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EVALUATION OF RESPONSE MODIFICATION FACTOR FOR SHEAR WALL WITH OPENINGS IN MULTI-STOREY BUILDINGS

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الملخص العربى:

تعتبر حوائط القص النظام الانشائى الاكثر فاعلية فى مقاومة الاحمال الزلزالية. و نتيجة الى التقدم فى الهندسة المعمارية ظهرت الحاجة الى تنفيذ فتحات فى حوائط القص. و يتم اهمال تاثير تلك الفتحات اذا ما كانت ابعادها صغيرة مقارنة بالابعاد الكلية للحائط. ولكن فى حال الفتحات الكبيرة نسبيا او التى تتواجد فى المناطق الحرجة فقد تؤثر على سلوك هذه الحوائط فى مقاومة الاحمال الزلزالية. يتناول هذا البحث تعيين معامل تخفيض رد الفعل لحوائط القص ذات الفتحات فى المبانى متعددة الطوابق و التى تم تصميمها طبقا للكود المصرى مع مراعات اختلاف المنطقة الزلزالية باستخدام نماذج فراغيه عن طريق برنامج الايتابس (ETABS). تم عمل تحقيق ومقارنة للنتائج على نموذجين فراغين احداهما مكون من دورين و الاخر مكون من ثمانية ادوار. تم عمل تحليل لا خطى على اربعة نماذج قبل و بعد عمل فتحات و كمحاولة لتحسين قيمة معامل رد الفعل تم اضافة تسليح جانبى للفتحات مساوى للتسليح المقطوع نتيجة تنفيذها. و تم رسم منحنى قوى القص القاعدى و الازاحة وكذلك تم تعيين قيمة معامل رد الفعل بأستخدام ثلاثة طرق مختلفة. تبين ان قيمة معامل رد الفعل تنخفض بزيادة مساحة الفتحات و ان ارتفاع الفتحة يؤثر بطريقة اكبر من عرضها. كما ان اضافة تسليح جانبى للفتحات ساهم فى زيادة معامل تخفيض رد الفعل. اوضحت النتائج ان قيمة معامل رد الفعل فى الكود المصرى لا تتناسب مع حوائط القص ذات الفتحات فى المبانى متعددة الطوابق.

الكلمات الدالة : معامل تخفيض القوى الزلزاليه – التحليل اللاخطى – حوائط القص ذات الفتحات – حوائط القص المترابطة – الكمرة الرابطة بين حائطى قص مترابطين.

ABSTRACT:

Reinforced concrete structural shear walls (RC Walls) have been recognized as a main effective lateral force resisting systems. Often, as result of the continuing evolution in the architectural engineering and the need to implement openings in shear walls. The effects of these openings are usually neglected when their sizes are relatively small compared to the wall dimensions.[1] But, in the case when these openings are relatively large or located within a critical region, they may influence the seismic behavior of RC walls significantly. The present study Evaluate Seismic Response Modification Factor for Shear Wall with and without Openings in Multi-Storey Frame Buildings designed according to the Egyptian code of loads ECP-201 (2012) .[2] Two verified comparative examples are presented: 2-storey reinforced concrete frame and eight-storey building. Then, a numerical study had been conducted on four different models Shear Wall with and without Openings in Limited ductility Multi-Storey Frame Buildings which has been well designed according to Egyptian code for two ground motions ag/g = 0.15 and 0.25, spectrum type (2) and (1) respectively. Pushover analysis conducted into the four models without openings and after conduction different openings on every model. To improve the performance of the shear-wall, half of reinforcement bars terminated to conduct openings has been added at the both openings sides. Pushover curves plotted and response reduction factor evaluated through three different methods. Response reduction factor decreased by increasing the opening area. The opening height impact on the response reduction factor is greater than the opening width. Adding half of reinforcement bars case some increasement in the response reduction factor. ECP-201 (2012) [2] R-factor value is un-conservative value for shear walls structures with opening.

KEYWORDS: Reduction / Modification Factor (R); Pushover analysis; Nonlinear static analysis; Shear walls with openings; Spandrel; Coupled wall.

INTRODUCTION

The lateral and gravity load resisting system consist of reinforced concrete walls and slabs. Shear walls consider the main vertical structure elements which resist both the gravity and seismic loads. Its thickness depends on the number of stories. Shear walls reinforced continuously throughout its height.

As result of the continuing evolution in construction, some architectural constrains force engineers to install openings in shear-walls to accommodate windows, doors or utility ducts. These openings effect is usually ignored when their sizes are relatively small compared to the wall dimensions.[1] But, in the case when these openings are relatively large or located within a critical region, they may influence the seismic behavior of RC walls significantly. Shear walls are typically regular in plan and elevation as shown in Figure 1 and Figure 2 its efficiency described in terms of stiffness. Solid shear walls are most efficiency. Openings may be required due to architectural demands. Shear walls with openings are called coupled walls. These walls perform as a cantilevered wall connected by coupling beams. Coupling beams can be a spandrel or lintel for bending and shear effect. In Figure 3 it’s obviously that openings in the shear wall can influence its capacity. Failure may happen due to these openings. And also, these opening represent a weak area which the crack can pass by due to its low stiffness. So, these openings might have an influence in the modification factor value.

Many researches “Lin & Kuo (1998) [3], Khatami et al. (2012) [4], Rajesh & Prasad (2014) [5], Mohan & Arathi (2017) [6], Swetha & Akhil (2017) [7], Kalbouneh (2020) [8], Alasani, M. R. et al. (2021) [28], and Elwi, M. A., & Hussein, W. G. A. (2021) [29]” have conducted experimental and finite element studies to illustrate the effect of these openings, most of these studies which performed on the shear-walls with openings focus on the relation Between openings characteristics and the displacement and didn’t touch upon the effect of these opening parameters on the response reduction - modification factor (R). So, there was an over-whelming urge to know to what extent these opening can influence the response modification factor and therefore the design. The Egyptian code hadn't mentioned any factor can be used in order to take into consider the impact of these opening and satisfied with a fixed factor for building which have shear walls systems.

In this research the influence of these openings in the response modification factor of the system is clarified, and calculate (R) factor using the nonlinear behavior. Nonlinear pushover analysis applied in a finite element model using a finite element program (ETABS) [9] to determine the (R) factor value. A finite element model for two-storey one bay building has been generated by using ETABS [9] and SAP2000 [10] software and the results discussed and compared with the experimental results. Eight-storey building has been studied and modelled by using ETABS [9] and SAP2000 [10] software. In this models Walls defined as a fiber shell element and layered shell element as an alternative method. Pushover analysis conducted. The results concluded and compared with the original paper result [11]. Numerical study is carried out for four different limited ductility buildings which have been well designed for two ground motions ag/g = 0.15 and 0.25, spectrum type (2) and (1) respectively according to Egyptian code. Then, pushover analysis conducted into the four models without openings and after conduction different size of openings on every model to assess the effect of the opening. Lintel beam above opening has been modelled by two different methods. The difference between modeling lintel beam as a wall segment and spandrel has been clarified. In addition, half of reinforcement bars terminated to conduct openings has been added on either side of the opening to explore its influence on the response reduction factor. A numerical study conducted to evaluate the impact of opening area, width and height on the (R) factor for RC shear wall systems. The results are discussed and recommendations are given.

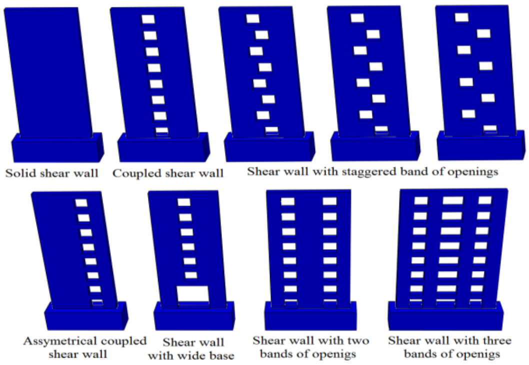


Figure 1 : Typical types of shear walls.

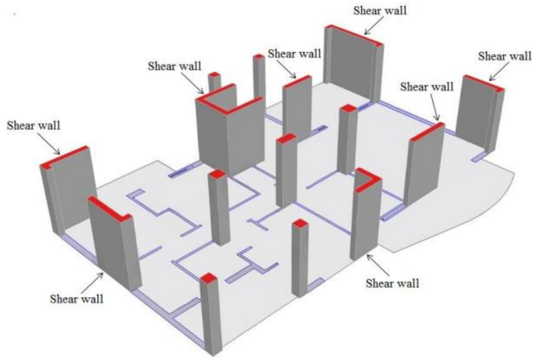


Figure 2 : 3-D view of a building with different types of shear walls.

Figure 3 : Performance of reinforced concrete buildings during Chile earthquake.

CONCEPT FOR DETERMINING RESPONSE MODIFICATION FACTOR (R)

Response modification factors (R) are main-seismic design tool, which illustrates the expected inelasticity level in structural systems. Seismic codes depend on reserve strength and ductility to justify this reduction, which improves the ability of the structure to dissipate and absorb energy. Hence, the role of the (R) factor and the parameters influencing its evaluation and control are essential elements of seismic design according to codes. The values assigned to the response modification factor (R) of the US-codes, FEMA, 1997 [12]; IBC, 2018[13], are aiming to account for reserve strength and ductility too (ATC, 1996) [14]. Some literature also mentions redundancy in the structure as a separate parameter. ATC-40 [14], calculates the response modification factor as an equation of three parameters that affect the seismic response of the structure (Ductility, overstrength and redundancy).

The main objective of the earthquake design is to get a system resist earthquake without completely collapse, but with some damage. In the same vein, the structure is designed for much less base shear forces than would be required if the building is remained elastic during severe shaking at a site. Such large reductions are mainly due to two factors: the ductility reduction factor ($R\mu$), which reduces the elastic demand force to the level of the maximum yield strength of the structure, and the over-strength factor, (Ω), which accounts for the over-strength introduced in code-designed structures. Thus, the response reduction factor (R) is:

$$R = R\mu \times \Omega \quad (1)$$

The relation between the base-shear of a structure and its roof displacement which can be calculated by a nonlinear static analysis has been illustrated in Figure 4 and Figure 5.

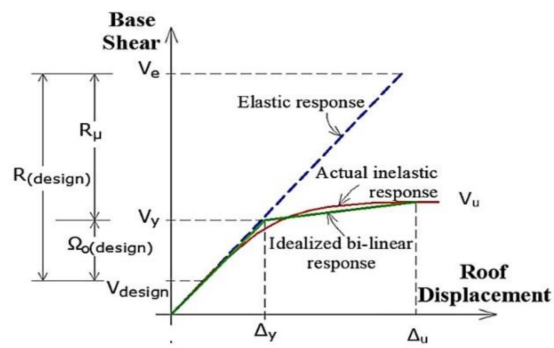


Figure 4 : Force displacement response of elastic and inelastic systems [27].

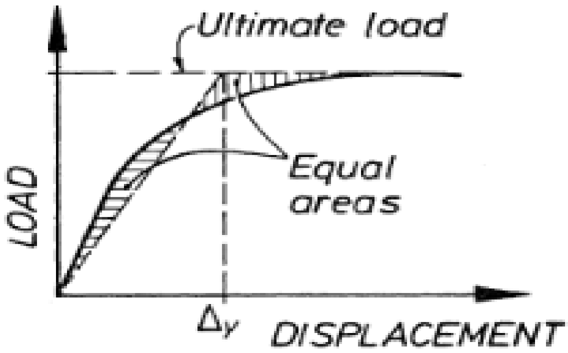


Figure 5 : Force displacement response based on equivalent elasto-plastic energy absorpation [26].

OVER-STRENGTH FACTOR Ω

The over-strength factor (Ω) can be defined as the ratio of the actual to design level strength (Elnashai and Mwafy, 2002 [21]). It can be expressed as:

$$\Omega = Vy / Vd \quad (2)$$

where Vy is the yield strength and Vd is the design strength

The main sources of the structural over-strength results from sequential yielding of critical regions, material over-strength, strain hardening, capacity reduction factors, member size, nonstructural elements and special ductile detailing (Elnashai and Mwafy, (2002) [15]; Rodrigues et al., (2012) [16]).

DUCTILITY REDUCTION FACTOR, Rμ

The extent of inelastic deformation experienced by the structural system subjected to a given ground motion or a lateral loading is given by the displacement ductility ratio "μ" (FEMA-451, (1999) [12]). The inelastic behaviors of a structure can be idealized as:

$$\mu = \Delta u / \Delta y \quad (3)$$

where μ is the displacement ductility ratio, Δu is the ultimate displacement and Δy is the yield displacement.

Yield displacement and yield base shear are judged through an idealization of the capacity curve.

Ductility reduction factor Rμ is a function of structural characteristics such as ductility, damping and fundamental period of vibration (T), and the characteristics of earthquake ground motion (Maheri and Akbari, (2003) [17]). Researchers proposed different formulations in order to determine the ductility reduction factor Rμ, (Newmark and Hall, (1973) [18]; Elnashai and Mwafy (2002) [15]).

In this study, the formulation proposed by Newmark and Hall (1982) [18] is used

$$R\mu = 1.0 \qquad T \leq 0.03 \qquad (4)$$

$$R\mu = 1 + \frac{(T-0.03).(\sqrt{(2\mu-1)}-1)}{0.09} \qquad 0.03 < T < 0.12 \qquad (5)$$

$$R\mu = \sqrt{(2\mu - 1)} \qquad 0.12 \leq T \leq 0.5 \qquad (6)$$

$$R\mu = \sqrt{(2\mu - 1)} + 2(T - 0.5) \times \left(\mu - \sqrt{(2\mu - 1)}\right) \qquad 0.5 < T < 1.0 \qquad (7)$$

$$R\mu = \mu \qquad T \geq 1.0 \qquad (8)$$

where $R\mu$ is the ductility reduction factor and μ is the displacement ductility.

The target displacement δ_t is calculated from the idealized pushover curve, idealization of pushover curve can be made using ASCE 41-13 [19] coefficient method through the following relation:

$$\Delta_u = \delta_t = C_0 C_1 C_2 C_3 S_a \frac{{T_e}^2}{4\pi^2} g \qquad (9)$$

C_0: modification factor to relate spectral displacement of an equivalent SDOF system to the roof displacement of the building MDOF system.

C_1: modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response.

C_2: modification factor to represent the effect of pinched hysteretic shape, stiffness degradation and strength deterioration on maximum displacement response

C_3: modification factor to represent increased displacements due to dynamic P-Δ effects.

Sa: response spectrum acceleration, at the effective fundamental period and damping ratio of the building in the direction under consideration. g: acceleration of gravity.

Te: the effective fundamental period of the building in the direction under consideration in seconds.

PROVISIONS OF ‘R’ FACTOR IN INTERNATIONAL CODES AND GUIDELINES

The response reduction factor in different codes and guidelines varies depending on the type of structural system and ductility class of the structures. For Shear wall-frame, values of “R” as specified in IBC 2018 [13], Eurocode-8 [20], ECP 2012[2] are presented in Table 1.

IBC 2018 [13], and ASCE7-16 [21] gives a value of “R” equal to 4.5 for Shear wall-frame interactive system with ordinary reinforced concrete moment frame and ordinary reinforced concrete shear walls. Eurocode-8 gives the behavior factor for regular Frame system, dual system, coupled wall system for two ductility classes. Eurocode-8 (2004) [20] specified the over-strength factor (the ratio of Vu/Vy) as 1.30 in multi-story multi-bay frames. ECP (2012) [2] gives a value of “R” equal to 5.0 for Dual system from Moment Resisting Frames and Shear Walls with limited ductility.

Table 1 : R values allocated in different codes for concrete shear wall-frame structures.

Structural System		R-Value		
		IBC 2018 ASCE7-16	**Eurocode-8**	**ECP 2012**
Frame system, dual system, coupled wall system	Medium ductility class (DCM)		3.0 Vu /Vy	
	High ductility class (DCH)		4.5 Vu /Vy	
Dual system from Moment Resisting Frames and Shear Walls	Limited ductility frame			5.0
	Sufficient ductility frame			6.0
Shear wall-frame interactive system with ordinary reinforced concrete moment frame and ordinary reinforced concrete shear walls		4.5		

For multi-bay multi-story Vu /Vy = 1.3, and for single-bay multi-story Vu /Vy = 1.2

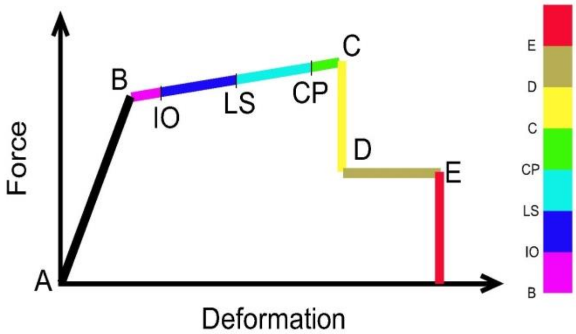
NONLINEAR STATIC ANALYSIS (PUSHOVER ANALYSIS) AND PERFORMANCE LEVEL

Nonlinear static analysis (Pushover analysis) was used in the current study to evaluate the global limit states of the RC MRF in terms of drift and force level. In this analysis, the increasing forcing function, either in terms of horizontal forces (representation of inertial forces along the structure height) or displacements imposed on a mathematical model of a building. The analysis is terminated when the target displacement or ultimate limit state is reached.[22] The target displacement or drift represents a maximum building displacement or drift during earthquake shaking. This kind of analysis can evaluate the maximum strength and deformation capacity of the building. They also help in identifying potential weak and soft stories in the building.

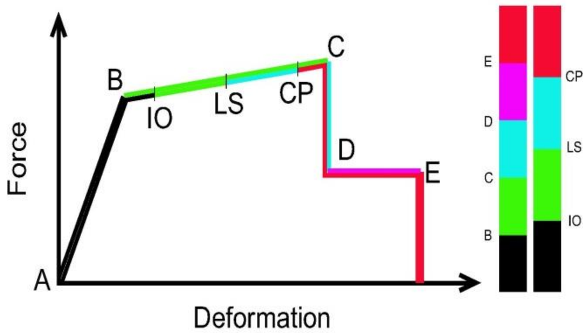
Generally nonlinear static analysis is integrated into following steps, as follows:
Develop 3D structural model of the building.
Impose gravity loads and apply static lateral loads or displacements in the pattern that approximately captures the relative inertial forces developed at locations of substantial mass or where the mass of every floor is lumped in the model.
Push the structure using the load pattern of step 2 to a target displacement level (i.e., the displacement of the target node reaches the target displacement).
Estimates the forces and deformations in every element at the level of displacement corresponding to the target displacement.
Plot the base shear Versus top storey displacement or storey shear vs storey displacement.

Performance based seismic design is an alternative approach for analysis and design of tall buildings. Different codes and standards allow the use of alternative procedures which based on a well-established principle in design and analysis. To attain a more refined structural behavior incorporating in elastic analysis it is necessary to present the structural inelastic seismic response and adequately account for damage loss in both structural and nonstructural elements during earthquakes.

Performance-based engineering yields structures with predictable performance within defined levels of risk and reliability (FEMA 356 [12] and ATC 40 [14]). The critical outcome is the prevention of total structural collapse. This means that the upper level withstands total collapse (CP); the sub level, for the crucial structures, may be slightly damaged but remains fit for immediate occupancy (IO). Between the sub and upper levels there is Life Safety (LS) level situation. The nonlinear procedures of FEMA require definition of the nonlinear load deformation relation. Such a curve is given in Figure 6.



SAP2000.



ETABS.

Figure 6 : Typical load – deformation relation and target performance levels.

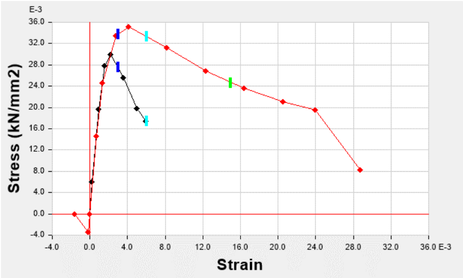
The five points (A, B, C, D and E) are used to define the hinge rotation behavior of RC members according to FEMA. Three more points Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP), are used to define the acceptance criteria for the hinge. The illustrative damage for concrete frames at different levels structural performance levels mentioned in ASCE, 2017b [23].

COMPARISON EXAMPLES

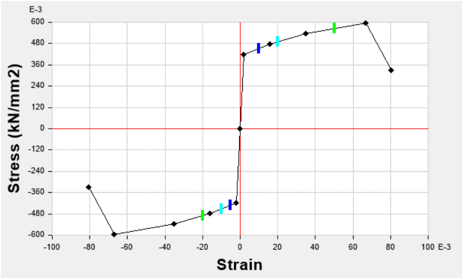
An investigation has been carried out for two models first, a one-span, two-storey one bay reinforced concrete frame which experimentally tested by Vecchio and Emara (1992) [24] and verified numerically by Serhan Güner (2008) [25] has been modelled using ETABS and SAP2000 software second, 8-storey Dual System building which studied by Ibrahim Yasser et al. [11] modelled using both SAP 2000 [10] and ETABS [9] software.

COMPARISON EXAMPLE MODEL (1): TWO-STOREY R FRAME

A one-span, two-storey reinforced concrete frame which constructed with a center-to-center span 3500 mm with storey height of 2000 mm and total height 4600 mm. All beams and columns were 300 mm width and 400 mm depth. The frame built integral with a large, heavily reinforced concrete base to make a fixed footing as shown in Figure 8 Material properties were determined from cylinder tests and steel coupon tests. Stress-strain curves for concrete columns illustrated in Figure 7 (a). where steel longitudinal bars are illustrated in Figure 7 (b). The model laboratory tested by applying a constant axial load of 700 KN to each column while monotonically applying a lateral load to the second storey beam until the ultimate-capacity of the frame was reached. The column loads were provided by two pairs of 450 KN capacity hydraulic jacks, applied through two transverse beams in the force-controlled mode. The lateral load was provided by a 1000 KN capacity actuator, mounted laterally against a reacting strong wall, in a displacement mode. And the Base shear Versus Displacement Curves were plotted.



Stress-strain curve for concrete Column.



Stress-strain curve for Longitudinal Bars.

Figure 7 : Stress-strain curves introduced in ETABS.

Pushover analysis are applied on the buildings and displacement control analysis used with targeted monitored displacement at top storey about 4% from total building height. The results compared with the experimental results and RUAUMOKO software results [24]. The results obviously showed that the result from ETABS [9] and SAP 2000 [10] software almost the same the experimental and other software results. Ultimate and yield steps base shear and displacement very close and almost identical for the both.

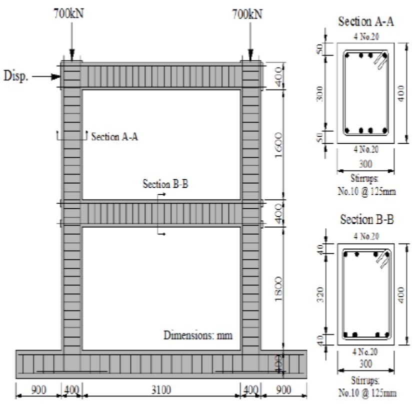


Figure 8 : Details of Frame Vecchio and Emara (1992) [24].

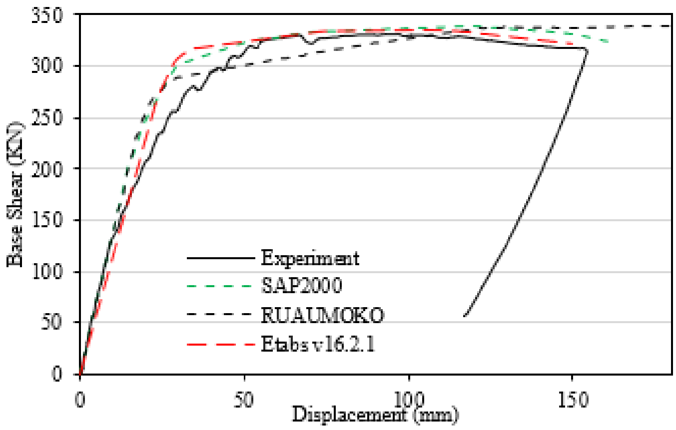
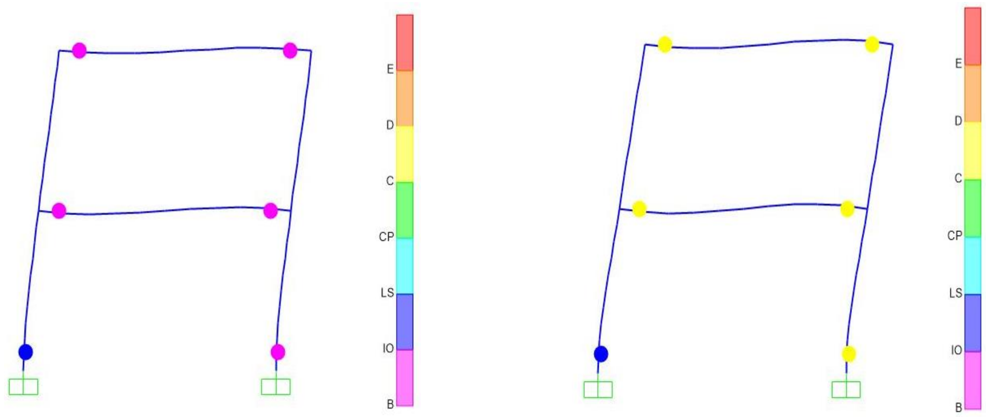
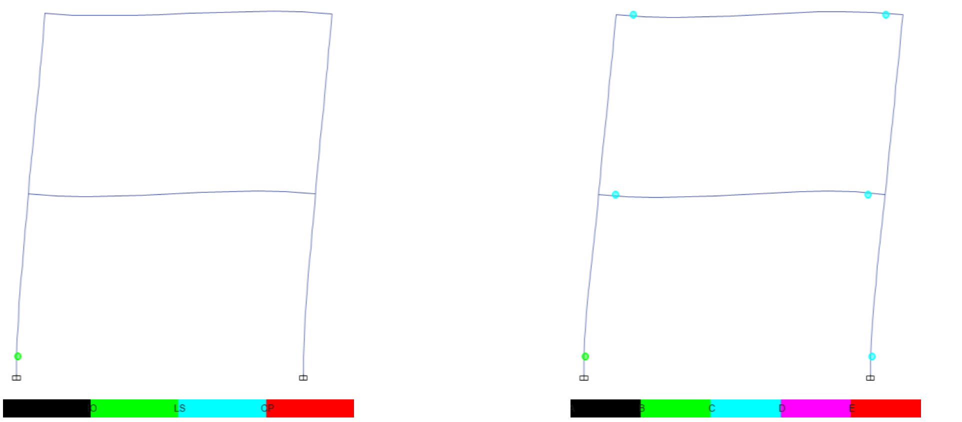


Figure 9 : Base shear Vs. Displacement Curves.



Yield Step (Step 27) SAP 2000
Vy = 323.97 kN Δy= 48.99 mm

Ultimate Step (Step 71) SAP 2000
Vu = 340.66 kN Δu= 131.46 mm



Yield Step (Step 21) ETABS
Vy = 320.75 kN Δy= 49.28 mm

Ultimate Step (Step 55) ETABS
Vu = 337.06 kN Δu= 131.05 mm

Figure 10 : Yield and Ultimate steps using SAP2000 & ETABS.

From Figure 9 and
Figure **10** it's obviously that experimental and numerical RC structures illustrate that finite element software ETABS and Sap2000 can be used efficiently for predicting the nonlinear seismic performance of RC concrete structures. Ultimate and yield base shear as well as displacement almost identical.

COMPARISON EXAMPLE MODEL (2): EIGHT STORER DUAL SYSTEM BUILDING

Eight storey Dual system building have 5 bays for both X-direction and Y-direction. Storey height is 3.2 m and the total width of the building in both X-direction and Y-direction is 26.3 m. The building plan and 3D-view is shown in

Figure 12. Material stress-strain curves for concrete and steel are illustrated in

Figure **11**. Two different modeling methods (Fiber and Layered) used to model the shear walls. Also, fiber shear walls divided into 2x2, 4x4 and 8x8 parts. Performance base design are applied on the buildings using Nonlinear static pushover analysis as per ATC-40 and FEMA 356. Plastic hinges are assigned at the locations where yielding is expected under seismic forces at both ends of the beams and columns with start and end relative distances of 0.05 and 0.95 respectively. Also, hinges are assigned at walls in fiber model. Plastic hinge type assigned to columns is interacting (P-M2-M3) and assigned to beams is M3 type which is single moment rotation type as per ASCE 41-13. And for walls in fiber model is (P-M3). Nonlinear static gravity load case; containing own weight multiplied with scale factor equal (1), super dead load multiplied with scale factor equal (1) and live load with scale factor equal (0.25), with zero initial condition, the mass source is (Dead Load + Super Dead Load + 0.25 Live Load). Nonlinear static pushover load cases in global X-Direction with static lateral load pattern is applied to the structure starts from the end of the nonlinear gravity load case with target displacement equal 4% from the total building height.

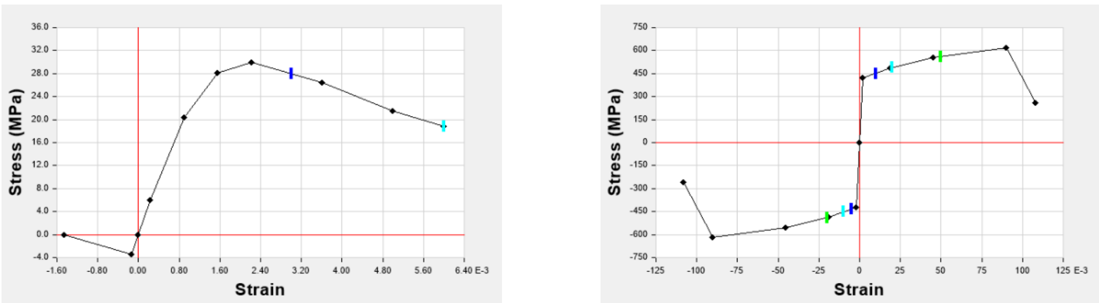
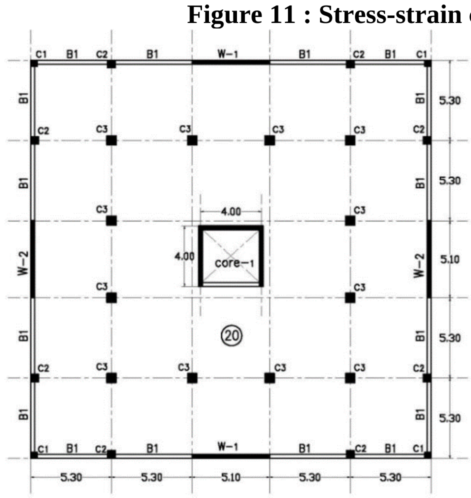
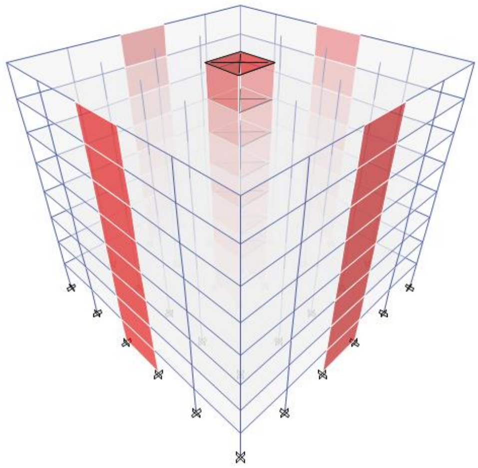


Figure 11 : Stress-strain curve for concrete and rebar material.



Building plan view.



Building 3D-view.

Figure 12 : Building Configuration.

ETABS 2016 v16.2.1 [9] Layered shell, Fiber models, and Layered shell SAP2000 v19.2.1 [10] model Compared with the original paper Layered shell SAP2000 v19.2.1 model. Figure 13 present the base shear-displacement curves.

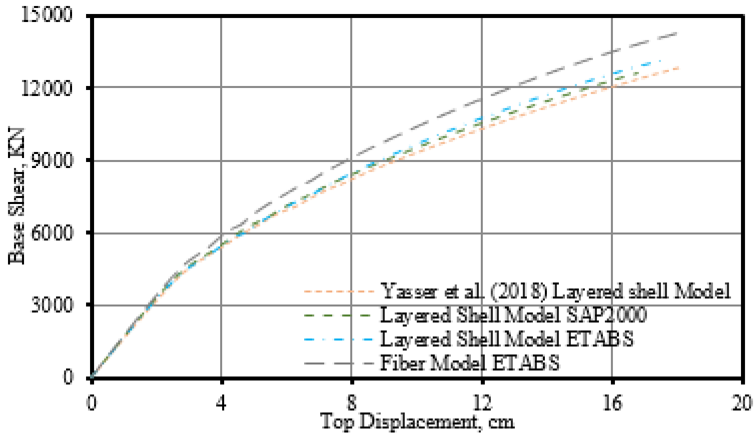


Figure 13 : Pushover Curves for verification and original models.

NUMERICAL STUDY FOR SEISMIC RESPONSE MODIFICATION FACTOR FOR SHEAR WALL WITHOUT OPENINGS IN MULTI - STOREY FRAME BUILDING

Eight-storey reinforced concrete building have 5 bays for both X and Y directions with a storey height equal 3.2 m which is 25.6 m tall with 26.3 x 26.3 m2. The Plan, 3D- View, Design zones and Spectrum is shown in

Figure 15. The seismic load resisting system consists of dual system shear walls and frames, whereas the gravity load carrying system comprises 200 mm thickness concrete flat slab resting on reinforced concrete columns and shear walls. Shear walls and core thicknesses in both X and Y directions equal 200 mm in model type (A), 350 and 300 mm respectively in the lower four stories, 300 and 250 mm respectively in the upper four stories in model type (B), 200 mm walls in model type (C), and 350- and 300-mm walls for the lower and upper four stories respectively in model type (D). Material properties and stress-strain curves for concrete and steel are illustrated in Table 2 and Figure 14.

Table 2 : Material properties for models.

F’c	30 MPa	Concrete strength
Fy	420 MPa	Rebar yield strength
Ec	24100 MPa	Modulus of elasticity of concrete
Es	200000 MPa	Modulus of elasticity of Rebar
G	10041.58 MPa	Shear modulus
Y	0.2	Poisson’s ratio

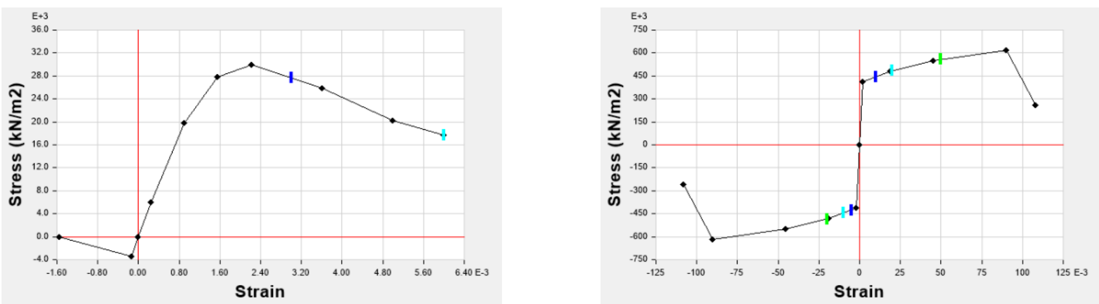
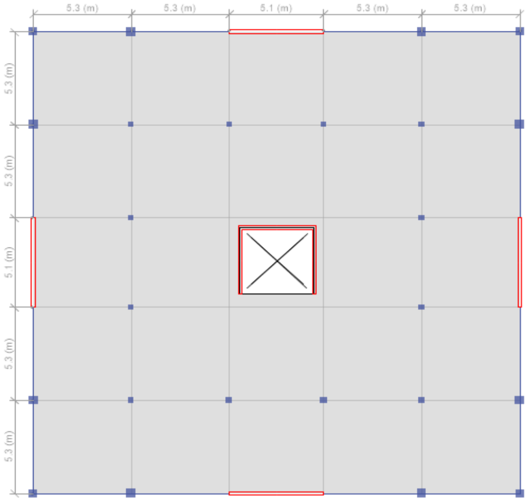
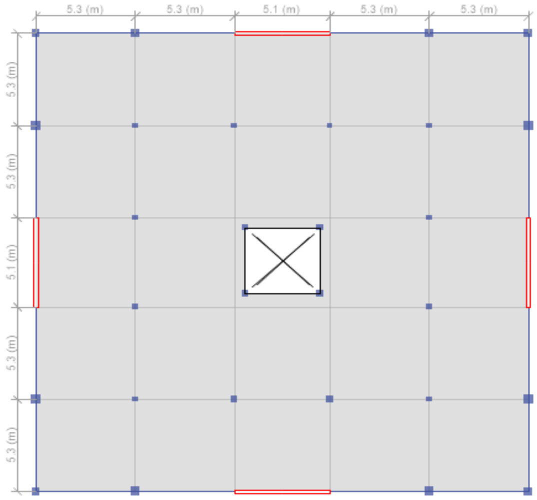


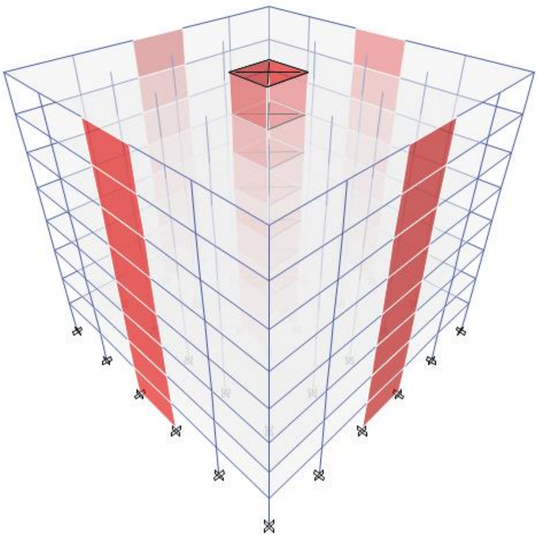
Figure 14 : Stress-strain curve for concrete and rebar material.



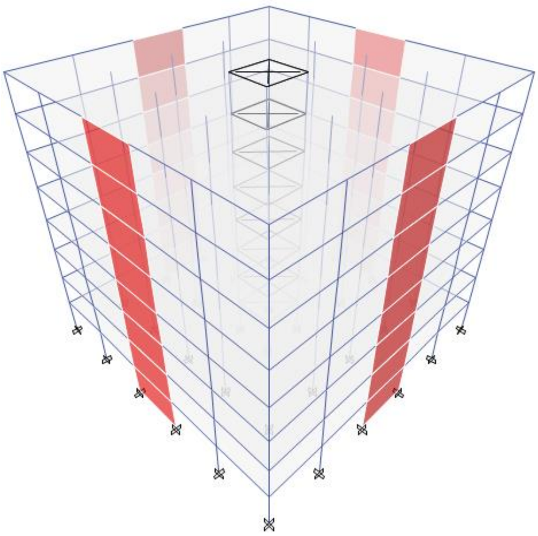
Model Plan (A), (B).



Model Plan (C), (D).



Model 3D-View (A),(B).



Model 3D-View (C),(D).

Model	Design zone	Spectrum type
A	0.15 g	2
B	0.25g	1

Model	Design zone	Spectrum type
C	0.15 g	2
D	0.25g	1

Figure 15 : Layout of studied buildings Models.

MODELS DESCRIPTION

The four buildings have been well designed according to the Egyptian code. Columns P-M-M interaction ratios and walls D/C ratios should be less than coefficient of (1) to ensure that columns are safe and compatible with the Egyptian code requirements. Performance base design are applied on the four models using Nonlinear static pushover-analysis as per ASCE 41-13 before opening conducted. Seismic load defined as per the Egyptian code in two cases. First, by using ground acceleration equal 0.15 ag/g and spectrum type (2) for models (A) and (C). Second, by using ground acceleration equal 0.25 ag/g and spectrum type (1) for models (B) and (D). The following table present columns, beams, and wall section for model (A), (B), (C), and (D).

The following loading assumptions have been considered:

Total Dead Load (D) is equal to DL+SDL+CL

Dead Load (DL) is equal to the self-weight of the members and slabs.

Super-imposed Dead Load (SDL) equals to 1.5 kN/m². SDL not included partitions weight.

Live Load (L) equals to 2.0 kN/m².

The studied buildings are subjected to different types of load combinations according to ECP 2012. These combinations are applied by the following terms:

$$U = 1.40\ D + 1.60\ L \tag{10}$$

$$U = 1.12\ D + \alpha\ L \pm S \tag{11}$$

Where D is the dead load, L is the live load; S is the seismic load and superposition factor of the structure’s the residential buildings.

Nonlinear static gravity load case; containing own weight multiplied with scale factor equal (1), super dead load multiplied with scale factor equal (1) and live load with scale factor equal (0.25), with zero initial condition, the mass source is (Dead Load + Super Dead Load + 0.25 Live Load). Nonlinear static pushover load cases in global X-Direction with static lateral load pattern is applied to the structure starts from the end of the nonlinear gravity load. While the target displacement equal 4% from the total building height.

Plastic hinges are assigned at the locations where yielding is expected under seismic forces at both ends of the beams and columns with start and end relative distances of 0.05 and 0.95 respectively.

Table 3 : Designed sections for type (A).

Columns Sections		
Column ID	**Cross-sec (mm x mm)**	**Main bars**
C1	450 x 450	16T14
C2	500 x 500	16T16
C3 (1-2)	600 x 600	20T16
C3 (3-4)	500 x 500	16T16
C3 (5-6)	400 x 400	12T14
C3 (7-8)	300 x 300	8T14
Beams Sections		
Beam ID	**Cross-sec (mm x mm)**	**Reinforcement at supports Upper & lower**
B1	250 x 650	11T16
Walls Sections		
Wall ID	**Thickness(mm)**	**Shear wall sections and Reinforcement VL / HL RFT**
Core-1	200	T12@200 / T12@200
W-1	200	T12@165 / T12@200

Table 4 : Designed sections for type (B).

Columns Sections		
Column ID	**Cross-sec (mm x mm)**	**Main bars**
C1 (1-2)	800 x 800	28T20
C1 (3-4)	700 x 700	24T18
C1 (5-6)	650 x 650	20T18
C1 (7-8)	600 x 600	20T16
C2	750 x 750	24T20
C3 (1-2)	600 x 600	20T16
C3 (3-4)	500 x 500	16T16
C3 (5-6)	400 x 400	12T14
C3 (7-8)	300 x 300	8T14
Beams Sections		
Beam ID	**Cross-sec (mm x mm)**	**Reinforcement at supports Upper & lower**
B1	250 x 1150	19T16
Walls Sections		
Wall ID	**Thickness(mm)**	**Shear wall sections and Reinforcement VL / HL RFT**
Core-1 (1-4)	300	T20@200 / T12@200
Core-1 (5-8)	250	T16@200 / T12@200
W-1 (1-4)	350	T20@150 / T12@200
W-1 (5-8)	300	T18@200 / T12@200

Table 5 : Designed sections for type (C).

Columns Sections		
Column ID	**Cross-sec (mm x mm)**	**Main bars**
C1	450 x 450	16T14
C2	500 x 500	16T16
C3 (1-2)	600 x 600	20T16
C3 (3-4)	500 x 500	16T16
C3 (5-6)	400 x 400	12T14
C3 (7-8)	300 x 300	8T14
C4 (1-2)	550 x 550	20T16
C4 (3-4)	500 x 500	16T16
C4 (5-6)	450 x 450	16T14
C4 (7-8)	350 x 350	8T16
Beams Sections		
Beam ID	**Cross-sec (mm x mm)**	**Reinforcement at supports Upper & lower**
B1	250 x 650	11T16
Walls Sections		
Wall ID	**Thickness(mm)**	**Shear wall sections and Reinforcement VL / HL RFT**
W-1	200	T16@200 / T12@200

Table 6 : Designed sections for type (D).

Columns Sections		
Column ID	**Cross-sec (mm x mm)**	**Main bars**
C1 (1-2)	800 x 800	28T22
C1 (3-4)	700 x 700	24T20
C1 (5-6)	600 x 600	20T18
C1 (7-8)	500 x 500	20T16
C2 (1-6)	750 x 750	24T20
C2 (7-8)	700 x 700	24T20
C3 (1-2)	650 x 650	16T18
C3 (3-4)	550 x 550	16T16
C3 (5-6)	450 x 450	12T16
C3 (7-8)	350 x 350	8T14
C4 (1-2)	800 x 800	28T22
C4 (3-4)	700 x 700	24T20
C4 (5-6)	600 x 600	20T18
C4 (7-8)	500 x 500	12T16
Beams Sections		
Beam ID	**Cross-sec (mm x mm)**	**Reinforcement at supports Upper & lower**
B1	250 x 1150	19T16
Walls Sections		
Wall ID	**Thickness(mm)**	**Shear wall sections and Reinforcement VL / HL RFT**
W-1 (1-4)	350	T16@125 / T12@200
W-1 (5-8)	300	T16@200 / T12@200

RESULTS AND DISCUSSION

To evaluate the yield and ultimate forces and displacement a pushover nonlinear analysis conducted after finalizing the design and assigning the hinges as mentioned before. The ultimate and yield step were determined by using Acceptance Criteria Limits for Hinges Deformation, Park Definition [26] for Ultimate and Yield Deformation, and ASCE 41-13 Idealized Bilinear Curve after plotting the pushover curve.

Table 7 : Acceptance Criteria Limits for Hinges Deformation Results.

Model	**Time Period**	**Δu**	**Δy**	**μ**	**Rμ**	**Vy**	**Vd**	**Vu**	**Rs**	**R**
	(sec)	**m**	**m**			**kN**	**kN**	**kN**		
1-A	0.932	0.30	0.12	2.52	2.45	12125.53	3424.89	18634.32	3.54	8.66
1-B	0.632	0.243	0.104	2.33	2.03	25957.97	9467.39	33737.89	2.74	5.56
1-C	1.098	0.265	0.124	2.14	2.21	10451.01	3469.43	15009.81	3.01	6.65
1-D	0.697	0.27	0.11	2.57	2.25	23114.73	9627.95	32401.81	2.40	5.39

Table 8 : Park Definition [26] for Ultimate and Yield Deformation Results.

Model	Time Period	Δu	Δy	μ	Rμ	Vy	Vd	Vu	Rs	R
	(sec)	m	m			kN	kN	kN		
1-A	0.932	0.47	0.27	1.71	1.69	17931.00	3424.89	21594.10	5.24	8.84
1-B	0.632	0.32	0.16	1.95	1.77	30290.81	9467.39	36159.32	3.20	5.66
1-C	1.098	0.45	0.27	1.67	1.70	15122.89	3469.43	18045.78	4.36	7.42
1-D	0.697	0.28	0.15	1.83	1.71	27658.78	9627.95	32401.81	2.87	4.92

Table 9 : ASCE 41-13 Idealized Bilinear Curve Results.

Model	Time Period	Δu	Δy	μ	Rμ	Vy	Vd	Vu	Rs	R
	(sec)	m	m			kN	kN	kN		
1-A	0.932	0.18	0.05	3.91	3.73	7921.86	3424.89	14949.96	2.31	8.63
1-B	0.632	0.19	0.06	3.03	2.45	22571.17	9467.39	31404.12	2.38	5.85
1-C	1.098	0.21	0.08	2.70	2.82	8495.46	3469.43	13596.99	2.45	6.91
1-D	0.697	0.19	0.07	2.72	2.35	21743.17	9627.95	29527.75	2.26	5.30

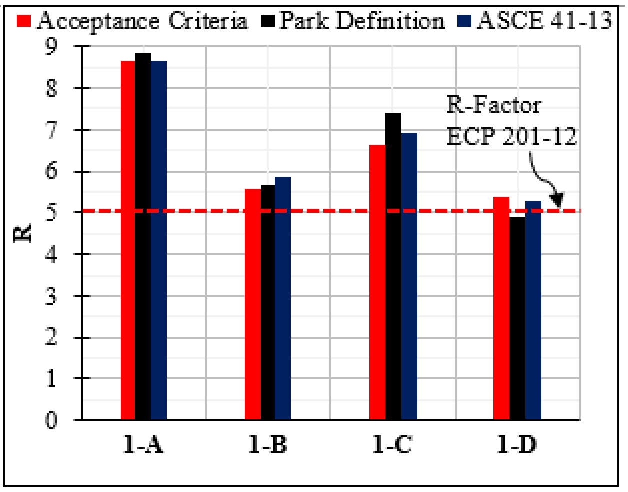


Figure 16 : Relation between R for Different Models and Various Calculation Method.

Table 7,

Table 8,

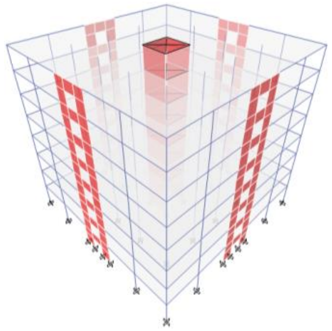
Table 9, and Figure 16 present R-factor value calculated by three different methods. The three different procedures for calculation of seismic Response modification factor from pushover curve give results with average variation between them less than ±10%. i.e., Acceptance criteria limit for hinge deformation, Park definition for ultimate and yield steps and ASCE41-13 Idealized bilinear curve. which means that any of them could be used for evaluating R-factor. calculated R-factor values comply with the given value of R-factor at ECP-201 (2012).

NUMERICAL STUDY FOR SEISMIC RESPONSE MODIFICATION FACTOR FOR SHEAR WALL WITH OPENINGS IN MULTI-STOREY FRAME BUILDINGS

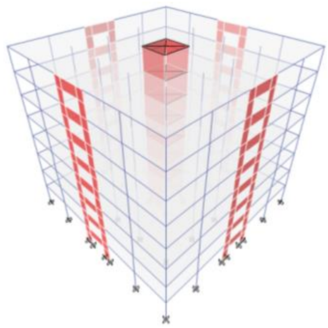
A numerical study has been conducted using the same method to evaluate (R) factor by applying openings with different dimensions, defined the lintel beam as a spandrel, wall segment and putting additional steel around openings. Acceptance criteria limits for hinges deformation method will be used to evaluate the R-Factor. Table 10 show the models numbering and the openings dimensions.

MODEL DESCRIPTION

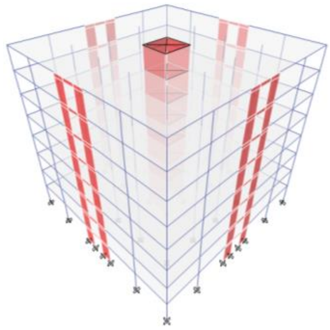
Figure 17, Figure 18, and Figure 19 present the openings dimensions and arrangement in 3D-View and elevation respectively. The ground acceleration, spectrum type and opening dimensions had been mentioned in Table 10.



Model with Opening 1.80 Width and 1.80 Height, (20% opening).

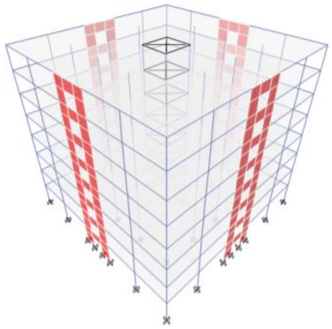


Model with Opening 2.75 Width and 1.80 Height, (30% opening).

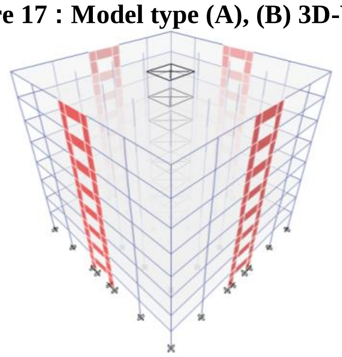


Model with Opening 1.80 Width and 2.75, (30% opening).

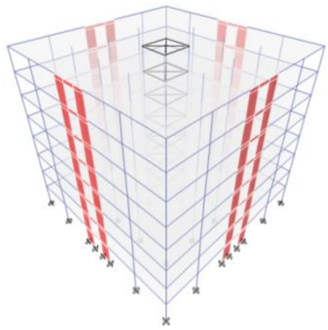
Figure 17 : Model type (A), (B) 3D-View.



Model with Opening 1.80 Width and 1.80 Height, (20% opening).

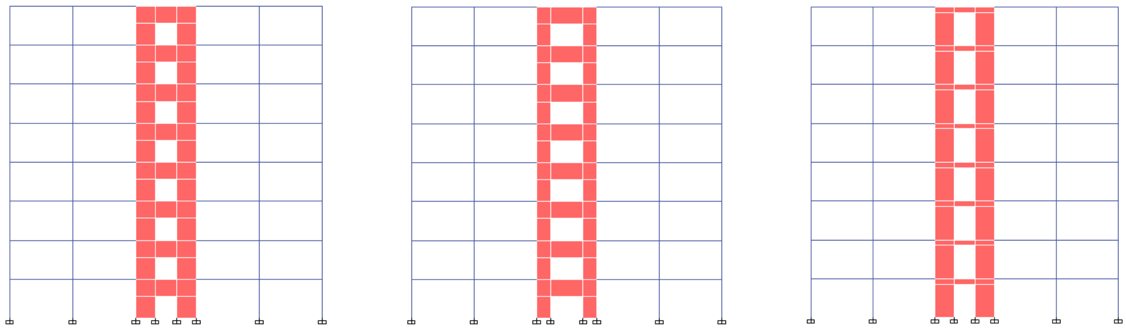


Model with Opening 2.75 Width and 1.80 Height, (30% opening).



Model with Opening 1.80 Width and 2.75 Height, (30% opening).

Figure 18 : Model type (C), (D) 3D-View.



Model with Opening 1.80 Width and 1.80 Height, (20% opening). Model with Opening 2.75 Width and 1.80 Height, (30% opening). Model with Opening 1.80 Width and 2.75 Height, (30% opening).

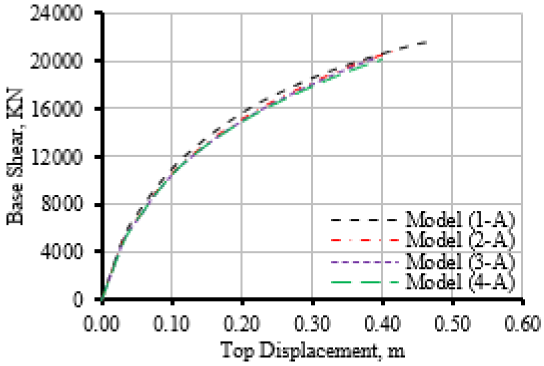
Figure 19 : Elevations of openings.

Table 10 : Numerical and Their Range (Lintel beam defined as a wall Segment).

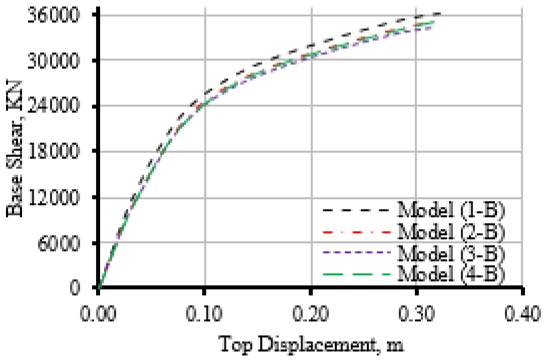
Model	Ground Acceleration ag/g	Spectrum Type	Opening Dimension B (m)	x	H (m)
1-A	0.15	2	-	x	-
2-A			1.80	x	1.80
3-A			2.75	x	1.80
4-A			1.80	x	2.75
1-B	0.25	1	-	x	-
2-B			1.80	x	1.80
3-B			2.75	x	1.80
4-B			1.80	x	2.75
1-C	0.15	2	-	x	-
2-C			1.80	x	1.80
3-C			2.75	x	1.80
4-C			1.80	x	2.75
1-D	0.25	1	-	x	-
2-D			1.80	x	1.80
3-D			2.75	x	1.80
4-D			1.80	x	2.75

RESULTS AND DISCUSSION

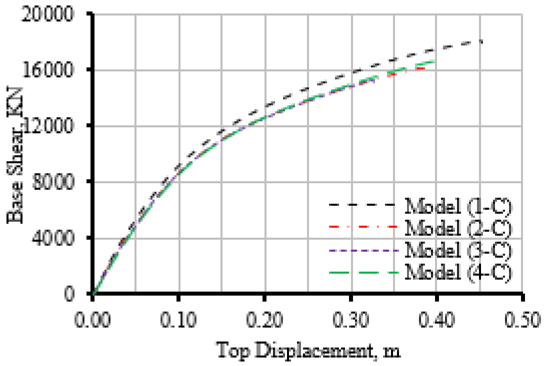
After pushover analysis a pushover curve has been plotted which used to calculate the response reduction / modification factor parameters (ultimate displacement, yield displacement and yield base shear) and evaluate (R) factor. as shown in Figure 20. Some degradation was observed in base shear related to the same displacement while opening conducted. This reduction increased by increasing the opening size. This decrease appears clearly in the models which haven't a core inside and also have openings in whole shear walls. Table 11 show the calculations of R-factor by using acceptance criteria limits for hinges deformation method and also, by defining the lintel beam as a wall segment.



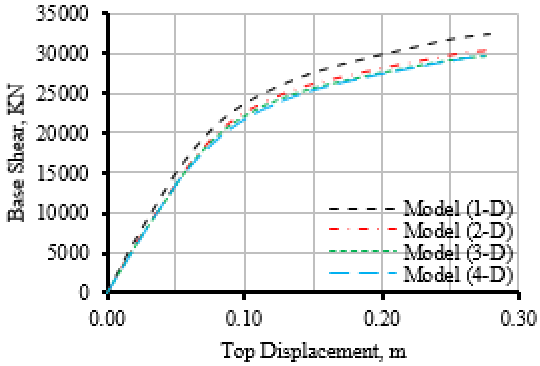
Models Type (A)



Models Type (B)



Models Type (C)



Models Type (D)

Figure 20 : Pushover Curves for Models A,B,C and D.

Table 11 : Calculation of R according to Acceptance Criteria Limits for Hinges Deformation (Wall segment).

Model	Time Period (sec)	Δu m	Δy m	μ	Rμ	Vy kN	Vd kN	Vu kN	Rs	R
1-A	0.932	0.30	0.12	2.52	2.45	12125.53	3424.89	18634.32	3.54	8.66
2-A	0.958	0.30	0.12	2.43	2.39	11935.52	3402.01	18287.35	3.51	8.38
3-A	0.979	0.25	0.13	1.99	1.97	11931.88	3583.24	16703.16	3.33	6.58
4-A	0.990	0.23	0.14	1.72	1.71	12389.41	4303.45	16183.77	2.88	4.94
1-B	0.632	0.24	0.10	2.33	2.03	25957.97	9467.39	33737.89	2.74	5.56
2-B	0.659	0.24	0.11	2.14	1.92	25761.12	9373.14	32562.81	2.75	5.27
3-B	0.666	0.21	0.11	1.83	1.70	25333.94	9717.77	30659.16	2.61	4.43
4-B	0.675	0.22	0.13	1.63	1.55	27151.57	11534.00	31456.53	2.35	3.65
1-C	1.098	0.26	0.12	2.14	2.21	10451.01	3469.43	15009.81	3.01	6.65
2-C	1.143	0.26	0.13	2.03	2.12	10074.52	3446.56	14022.65	2.92	6.18
3-C	1.181	0.25	0.13	1.97	2.06	10064.76	3836.33	13824.82	2.62	5.41
4-C	1.208	0.31	0.13	2.39	2.58	10233.18	5287.65	15307.96	1.94	4.99
1-D	0.697	0.27	0.11	2.57	2.25	23114.73	9627.95	32401.81	2.40	5.39
2-D	0.734	0.23	0.10	2.33	2.11	22543.69	9533.69	29329.45	2.36	4.99
3-D	0.747	0.22	0.10	2.25	2.06	22090.70	10135.00	28436.66	2.18	4.48
4-D	0.763	0.24	0.11	2.22	2.05	22239.47	12965.00	28557.28	1.72	3.51

As an attempt to improve the system performance in case of openings conducted. Half of reinforcement bars terminated to conduct openings has been added on either side of the opening.

The results mentioned in Table 12 show two methods used in modeling the lintel beam above openings. The first, by considering the lintel beam as a segment of the wall (the openings have been conducted without changing the definition of the upper lintel). The second, by defining the upper lintel as a spandrel. And also, the study conducted by using two different seismic zones, ground acceleration (ag/g) equal 0.15 and 0.25 when the spectrum type is (2) and (1) respectively.

Table 12 : Response Reduction Factor (R) for Model A, B, C, and D (Different Modeling Methods).

Model	Response Reduction Factor (R)		
	Wall Segment	Spandrel	Additional Steel
1-A		8.66	
2-A	8.38	8.41	8.49
3-A	6.58	6.54	6.76
4-A	4.94	4.94	4.97
1-B		5.56	
2-B	5.27	5.19	5.42
3-B	4.43	4.45	4.59
4-B	3.65	3.65	3.96
1-C		6.65	
2-C	6.18	6.16	6.31
3-C	5.41	5.42	6.11
4-C	4.99	4.96	5.12
1-D		5.96	
2-D	4.99	4.99	5.27
3-D	4.48	4.56	4.58
4-D	3.51	3.49	3.70

Figure **21**

Figure **21** present the relation between the opening's dimension and the response reduction factor and clarify the decrement in R-Factor due to the openings. It's obviously that by increasing the opening dimension a reduction in R-factor observed. Opening height influence is critical than width

.

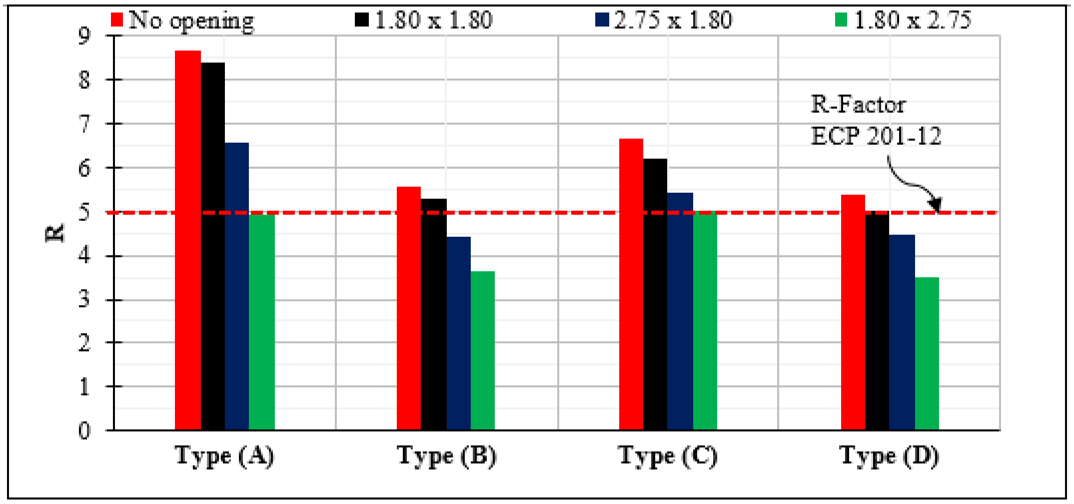
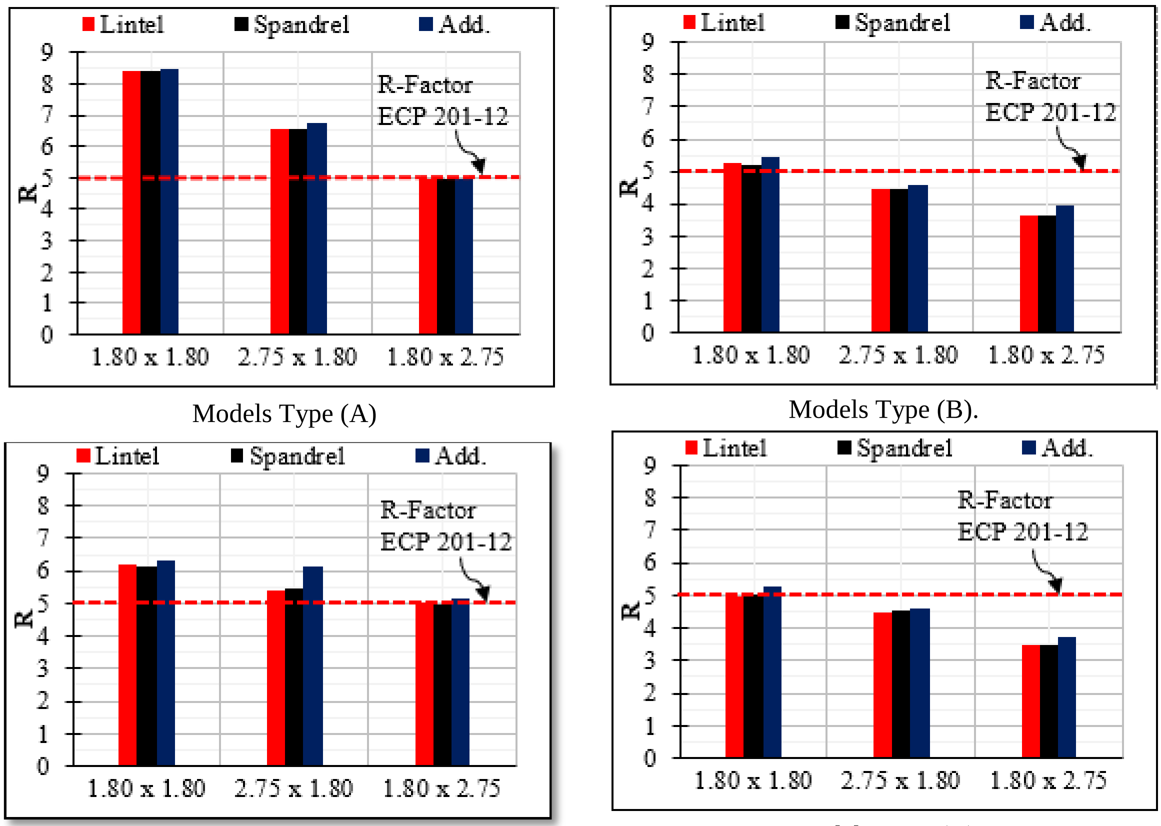


Figure 21 : Relation between R and opening Dimension in different models (Wall Segment).



Models Type (A)

Models Type (B).

Models Type (C)

Models Type (D)

Figure 22 : Additional Steel Improvement for Models A,B,C, and D.

From Figure 22 it's clear that additional steel beside openings case some improvement in R-factor up to 12%.

The given value of R-factor at ECP-201(2012) equals 5.0 for limited ductility class of Dual system from Moment Resisting Frames and Shear Walls with opening is un-conservative value; as the accurate value of R-factor is less than the given value. It may be noted that

Eurocode-8 (2004) specify values response reduction factor range between 3.0 and 3.9 for medium ductility reinforced concrete Frame system, dual system, and coupled wall system according to the configuration (One – story buildings, multi-story one-bay and multi-story multi-bay). IBC 2018 identify for Shear wall-frame interactive system with ordinary reinforced concrete moment frame and ordinary reinforced concrete shear walls response reduction factor equal 4.5.

CONCLUSIONS

In the present study Seismic Response Modification Factor was evaluated for Shear Wall with and without Openings in Multi-Storey Frame Buildings designed according to the Egyptian code of loads ECP-201 (2012). Two verified comparative examples are presented. Then, a numerical study had been conducted on four different models Shear Wall with and without Openings in Limited ductility Multi-Storey Frame Buildings which has been well designed according to Egyptian code of practice. The significant outcomes of works are summarized as follows:

The response reduction factor is considerably affected by the seismic zone and fundamental time period of the structure. It reduces as the seismic zone increases and increases as the fundamental time period increases. Openings affect the maximum base shear and maximum displacement that causes a decrease in the response reduction factor.

There is an inverse-relationship between the opening area and the response reduction factor as (R) factor decreased by increasing the opening area. The opening height impact on the response reduction factor is greater than the opening width.

The ratio between opening sizes to the area of the shear walls affects the response reduction factor (R) in which the ratio was more than 20%, decreasing the value for response reduction factor more than the recommended code.

To improve the system performance in case of openings conducted. Half of reinforcement bars terminated to conduct openings has been added on either side of the opening which case some increasement in the response reduction factor up to 12% for the studied cases.

The given value of R-factor at ECP-201 (2012) equals 5.0 for limited ductility class of reinforced concrete dual system from moment resisting frames and shear walls structures with opening is un-conservative value; as the accurate value of R-factor is less than the given value.

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