beam-column joint rigid offsets, but retaining the material variation and sectional properties of the slab and dropped beams. For simple solid slab/beam, the shell element model is sufficient and analysis is quite fast and accurate too compared to the solid element model. Shell element with six degrees of freedom at each joint gives more accurate results than the solid element. On the other hand, solid element overestimates the stiffness as it doesn't provide a rotational degree of freedom. From the analyses performed here, model 5, which assumes many refinements in modeling the various structural components, is anticipated to be the most accurate among proposed models and hence will be used as the reference model.

Table 2 Period for fundamental of vibration for studied building Models

Building	Vibration Modes	Period of vibration mode, T (sec)					
		Model1	Model2	Model3	Model4-1	Model4-2	Model5
4-Story building	1st Lateral Mode	0.76	0.71	0.61	0.57	0.59	0.59
	2nd Torsional Mode	0.66	0.62	0.54	0.50	0.51	0.51
	3rd Lateral Mode	0.24	0.22	0.19	0.17	0.18	0.19
8-Story building	1st Lateral Mode	1.21	1.08	0.99	0.88	0.92	0.92
	2nd Torsional Mode	1.06	0.95	0.87	0.77	0.81	0.81
	3rd Lateral Mode	0.39	0.35	0.32	0.29	0.30	0.30
12-Story building	1st Lateral Mode	1.52	1.30	1.26	1.08	1.15	1.15
	2nd Torsional Mode	1.34	1.14	1.10	0.95	1.01	1.01
	3rd Lateral Mode	0.49	0.42	0.41	0.35	0.38	0.38

5. Numerical and results discussion

Three-dimensional finite element models of the studied buildings are developed, where different levels of modeling refinement that range from centerline, rigid offset to shell element modeling are considered. A semi-rigid diaphragm is assigned at each floor level to simulate actual in-plane stiffness properties and behavior. The problem is investigated by analyzing alternative models of a typical 4-story, 8-story and 12-story RC-MRF buildings, which has been designed for moderate seismicity using ASCE 7 code seismic provisions (ASCE 2010). A range of practical and more detailed finite element idealizations are established. Following seismic analysis, key performance global response indices including: lateral displacement, inter-story drift, and story shear force response are estimated, to quantify the effects of modeling assumptions. A simple finite element modeling technique, which is accurate enough for common practice will be formulated and recommended for use in the construction industry.

5.1 Free vibration analysis

Although the fundamental period of a building is a key parameter for the seismic design of buildings structure, the building period cannot be analytically calculated before the building is designed. Therefore, the periods can be calculated either from the empirical period formulas recommended in seismic design codes or from finite element analysis with assumed mass and stiffness that are used during the preliminary design stage. In most building design projects, empirical building period formulas are used to initiate the design process. The period from the empirical period formula also serves as a basis to limit the period from a finite element model by applying the upper bound factor suggested in the 2003 NEHRP Recommended Provisions for Seismic Regulations for New Buildings (BSSC 2003, ASCE 2005, Abdel Raheem 2013). A modal analysis is undertaken to obtain the natural periods, mode shapes, and modal mass participation factors for the first three vibration modes, Table 2. It is realized that structural modal behavior of the building is significantly affected by the structure modeling level of complexity. It is found that higher modes of vibration were less sensitive to the different modeling assumptions than the lower modes. The natural period is shorter when the T-beam and beam-column joint effect are included.

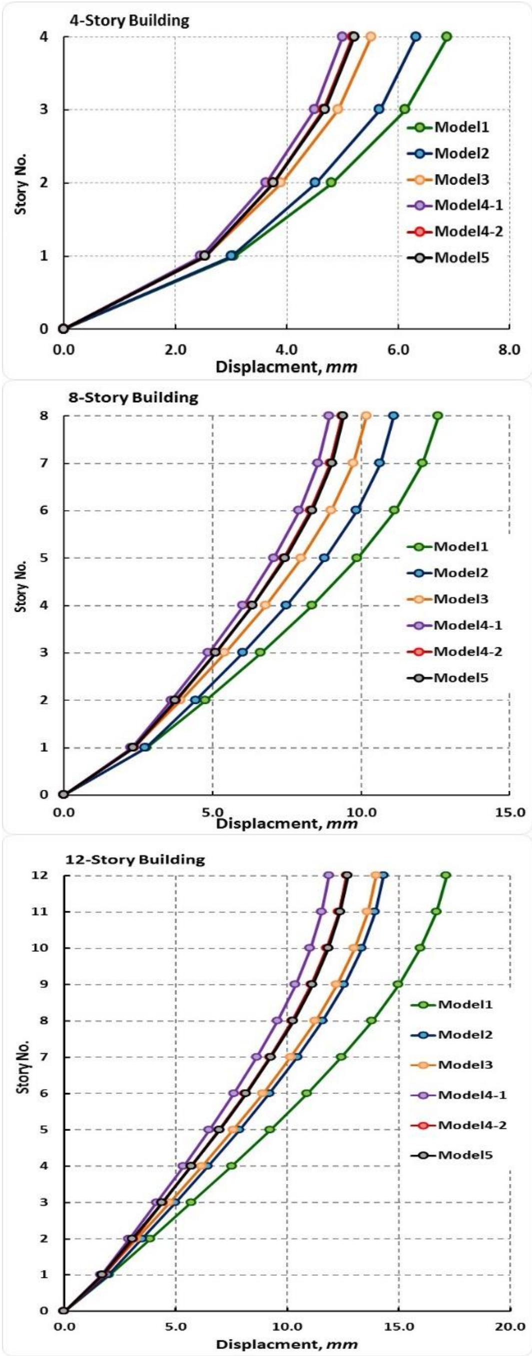


Fig. 8 Displacement response profile for different models

The centerline based model; Model1 underestimates the story stiffness, hence overestimates the natural vibration period with 28, 31 and 32% higher for 4-story, 8-story, and 12-story buildings, respectively, compared to that of the refined model; Model5. Through the upgrade of Model1 with

T-beam effect; Model2 could improve the estimation of natural vibration period, overestimate the natural vibration period with 20, 17 and 13% higher for 4-story, 8-story, and 12-story buildings compared to that of the refined model, respectively. Furthermore, the upgrade of model1 with beam-column rigid joint offset effect; Model3 could improve the estimation of natural vibration period, overestimate the natural vibration period with 3, 8 and 10% higher for 4-story, 8-story, and 12-story buildings compared to that of the refined model, respectively. The upgrade with the combined T-beam and beam-column rigid joint offset effects slightly underestimate the natural vibration period; Model4-1. So, an adjusting end zone factor is explicitly introduced through the calibration with the refined model, Model5. The values of end zone correction factor are calculated for 4-story, 8-story, and 12-story buildings; α = 0.85, 0.80 and 0.65, respectively.

5.2 Seismic design demands

The results of dynamic analyses are evaluated to identify the modeling parameters and modeling assumptions that have the most significant impact on the variability of seismic design demands. ASCE 41 (2006) provides acceptance criteria in terms of deformation and force demands on individual structural components. Other global response demand parameters, especially story drifts and floor accelerations are also important damage indicators to nonstructural components and building performance (PEER 2010, Willford *et al.* 2008, PEER/ATC 2010, ATC 2009). Structural models are used to determine force and deformation demands to design new structures and evaluate the performance of existing structures. The seismic structural design demand parameters include peak shear forces and deformations in structural components, inter-story drifts, and floor lateral displacement.

5.3 Displacement response demand profile

The estimation of drift during the design stage is essential for checking stability and damage limitation to non-structural elements as well as proper estimation of the separation distance between buildings. Also, excessive drift can affect the vertical stability of a building, especially flexible massive buildings, potentially leading to collapse due to P-Δ effects. Thus, the possibility of reducing the lateral drift at each story of a building entails minor costs for rehabilitating its functionality after a strong seismic shaking. Moreover, large lateral displacement response magnifies the internal force and moment demands, causing a decrease in the effective lateral stiffness. With the increase of internal forces, a smaller proportion of the structure’s capacity remains available to sustain lateral loads, leading to a reduction in the effective lateral strength. In order to evaluate the level of accuracy obtained with the different modeling strategies, the lateral displacement response profile trends for the studied models are depicted in Fig. 8. Displacement demands are shown to be affected significantly by the variation of the global stiffness of the building for different modeling strategies. In stiffness calculations, the use of centerline dimensions gives a much-distorted picture of the relative importance of beam versus column stiffness in drift control. If centerline dimensions are used for columns rather than clear span dimensions, the contributions of the column flexural deformations to inter-story drift can easily be overestimated, as the contribution of the columns to the story drift is proportional to the cube of the column length.

The seismic response of the analyzed building models is most influenced by rigid offsets vs. centerline element dimensions. The Model1 consistently overestimates lateral displacement

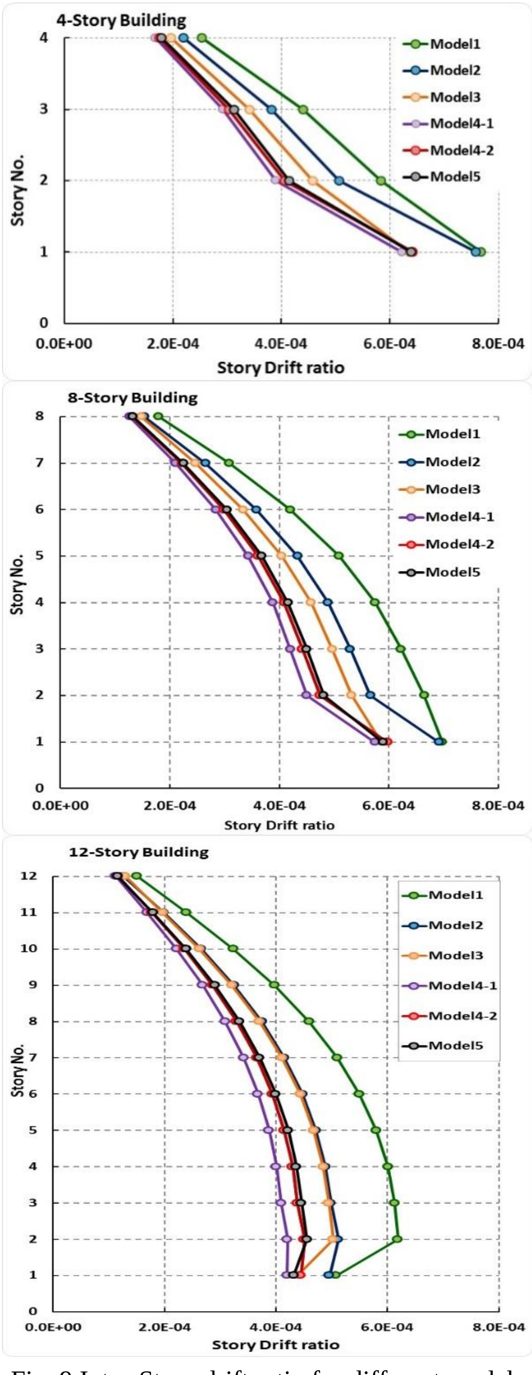


Fig. 9 Inter-Story drift ratio for different models

demands; while the model4-1 provides a sound estimate and could be improved through a calibration with shell element based refined model; model5 through the introduction of an adjusting end zone factor, Model4-2. The contributions to the drift vary with consideration of T-beam and beam-column joint effects. The beam-column joint effect is a significant contributor to the lateral displacement response demand for low rise-building, this effect decreases gradually for the higher buildings. In contrary, the T-beam effect has a slight contribution to the lateral displacement response for low-rise building; this effect increases for the higher building. The contribution of panel zone deformation to the story drifts of RC moment frames is usually significant and should be taken into consideration using appropriate mechanical models, especially for low-rise buildings. The lateral displacements are reduced when the T-beam or beam-column joint dimensions or both are included in the modeling refinement.

5.4 Inter-Story drift ratio demand

Inter-story drift represents the most important parameter to be analyzed as they are strictly connected to the damage suffered by both structural and non-structural elements. The inter-story drift has been employed as an index to evaluate the deformation capacity of a building and to further determine its performance. The recommended inter-story drifts to be considered for the serviceability check range from 0.2 to 0.5% the story height, depending on the type of partitions used. The Egyptian code, ECP-201 (ECP 2008), specifies a value of 0.7 for the ratio between the maximum displacement and the calculated elastic design displacement using the equivalent static load method of analysis. The code allows designing for drift up to 70% of the drift in a theoretically elastic structure. UBC 1997 section 1630.10 (ICBO 1997) gives the guidelines for calculating the maximum inelastic response drift;$\Delta_{\mathrm{M}}= 0.7R\Delta_{\mathrm{s}}$, where$\Delta_{\mathrm{s}}$ is the elastic deflections due to strength-level design seismic forces are called design-level response displacements.

The story and roof drift demands and their patterns for buildings of different heights are investigated. Figure 9 indicates that the modeling assumption significantly affects Inter-story drift ratio “IDR” demands. The Model1 consistently overestimates IDR demands; while the model4-1 provides a reasonable estimate of more 92% matching to that of the refined model, and could be calibrated with the refined model through adjusting end zone factor, Model4-2. Consideration of rigid offsets for either T-beam or Beam-column rigid joint effects as a model refinement to the centerline based model led to important changes in global structural strength and stiffness, as well as a change of relative story shear strengths. Thus, besides higher displacement demands of the centerline based model; Model1, there is a significant change in the distribution of inter-story drift demands along the building’s height. The beam-column joint effect is a significant contributor to the IDR demand for low rise-building, this effect contribution decreases gradually for high buildings. The IDR has linear variation along the building’s height, while it has a nonlinear variation for the high-rise building due to the significant contribution of higher modes of vibration. The IDR plots in Fig. 9 also indicate a decrease in IDR demands with the increase in the total number of stories. The location of the maximum IDR demands shows significant contributions from the higher mode in the 12-story building resulting in the migration of the peak inter-story drift from the lower to the upper stories. The importance of higher modes is larger at the upper portions of the building and increases as the height of the building increases

5.5 Story shear force response demand

The level of accuracy in the estimation of member rigidity plays a very important role in determining realistic values for the structural stiffness and hence the seismic forces imposed. The story shear force response demand for different models is shown Fig. 10. It could be noted that

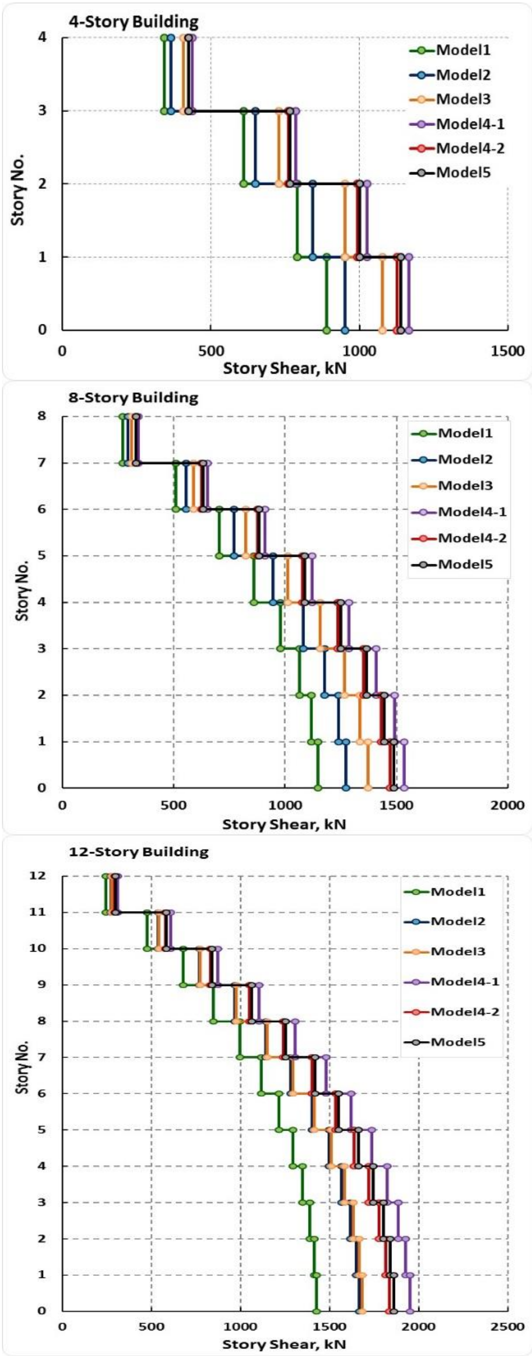


Fig. 10 Story Shear force response demand for different models

the centerline modeling approach; Model1 underestimates the story shear force response demands and could lead to an un-conservative design. This important finding prompted to conduct more investigation on the comparative stiffness of the centerline modeling approach to the more accurate refined modeling approach using T-beam and beam-column rigid joint effects. The shell based modeling approach, reference model Model5; shows larger story shear force response demands over the building's height, indicating that the centerline modeling approach underestimates the response demands. However, the use of combined T-beam and rigid beam-column joint effects; Model4-1 properly represents the stiffness of the structural MRF Building as well as the story drift and the overall deflection of the multi-story buildings with more than 92%of the refined model.

5.6 Seismic responses comparison

Evaluation of the seismic response of RC buildings is subjected to a considerable degree of approximation and simplification of the real behavior. Based on experimental investigations and their response in past earthquakes, as well as capacity design principles assure the validity of a considerable number of simplifications in the structural model. It is always cost effective for the design engineer to simplify the modeling approaches that employed to estimate the structural responses of the components within the buildings. However, these simplifications compromise one or more aspects of the real building behavior. The sophistication of the structural analysis and modeling affect both the detail of the analysis results and the design cost. The simple centerline-based model may provide a reasonable prediction of the seismic behavior that enables rapid assessment of the expected building performance and could be used in the early stages of the design process. Although the refined shell element-based model would provide more accurate information about the seismic response, the model preparation and the computational time are much longer. The importance of the structure, the designer experience, and the level of needed accuracy affect the selection of modeling approach.

Table 3 compares the maximum roof displacement, inter-story drift ratio, base shear force predicted by different modeling approaches. The compiled results demonstrate that the consideration of T-beam or beam-column joint dimension while modeling MRF-building could improve the overall lateral stiffness of buildings, which leads to effectively simulate the realistic behavior of RC-MRF buildings under lateral loads. Therefore, the detailed slab-beam-column models would be recommended to evaluate and predict the seismic performance of MRF buildings. The results of the response demands ratios calculated using the different modeling approaches are compared to the results of the reference shell-based refined model, Model5. The centerline modeling approach; Model1 predicted larger lateral displacement response (30-35%) and inter-story drift ratio response (18-35%) compared to the refined model, hence overestimates drift and ductility demands. In the meantime, Model1 underestimates the story shear response demands (22%) along the building height. The refinement of the model by including the T-beam and beam-column joint offset significantly affect the response demands as shown in Fig. 11. The effect of the flexural stiffness due to T-beam effect is relatively significant, especially in taller buildings. Model2 predicts slightly larger lateral displacement and inter-story drift ratio responses of 10% for 12 story building and 20% for 4-story building compared to the reference model, while it underestimated the story shear response demands (10-15%) for the 8 and 12 stories buildings.

The effect of the beam-column joint dimension effect is relatively significant, especially in shorter buildings. Model3 gives slight larger lateral displacement and inter-story drift ratio responses of 10% for 12 story building and 5% for 4-story building compared to the refined model, while it underestimates the story shear response demands (5-10%) for the 8 and 12 stories buildings. The FE model refinement with both T-beam and beam-column joint dimension effects

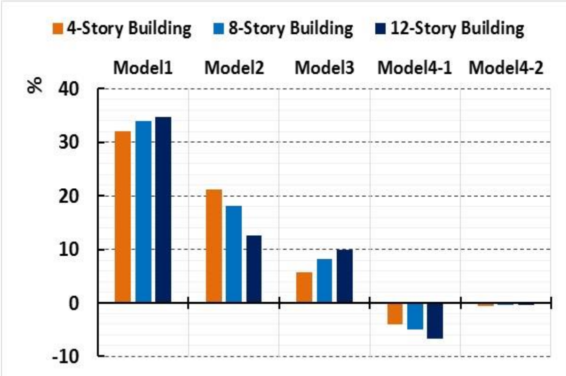
Table 3 Peak response demands for studied building Models

Building	Seismic response	Peak response demands					
		Model1	Model2	Model3	Model4-1	Model4-2	Model5
4-Story building	Lateral displacement, mm	6.9	6.3	5.5	5.0	5.2	5.2
	Inter-story Drift × 10-3	0.77	0.76	0.64	0.62	0.64	0.64
	Base Shear, kN	890	951	1078	1166	1128	1139
8-Story building	Lateral displacement, mm	12.6	11.1	10.2	8.9	9.4	9.4
	Inter-story Drift × 10-3	0.70	0.69	0.59	0.57	0.60	0.59
	Base Shear, kN	1147	1274	1372	1535	1471	1487
12-Story building	Lateral displacement, mm	17.2	14.3	14.0	11.9	12.7	12.7
	Inter-story Drift × 10-3	0.62	0.51	0.50	0.42	0.45	0.46
	Base Shear, kN	1426	1665	1684	1949	1836	1860

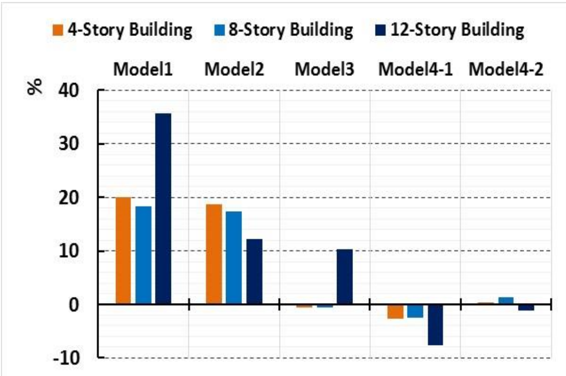
display close matching results on the seismic response demands to that of the reference model, its accuracy could be more than 92% for all the design demands. Accordingly, Model4-1 is the most conservative model and best alternative model for the design purpose. Results from Model4-1 show close matching in the inter-story drift, lateral deflection, and story shear force responses demands of three buildings considered. To compensate for the slight difference from the perfect matching, Model4-1 could be calibrated for the shear zone effect through response comparison with that of the reference model, where an adjusting end zone factor is explicitly introduced; Model4-2. The end zone factor is calculated for different the building models. The correction factor is derived using the least squares regression analysis for the critical peak responses between the modified Model4-1 compared to the refined model5. The correction factor has values of α = 0.85, 0.80 and 0.65 for 4-story, 8-story, and 12-story buildings, respectively.

6. Conclusions

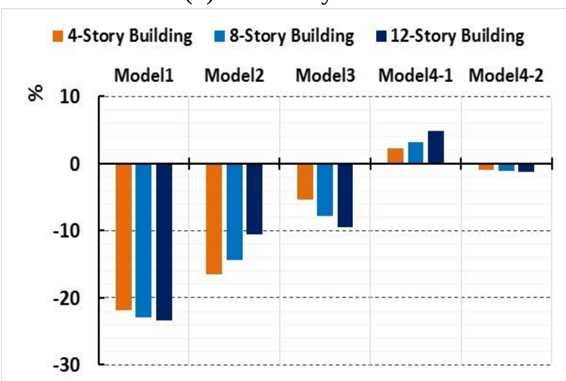
Structural modeling and analysis are essential stages of the design process and their accuracy are essential in estimating the structural stiffness and in achieving safe seismic designs. The structural stiffness plays a vital role in defining the natural periods of structures from which seismic demands ensue. Therefore, there is still a dire need to formulate robust modeling techniques for the practical seismic design of MRF-buildings. So, the objective of the study presented herein is to perform a thorough evaluation of different modeling techniques ranging from simple to more refined modeling technique to produce a robust FE model that could give results in the structural analysis similar to the real structural behavior of RC-MRF buildings. A quantitative measure of the importance of modeling assumptions and FE modeling refinement level on the predicted response is formulated through a comparison among simplified and refined finite element models. In order to quantify the uncertainty introduced strictly by the modeling assumptions, key performance global response indices are estimated and compared. A series of three-dimensional FE models were created to study various approaches to improve the accuracy of FE models quantitatively. Five different numerical models have been created and compared using ETABS finite element package. The finite element models are based on different modeling refinement levels: centerline element dimensions; T-beam effect; beam-column joint dimension;



(a) Top floor lateral displacement



(b) Inter-story drift



(c) Base shear force

Fig. 11 Variation of response demands for different models

and shell element based model. Seismic performance is defined through accuracy and efficiency comparison for seismic design demands among different models. The seismic response is investigated for centerline element dimensions based model vs. slab-beam floor flexural stiffness and beam-column rigid joint offsets vs. shell element based model.

Consideration of rigid offsets for T-beam effect or/and beam-column rigid joint offset as a substitute to the centerline dimensions of elements modeling approach led to important changes in global structural strength and stiffness, as well as a change of relative story shear demands. Thus, besides higher displacement response demands of the centerline model, there is a significant change in the distribution of inter-story drift demands. The model with both the T-beam effect and beam-column rigid joint effect is the best for engineering practice. Since it introduces the seismic response demands with a close matching of more than 92% to that refined model based on the shell elements modeling approach. While responses of the centerline based model, model1 have 40% deviation from that of the refined model for most of the seismic design demands. Furthermore, a correction factor could be introduced to compensate for this slight deviation from the perfect matching through the end zone factor adjustment, hence could improve the accuracy up to 98%; the correction factor is derived from a least squares regression analysis for the critical peak responses between the modified Model4 compared to the refined model5. Model4-2 is formulated through the calibration with the refined model. The end zone factor is calculated as 0.85, 0.80 and 0.65 for 4-story, 8-story, and 12-story buildings, respectively. The proposed model, Model4-2 could afford seismic response of the building structures with significant reduction in the computational times while the accuracy in the analysis results such as vibration periods, different response demands are close matching to those obtained from the refined model. Improving FE modeling will result in more cost effective structures and more reliable seismic design.

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