Seismic strengthening of existing reinforced concrete buildings by providing shear walls

Mohie El-Din Salah Shoukry, Tarek I. Ebeido, Gehad Ez El-Din Rashad and Sanaa Mohamed Abd El-Reheim

Structural Eng. Dept., Faculty of Eng., Alexandria University, Alexandria, Egypt

In the recent years, shear walls have proven to be the best system in strengthening reinforced concrete structures with bad performance during earthquakes. Shear walls may be placed on the exterior sides of the building or at the middle if it is convenient. Additional shear walls are to be connected to the existing columns. This method increases the seismic resistance of the building. In this paper a numerical analyses were conducted on three models of buildings low-height, high buildings and buildings with setback. Reductions in column's cross-sections were also considered. Such investigations were performed using pushover nonlinear analysis. Models were first analyzed without shear walls. Because these models were not designed to resist seismic loads the results showed several weak points. The first model consisted of four stories whereas the second model comprised ten stories. The third model considered represented a setback structure. The three models were strengthened by providing shear walls in order to investigate their effects on the seismic behavior of the structures. In some cases column's sections were kept constant along the height without any reduction whereas in other cases columns sections were reduced along the height. Furthermore, cases of partial shear walls located at the weak or abrupt drifts where the shear walls were placed in some floors above the ground floor were considered. Results revealed that the presence of shear walls significantly improved the strength and stiffness of the building when subjected to seismic forces. The presence of shear walls resulted in a significant decrease in the roof displacements and inter-story drifts. Furthermore, it was found that the collapse mechanisms of RC buildings provided with shear walls differ significantly than those without shear walls.

فى السنوات الأخيرة أثبتت الدراسات ان استخدام حوائط القص يعتبر من افضل الانظمه لتقوية المنشآت الخرسانية المسلحة ذات الأداء الغير مرضى أثناء تعرضها لاحمال الزلازل. و عموما فان حوائط القص يمكن أن توضع عند أطراف المبنى أوفى منتصف المبنى إذا كان ذلك مناسبا كما يجب توصيل هذه الحوائط المضافة بالأعمدة القائمة حيث ان هذه الطريقة تزيد المقاومة الزلزالية للمبنى. فى هذا البحث تم عمل تحليل عددى على ثلاث نماذج للمنشآت قليلة الارتفاع، عاليه، وكذلك المبانى ذات الردود و تم كذلك دراسة تأثير تخفيض قطاعات الأعمدة مع الارتفاع و تم تحليل المنشآت أولاً بدون حوائط قص. لماكانت هذه المبانى لم يتم تصميمها لمقاومة أحمال الزلازل فقد أظهرت النتائج عدة نقاط ضعف. يتكون النموذج الأول من أربعة طوابق فيما يتكون النموذج الثانى من عشرة طوابق. مثل النموذج الثالث مبنى بردود. تم تقوية النماذج بإضافة حوائط القص من أجل دراسة تأثيرها على السلوك الزلزالى للمنشآت و فى بعض الحالات تم الاحتفاظ بقطاعات الأعمدة ثابتة بكامل ارتفاع المبنى وفى بعض الحالات الأخرى تم تخفيض قطاعات الأعمدة مع الارتفاع. أيضاً تم إعتبار حالات إضافة حوائط قص جزئية تم وضعها عند نقاط الضعف فى بعض الطوابق دون الأخرى. وقد أوضحت النتائج أن وجود حوائط القص أدى إلى تحسن كبير فى مقاومة وجساءة المبنى عند تعرضه لأحمال زلازل و إن وجود هذه الحوائط أدى إلى خفض كبير فى الازاحة الكلية للمبنى عند قمته وكذلك فى الازاحات النسبية بين الطوابق. كذلك فقد وجد إن وجود حوائط القص قد أدى إلى إختلافاً كبيراً فى شكل الانهيار بالمقارنة بحالة عدم وجود حوائط قص.

Keywords: Drift, Earthquakes, Pushover, Reinforced concrete, Shear walls, Seismic

1. Introduction

Shear walls are essentially columns with large strength and small thickness. Generally, reinforced concrete columns resist horizontal external forces with their axial and flexural stiffness. However, reinforced concrete shear walls exhibit shear stiffness, in addition to the axial and flexural stiffness, which provides higher resistance against lateral forces. Shear walls are quite stiff in their own planes and are flexible in the perpendicular planes.

Therefore, shear walls transfer seismic forces in their own planes by developing adequate moment and shear resistance. The required reinforcement for shear walls is not excessive. A shear wall may behave as a shear beam if the height- to-length ratio H/L is small and it is called in this case squat wall; H/L < 1.5 [1]. On the other hand, if H/L is greater than 1.5, the shear wall will behave as a beam with significant flexure and shear deformations.

Generally, reinforced concrete shear walls are vertical seismic elements that resist lateral loads in their plane. They are like vertical diving boards extending upward from the foundation. The earthquake forces act horizontally in the plane of this vertical cantilever. After the shear force has been transmitted into the shear wall, it behaves like an almost rigid diaphragm to resist these forces. In reinforced walls, the reinforcing bars are usually described in a regular rectangular pattern, with bars running in both horizontal and vertical directions at uniform spacing. Shear walls develop bending moments as well as shear forces, and all forces are transmitted to the foundations, which resist the tendency of the seismic forces to push the wall over in its own plane. This moment, which tends to rotate the shear wall, is called the overturning moment. It increases from the top to the bottom of the building. This is why reinforced shear walls have extra vertical bars placed at the ends. This boundary reinforcing resists the bending forces, alternating vertical tension and compression, in the wall. In new construction, there are usually smaller bars placed like column ties around the boundary reinforcing to ensure confinement and ductility of the concrete. Bending forces can also develop around large openings in walls. This is why additional trim bars are added at the edges of wall openings. Horizontal construction joints in walls rely on shear transfer mechanisms such as built-in bumps or blocks, like the vertical joints in rigid floor diaphragms. Damage patterns in reinforced walls following earthquakes reflect their relative strengths in shear and bending. Walls which are stronger in bending than shear exhibit shear damage [2].

Reinforced concrete buildings provided with a combined system of columns and shear walls have behaved relatively well during earthquakes. The horizontal displacement, at the direction of earthquake, will be ensured to be uniform and small. This means that shear walls have the capability to control of excessive undesirable drift and inter–storey drifts. Shear walls and dual system buildings have also performed well except for buildings having wall plan irregularities that have exhibited excessive torsional demands on columns [3].

In the recent years, shear walls have proven to be the best system in strengthening reinforced concrete structures with bad performance during earthquakes. In the following sections, different ways of strengthening reinforced concrete buildings using shear walls will be presented and discussed.

1.1. Seismic strengthening of RC buildings

A higher degree of damage in a building is expected during an earthquake if the seismic resistance of the building is inadequate. The decision to strengthen it before an earthquake occurs depends on the building's seismic resistance. The seismic evaluation procedure may give a measure to the seismic resistance of the structure. The structural system of deficient building should be adequately strengthened in order to attain the desired level of seismic resistance. The term "strengthening" comprises technical interventions in the structural system of a building that improves its seismic resistance by increasing the strength, stiffness and/or ductility.

The first step towards the process of seismic strengthening is to determine the basic construction characteristics and earthquake resistive capacity of the existing building. The performance objectives for rehabilitation are then decided and the corresponding seismic hazard level is determined. The strategy of strengthening is discussed in the following sections [4].

1.1.1. Local modifications of building components

A few components (such as beams, columns, connections, shear walls, diaphragms, etc.) in an existing building may not have adequate strength or deformation capacity, though the building in whole may have substantial strength and stiffness. For such components, local modifications can be performed, while retaining the basic configuration of the building's lateral force resisting system. The local modifications considered are component connectivity, their strength, and/or deformation capacity. FEMA 273 [5] and NZDC [6] explain that the component is allowed to resist large deformation levels without failure by improving the deformation capacity or ductility of the component, without necessarily increasing the strength. For example, placement of a jacket around a reinforced concrete column to improve its confinement increases its ability to deform without spalling or degrading reinforcement splices. As per FEMA 273 [5], the cross section of selected structural components can be reduced to increase their flexibility and response displacement capacity. According to Eurocode 8 [7] local or overall modification of damaged or undamaged elements (repair or strengthening) can be done, considering their stiffness, strength and/or ductility. It also suggests full replacement of inadequate or heavily damaged elements. Structural rehabilitation, as defined in UNIDO manual [8], may also consist of a modification of the existing structural members so that their individual strength and/or ductility are improved. As a result, the respective characteristics of the structure are influenced (e.g., jacketing of the columns), even though the overall structural scheme is unmodified.

1.1.2. Removal or lessening of existing irregularities

Irregularities of strength, stiffness and mass have major contribution in unsatisfactory earthquake performance. Distribution of uneven structural displacements, with large concentrations of high values within one storey or at one side of a building, indicates the presence of an irregularity. Asymmetrical plan distribution of resisting members, abrupt changes of stiffness from one floor to the other, concentration of large masses, large openings in walls without a proper peripheral reinforcement are further examples of such irregularities. Such features that are sources of weakness or that produce concentrations of stresses in some members should be eliminated. FEMA 273 [5] and NZDC [6] provide some corrective measures for removing such irregularities. As per these documents, irregularities such as soft or weak stories can be removed by addition of braced frames or shear walls, whereas torsional irregularities can be removed by addition of moment frames, braced frames or shear walls to balance the distribution of stiffness and mass within a storey. Components such as columns or walls which abruptly end at certain floors can be extended through the zone of discontinuity for smooth transfer of forces to the foundation. An irregular building can be transformed into a number of simple regular structures by isolating them through the provision of movement joints. However, this should be done with due consideration to problems associated with the provision of insufficient gap, which can lead to damage due to pounding. Eurocode 8 [7] considers the modification of the structural system, like elimination of some structural joints, elimination of vulnerable elements, and modification into more regular and/or more ductile arrangements.

1.1.3. Global structural stiffening and strengthening

Large lateral deformations induced in the structure due to ground shaking, impose high ductility demand on the components of the structure. Also flexible structures with components having inadequate ductility behave poorly. It is essential that such structures be stiffened at a global level. FEMA 273 [5] and NZDC [6] propose the addition of new braced frames or shear walls within an existing structure for increasing the stiffness. While some existing structures have inadequate strength, which result into inelastic behavior at very, low levels of earthquake forces and cause large inelastic deformation demands throughout the structure. By strengthening the structure, the threshold of lateral force at which the damage initiates, can be increased. Moment resisting frames can be provided as they are more flexible and add strength to the structure without significantly increasing its stiffness. Eurocode 8 [7] suggests addition of new structural elements like bracings or infill walls; steel, timber or reinforced concrete belts in masonry construction; etc. or addition of a new structural system to take the seismic action. As per UNIDO manual [8], strengthening of the whole structure can be undertaken to improve its lateral force resistance, stiffness and ductility. This can be achieved through the addition of new structural members to increase the respective characteristics of the structure, like bracing in a frame or skeleton structure or new shear walls in a shear wall structure in order to reduce the eccentricity of the masses. A new lateral force resistant structure can be introduced to act integrally with the existing system to resist seismic forces (e.g., stiff shear walls introduced in a flexible frame or skeleton structure). Such an intervention produces significant changes of the stress distribution in the structure as well as in the structural layout. FEMA 273 [5] and Eurocode 8 [7] also suggest mass reduction of the structural system, wherever possible.

1.2. General strengthening techniques of RC buildings

A strengthening scheme consists of one/many strengthening techniques to remedy structural deficiency. Such schemes are specific to structural system and material type. Following is a brief description of major techniques that are used for reinforced concrete and masonry buildings. The provisions presented in different documents for the strengthening of reinforced concrete elements are as follows: (i) jacketing existing beams, columns, or joints with new reinforced concrete, steel, or fiber wrap overlays; (ii) post-tensioning existing beams, columns, or joints using external post-tensioned reinforcement; (iii) modification of the element by selective material removal from the existing element; (iv) improvement of deficient existing reinforcement details; (v) changing the building system to reduce the demands on the existing elements; (vi) changing the frame element to a shear wall, infilled frame, or braced frame element by the addition of new material; (vii) strengthening of individual diaphragm components by the addition of additional reinforcement and encasement; (viii) increasing the diaphragm thickness; and finally (ix) reducing the demand by adding lateral-force-resisting elements, introducing additional damping, or base isolating the structure.

1.3. The current research

It was found from the literature review that all documents of structural strengthening provide a general framework of rehabilitation process and do not provide much specific design/detailing procedure. In the recent years, shear walls have proven to be the best system in strengthening reinforced concrete structures with bad performance during earthquakes. Shear walls may be placed on the exterior sides of the building or at the middle if it is convenient. Shear walls are to be connected to the existing columns. This method increases the seismic resistance of the building. In this paper a numerical analyses were conducted on three models of buildings low-height, high buildings and buildings with setback. Reductions in column’s cross-sections were also considered. Such investigation were performed using pushover nonlinear analyses. Models were first analyzed without shear walls and the results are not presented herein for brevity. Because these models were not designed to resist seismic loads the results showed several weak points.

The first model consisted of four stories whereas the second model comprised ten stories. The third model considered represented a setback structure. The model consisted of ten stories and had a regular setback in the last five stories. The setback was done by dispatching the first and last bays in the longitudinal direction and the first bay in transverse direction. The three models were strengthened by providing shear walls in order to investigate their effects on the seismic behavior of the structures. In some cases column’s sections were kept constant along the height without any reduction whereas in other cases columns sections were reduced along the height. Therefore the effect of the reduction in column’s sections were detected. Furthermore, cases of partial shear walls located at the weak or abrupt drifts where the shear walls were placed in some floors above the ground floor were considered. Such cases were compared to those having complete shear walls from the base to the top.

2. Description of models

A numerical analyses were conducted on three models of buildings low-height, high buildings and buildings with setback. Reductions in column's cross-sections were also considered. Such investigation were performed using pushover nonlinear analyses. Because these models were not designed to resist seismic loads, the results showed the following weak points: (i) the size and reinforcement of beams were not sufficient to resist the lateral seismic loads, premature plastic hinges occurred and as a result the first yield point was too less than the significance yield point and consequently the yield strength of the structure decreased; (ii) for models with sudden reduction in column sections, plastic hinges occurred in non-desirable locations, which resulted in either a decrease in the strength of the building or, a higher inter-story drift or both; and finally (iii) the models of setback structures were vulnerable to disagreeable plastic hinges which outcome weakness in the strength and drift.

The three models which represent reinforced concrete residential buildings were designed using the Egyptian code for loading [9] and the Egyptian code for design and construction of concrete structures [10]. The structural elements for the models consisted of solid slabs, beams, and columns. Each model consisted of 3 x 5 bays each having a width equal to 6000 mm. The height of the ground floor was 5000 mm whereas such height was 3000 mm for typical floors. The buildings were designed to resist gravity loads only: dead loads and live loads. The dead loads included the self weight of the structural elements and covering materials of 2.0 kN/m^2. The load of partitions was considered on all beams at floor levels except the roof with a value of 6.6 kN/m'. The live load was taken equal to 2.0 kN/m^2. The characteristic strength of concrete was taken equal to 25 Mpa. High tensile steel (360/520) was used. The analysis was carried out using the commercially available finite element package (SAP-2000-V9) [11]. The unit volume of the mass of concrete was taken equal to 2.5 kN/m^3 and the weight per unit volume was equal to 25 kN/m^3. Poisson's ratio for concrete was taken equal to o.2, modulus of elasticity of concrete and steel was taken equal to 20000 Mpa and 200000 Mpa, respectively.

The first model "SA" consisted of four stories whereas the second model "SB" comprised ten stories. The third model considered "SC" represented a setback structure. The model consisted of ten stories and had a regular setback in the last five stories. The setback was done by dispatching the first and last bays in the longitudinal direction and the first bay in transverse direction. More details of the models considered are presented elsewhere [12]. The three models were strengthened by providing shear walls in order to investigate their effects on the seismic behavior of the structures. The models "SA" and "SB" were provided with two parallel shear walls in the weak direction (y-direction). The shear walls were placed at the two ends axes (1 and 6) in the intermediate span. Such cases were called "SA-Y" and "SB-Y". Another case was studied for model "SB" where the two shear walls were placed in the two mid-span axes (3 and 4), similar to that of the core shear walls. This case was called "SB-Y_{core}" The model "SC" were provided with shear walls at different places in the x-and y-directions in order to investigate the effect of shear wall location on controlling the horizontal drifts. In the X-direction, the shear walls were placed at the first and end frame panels of setback; axes "A" and "C", and the case was labeled "SC-X", whereas the model was strengthened with shear walls in the y-direction placed at axes 2 and 5, and the case was labeled "SC-Y". Furthermore, two additional cases were studied for model "SC" as follows: (i) case of partial shear wall located at the weak or abrupt drifts, "SC1-X" and "SC1-Y", where the shear walls were added in the fifth and sixth floors above the ground floor, cases "SC2-X" and "SC2-Y" where the shear walls were placed in the fourth and fifth floors above the ground floor and, cases

"SC3-X" and "SC3-Y" where the shear walls were in the fifth, sixth, seventh, eighth and ninth floors above the ground floor; and (ii) case of complete shear wall, from the base to the top; "SC4-X" and "SC4-Y". It should be noted that for models "SA1-Y" and "SB1-Y" columns sections were kept constant along the height without any reduction whereas in the case of models "SA2-Y" and "SB2-Y" columns sections were reduced along the height. Furthermore for models "SC" no reduction in columns sections were considered in the analyses. Figs. 1 and 2 show plans and three-dimensional views for models considered in the analyses, respectively.

3. Non-linear pushover analysis

To carry out the non-linear analyses using the pushover process, it was necessary to assume a pattern for the applied pushover load distributed along the stories levels. In the current analyses the pattern was assumed similar to that given by the (ECL-201-2003) [9]. The total lateral load acting on the structure was calculated as follows:

$$V = Z.I.K.C.S.W. \tag{1}$$

Where V is the total lateral load, Z is the zone factor =0.30 (higher zone in the seismic Egyptian map), I is the importance factor = 1.00 for the ordinary and dwelling buildings, K is the structural factor (non-ductile frames) = 0.80, C is the flexibility factor $=1/\ (15\sqrt{T}) \leq 0.12$, T is the fundamental natural period and $T = 0.09\ H\ /\ \sqrt{B}$, H is the total height of the building, B is the width of the building in the force direction, and S is the soil factor were taken equal to 1.15 (medium to stiff soil) and W is the total weight of the structure. The magnitude of lateral load applied at the center of mass at a given floor was calculated as follow:

$$F_j = \frac{W_j H_j (V - F_t)}{\sum_{(i=1,n)} (W_i H_i)}. \tag{2}$$

Where F_j is the lateral force at the j^{th} floor above the foundation level, W_j is the weight of the jth floor, H_j is the height of the j^{th} floor measured from foundation level, n is the number of floors and, F_t is the additional top load at roof level and is given as:

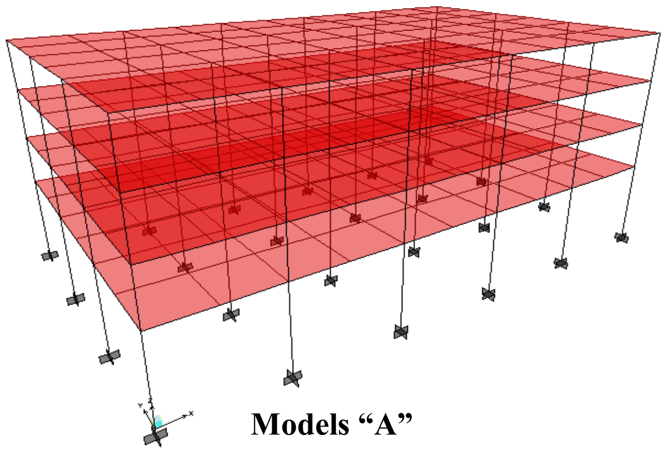
$$F_t = 0.07\ TV \leq 0.25\ V. \tag{3}$$

The computation process was carried using the (SAP-2000-V9) [11], considering a reduction to the acting dead load to 90%. All columns and beams were modeled using three-dimensional two-node frame elements with six degrees of freedom at each node. The reinforced concrete slabs were modeled using a four-node shell elements having six degrees of freedom at each node. The results obtained were the pushover curve for each model (base shear versus roof displacement), the sequence of plastic hinges, and the stories displacements at each incremental step.

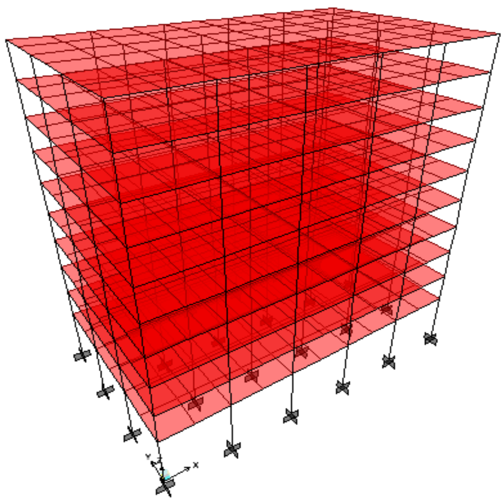
4. Results

All cases described above for the three models of low-height, high buildings and buildings having setback were investigated using pushover nonlinear analysis. The analysis was carried out using the commercially available finite element package (SAP-2000-V9) [11]. Models were first analyzed without shear walls and the results are not presented herein for brevity and can be found elsewhere [12]. Because these models were not designed to resist seismic loads the results showed several weak points as previously described. The behavior of such models were significantly enhanced when providing shear walls which will be discussed in the following sections. Table 1 shows linear and nonlinear properties for models "SA". Table 2 presents linear and nonlinear properties for models "SB", "Sb_{core}", and "SC". Table 3 shows seismic force evaluation for the considered models. Fig. 3 presents pushover curves for the models considered in the analysis. Figs. 4 to 8 (a and b) show collapse mechanisms for the models considered in the analysis. Figs. 9 to 13 show story displacement and inter-story drift for the models considered in the analysis.

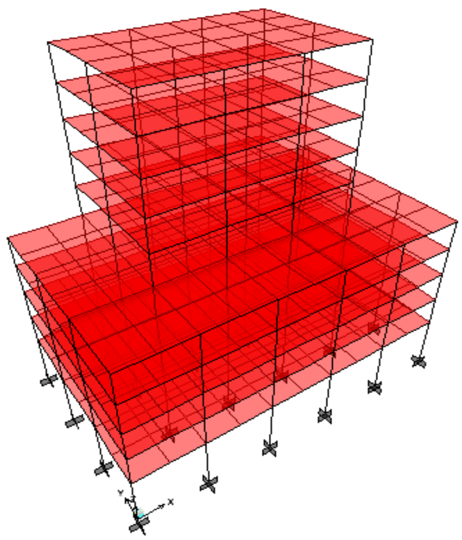
Fig. 1. Plans for the models considered in the analysis.



Models "A"



Models "B"



Models "C"

Fig. 2. Three-dimensional views for the models considered in the analysis.

Table 1
Linear and non-linear properties for models "SA"

Model	K_i	K_y	V_{y1}	V_y	Δ_y	V_u	Δ_u	μ_s	R_u
	(kN/mm)	(kN/mm)	(kN)	(kN)	(mm)	(kN)	(mm)	Δ_u/Δ_y	$\sqrt{2\mu_s - 1}$
SA1-Y	452.88	424.42	5462.79	5462.79	12.06	7970.52	18.40	1.52	1.43
SA2-Y	432.64	414.75	5145.22	5145.22	11.89	8624.97	20.66	1.73	1.57

* T= 0.542 sec.

Table 2
Linear and non-linear properties for models "SB", "SB_{core}" and "SC"

Model	K_i	K_y	V_{y1}	V_y	Δ_y	V_u	Δ_u	$R_u = \mu_s$
	(kN/mm)	(kN/mm)	(kN)	(kN)	(mm)	(kN)	(mm)	Δ_u/Δ_y
SB1-Y	103.91	103.91	3066.99	3066.99	29.52	6863.79	73.48	2.49
SB2-Y	86.06	86.06	2816.14	2816.14	32.72	6415.11	53.43	1.63
SB1-Y_{core}	122.22	119.27	3352.35	4124.83	33.91	7938.61	72.60	2.14
SB2–Y_{core}	104.34	102.31	3408.01	4050.49	38.94	7469.72	79.84	2.05
SC1-X	64.60	44.53	1880.69	4082.54	72.56	5673.64	160.46	2.21
SC2-X	64.96	44.06	1976.22	4276.24	73.97	5876.98	155.62	2.10
SC3-X	68.37	46.17	1880.32	4077.73	69.25	5670.00	155.23	2.24
SC4-X	139.90	139.90	3827.37	3827.37	27.36	9816.43	88.88	3.25
SC1-Y	24.96	26.30	1152.47	2235.56	71.30	3211.27	165.59	2.32
SC2-Y	24.85	26.21	1147.49	2385.60	77.64	3205.76	163.41	2.10
SC3-Y	29.97	29.10	1154.71	2269.56	63.22	3197.63	150.75	2.38
SC4-Y	413.79	393.06	7285.52	7285.52	18.25	16209.79	43.64	2.39

* T= 1.0 sec.

Table 3
Seismic force evaluation for the considered models

Model	Proposed system of evaluation				Evaluation according to ECL-201-2003			
					Limiting Ω_s to 1.60		Without any limits to Ω_s	
	R_m*	Ω_s* = V_u / V_{y1}	R* = $R_u . R_m . \Omega_s$	E* = V_u/R	R* = $\mu_s \Omega_s$	E* = V_u/R	R* = $\mu_s \Omega_s$	E* = V_u/R
SA1-Y	0.98	1.46	2.06	3876.02	2.43	3280.00	2.21	3606.56
SA2-Y	0.98	1.68	2.59	3336.47	2.77	3113.63	2.91	2963.90
SB1-Y	0.88	2.24	4.90	1400.63	3.98	1723.34	5.57	1232.08
SB2-Y	0.93	2.28	3.46	1851.47	2.61	2455.40	3.72	1724.62
SB1-Y_{core}	0.90	2.37	4.55	1744.40	3.43	2317.16	5.07	1565.60
SB2–Y_{core}	0.90	2.19	4.06	1841.18	3.28	2276.98	4.49	1662.17
SC1-X	0.89	3.02	5.96	951.64	3.54	1603.41	6.67	850.39
SC2-X	0.90	2.97	5.63	1044.12	3.37	1745.89	6.26	939.33
SC3-X	0.89	3.02	6.03	940.37	3.59	1580.85	6.76	838.80
SC4-X	0.85	2.56	7.08	1386.26	5.20	1888.40	8.33	1178.04
SC1-Y	0.89	2.79	5.75	558.93	3.72	864.17	6.47	496.21
SC2-Y	0.90	2.79	5.29	606.09	3.37	952.01	5.88	545.23
SC3-Y	0.88	2.77	5.84	547.39	3.82	838.15	6.60	484.27
SC4-Y	0.88	2.22	4.70	3445.62	3.83	4237.39	5.32	3047.20

*R_m = maximum reduction in strength.
Ω_s = the overstrength reduction factor.
R = response reduction factor.
E = computed seismic strength.

4.1. Strength and collapse mechanisms

The collapse mechanism of RC structures provided with shear walls differs significantly from those without shear walls. The high shear stiffness of the shear walls will attract nearly all the lateral seismic force. Therefore, the columns and beams will be strained with the bending moments caused by small amount of lateral seismic forces. Subsequently, it is not expected that plastic hinges take place either in columns or beams at low lateral forces. On the other hand, the RC wall may slacks too early its flexure stiffness, but the shear stiffness will start later to degrade near failure.

The collapse mechanisms of models of low-height buildings “SA1-Y” and “SA2-Y” are approximately the same; only the flexural stiffness of the shear walls, at the side of lateral excitation, failed in the ground and first floors. The pushover curves of both models were approximately the same; whereupon the reduction of columns did not have any effect in the presence of shear walls.

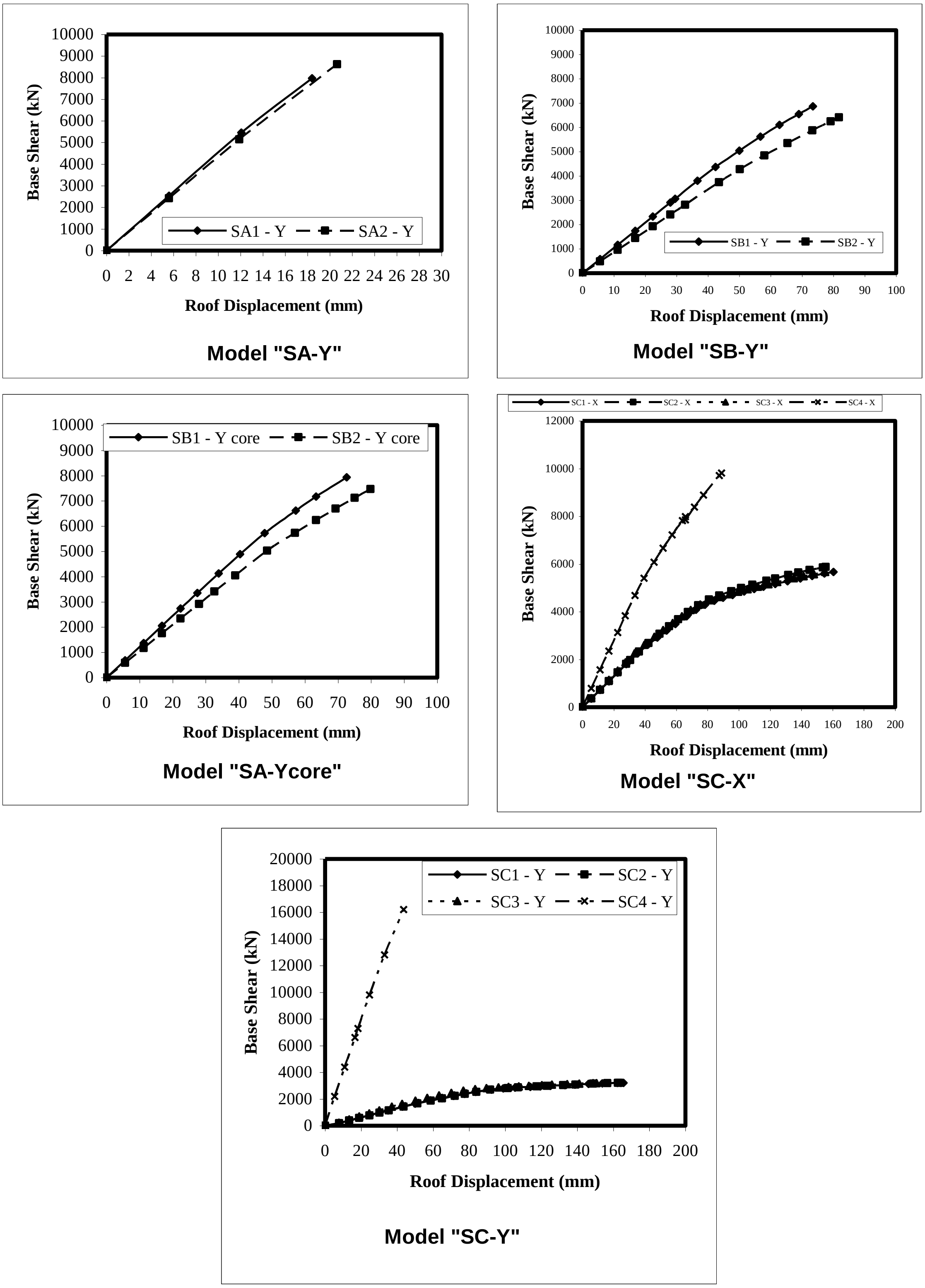
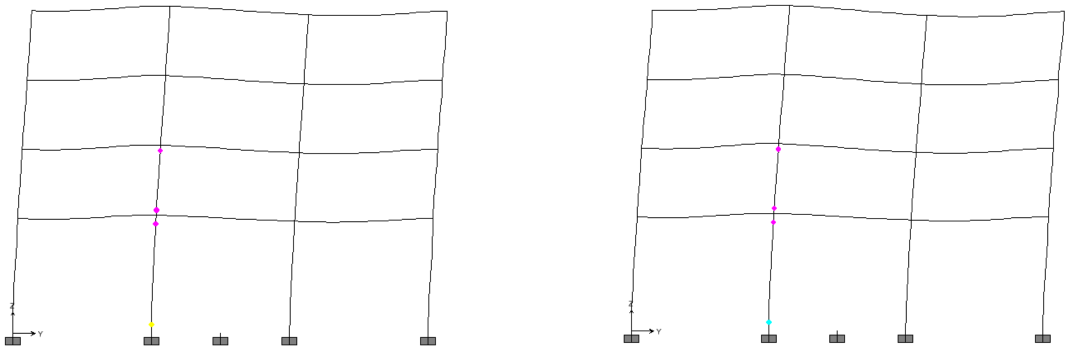
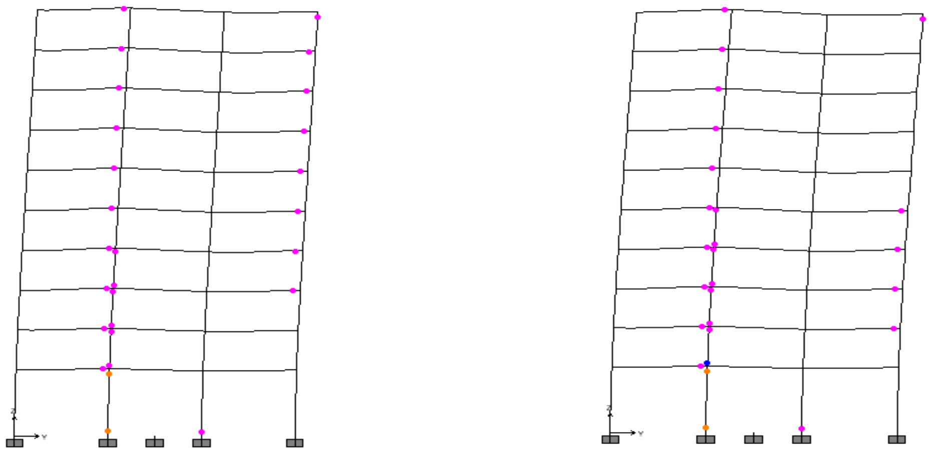


Fig. 3. Pushover curves for the models considered in the analysis.



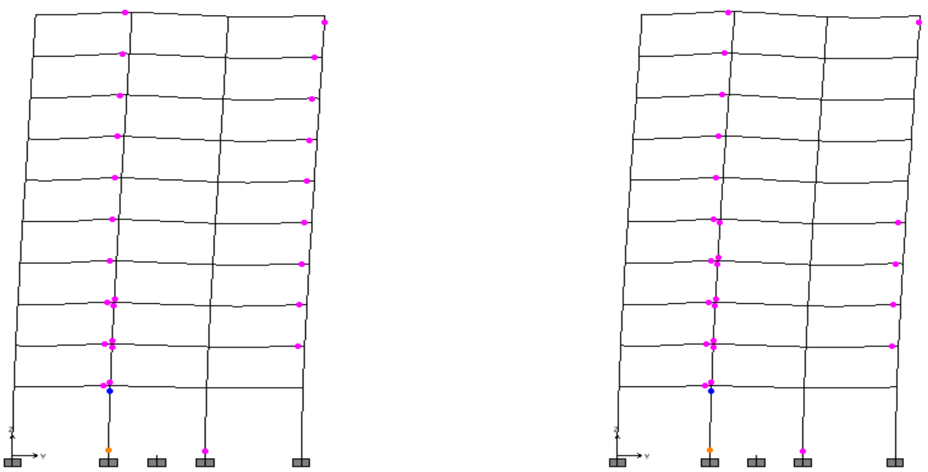
a- SA1-Y **b- SA2-Y**

Fig. 4. Collapse mechanisms for models "SA-Y".



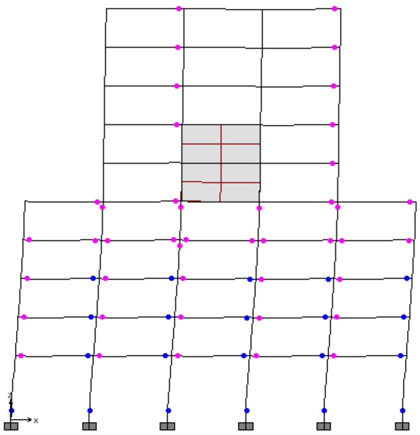
a- SB1-Y **b- SB2-Y**

Fig. 5. Collapse mechanisms for models "SB-Y".

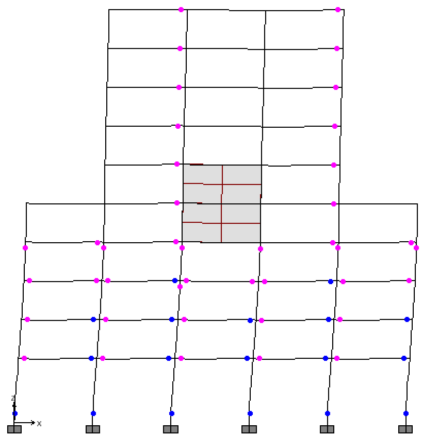


a- SB1-$\mathbf{Y_{core}}$ **b- SB2-$\mathbf{Y_{core}}$**

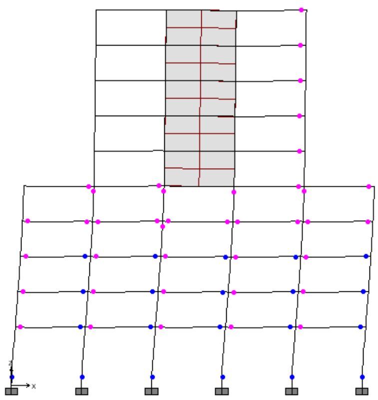
Fig. 6. Collapse mechanisms for models "SB-Y_{core}".



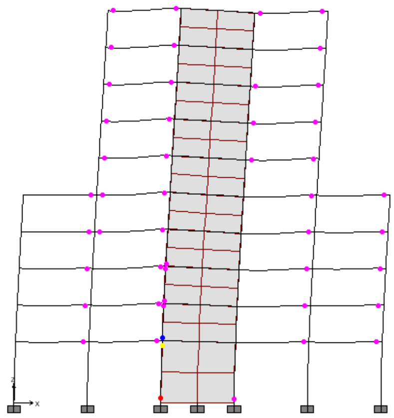
a- SC1-X



b- SC2-X

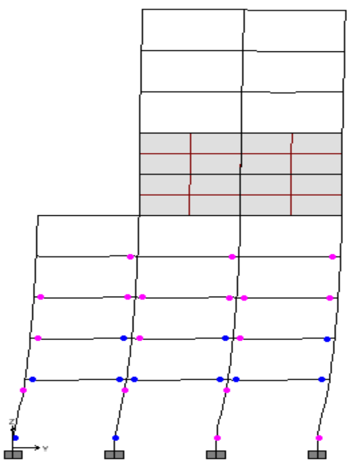


c- SC3-X

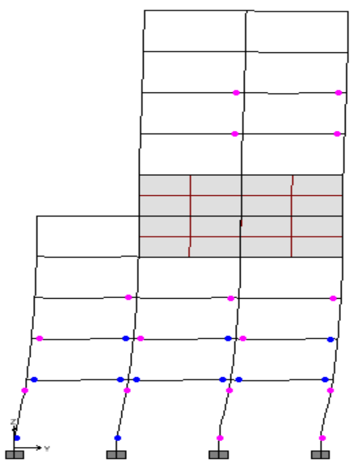


d- SC4-X

Fig. 7. Collapse mechanisms for models “SC-X”.



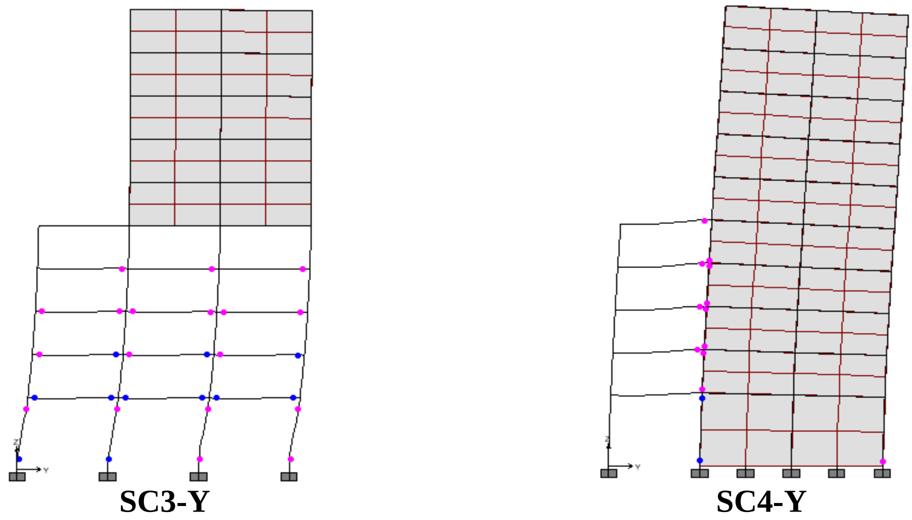
SC1-Y



SC2-Y

(a)

Fig. 8-a. Collapse mechanisms for models “SC1-Y and SC2-Y”.



(b)

Fig. 8-b. Collapse mechanisms for models "SC3-Y and SC4-Y".

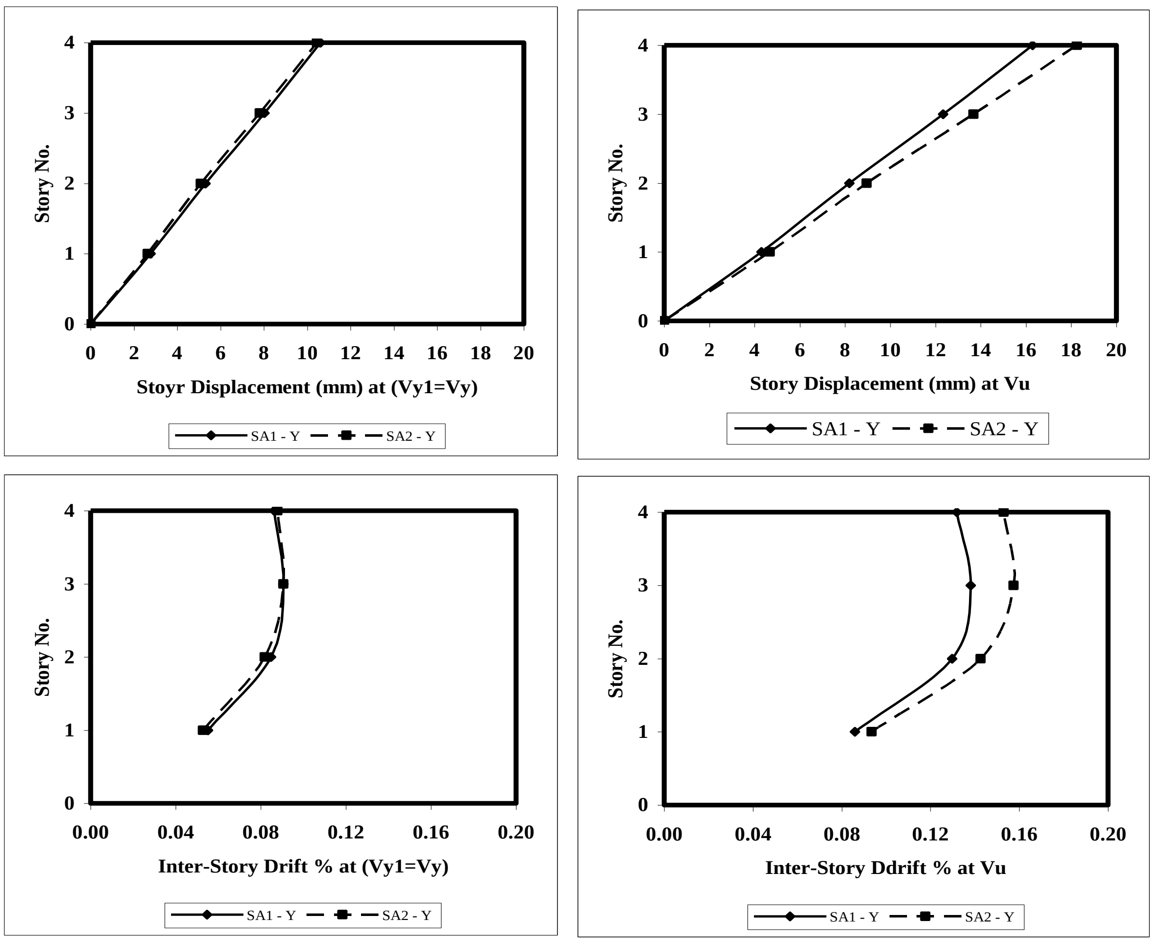


Fig. 9. Story displacement and inter-story drift for models "SA".

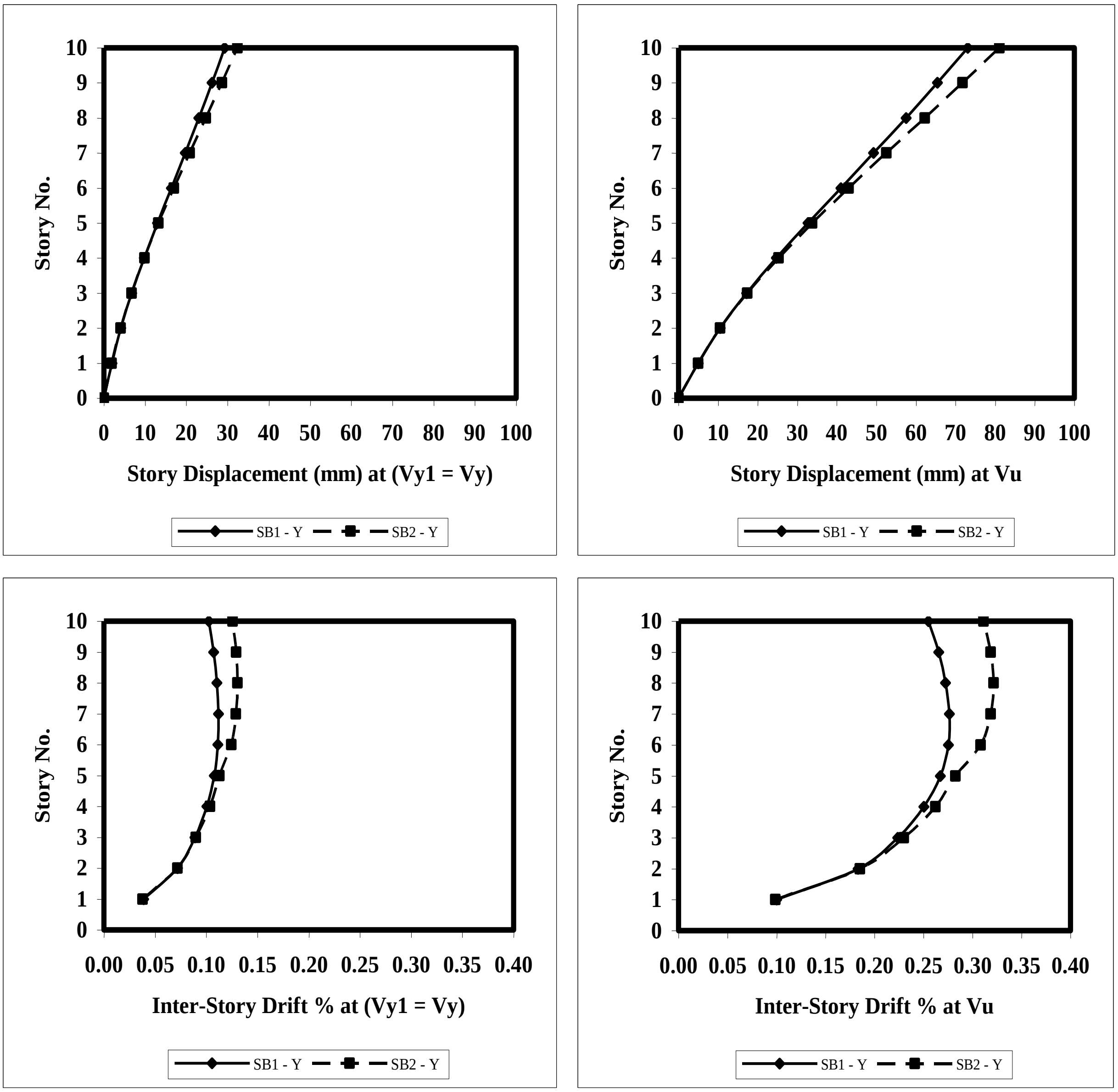


Fig. 10. Story displacement and inter-story drift for models "SB".

In all models of "SB" of high buildings, the collapse mechanism started by slacking the flexural stiffness of shear walls at side of lateral loads excitation, so later different plastic hinges occurred only in beams. The variation between the models with or without reduction in columns' sections is as follows: (i) in models with no reduction, shear walls had flexural plastic hinges in the ground floor and three floors above, whereas in models with reduction plastic hinges occurred in the ground and four floors above; and (ii) the strength of models with no reduction is slightly higher than that for models with reduction. Furthermore, it was found that models with shear walls placed in the middle of the building (core) exhibit more strength than those having shear walls placed at the edges of the buildings.

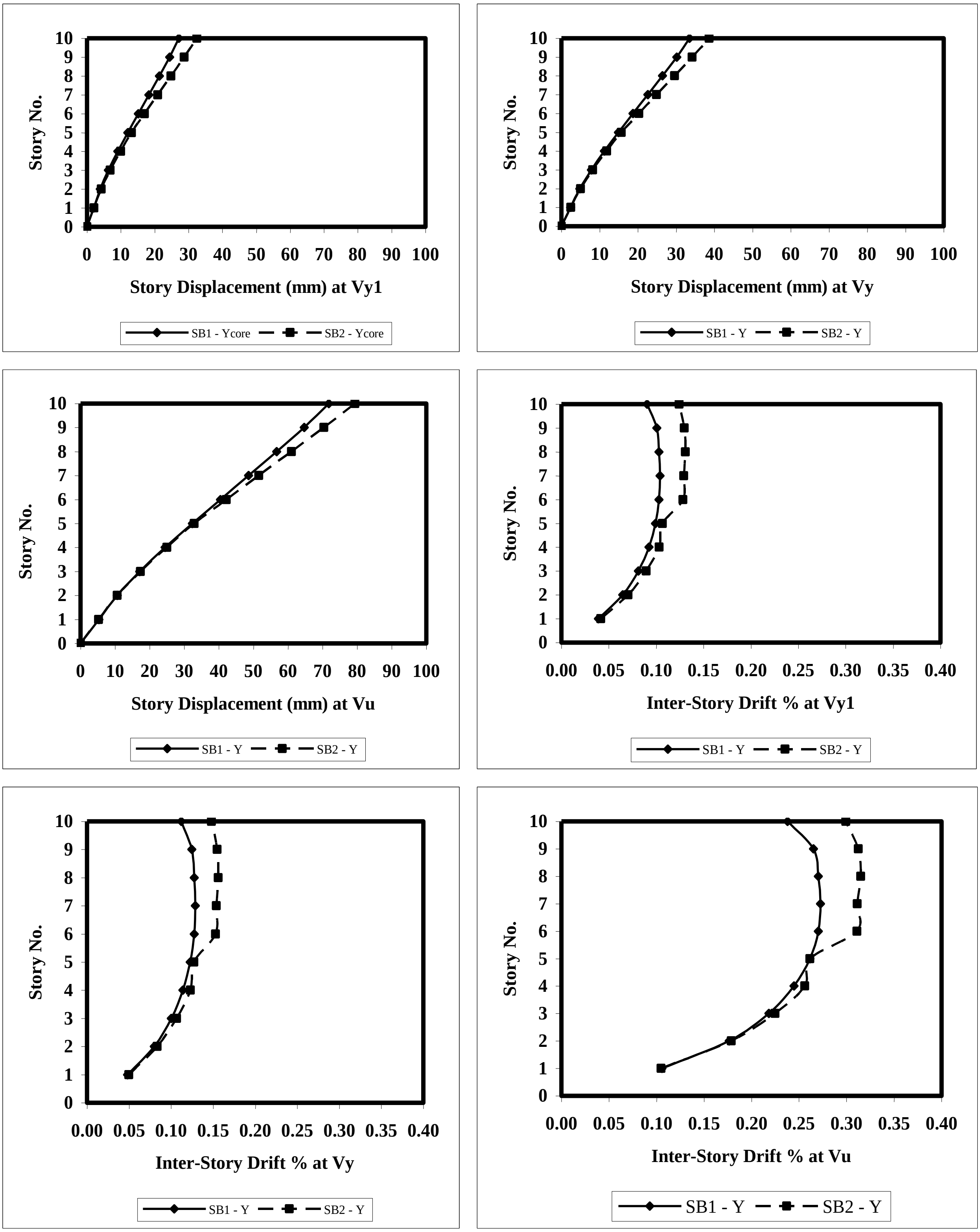


Fig. 11. Story displacement and inter-story drift for models "SB_{core}".

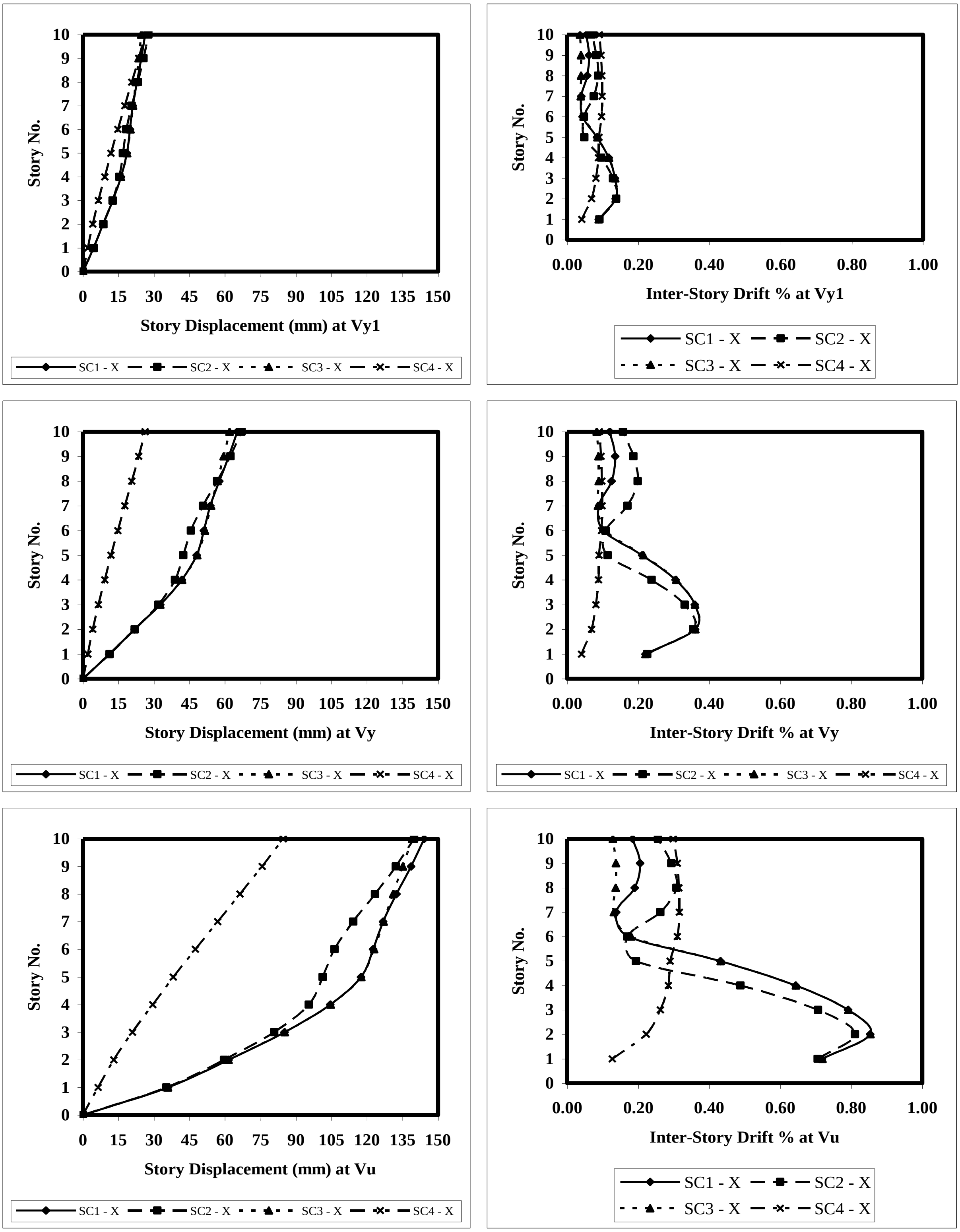


Fig. 12. Story displacement and inter-story drift for models "SC-X".

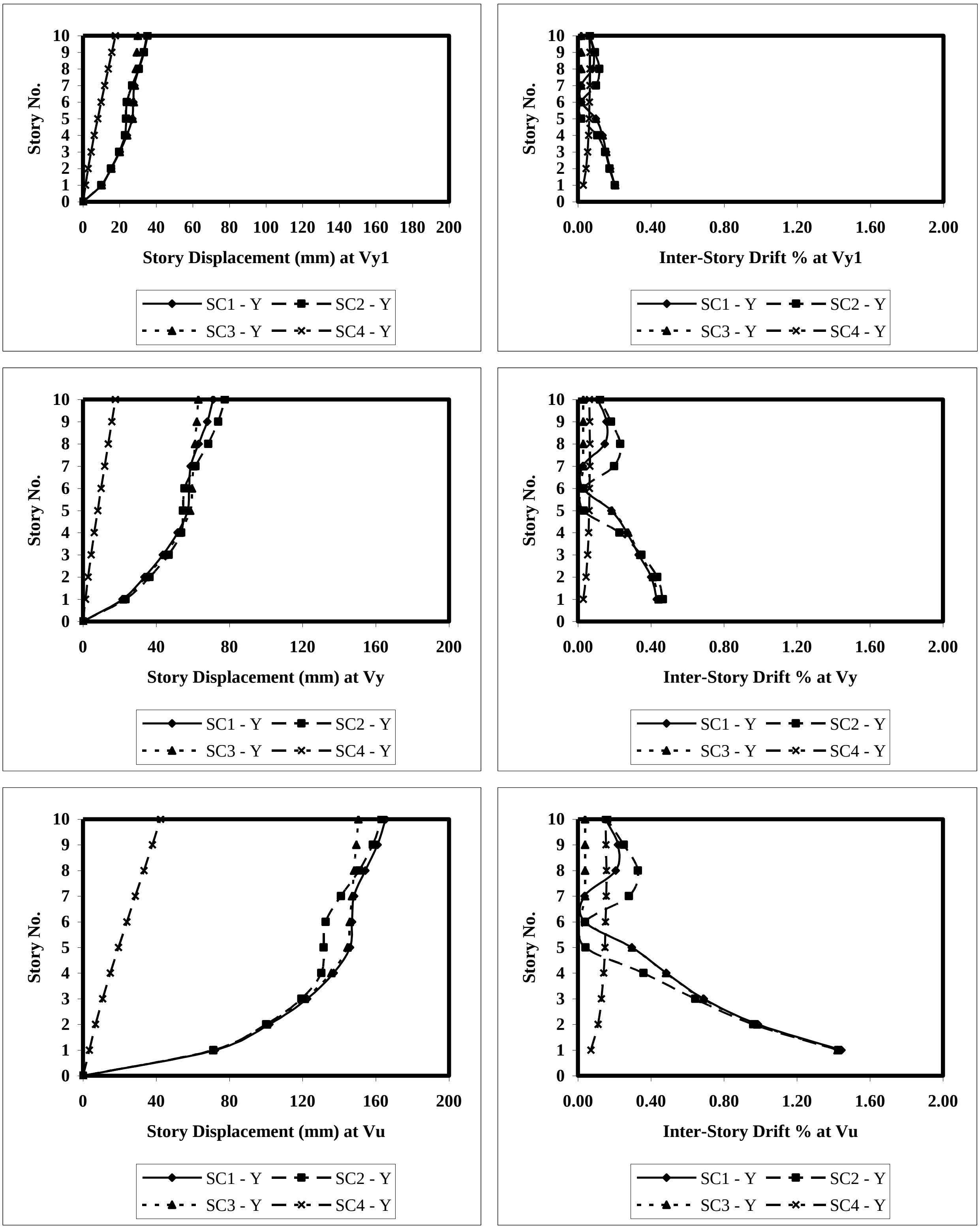


Fig. 13. Story displacement and inter-story drift for models "SC-Y".

For models of buildings having setback "SC-X" and "SC-Y", where the shear walls were added at the weak levels in the mid-height of the buildings, plastic hinges started to form at ends of beams and columns up to the level where the shear walls staged. The failure occurred due to beams collapse at floors beneath the level of shear walls. However, for models "SC4-X" and "SC4-Y", where the shear walls were added along the height of buildings, the collapse mechanisms differed substantially. In this case the RC walls started to loose flexural stiffness at edges at the side of lateral loads direction and also at both edges at base. Furthermore, plastic hinges formed in beams along the floor levels without any plastic hinges at columns. The strength of models with complete shear walls from base to roof level showed considerable higher strength than those of partial shear walls.

The main characteristic structural properties, extracted from the pushover curves, were presented in tables 1 and 2. The tables demonstrated the initial stiffness (Ki), the yield stiffness (Ky), the loads corresponding to the occurrence of first plastic hinge (Vy1), significant yield load (Vy), and the ultimate load (Vu). Also, the tables present the overall displacement at yield (Δy), at ultimate (Δu), and the calculated ductility (μs). Comparing those, with the results of same models without shear walls [12] the following can be observed for models "SA" and "SB": (i) the initial and yield stiffness of models "SA" and "SB" with shear walls were enormously higher than the same without shear walls; (ii) the first and significant yield point for models "SA" and "SB" with shear walls are the same which eliminated the awful property in calculating the seismic reduction factor; (iii) the ultimate strength increased substantially for models with shear walls; (iv) though the variation of the structural properties for models with or without shear walls, the calculated ductility were approximately the same; (v) the stiffness and the strength for models with shear walls at middle (core) are considerably higher (about 20%) than that having the shear walls placed at ends. This may be attributed to the fact that when the walls are placed close to the circumference of the center of mass they attract rapidly the excited lateral seismic load more than that when the same are placed far from the center of mass; and finally (vi) the effect of the reduction in column's sections will not appear in low-height building. However, on the other hand higher buildings will be significantly affected by a slight decrease in the stiffness and strength.

Furthermore, the following points were observed when comparing the results of the setback models "C" without shear walls presented elsewhere [12] with those provided with shear walls in the current study: (i) generally, for both directions of lateral loads excitations, the performance of models with complete shear walls from base to roof were significantly better than those with partial shear walls; (ii) for models with partial shear walls, the base shear versus roof displacement curves were too less than those with complete shear walls; (iii) because plastic hinges started in beams and columns before reaching the shear walls, the first yield point was still much lesser than the significant yield point for models with partial shear walls whereas for models with complete shear walls the first yield point was close to the significant yield point, and was approximately equal to twice the values obtained from models with partial shear walls; (iii) though the performance of models, provided with shear walls, were better, but it may be concluded that it should not be used in order to extensively increase the strength.

Table 3 shows seismic force evaluation for the considered models. The extracted values of the overstrength reduction factor (Ω_s) and the values of the seismic reduction factors (R) calculated using two different methods are shown in the table. These methods are: (i) the method proposed by the author after modification of Miranda's method [12 and 13]; and (ii) according to the (ECL-201-2003) [9], once by limiting the overstrength factor to 1.6 and once more after keeping the overstrength factor as computed. The determined values of "R" by the proposed method ranged between 2.06 and 4.90 for models "SA" and "SB", and ranged from 4.07 to 7.08 for models "SC". The (ECL-201-2003) [9] method for models without shear walls gave un-conservative results because the first yield point were far less than the significant yield point. However, in the case of models provided with shear walls the first yield point was too close to the significant yield point and the (ECL-201-2003) [9] method becomes more correct.

Comparing the results obtained by the proposed method [12] to the method of (ECL-201-2003) [9] without fixing the value of Ω_s, it can be observed that the proposed method gives higher values ranging between 10% and 13%. However, comparing the results of the proposed method [12] to the method of (ECL-201-2003) [9] with fixing Ω_s to 1.60, the variation was not identified; some models were slightly higher and other models were 40% less. It can be concluded herein that the proposed method for evaluating the seismic strength is more reliable.

4.2. Displacements and inter-story drifts

Figs. 9 to 13 show the extracted displacements and inter-story drifts at three stages; ultimate, yield and, first plastic hinge levels. The pattern of the inter-story curves may be explained as follows: (i) comparing the drifts resulted from models provided with shear walls and the models without shear walls, the entity of shear walls significantly increased the stiffness of the buildings which resulted in low drift values; (ii) smooth curves are observed for all models of "SA" and "SB". The drift of models having column's section reduction was slightly greater than that for models with no reduction; and (iii) dented curves are observed for models "SC" with partial shear walls, higher values of drift are observed beneath the levels of shear walls and low values of drifts were observed starting from the level of shear walls to the roof. On the other hand, smooth curves and low values of drifts were obtained in models "SC4-X" and "SC4-Y" having complete shear walls from the base to the roof. It should be noted that all drift values were accepted according to the (UBC-1997) [14] and the (ACI 318-2003) [15] codes, but were not accepted by the (ECL-201-2003) [9].

5. Summary and conclusions

The seismic evaluation of the structural systems of existing buildings needs to be performed in order to determine the nature and extent of deficiencies, which can cause poor performance during future earthquakes. This evaluation also helps to decide whether structural modifications are required at few locations in the structure for deficient components only or interventions are needed at the structure level so that its global behavior is improved and thus seismic demands on components are reduced. The success of strengthening scheme is very much dependent on the choice of strengthening techniques, which are very specific to structural type and materials of construction. Furthermore, the design and analysis of such schemes/techniques are quite complex and require a great level of sophistication than that ordinarily required for new components/ elements. All documents of structural strengthening provide a general framework of rehabilitation process and do not provide much specific design/detailing procedure.

One of the most efficient strengthening approach is the addition of shear walls on the exterior sides, or at the middle if it is convenient. Shear walls are to be connected to the existing columns This method increases the seismic resistance of the building. In this paper a numerical analyses were conducted on three models of buildings low-height, high buildings and buildings with setback. Reductions in column's cross-sections were also considered. Such investigation were performed using pushover nonlinear analyses. Models were first analyzed without shear walls and the results are not presented herein for brevity and can be found elsewhere [12]. Because these models were not designed to resist seismic loads the results showed several weak points as previously described.

The first model consisted of four stories whereas the second model comprised ten stories. The third model considered represented a setback structure. The model consisted of ten stories and had a regular setback in the last five stories. The setback was done by dispatching the first and last bays in the longitudinal direction and the first bay in transverse direction. The three models were strengthened by providing shear walls in order to investigate their effects on the seismic behavior of the structures. In some cases column's sections were kept constant along the height without any reduction whereas in other cases columns sections were reduced along the height. Therefore the effect of the reduction in column's sections were detected. Furthermore, cases of partial shear walls located at the weak or abrupt drifts where the shear walls were placed in some floors above the ground floor were considered. Such cases were compared to those having complete shear walls from the base to the top. Based on this study the following conclusions may be drawn:

1. Reinforced concrete shear walls improve building strength and stiffness and, consequently the entire behavior of the building when subjected to seismic forces. The presence of shear walls results in a significant decrease in the roof displacements, and inter-story drifts.
2. The collapse mechanism of RC buildings provided with shear walls differs significantly from those without shear walls. The high shear stiffness of the shear walls attracts nearly all the lateral seismic force. Subsequently, it is not expected that plastic hinges take place either in columns or beams at low lateral forces. On the other hand, the RC wall may slacks too early its flexure stiffness, but the shear stiffness will start later to degrade near failure.
3. The collapse mechanisms of models of low-height buildings with and without reduction in column's sections are approximately the same when they are provided with shear walls; only the flexural stiffness of the shear walls at the side of lateral excitation, failed in the ground and first floors. The pushover curves of both models were approximately the same; whereupon the reduction of columns did not have any effect in the presence of shear walls.
4. For models of high buildings with no reduction in column's sections, shear walls had flexural plastic hinges in the ground floor and three floors above, whereas in models with reduction in column's sections plastic hinges occurred in the ground and four floors above.
5. The strength of models of high buildings with no reduction in column's secctions is slightly higher than that for models with reduction. Furthermore, models with shear walls placed in the middle of the building (core) exhibit more strength than those having shear walls placed at the edges of the buildings.
6. For models of buildings having setback, where the shear walls were added at the weak levels in the mid-height of the buildings, plastic hinges started to form at ends of beams and columns up to the level where the shear walls staged. The failure occurred due to beams collapse at floors beneath the level of shear walls.
7. For models of buildings having setback, where the shear walls were added along the height of buildings, the collapse mechanisms differed substantially. In this case the RC walls started to loose flexural stiffness at edges at the side of lateral loads direction and also at both edges at base. Furthermore, plastic hinges formed in beams along the floor levels without any plastic hinges at columns. The strength of models with complete shear walls from base to roof level was considerably higher than those of partial shear walls.
8. The stiffness and strength of models with shear walls at middle (core) are considerably higher (about 20%) than that having the shear walls placed at ends. This may be attributed to the fact that when the walls are placed close to the circumference of the center of mass they attract rapidly the excited lateral seismic load more than that when the same are placed far from the center of mass.
9. Generally, for both directions of lateral loads excitations, the performance of models with complete shear walls from base to roof were significantly better than those with partial shear walls. For models with partial shear walls, the base shear versus roof displacement curves were too less than those with complete shear walls.
10. Because plastic hinges started in beams and columns before reaching the shear walls, the first yield point was still much lesser than the significant yield point for models with partial shear walls whereas for models with complete shear walls the first yield point was close to the significant yield point, and was

approximately equal to twice the values obtained from models with partial shear walls. Although the performance of models, provided with shear walls, were better, but it may be concluded that it should not be used in order to extensively increase the strength.

References

[1] M.B. Hueste, "Estimating Seismic Damage and Repair Cost" August.mceer.buffalo.edu/education/reu/04Proceedings/08Foltz.pdf (2004).

[2] ATC / SEAOC Joint Venture Training Curriculum, "Seismic Response of Concrete and Masonry Buildings part C: The Role of Shear Wall and Frames", c/o Applied Technology Council, 555 Twin Dolphin Drive, Suite 550, Redwood City, California, U.S.A., pp. 1-4. www.atcouncil.org (1997)

[3] M. Wernli, C.E. Ospina and S.H. Quan, "Seismic Evaluation of 32 Industrial Buildings by Screening Process and Analysis" the 13th World Conference on Earthquake Engineering, Vancouver, B.C., Canada, August, p. 999 (2004).

[4] D.C. Rai, Department of Civil Engineering Indian Institute of Technology Kanpur, Document No. IITK-GSDMA-EQ07-V1.0, Final Report: A-Earthquake Codes, IITK-GSDMA Project on Building Codes (2005).

[5] Federal Emergency Management Agency, "FEMA 273-Guidelines for Seismic Rehabilitation of Buildings", P.O.Box70274, Washington, D.C. 20024, U.S.A. (1997).

[6] Standards Association of New Zeland, "Code of Practice and Commentary on: The Design of Concrete Structures (NZS 3101: Parts 1 and 2", Wellington, New Zeland (1985).

[7] Eurocode (prEN 1998-1:2003), Design of Structures for Earthquake Resistance - Part 1: General Rules, Seismic Actions and Rules for Buildings (1998).

[8] UNDP/UNIDO, "Post-Earthquake Damage Evaluation and Strength Assessment of Buildings Under Seismic Conditions", Vol. 4, Vienna. www.acs-aec.org/Documents/Disasters/Projects/ACS_ND_001/COLOMwce.pdf (1985).

[9] Housing and Building Research Center, Ministry of Housing and Utilities, "The Egyptian Code for Calculation of Loads and Forces in Structures and Buildings", Cairo, Egypt, pp. 76, ECL (2003).

[10] Housing and Building Research Center, Ministry of Housing and Utilities, "The Egyptian Code for Design and Construction of Reinforced Concrete Structures", Cairo, Egypt, ECCS 203-(2001).

[11] SAP 2000 Nonlinear Computer Program, Version 9.0.3, Computer and Structures, Inc., Berkeley, California, U.S.A. (2004).

[12] S.M. Abd El-Reheim, "Evaluation and Strength Assessment of Existing Reinforced Concrete Frames", A thesis Presented to the Department of Civil Engineering, Faculty of Engineering, Alexandria University, In Partial Fulfillment of the Degree of Master of Science, Alexandria, Egypt (2007).

[13] E. Miranda, "Strength Reduction Factors in Performance-Based Design", The EERC-CURE Symposium in Honor of Vitelmo V. Bertero, January 31-February 1, Berkeley, California, U.S.A. (1997).

[14] International Conference of Building Officials (ICBO), Uniform Building Code (UBC), Whittier, California (1997).

[15] American Concrete Institute, "Building Code Requirements for Structural Concrete and Commentary, ACI 318-05", Detroit, Michigan, U.S.A. (2005).

Received November 25, 2007
Accepted January 23, 2008