Reliability-based evaluation of seismic design and numerical studies of Self-centering moment resisting steel frames

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ABSTRACT

Recently developed self – centering moment resisting steel frames have been analytically and experimentally validated as having the potential to eliminate structural damage under a design basis earthquake and return to their original vertical position following a major earthquake. The objective of this research is to develop the understanding of behavior and performance of steel Self Centering – Moment Resisting Frame (SC – MRF) systems. This study examines the response numerically simulated of a 3, 8, and 19 story SC-MRF subject to dynamic time – domain analysis, using a database of different terms, as for instance the recorded earthquake of El Centro 1940, one of the most powerful recorded in history. Peak structural demand responses such as story drift and beam-column relative rotation has been evaluated. This data is used to examine the sensitivity of the SC – MRF behavior to structural properties and geometry. The results could be used to develop a reliability-based seismic design procedure for these SC-MRF connection details.

Keywords: Steel frame, Self-centering moment resisting frame, frame ductility, seismic design

Introduction

In both U.S. and European codes, kept that some necessity on strength and ductility are convinced, yielding is admitted to appear either in beam, panel zone or connections. The development of plastic hinges in columns is forbidden, made exclusion for base plates, column ends at the top of multistory frames, and in case of single storey MRFs. In the logic of the global ductility approach, both AISC 2005 and Eurocode 8 contribute some necessities concerning inelastic capacities of the dissipative parts. It should happen, in case of steel frame in high class of ductility (DCH/SMF), that the structural system is drew to ductile behaviour if under strong earthquake. The difference between "rigid" node and ductile node is shown in the following fig.1 and fig.2.

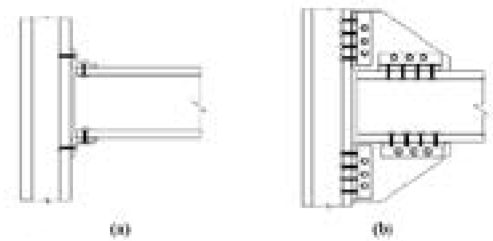


Fig.1 Typical Early Beam-Column Connections: (a) pinned connection and

(b) rigid connection (FEMA 351, 2000).

Among the several connection suggested by experiencing strong earthquakes, one that has been accepted as considered trustworthy and practical (economical) is the Reduce Beams Section (RBS) – or "dogbone" detail (Englehardt et al. 1998). In the RBS connection the section of the beam decreases at a span from the beam-column connection so that yielding is fixed in that diminished area at moments kind of lower than those that activate the full inelastic requirement on the connection.

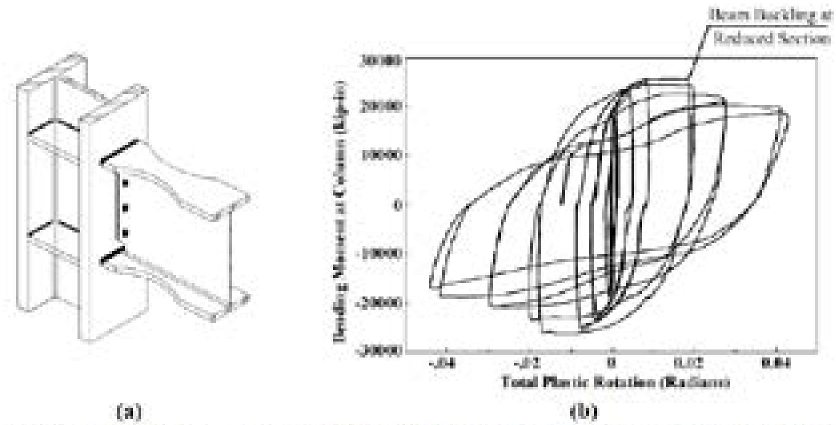


Fig. 2. Reduced beam section connection (RBS): (a) connection arrangement and

(b) moment-rotation relationship (FEMA 355D, 2000).

After that experience the research moved toward and other promising concept. This is the approach of the Self-centering Moment Resisting Frames (SCMRF). Post Tensioned Devices (PTD) and Self-Centering Moment-Resisting Frames (MRFs) have recently been developed as a viable alternative to welded moment frames in high seismicity areas. This connections are designed to prevent brittle fractures in the area of the nodes of the frames, which can cause severe reduction in their ductility. High-strength post- tensioned (PT) steel bars, which clamp beams to the columns, are adopted, and then main energy dissipation is due to the yielding or friction-based energy-dissipation devices (EDs). The pre-stressing elements are designed to remain elastic even when the system experience large lateral deformations. Also, such a devices have the property to assures a fully re-centered structure after strong earthquake (Christpoulous et. al. 2002).

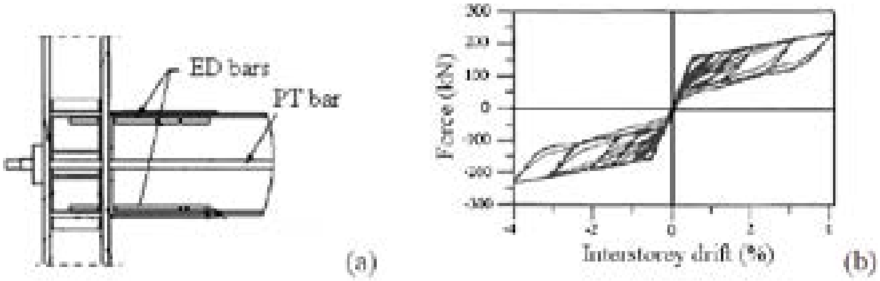


Fig. 3 - PTED connection with PT and ED bars: (a) typical node; (b) experimental force-displacement curve (Christopoulos et al., 2002a, b).

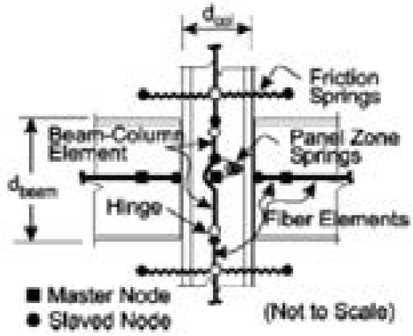


Fig.4 - Numerical modeling of the typical PTED connection with PT and ED element (Rojas et al., 2005)

The object being taken into consideration in the paper for the study of the global effects of self-centering methods to moment resisting frames is conceived as a multi-bay portal frames for three case studies, for buildings whose fundamental period of vibration fall into constant accelerations, constant speeds and constant displacements respectively: 3 stories above ground, 8 stories above ground and 19 floors ground the ground. The frames and their elements are distributed in a regular manner, both in height and in plan. The mass and stiffness distributions do not vary in height, therefore the structural elements are the same on all floors, both vertical and horizontal elements. For simplicity, it is assumed that the distance between the axis of the columns is 600 cm. Mathematical models are built for the Moment Resisting Frames (MRF) with Rigid Joints and Self-Centering Moment Resisting Frames (SCMRF). Pinned-based structures with hinged joints, which are not taken into account in this study, may cause large displacements, residual deformations and hinge displays on the floor that could lead to the loss of durability of the entire structure by turning the structure into a mechanism. Pinned-based structures with rigid joints on the other hand (MRF) during the earthquakes may lead to moderate displacements, develop residual deformations but provide structural sustainability. But the deformations that this system exerts can have very high cost of reinforcement or retrofit. On the other hand, the other structural type studied is SCMRF, which is characterized by very large displacement, no residual deformation, structural stability and above all, no major repair costs. The models were subjected to dynamic time-domain analysis, using a database of different terms, where the term references for the design of the results in this study is the El Centro 1940's earthquake (http://www.vibrationdata.com/) elcentro.htm), one of the most powerful registered earthquakes in history.

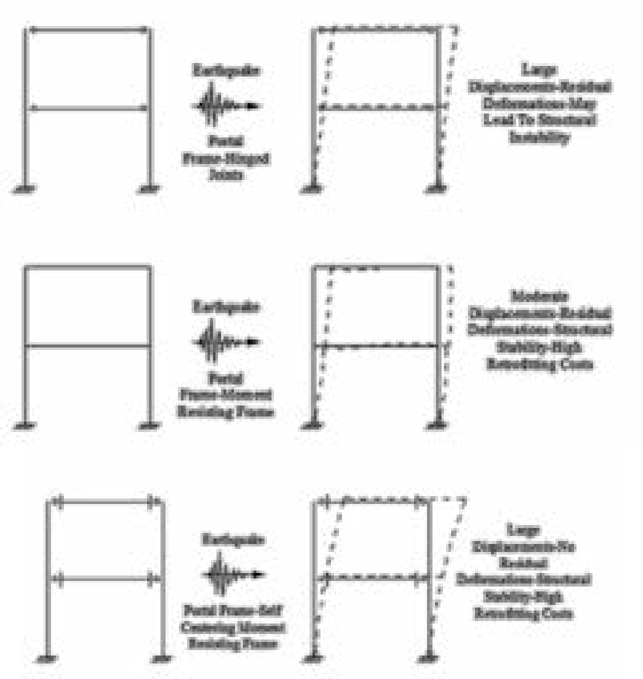


Fig. 5 - Basic behavior of the three models taken into consideration.

The preliminary dimensioning of the structural elements, and their final solution is made with regard to the types of structural steel constructions and the codes that govern them. The object for simplicity is represented as an equivalent two-dimensional frame portal. The conceptualization of the elements of the structure is made based upon the capacitive design of the structural elements. The foundation of the object is assumed as infinitely rigidly while the link of the column element with the foundation is accepted as fully fixed with all of the six rotational degrees restrained.

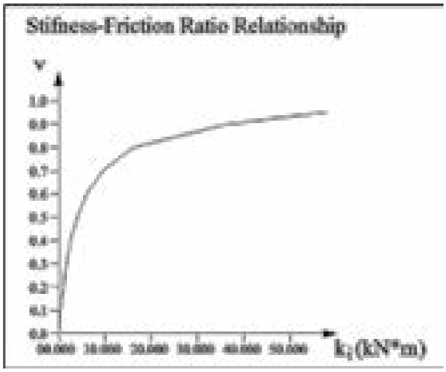


Fig. 6 - The chart of rigidity change of the beam-column node.

Model description

Frame elements and slabs

The object columns are selected with HEA 300 section. The column section does not change in height. Column sizes have been chosen in accordance with design requirements for seismic zones according to European recommendations for their normal ductility, according to Eurocode 3 and Eurocode 8.

The beams of the structure have been preselected as IPE 300 section. Decks are conceived as rigid diaphragms in the horizontal plane with a thickness of 20 cm. In the mathematical model, the weight of the slabs is centered on the joints of the structure.

Material used is the Fe360 steel grade for the entire structure.

SAP2000 Nonlinear calculator was used for calculating the object. The preliminary calculation was done based on Eurocode EC1, EC2, EC5, EC8. The calculation of the objects in seismicity was done according to EC8 per D-type of soil category, PGA =0.27g, for buildings of normal class of significance.

Calculation by multi-modal analysis of seismic forces at the floor levels, according to EC8 is:

$$E_{ki}=m_k*S_{a,i}*\eta_{ki}$$

where $S_{a,i}$ is the response design spectral value.

The calculation of elements according to the method of the limit states includes the ultimate limit state and the serviceability limit state. According to the method of the limit states for the loads, the meaning representative value is introduced, whereas for the resistance the meaning of the characteristic value is introduced. The above values are added to the partial security coefficients for the purpose of obtaining the calculation values. Partial security coefficients cover inadequate load shifts and material resistance. The values of partial security coefficients are also obtained through probabilistic methods.

For the calculation of reinforced concrete structures according to the limit state method we distinguish two types of loads: nominal loads derived by different tables in design codes and design loads obtained by multiplying the loads with safety coefficients. Loads are divided into permanent, temporary and special loads. In the absence of statistical data, the representative values for permanent action can be replaced by nominal values from the norms. Representative values for temporary loads are obtained from the characteristic values of the load multiplied by the coefficient of combinations ψ. Characteristic values in the absence of statistical data are replaced by nominal values, which are derived from experience, evaluation and forecast for future development.

In this presented analysis “nominal loads” include:

a. Permanent loads (self-weight of the object, including frame skeleton, finishing layers and permanent partition walls.)

b. Live loads (long-acting live loads, short-acting live loads: mobile lifting-carrying equipment, weight of people in different furniture in residential or social buildings, wind, snow)

Mathematical modeling with Finite Elements

Mathematical modeling with finite elements was carried out using the structural code SAP2000 Nonlinear version. The three-dimensional model was accepted to be fully restrained at the base without taking into account the soil reaction. The decks were conceived as horizontal rigid diaphragms. Columns and beams were modeled as elements of the frame type, while the decks as lumped masses in storey level nodes. The seismic measures were applied in the form of horizontal pseudo static loads at the levels of floors. The elasticity modulus of steel was assumed E = 200000 MPa while the unit volume weight γ = 78kN/m3. The seismic analysis was carried out with the multimodal response spectrum method for each of the directions with its own specifics. The damping for all modes was accepted equal to 5%. Since individual periods of vibration resulted very close to each other, modal superposition was made according to the Complete Quadratic Combination method. In the analysis according to EC8 it was accepted that the type of soil is D and the reference acceleration of ag,R=0.27g. This analysis for the conditions of EC8 was performed for high class of ductility.

Three storey model numerical analysis results

The three storey model consists of a portal frame. In the nodes of the portal are concentrated the lumped seismic masses. In each level is applied a tendon which represent the strands effect and the linear behavior during earthquake. Also, in the beam-column interface the damping properties of the plates and the nonlinear rotational spring properties are localized. All of the three components are developed in the frame of the gap opening of the semi-rigid connection which has been assumed to represent the ideal joint of SCMRFs beam-column connection. For initial fixity factor ν=0.99 (full semi-rigid connection) the correspondent ki value is ki=405197 kN/m.

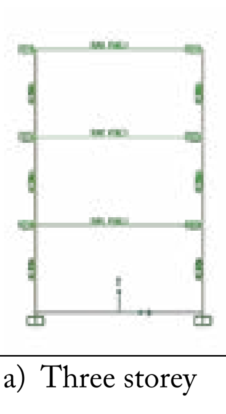
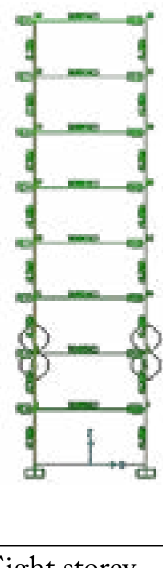
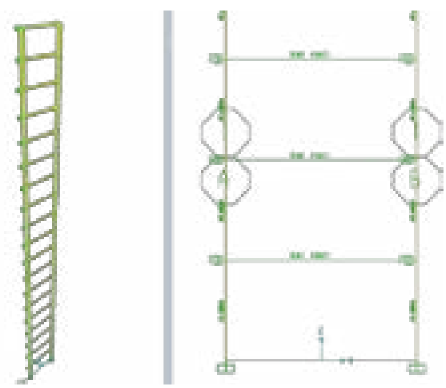
		
a) Three storey plan model	b) Eight storey plan model	c) Nineteen storey plan model

Fig. 7 – Three different storey frame, with damper, non linear springs and tendons

Input parameters for the time domain dynamic analysis are taken those of El Centro (1940) earthquake (Fig.6), with proportional damping , time integration as for Hiber-Hughes-Taylor method and direct integration of motion equations.

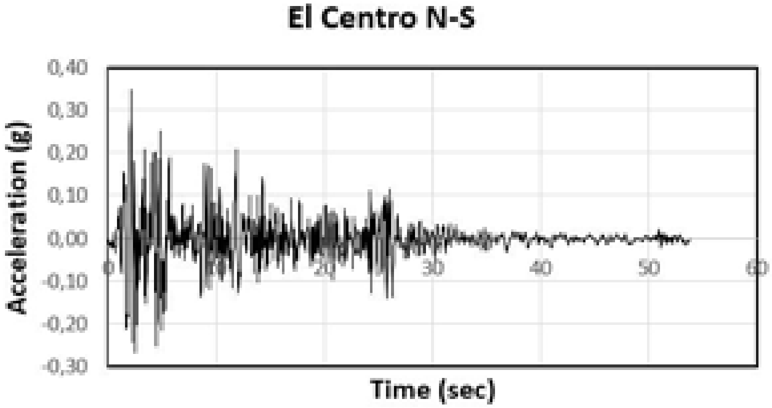
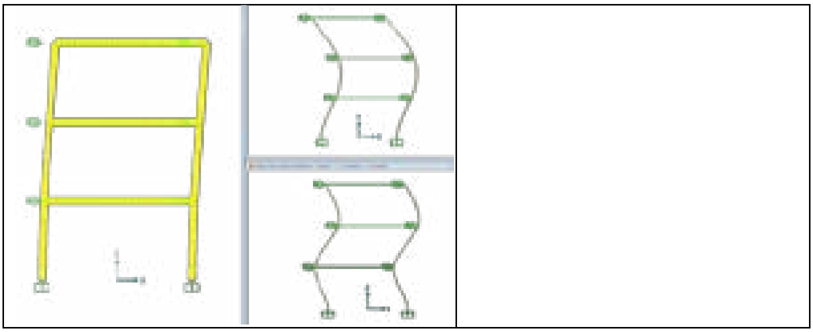


Fig. 8 - El Centro (1940) earthquake accelerogram, North-South component.



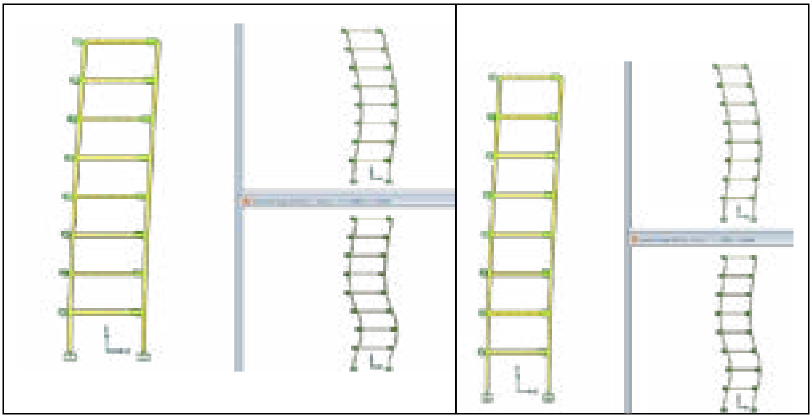


Fig. 9a, 9b, 9c: Three fundamental periods of vibration of the three, eight and nineteen storey models.

Modal analysis conducted for the three storey SCMRF model shows that the fundamental period of T1=0.6s (Fig.9a) is approximately twice the empirical approach of conventional MRFs. Also the second and third period of vibration are 0.55 s and 0.42 s respectively, almost twice the respective periods of conventional MRFs. Compute of the frequency of vibrations vs. pseudo-accelerations shows that the pseudo-accelerations obviously increase much more then proportionally, by increasing the damping of the connection. The difference comes by the changing of the stiffness of SCMRFs semi-rigid interface between columns and beams.

The comparison between the displacements of SCRF and conventional MRF, clearly shows that the SCMRF system provides larger displacements. The comparative graphic (Fig. 10) shows that SCMRF exhibits less cycles than conventional MRF, while the amplitude of displacement at the top of the building, which at the beginning of time history tend to have a great difference, at the second half of the time history tend to be equal in both cases. This result of course reflects the fact that for weak motions no gap opening will occur due to tensile force of the strands and the stiffness of the systems is almost equal in both cases.

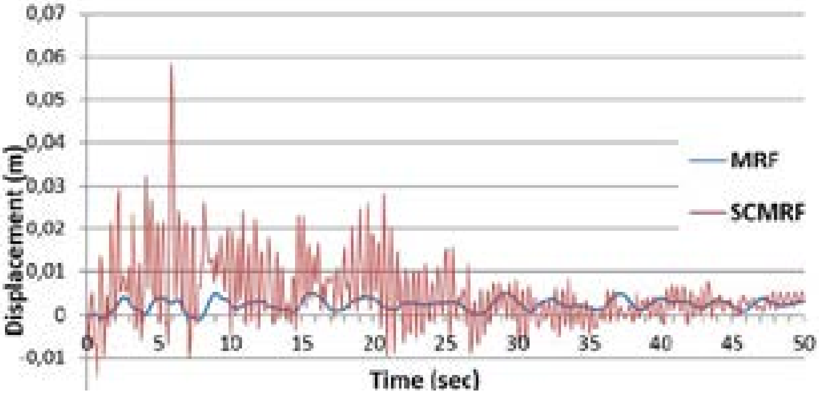


Fig. 10 Comparison of spectral displacements of MRF and SCMRF for the three storey model.

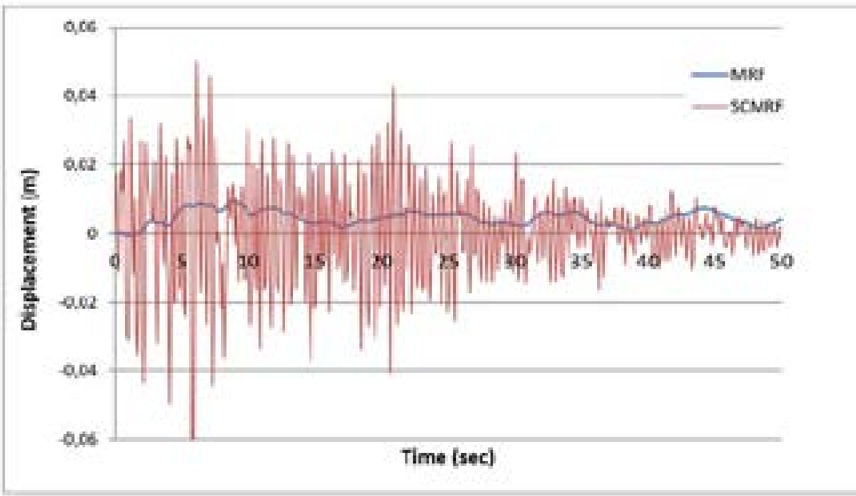


Fig. 11 - Comparison of spectral displacements of MRF and SCMRF for the eight storey model.

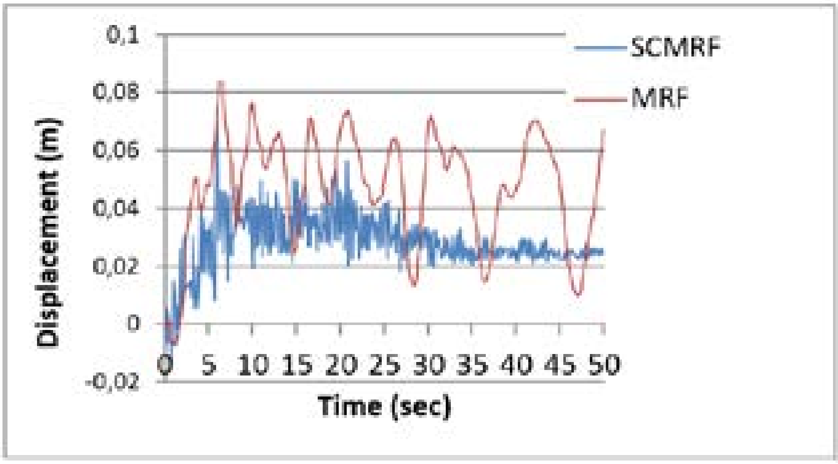


Fig. 12 - Comparison of spectral displacements of MRF and SCMRF for the nineteen storey model.

Nonlinear "pushover" analysis for the conventional MRFs and SCMRFs show a significant increase in the overall capacity of post-tensioned system. The initial stiffnesses in the linear phase of the pushover curves are comparable, especially for systems with high values of post-tensioning. In the nonlinear phase the results show that in SCMRF this phase begins for greater lateral forces, while the continuing curve shape will depend on the nonlinear capacity of plates, as the post tensioned strands must remain in the elastic phase. even the collapse point of the systems are clearly visible in all the models it must be noted that for well designed SCMRF (adequate post tensioned force)systems theoretically it cannot be reached, as the capacity of the system guarantees that the flag shape behavior has no residual displacements.

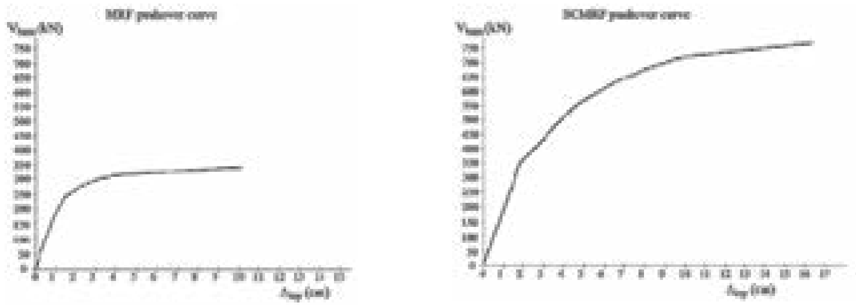


Fig. 13 - Pushover curves for MRF and SCMRF, three storey model.

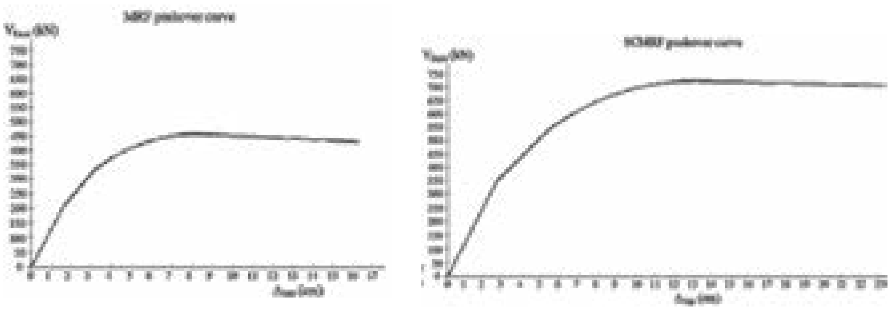


Fig. 14 - Pushover curves for MRF and SCMRF, eight storey model.

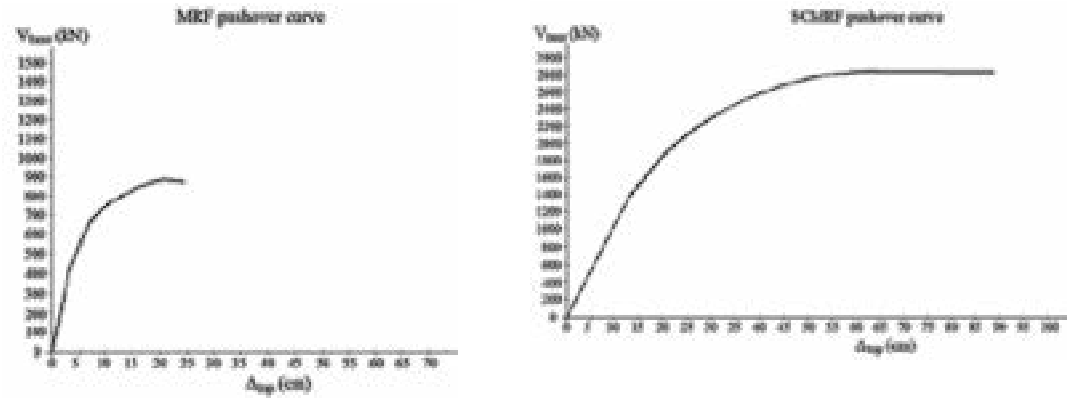


Fig. 15 - Pushover curves for MRF and SCMRF, nineteen storey model.

Summary and Conclusions

Self-Centering Moment Resistant Frames SCMRF's is a big improvement in structural steel frame design. Earthquake action is resisted by the ductility of the frame which is represented by the ability to perform without excessive deformations. This ductility is due to the actual movement of frame and not by the permanent or elastic deformation of the steel material. On the other hand, the sway of the SCMRF frame is definitely much higher than that of a conventional frame MRF. Of course the great advantage of SCMRF is its capacity to preserve its structural integrity after big earthquakes. After conducting the nonlinear dynamic analysis for three models of 3, 8 and 19 stories, the conclusions are as described below:

1. Deformations during earthquakes are greatly increased in all models. The difference is greater as the number of stories rise, as the structural systems due to their dynamic properties tend toward the constant displacements zone in the response spectra. These deformations, even the structural system guarantees they provide safe behavior, are beyond the limits set for conventional MRF's in actual codes throughout Europe.

2. The natural periods of vibrations are significantly higher in SCMRF's, rather than in conventional MRF's. This is caused by the lowering of the rigidity of the nodes and as a result the increase of the flexibility of the system. As a result, SCMRF may maybe affected by disturbances during building occupation due to impulsive vibrant forces which can trigger first mode vibrations. The study over the possible effects that the increase of natural periods may cause to SCMRF's in short and long time loading has not been part of this work and further studies on this topic may be made in the future.

3. The pushover curves generated for all of the models, show an increase in structural capacity. The presence of the tensile forces applied by the tendons during the opening of the gap earthquakes, brings into the equilibrium the structural system with very low deformations which occur in the thin plates. Because of this behavior larger forces are needed to bring the system to the unstable equilibrium condition (failure). Performance points in SCMRF where considerably higher than in conventional MRF's.

From the energetic point of view, the total amount of energy dissipation was higher in SCMRF's tan in MRF's.

4. Residual deformations of SCMRF's are insignificant compared to MRF's. In MRF's the behavior under cyclic loading in earthquake simulations has the classical form of hysteresis type, which is summed in the degradation "backbone" curve. On the other hand, in SCMRF's the pattern of the cyclic behavior is flag shape with no residual deformations. Actually, the models provide a moderate accuracy in the calculation of pushover increments and plot data, while the hysteretic and flag shape curves of respectively the material and joint behavior are not plot exactly as theoretical and experimental results.

5. The amount of energy damped in SCMRF's nodes is composed by the energy spent in the elastic deformation of the post-tensioned strands (tendons) and by the energy spent for the inelastic deformation of the thin plates. The actual work with the mathematical models with SAP 2000 Nonlinear has not evaluated the contribution of each of the components by numerical values but it was used for capturing the general behavior of the structure. Further investigations could also be performed with other powerful software.

The frame of this work has been the computer modeling, simulation of mathematical finite elements models and interpretations of results for the basic SCMRF model with nodes composed by post tensioned strands, nonlinear helicoidally springs and hook damping properties. Future developments in this area with the aid of specific tools, laboratories and new technologies are the basis for further mini scale and full scale experiments. Experimental data and theoretical results achieved in this work must be compared and must be calibrated in order to provide a more accurate model for the study of the behavior of Self Centering Moment Resisting Frame's, and may be after that for the implementation of the design of this structural system to civil engineering codes.

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