

NONLINEAR MODELLING AND SEISMIC BEHAVIOUR OF PRECAST CONCRETE STRUCTURES WITH STEEL SHEAR WALLS

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ABSTRACT

A new structural system consisting of precast concrete frames and steel shear walls (SSW's) is introduced and studied numerically in this paper. Two different models, first using "exact" FEM and second using approximate equivalent strip model (ESM), are utilized for analysis of such a system with nonlinear static (pushover) procedure. In the FEM model use is made of shell elements while the ESM benefits from simple links that replace the wall panels in the model and are oriented such that they work in tension. Because of good agreement observed between the results of the models in smaller structures, for taller buildings only the ESM approach is followed where computationally applying the FEM approach is impractical. The lateral behaviour of the systems under consideration is investigated with regard to parameters such as number of stories and beam-column connection type. As a result, the ductility, overstrength and response modification factors are calculated for this new structural system as quantities required for their practical design.

INTRODUCTION

Use of steel shear walls (SSW's) as lateral load resisting system in steel structures goes back to early 70's. Since then, there has been many analytical as well as experimental research works accomplished on this system. Because the present paper embarks on numerical evaluation of the system, in the following literature review mainly analytical works has been focused on. Takahashi et al. performed two series of tests on a one storey and a two storey SSW having variable sizes and details and concluded that both specimens benefit from stable hysteresis loops with high energy dissipation capacity [1]. They also conducted FEM analyses on the same walls and determined their load- deformation curves which agreed well with the experimental results.

Mimura & Akiyama investigated the load-displacement behaviour of a steel frame with SSW in which the buckling load of wall plate was much smaller than its yield shear strength [2]. They assumed the total strength of such a combined system to be equal to the sum of the load required for elastic buckling of the SSW and the post-buckling strength of the SSW and its surrounding frame. To calculate the shear buckling load of the infill plate, use was made of the elastic theory of plate buckling and a simply supported wall. A tension field was considered to be existing for loads in excess of the elastic buckling load. Timler & Kulak tested a one storey, one bay steel frame infilled with an SSW [3]. Studying the developed tensile strains, it was conceived that the inclination angle of the tensile field in the wall was in the range of 44° to 56°. Since the thin SSW easily buckles in compression, a simple idea was developed to include only the inclined tensile strips of walls in their modelling and analysis. This idea was made it possible to replace the two-dimensional (2D) walls with simple one-dimensional (1D) links parallel to the tension field. The corresponding analytical model was called the equivalent strip model (ESM). Thorburn et al. presented a method for evaluating shear capacity of unstiffened thin SSW's [4]. To simulate the tension field, they

suggested and used the ESM and showed that replacing the wall panel with just 10 equivalent tension strips was sufficient.

Elgaaly et al. provided two analytical models to assess the lateral behaviour of SSW's under monotonic horizontal loads [5]. First, they performed a nonlinear FEM analysis for studying the post-buckling behaviour of different SSW's. Then a simplified model in which wall panels were replaced by equivalent diagonal tension strips was studied. The calculated ultimate strengths of the FEM models were considerably larger than the corresponding experimental results due to initial imperfections of test specimens. Elgaaly used NONSAP and ANSR-III softwares to study the post-buckling behaviour of steel plates [6]. Also he prepared models for investigating plate girders under in-plane shear and moment. A good agreement with experimental results was reported. In addition, models of slender multistorey SSW's under cyclic loads were examined. It was concluded that the post-buckling behavior of slender steel plates under in-plane loads can be predicted with high accuracy using the developed analytical models.

Studying the effectiveness of seismic strengthening of steel structures with SSW's, Bruneau & Bhagwagar analysed a 3-bay frame of a 20-storey hospital [7]. They used Drain-2DX for analysis and computed parameters such as maximum lateral displacements and ductility demands of stories. It was resulted that use of SSW can decrease the lateral displacements to a large extent and dissipate the input energy.

Berman & Bruneau suggested a new design process for SSW's [8]. They developed equations for calculating the required thickness of panels by nonlinear analysis of ESM. After examining the results of pushover analysis of multistorey SSW systems, it was observed that the actual failure mechanism of these systems located in between a soft storey mechanism and a uniform yielding of steel plates along building's height.

Behbahanifard & Grondin implemented an experimental work on a 3-storey SSW in the University of Alberta [9]. They used

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Abaqus to develop a companion FEM model for the test specimen and calculated the dissipated energy out of the stable hysteresis loops. The results of test and analysis were reported to be in good agreement. Rezai & Ventura analysed 4-storey SSW systems using LS-DYNA [10]. They concluded that generally strip, orthotropic, and FEM analytical models overestimated the elastic stiffness of walls compared with those from experiments. Kharrazi et al. developed a new analytical model called "the modified plate-frame interaction method" and demonstrated that this method could predict the seismic behaviour of multistorey SSW's with good accuracy [11].

Berman et al. tested six steel frames out of which two frames were with SSW and the other four were accommodated with bracings [12]. One of the SSW frames was equipped with plain and the other one with corrugated SSW's, the two diagonally braced frames were braced with pipes w/o concrete filling, and the two cross-braced frames again w/o concrete-filled pipes. The experimental specimen of the corrugated SSW demonstrated a very good behaviour against the applied loads. In the diagonally braced frames the one with a concrete-filled pipe had a superior behaviour over the other one with regard to the dissipated energy. Also, the cross-braced frame with concrete-filled pipes possessed maximum initial stiffness and the frame with plain SSW showed a maximum ductility.

Sabouri-Ghomi et al. investigated the interaction between a structural frame and the embedded SSW by developing an analytical model of the system [13]. The system included SSW's with different configurations regarding their thickness, the stiffeners, and openings. Using a low yield point (LYP) steel, Chen and Jhang constructed and tested several specimens of SSW's and studied the effects of width-thickness ratio, continuity of walls, and the beam-column connections. The LYP system proved to have a good level of ductility of about 3-6 percent [14]. Tsai et al. studied the effects of out-of-plane restrainers on seismic behaviour of SSW's at large drifts [15]. It was found that the restrainers are quite active in containing the out-of-plane buckling of walls and increase their ductility.

Tests on stiffened SSW's were implemented by Guo et al. using a 7-storey frame equipped with different SSW's including composite walls and walls stiffened against buckling with steel segments [16]. The shear capacity, ductility, and hysteretic behaviour of the walls were studied. It was reported that a combined stiffening method consisting of steel elements and reinforced concrete covering resulted in a desirable behaviour. The limitations of ASCE design regulations of SSW's were investigated by Berman [17]. It was noticed that the ductility demand of the web plate of SSW is much larger in shorter buildings. Also, the maximum demand on the side columns of SSW's in taller buildings from nonlinear time history analysis is considerably less than that computed by the capacity design concept of the code. SSW's with tension bracings were introduced by Kurata et al. [18] as a system for seismic rehabilitation of steel structures. In a large scale experiment, desirable seismic behaviour of a sample of the system was confirmed at large drifts.

The hysteretic behaviour of SSWs was studied by Sun et al. [19] using the equivalent strip method (ESM). Accuracy of the ESM was confirmed in modelling the hysteretic behaviour and the height to thickness ratio, shear strength of the steel plate, and stiffness of the boundary columns were identified as the main parameters affecting the nonlinear behaviour of the system. Seismic behaviour of SSWs was evaluated by Wang et al. by accounting for the exact practical details of the wall construction [20]. As a result, an advanced model was proposed for the stiffened low yield point steel plate shear walls.

A system composed of precast concrete frames and steel plate shear walls is clearly much rapid for construction and suitable for earthquake prone areas. In design of precast concrete frames in such seismic zones, one has the alternatives of designing a moment resisting frame with special (and expensive) joints, or a simple frame accommodated with shear walls. Use of the former system results in much heavier structural members because of the need to limit the storey drifts. In the latter case if use is made of steel plate shear walls, erection of the members and completion of the structural system can be made much more rapidly and some space saving can also be attained. Also, cost of construction of the joints is reduced considerably. Nonetheless, to the best knowledge of the authors, no reported research exists in the literature regarding the seismic behaviour of this innovative structural system. This is the motivation behind this paper in which seismic characteristics of this system including ductility, overstrength and response modification factors are derived through multiple pushover analysis of appropriate models of the system.

THE BUILDING MODEL

General

Precast concrete frames of 3, 5 and 10 stories height equipped with SSW's are to be assessed by nonlinear static analysis. All of the systems have 3-bay frames and each bay is 5 metres long. The SSW is located in the middle bay in all stories. The centre-to-centre storey height is 3 m. The common plan of the buildings and the elevation of the 3-storey building are shown in Figure 1. The buildings are located in a high seismicity zone and the site soil is of B type, according to the International building code [21]. For initial design, the response modification factor of this system is taken to be equal to that for precast concrete structures having intermediate concrete shear walls, i.e. 7 according to the PCI Handbook [22]. The buildings are to be used for residential purpose with a floor dead load of 5.0 kN/m² and live load of 2.0 kN/m². The storey diaphragms are assumed to be rigid. For the reinforced concrete columns and beams around the shear walls, the same stiffness limitations as required for the steel members are enforced in the design [11, 13, 15].

As results of design, for the 3-storey building, the wall thickness is 0.5 mm, the columns are 400×400 mm², the beams are 300×350 mm², the steel area (A_s) for columns is 9×10⁴ mm², top A_s of beams is 1×10⁴ mm², bottom A_s for internal and external beams are 11 and 10×10⁴ mm², respectively. For 5- and 10-storey buildings all the beams are 300×350 mm² with their top and bottom A_s being 3 and 7×10⁴ mm², respectively. The frame members' cross-section details for the 3-storey building are shown in Figure 2. The outcomes of design for the 5-storey and 10-storey buildings are summarized in Table 1 and Table 2, respectively.

Certain details are conventionally used for beam-column connections in precast concrete structures. To evaluate the connection type effects on the overall behaviour of the system, six types of connections varying in stiffness from simple to rigid are taken into account. These are: simple, lightly reinforced brackets, or LRB, heavily reinforced brackets, or HRB, 4-bolt connections, or 4BC, 8-bolt connections, or 8BC, and rigid. Details of the connections are shown in Figure 3.

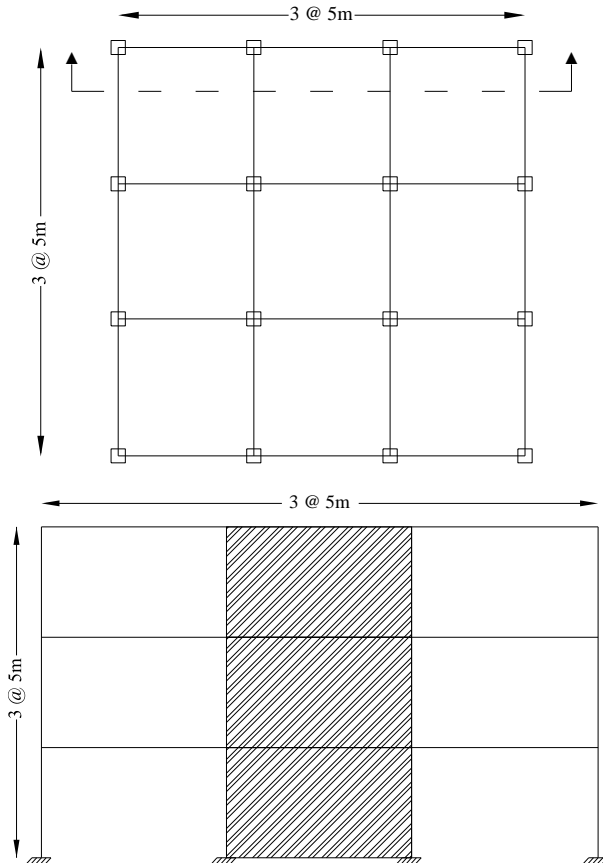


Figure 1: The common plan of the building and elevation of the 3-storey building.

Table 1: Design results for 5-storey frame.

Storey No.	Wall Thickness (mm)	Column Section (mm ²)	Steel Area	
			Internal Columns (×10 ⁴ mm ²)	External Columns (×10 ⁴ mm ²)
5	0.5	350×350	1200	1300
4	0.5	350×350	1200	1300
3	0.9	400×400	1200	2100
2	0.9	400×400	1200	2100
1	0.9	450×450	1900	3900

Table 2: Design results for 10-storey frame.

Storey No.	Wall Thickness (mm)	Column Section (mm ²)	Steel Area	
			Internal Columns (×10 ⁴ mm ²)	External Columns (×10 ⁴ mm ²)
10	0.5	300×300	1200	900
9	0.5	300×300	1200	900
8	1.0	350×350	3200	1200
7	1.0	350×350	3200	1200
6	1.3	400×400	4700	1600
5	1.3	400×400	4700	1600
4	1.5	450×450	7200	2000
3	1.5	450×450	7200	2000
2	1.5	500×500	10000	2500
1	1.5	500×500	10000	2500

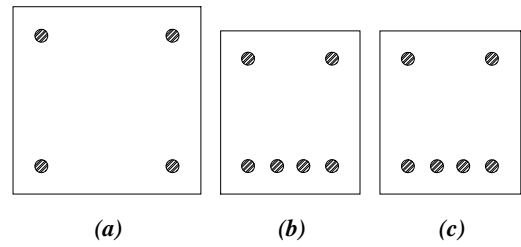
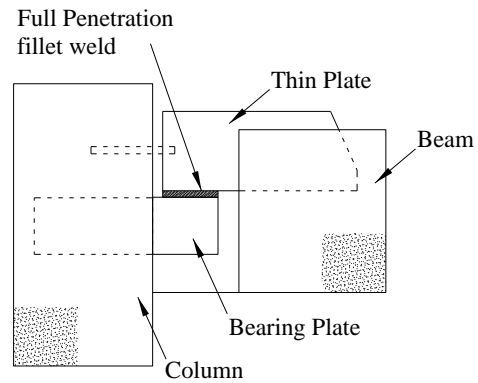
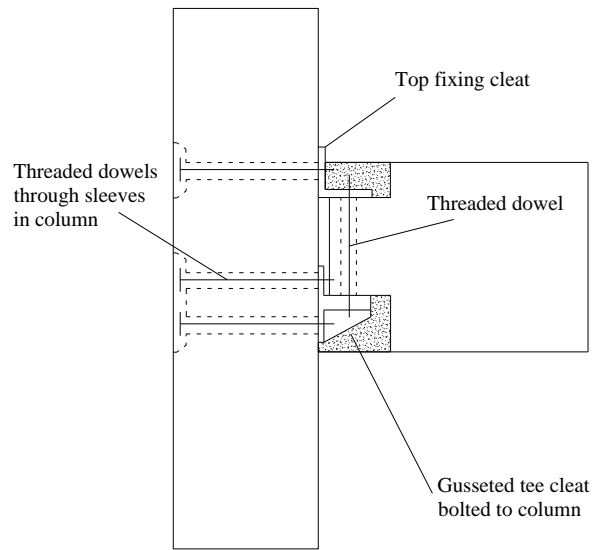


Figure 2: Section details of the beams and columns of the 3-storey building: (a) Column 400×400 mm, $A_s=9 \times 10^4 \text{ mm}^2$, (b) Beam 300×350 mm (internal), $A_{s,top}=1 \times 10^4 \text{ mm}^2$, $A_{s,bot}=11 \times 10^4 \text{ mm}^2$, (c) Beam 300×350 mm (external), $A_{s,top}=1 \times 10^4 \text{ mm}^2$, $A_{s,bot}=10 \times 10^4 \text{ mm}^2$.



(a) Reinforced bracket.



(b) Bolt connection.

Figure 3: The beam-column connections. (a) Reinforced bracket, (b) bolt connection.

In the following it will be shown why LRB to 8BC connections are considered to be semi-rigid. The simple and rigid beam-column connections are not modelled in this study.

These are only defined with suitable boundary conditions at the end of beams. On the other hand, an elastic-perfectly plastic moment-rotation behaviour is assumed for nonlinear modelling of the semi-rigid beam-column connections. The semi-rigid connections are modelled with rotational springs at the end of the beams.

The moment capacity (M_R) and the yield rotation (Φ_Y) of the semi-rigid connections have to be calculated. For determining M_R , value of the yield stress, f_y , for the reinforcement of the bracket connections and bolts of the bolted connections are taken to be 400 and 600 MPa, respectively. The yield rotation, Φ_Y , of the connections is calculated from Eq. 1 [22]:

$$\varphi_c = \frac{f_y l_e}{E_s d} + \frac{M_R l_p}{E_c I_b} + \frac{M_R h_{col}}{E_c I_c} \quad (1)$$

The first term on the right hand side of Eq. 1 shows the effect of development of a gap at the connection due to yielding of the top/bottom bars under a negative/positive moment. Therefore the gap is widest at the extreme fibers. In this term, f_y and E_s are the yield strength and the elastic modulus of the longitudinal reinforcements crossing the connection (if applicable) or the connecting steel segments, l_e is the anchorage length of the same bars in column (equal to 26 cm in this study), and d is the effective depth, or the vertical distance, of the steel bars/segments from the farther side of the beam.

The second and third terms on the right side of Eq. 1 are the contributions of beams and columns rotations at the connection due to the development of a plastic hinge at the beam's end and concrete cracking at the column's end. In these terms, E_c is the modulus of elasticity of concrete, I_b and I_c are the moments of inertia of the beam and column sections, respectively, l_p is the length of the plastic hinge at beam's end taken equal to the beam's depth, h_{col} is the depth of the column, and M_R is the moment capacity of the connection.

The bilinear moment-rotation relations of LRB, HRB, 4BC, and 8BC connections are depicted in Figure 4 along with the beam line. The beam line connects the fixed-end moment to the end rotation of a simple beam similar in bay length and loading to the one accommodated at the two ends with the studied connections. As seen in the figure, all of the moment-rotation curves intersect the beam line at a moment smaller than 90% of the fixed-end moment. Therefore, according to the common criterion, these connections are concluded to act as semi-rigid connections.

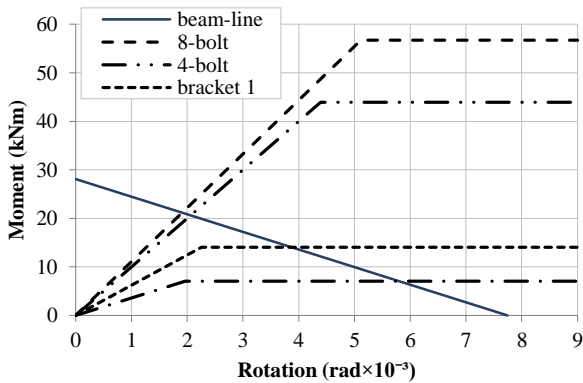


Figure 4: Moment-rotation diagrams for the connections used.

Use can be made of ANSYS and SAP2000 for modelling the wall panels with finite elements and equivalent strips, respectively [23, 24].

As discussed in the introduction, a major achievement of past investigations on SSW's is that the steel panels can be replaced with equivalent strips (rods) for modelling with good accuracy. This is a good chance, because modelling the panels with shell elements multiplies the number of DOF's of the FEM, and makes a nonlinear analysis of the frames containing SSW'S practically impossible.

Despite this fact, just to make it sure, only the modelling of the 3-storey frame is implemented both in Sap and Ansys using equivalent strips and shell elements, respectively. Later on, a very good agreement will be observed between the results of the two models, and the modelling of the taller building is only accomplished with the equivalent strips (rods).

The ductility, overstrength, and response modification factors of the systems are to be determined. Since the paper is only focused on the behaviour of the beam-to-column connection and the steel shear walls, the panel zone area and the base of the columns and shear walls are assumed to be strong enough to prevent any plastic behaviour at those places.

The Finite Element Model (FEM)

With regard to Figure 1, the following components of the 3-storey precast concrete frame with SSW's are modelled in Ansys:

1) Reinforced concrete: SOLID65 element is used to model the concrete medium. This is an element able to simulate concrete segments w/o reinforcement. The volume of element can sustain cracking against tensile forces and crushing under compression. The stress-strain relation of concrete is assumed to follow the Hognestaad equation, as Eq. 2 and Figure 5.

$$f_c = K_s f_c' \left[\frac{2\varepsilon_c}{\varepsilon_0} - \left(\frac{\varepsilon_c}{\varepsilon_0} \right)^2 \right] \quad (2)$$

In Eq. (2), f_c' is the uni-axial compressive strength of concrete, ε_0 is its associated ultimate strain, and ε_c and f_c are the strain and stress of concrete. The K_c factor is equal to 1.0, 0.97, 0.95, 0.93, and 0.92 for f_c' 's of 30, 25, 20, and 15 MPa, respectively. The cross sections of columns and beams are discretized by 4 and 2 SOLID65 elements, respectively, and each SSW panel by a 24×9 mesh of SHELL143 elements with the dimensions 30×20 cm². A sample of the finite element mesh is shown in Figure 7. The out-of-plane displacement of the studied frames is restrained at the storey levels.

2) Reinforcement: For modelling of reinforcements, LINK8 element is utilized. This is a truss-type element bearing only axial forces. Both the longitudinal and transverse bars are modelled with these elements at their exact locations. The stress-strain relation of steel bars is shown in Eqs. 3 - 5 and Figure 6.

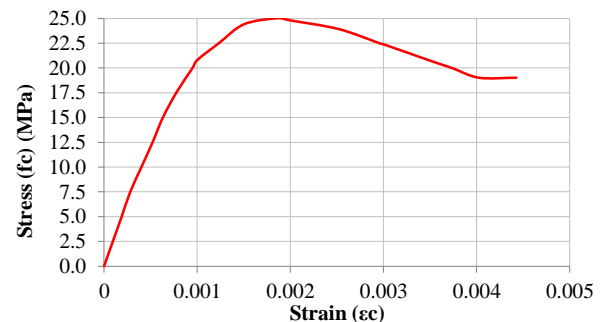


Figure 5: The stress strain relation of concrete.

$$\varepsilon_s < \varepsilon_y \quad f_s = E_s \varepsilon_s \quad (3)$$

$$f_s = f_y + (\varepsilon_s - \varepsilon_y) \left(\frac{f_{sh} - f_y}{\varepsilon_{sh} - \varepsilon_y} \right) \quad \varepsilon_y < \varepsilon_s \leq \varepsilon_{sh} \quad (4)$$

$$\varphi_c = \frac{f_y l_e}{E_s d} + \frac{M_R l_p}{E_c I_b} + \frac{M_R h_{col}}{E_c I_c} \quad (5)$$

In Eqs. 3-5, f_s , f_y , f_{sh} and f_u are the existing stress, the yield stress, the stress at the onset of strain hardening, and the ultimate stress of steel bars, respectively. Also, ε 's with similar indices represent corresponding strains.

3) Steel shear wall: To model an SSW, SHELL143 element is benefited from. This element is suitable for nonlinear modelling of thin to relatively thick shells being plane or curved, buckling in compression and yielding in tension. It is assumed that the steel panels are connected at close points, i.e., almost continuously, to the beams and columns, and the connections are the strongest parts of the system. Therefore the connections of the wall panels are not modelled explicitly, as this is not within the scope of this study. This is an important issue to be embarked on in a separate study.

4) The beam-column connections: The connections of beams to columns, when semi-rigid, are modelled as concentrated elasto-plastic springs as described in previous sections. The rigid and simple connections are not modelled. They are introduced only as boundary conditions.

The finite element model of the 3-storey frame is seen in Figure 7.

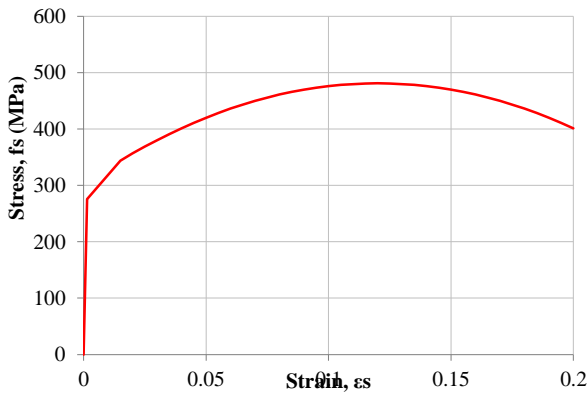


Figure 6: The stress-strain relation of steel bars.

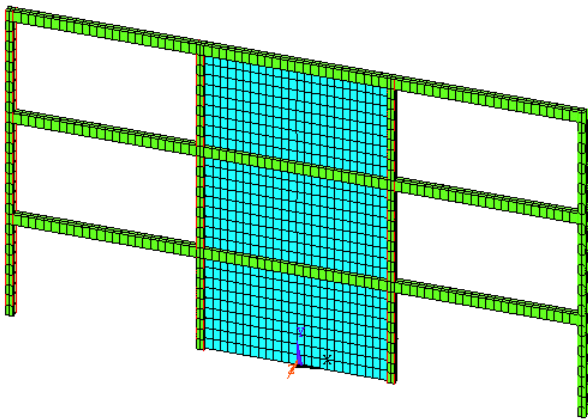


Figure 7: The FEM model of the 3-storey frame.

The Equivalent Strips Model (ESM)

Replacing 2D walls with a number of 1D links clearly decreases the computation time and cost drastically, and such a model can be developed in most conventional software. This is what is done in ESM.

As is recommended by Thorburn et al., number of equivalent strips for each panel is taken to be at least 10 and the angle of each strip with the vertical line, α , is calculated by Eq. 6 [4]:

$$\tan \alpha = 4 \sqrt{\frac{1 + \frac{Lt}{2A_c}}{1 + \frac{ht}{A_b}}} \quad (6)$$

in which t is thickness of the wall, A_c and A_b is the cross sectional areas of column and beam respectively, and L and h are width and height of panel, respectively.

Each strip is replaced by a nonlinear truss element simply connected to beams and columns. The nonlinear behaviour of this element is concentrated at the two ends as inelastic tension hinges. The properties of these multi-linear hinges are taken from ASCE41-06 [25] for different performance levels. The general tensile force-displacement behaviour of these truss elements is shown in Figure 8 after being normalized using suitable scale factors (SFs). Since the strips act as 1D elements, start of their nonlinear behaviour is not gradual. It is right at their tensile yield point. As seen, the very sharp linear branch is followed by a much softened branch with almost no stiffness. After being tensioned to the region of necking, the effective area and thus the capacity of the strip sharply decrease. This is conveniently modelled by a sudden decrease of the tensile capacity. The force-displacement path continues up to the point of rupture.

The beams and columns of the frames in ESM are modelled with nonlinear beam elements. Again, the nonlinear behaviour can only happen at the nonlinear hinges introduced at the ends of the member similar to Figure 8. The necessary parameters for defining the nonlinear behaviour are defaults of the software and are given in ASCE41-06. The beam-column connections are modelled similar to the FEM above. The ESM of the 3-storey frame is shown in Figure 9.

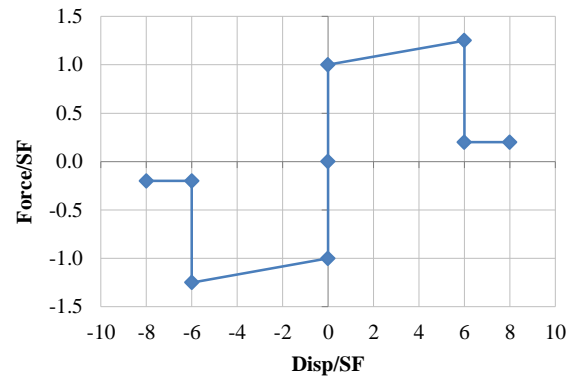


Figure 8: The force-displacement behaviour of truss elements in the ESM.

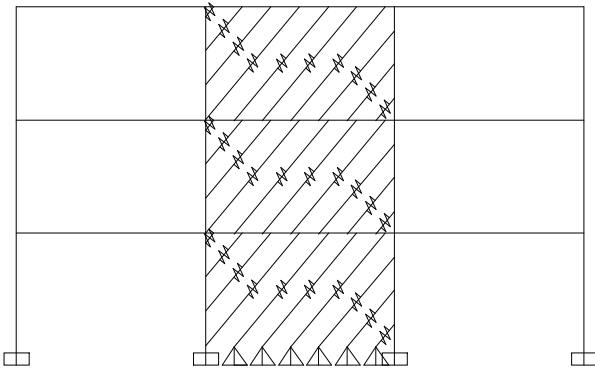


Figure 9: The ESM model of the 3-storey frame.

THE NUMERICAL RESULTS

In this section results of pushover analysis of the 3-storey, 5-storey, and 10-storey frames are presented, respectively. The pushover analysis follows the requirements of ASCE41-06 [25]. The focus is on the nonlinear behaviour up to the onset of failure.

The lateral load pattern is consistent with the fundamental mode shape of each building. The target displacement of the roof is calculated by Eq. 7:

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g \tag{7}$$

In Eq. 7, C_0 is a coefficient converting the spectral displacement of an SDF system to the roof displacement of the studied building, C_1 is used to modify the maximum displacement of an elastic system to its nonlinear counterpart, C_2 is the correction factor for systems with strength/stiffness degradation at plastic deformations, C_3 is used to consider $P-\Delta$ effects in the inelastic range, S_a is the spectral acceleration at the equivalent fundamental period T_e , and g is the acceleration of gravity. T_e is calculated using the slope of the first branch of an equivalent bilinear force-displacement curve of the system under study. To calculate the spectral acceleration, the design spectrum of the Iranian Code for Seismic Design of Buildings for very firm soil and a high risk seismic zone is used. It is shown in Figure 10.

The 3-Storey Systems

The pushover curves of the frames with simple connections and SSW calculated using FEM and ESM models are shown in Figure 11. The pushover curve for a bare frame with the same beams and columns but with rigid connections is also shown for comparison.

As is observed, the curve for the ESM model almost resembles that of the FEM. The initial elastic stiffness estimated by FEM is about 15% larger than that of ESM. This is expected, as in FEM the effect of compression field is also taken into account. Part of the elastic energy saved in the system then is due to shortening of compression strips. Observation of the initiation and development of plastic strain in the FEM of the system shows that the inelastic behaviour begins at the middle of the panels where the largest relative displacement exists between the two ends of a tension strip in ESM. It then extends toward two extreme points of the wall panel where the length of the tension strips tends to zero. At this state, only the frame would continue to contribute to the lateral stiffness of the system, if any. It depends on the type of the frame's beam-column connections. The lateral deformation of the frame when the wall panel practically has no stiffness, as is seen in the above figure, is more than 1% of the frame height. Therefore, it is

expected that even if the frame has rigid joints, after complete plasticization of the SSW, it quickly becomes a mechanism and loses its stability. In the above figure, this fact is evident as the ultimate lateral displacement of the SSW frame and the bare moment frame are almost the same.

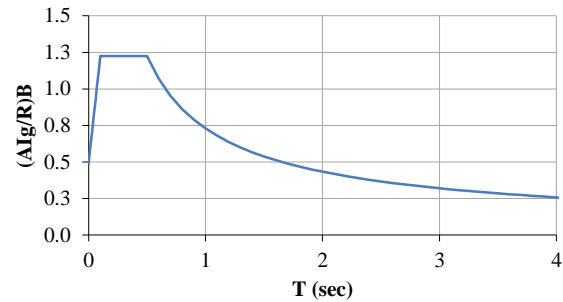


Figure 10: The design acceleration spectrum.

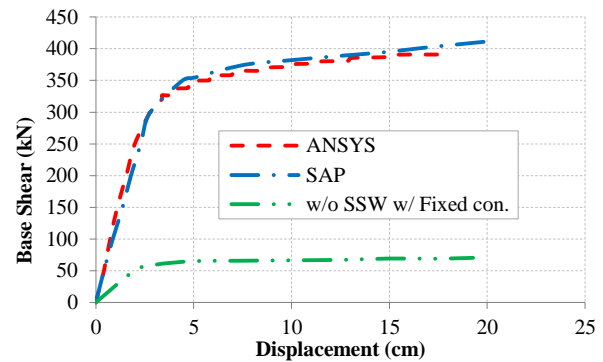


Figure 11: Pushover curves for the 3-storey frame.

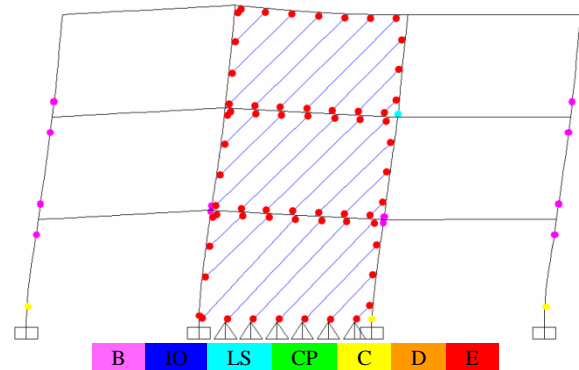


Figure 12: Distribution of plastic hinges in the 3-storey frame just before collapse.

Figure 12 shows the plastic hinges developed in the ESM at the last step of analysis before collapse. Here after yielding of all wall strips, the first storey is about to become a lateral mechanism. As in the analytical model the inelastic behaviour of each strip is concentrated at plastic hinges introduced only at strip ends, these are seen as end dots in the illustration. Colours of the dots in the figures of this paper all correspond to the legend of Figure 12. In the legend, performance levels of each hinge are defined with B for elastic region, IO (Immediate Occupancy) for near yield, LS (Life Safety) for post yield and a medium plastic action, CP (Collapse Prevention) for extensive plastic action, C (Collapse) for local rupture resulting in reduction of strength (see Figure 8), D for the flat region of Figure 8 where the section is progressing toward total failure, and E for a section at total rupture.

Another point to notice in the figure is that the ultimate capacity of the 3-storey frame with SSW is about 5.7 times that of the bare moment-resisting frame.

The 5-Storey Systems

Computationally, the lateral behaviour of these frames with FEM, cannot be followed to large displacements and maintaining the convergence of the analysis is a cumbersome task. Therefore, as was also stated in the previous section, 5- and 10-storey buildings are analysed only with the ESM. This is not a major limitation, as still generally applicable conclusions can be developed.

Distribution of plastic hinges for the 5-storey SSW buildings with different types of beam-column joints, just before collapse, is observed in Figure 13.

Part of the plastic behaviour of the system in more flexible, i.e. bracket, beam-column connections, is due to yielding of such connections. This should be because of small moment resistance of the joints. On the opposite, the bolted and rigid connections have not yielded up to collapse of the frame. Yielding of the SSW and the columns of the first storey is responsible for the collapse in all cases. It is an interesting point that in all cases of connection types, the collapse pattern is almost the same.

The pushover curves for the shear wall system along with those of the bare frame with different beam-column joints are shown in Figure 14. Also, the numerical values of important parameters calculated using the curves are mentioned in Table 3.

Table 3: Numerical values for the pushover curves of Figure 14.

Simple Con	Initial Stiffness (kN/M)	Yield Disp. (m)	Yield Strength (kN)	Ultimate Strength (kN)
LRB	10101	0.069	697	753
HRB	9884	0.069	682	708
4BC	10263	0.072	739	778
8BC	12405	0.069	856	874
rigid	12542	0.070	878	883

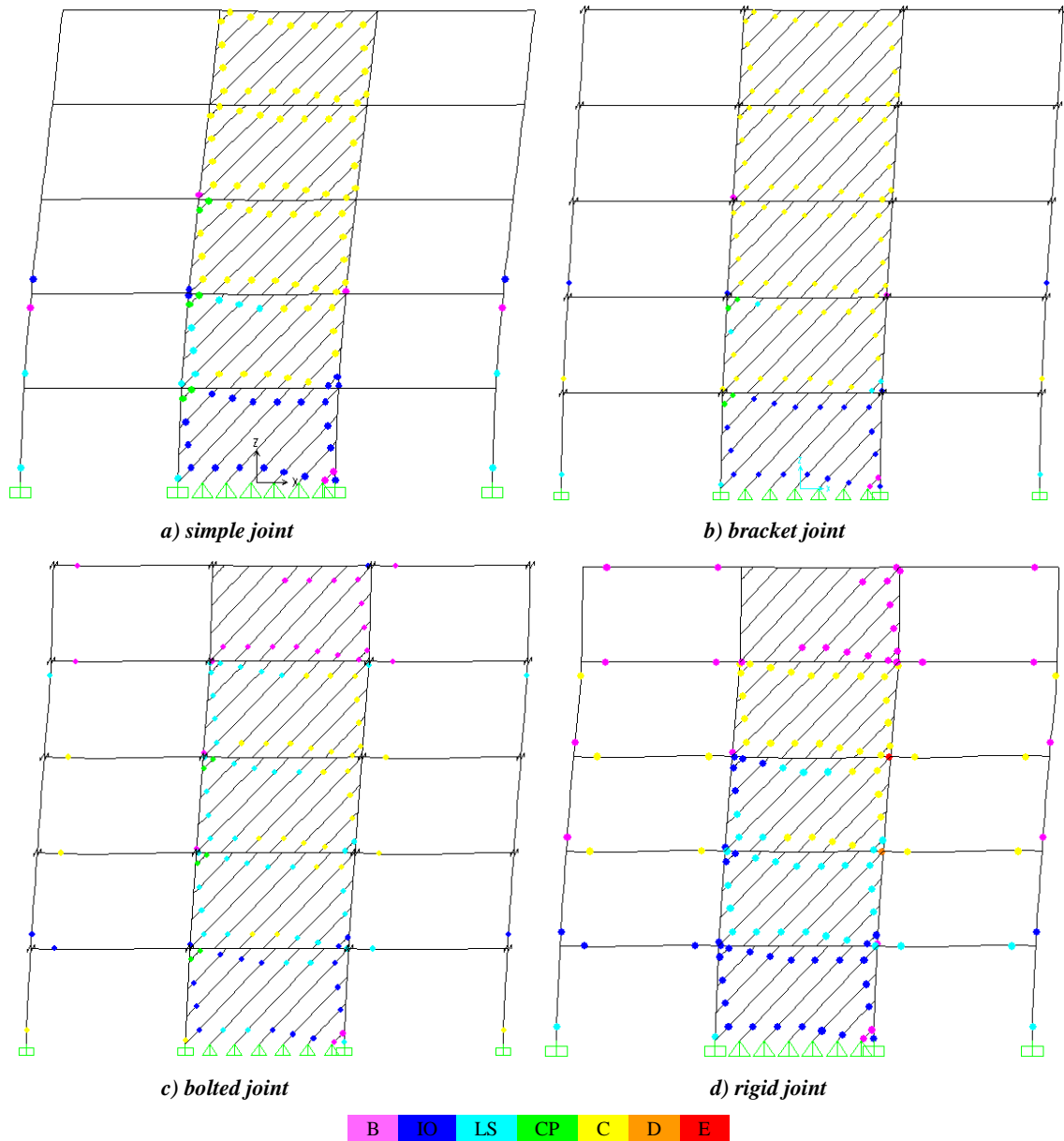


Figure 13: Distribution of plastic hinges in the 5-storey frame just before collapse. a) simple joint b) bracket joint c) bolted joint d) rigid joint.

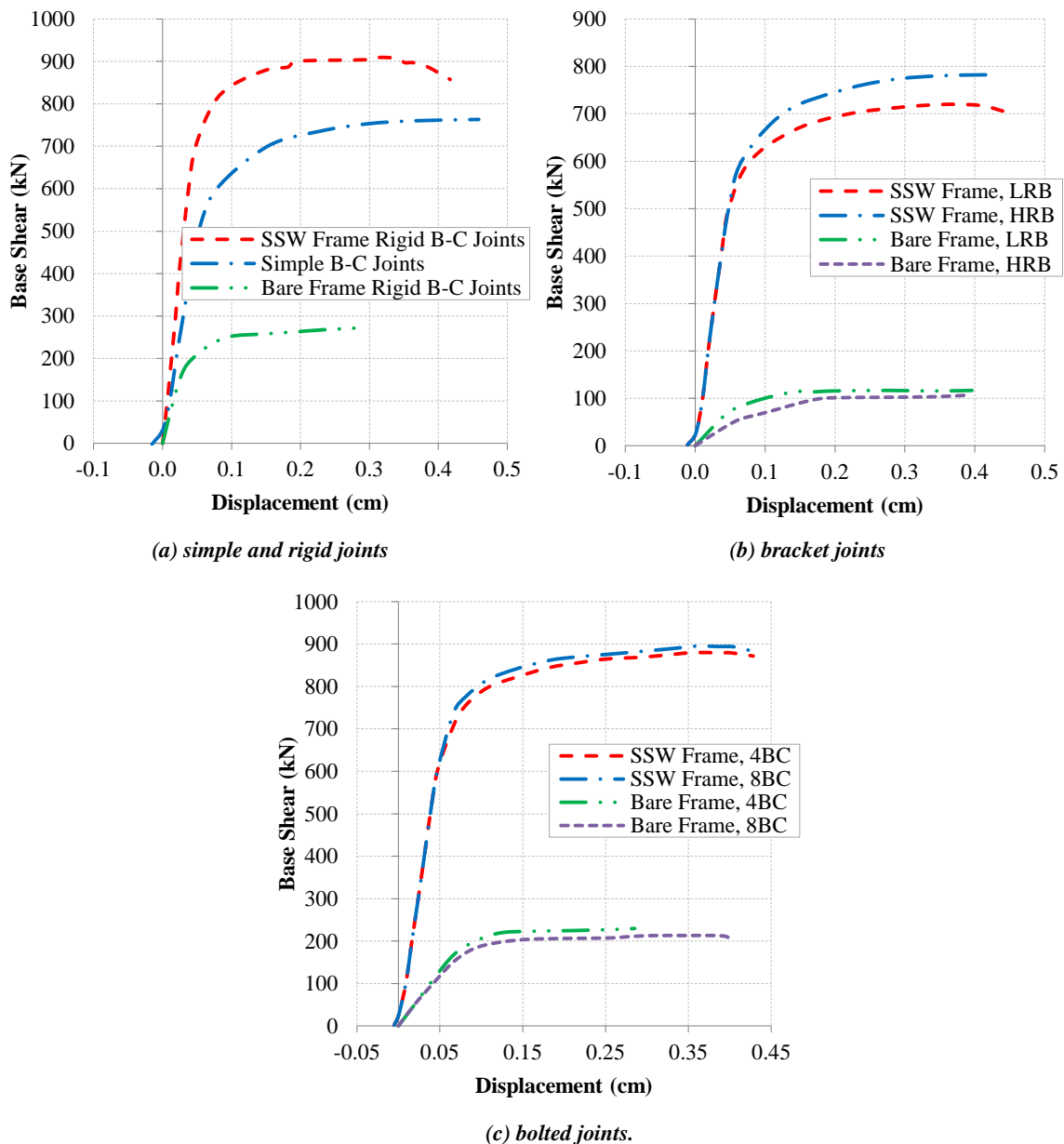


Figure 14: Pushover curves of the 5-storey systems with various beam-column joints.

As is predicted, a uniform trend of increasing of lateral stiffness and capacity is observed from the SSW system with simple beam-column joints to the one with rigid joints. The rate of increase is larger for stiffness. The stiffness of the SSW frames with rigid, bolted and bracket connections are about 54%, 23% and 10% larger than that of the SSW frame with simple joints. For the lateral strength, the variation is not as much considerable, as it changes only about 14% from simple to rigid joints.

On the other hand, the ductility (ratio of the ultimate to yield deformation) of the systems, is observed to be almost the same. Therefore, the type of the beam-column joints in this 5-storey SSW system is only important for small to medium lateral loads where no plastic behaviour has been initiated. For large loads, more or less only the SSW panels determine the behaviour of the frame regarding the lateral capacity and ductility.

In Figure 13, the lateral capacity of the 5-storey frame with SSW and bracket, bolted, and rigid beam-column joints are about 8, 4.4, and 3.2 times the lateral resistance of the bare frames with similar joints, respectively.

The 10-Storey Systems

Distribution of plastic hinges in the 10-storey SSW systems just before collapse is very similar to the 5-storey systems. This is illustrated in Figure 15.

The similarity is in the sense that tension failure of equivalent strips in many panels, especially in lower stories, results in rapid failure of columns in the same stories and total collapse of the system. In this sense, the SSW's in the precast concrete system behave just like shear walls in monolithic systems. Moreover, before collapse, much of the beam-column connections both of bracket or bolted types, have yielded but not collapsed. Two points can be embarked on here. First, from 3-storey to 10-storey frames, much increase is seen in the number of yielded beam-column joints. This is consistent with past data on shear wall moment frames, whether RC or steel, and shows the more contribution of the frame to the lateral behaviour as the frame's height increases. Second, the above conformance with reality verifies the use of equivalent strips for analysing the SSW frames.

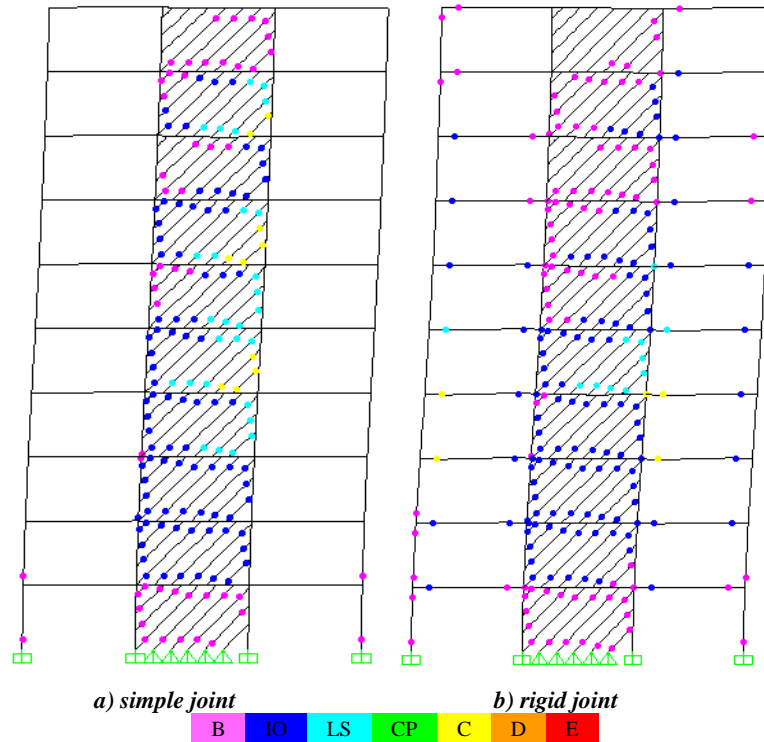


Figure 15: Distribution of plastic hinges in the 10-storey SSW systems just before collapse.

Pushover curves of the 10-storey systems for different cases are shown in Figure 16. Moreover, the numerical values important for interpretation of the results are summarized in Table 4.

Table 4: Key numerical values of pushover curves of 10-storey SSW systems.

Simple Con	Initial Stiffness (kN/M)	Yield Disp. (m)	Yield Strength (kN)	Ultimate Strength (kN)
LRB	7794	0.141	1099	1183
HRB	6303	0.155	977	1046
4BC	6098	0.152	927	1028
8BC	7006	0.145	1016	1112
rigid	5889	0.154	907	1009

The lateral stiffness of SSW systems with rigid, bolted, and bracket joints are about 27%, 12%, and 11% in excess of that of the SSW system with simple joints. At the same time while the lateral strength changes only about 20% between different systems, the ductility does not change practically. All of the mentioned three parameters, like the 5-storey building, again show that the behaviour of the system under large loads is mainly controlled by its SSW, while the increasing effect of the rigid joints on lateral stiffness is important under smaller lateral forces.

It is also observed in Figure 16 that the lateral resistances of a 10-storey SSW frame with bracket, bolted, and rigid beam-column joints are about 5, 8, and 17 times those of the bare frames with similar joints, respectively.

CALCULATION OF SEISMIC DESIGN FACTORS

One of the key parameters for seismic design of a structural system is its response reduction factor, R for ultimate state design, and, R_w for allowable stress design.

The R and R_w factors are determined by Eqs. 8 and 9, respectively:

$$R = R_\mu \Omega, R_w = RY \quad (8)$$

$$R_\mu = \frac{F_e}{F_y}, \Omega = \frac{F_y}{F_s}, Y = \frac{F_s}{F_w} \quad (9)$$

in which, R_μ is the response modification factor due to ductility, Ω is the overstrength factor, and Y is the analysis method factor. Also, F_e , F_y , F_s and F_w are the elastic demand, the equivalent yield strength, the first yield strength, and the allowable strength of structure, respectively. These parameters are introduced in Figure 17.

All of the quantities above, except F_e and Y , can be extracted from the pushover curves calculated. F_e is computed from the response spectrum assuming a unit R factor, then $F_e = S_a W$ in which S_a is the spectral acceleration and W is the seismic weight. Value of Y is 1.4-1.5 and is taken as 1.4 in this research.

Based on the above description, the values of R_μ , Ω and R are calculated and summarized in Table 5 for various SSW systems studied. Additionally, the ductility factor μ , computed from pushover curves as the ratio of the ultimate to the yield displacement ($\mu = D_{max}/D_y$, see Figure 17), is shown in Table 5.

The above factors can be used to compare the system introduced in this paper with other seismic resistant systems. Obviously they cannot be taken as design factors based only on the limited number of calculations as presented in this paper.

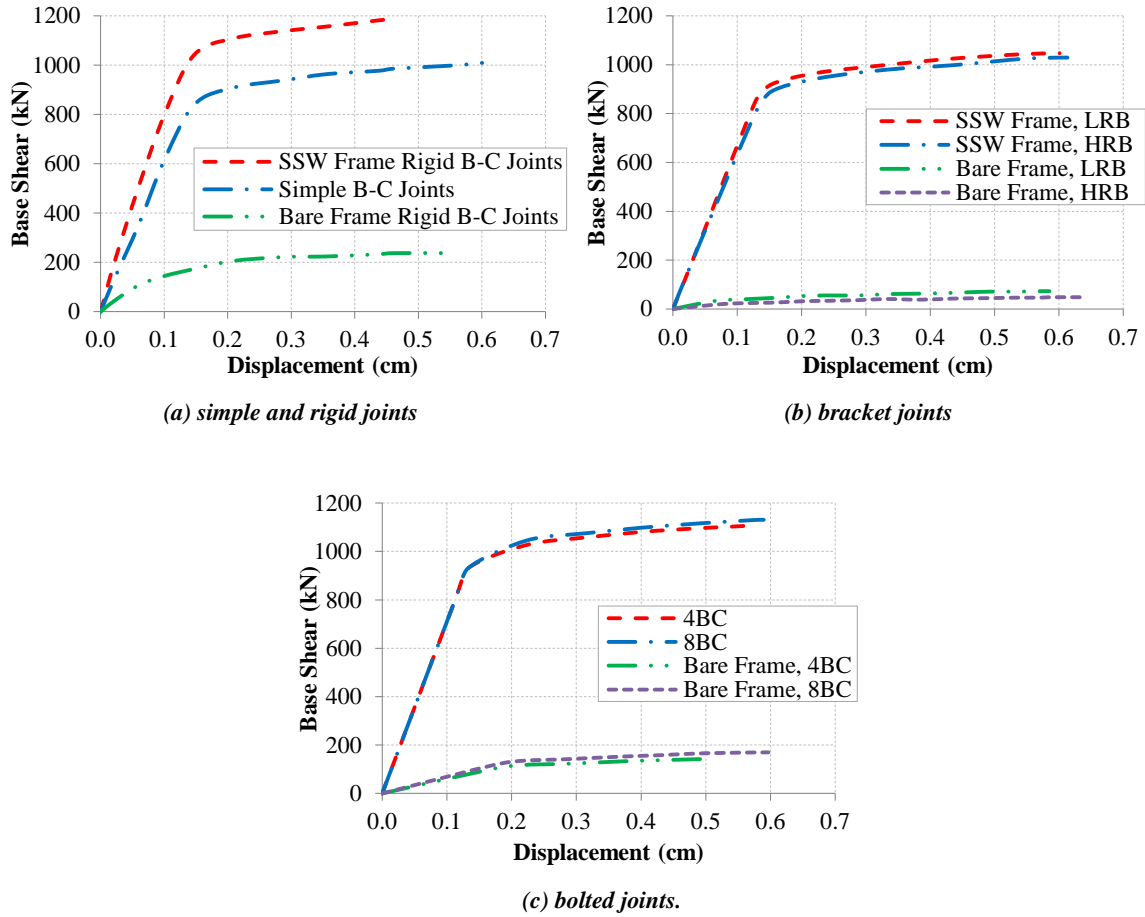


Figure 16: Pushover curves of 10-storey systems with various beam-column joints.

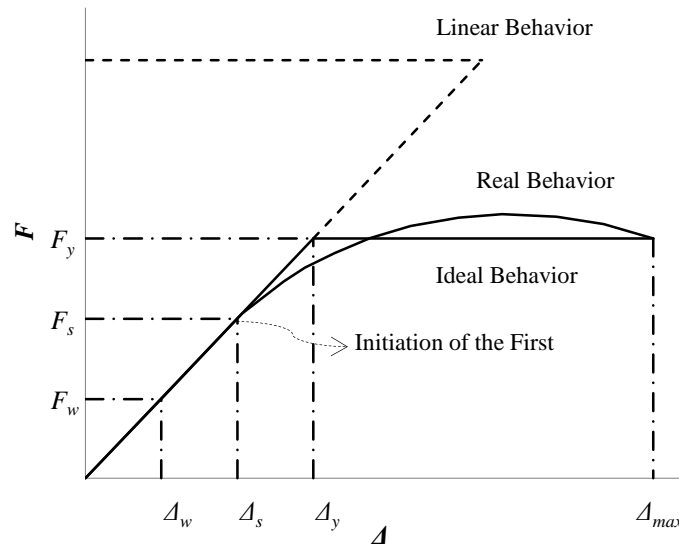


Figure 17: Seismic design factors.

Variation of ductility and overstrength factors for 5- and 10-storey SSW systems with different beam-column joints are shown in Figures 18 and 19.

Assessing Table 5 and Figures 18 & 19, it is seen that the ductility factor is not sensitive to the joint type in these systems, as described also in the previous section. It is resulted that at least up to 10 stories, the major part of energy dissipation in an SSW precast frame is provided by SSW's not

by member or joint plastic behaviour. It is noteworthy that as the stiffness of the connections increases, the ductility of the system decreases. Both of the above facts lead to the important result that in low- to mid-rise SSW precast frames there is no need to have costly rigid beam-column joints and use of simple connections will suffice from the view point of energy dissipation capacity. At the same time, rigid joint increases the lateral stiffness of the system considerably, as seen in the numerical results section. Therefore, if needs be, the

appropriate stiffness must be accommodated with increasing the panel thickness, when using simple joints.

For the overstrength factor also a similar conclusion is valid due to its small variations with the joint type.

Finally, looking into Table 5 for the R factors, again it is observed that fluctuation of the values of this factor about the mean value is relatively small. The average of R factors for bracket joints is larger than that of the bolted connections due to smaller system lateral capacity. Regarding the response modification factor of these systems, it can be concluded that a value of $R=4.5$ (or $R_w=6$) is enough conservative for all non-rigid joints including the recommended simple connections.

Table 5: Seismic design factors for various SSW precast concrete structures in this study.

Model	R_μ	Ω	R_w	μ	
3 Stories	3.48	1.37	6.72	2.71	
5 Stories	simple con	3.43	1.40	6.70	2.68
	LRB	3.44	1.24	5.98	2.53
	HRB	3.26	1.60	7.50	2.43
	4BC	3.33	1.30	6.14	2.42
	8BC	3.31	1.35	6.26	2.28
	rigid	3.47	1.43	6.50	2.41
Mean	3.37	1.38	6.51	2.45	
10 Stories	simple con	3.90	1.13	6.17	2.94
	LRB	4.58	1.20	7.60	2.34
	HRB	3.93	1.30	7.17	2.44
	4BC	3.90	1.10	6.15	2.41
	8BC	4.00	1.10	6.32	2.41
	rigid	3.22	1.15	5.23	2.26
Mean	3.92	1.16	6.44	2.46	

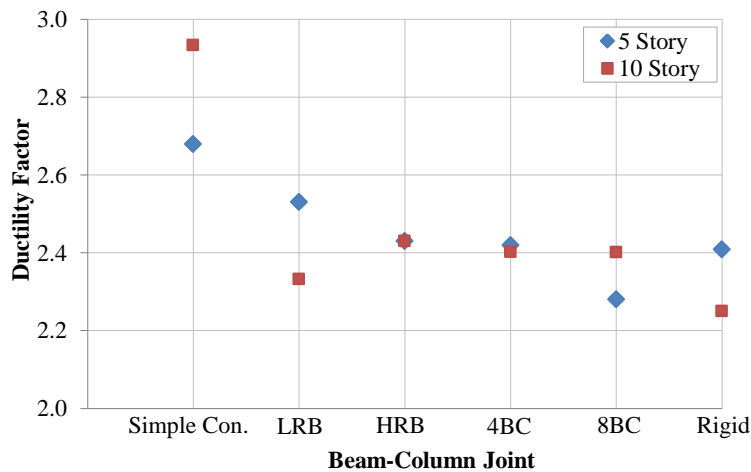


Figure 18: Variation of the ductility factor in SSW precast concrete frames with different beam-column joints.

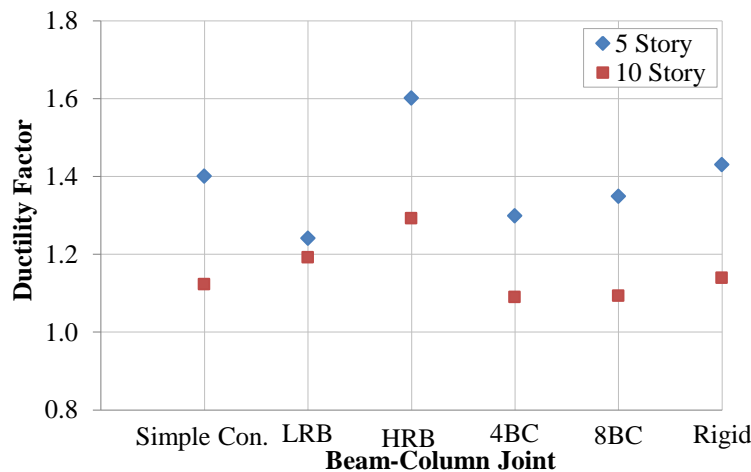


Figure 19: Variation of the overstrength factor in SSW precast concrete frames with different beam-column joints.

SUMMARY AND CONCLUSIONS

In this paper a new structural system consisting of steel shear walls and precast concrete frames was introduced. Based on the nonlinear static analysis of the system with different beam-column joints, the following conclusions can be derived:

- 1) For practical design of the introduced system, an equivalent strip model (ESM) is sufficient.
- 2) SSW's can largely increase the lateral stiffness and to same extent the lateral capacity of precast concrete frames based on the type of the beam-column joints used.
- 3) The paramount effect of the beam-column joints is on the elastic stiffness of the system, where this value can be increased up to 50% in shorter buildings by using stiffer joints. At the same time, the lateral capacity of the system with rigid joints is only about 20% larger than that with simple joints.
- 4) The joint type is almost ineffective regarding the ductility and overstrength factors of the systems studied. In other words, the share of the wall panels in the system's plastic behaviour and energy dissipation is overwhelming.
- 5) Despite the considerable effect of the joint type on the elastic stiffness of the system, it has a negligible role in providing for the system ductility and overstrength. Therefore, use of much more economical simple beam-column joints is recommended for SSW frames.
- 6) Based on the pushover analyses performed, a value of $R=4.5$ (or $R_w=6$) is recommended for the seismic design of SSW precast concrete frames, up to 10-storey buildings.

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