COMPARISON OF SEISMIC LOSSES IN STEEL BUILIDNGS USING CONVENTIONAL OR SELF-CENTERING MOMENT RESISTING FRAMES

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SUMMARY

Steel self-centering moment-resisting frames (SC-MRFs) are a class of resilient structural systems that avoid damage in beams and eliminate residual drifts under the design basis earthquake. In this paper, a building is designed using SC-MRFs or conventional steel moment-resisting frames (MRFs) and the monetary losses of both cases are compared with the aid of the FEMA-P58 methodology. The latter is a performance-based earthquake engineering methodology based on explicit determination of performance (e.g. monetary losses) in a probabilistic manner, where uncertainties in earthquake ground motion, structural response and losses are considered. The results show that SC-MRFs have significantly improved performance compared to conventional MRFs and result in lower seismic losses. The results also highlight the importance of considering residual drifts as a demand parameter controlling whether a building is repairable or needs to be demolished in the aftermath of a strong earthquake.

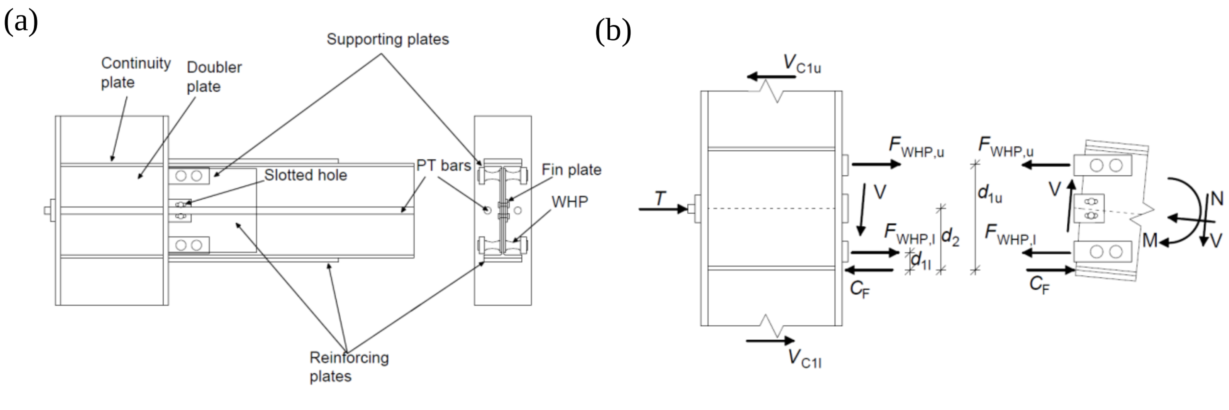
1. INTRODUCTION

Conventional steel moment-resisting frames (MRFs) are designed to sustain significant inelastic deformations in main structural members under the design basis earthquake (DBE; 475 years return period). Inelastic deformations result in damage and residual drifts, and so, in economic losses such as repair costs and downtime (time duration for repairs corresponding to loss of function). Steel self-centering moment-resisting frames (SC-MRFs) using post-tensioned (PT) beam-column connections are a new type of resilient structures. The advantage of SC-MRFs against conventional MRFs is the elimination of beam inelastic deformations and residual drifts as the result of gap opening developed in beam-column interfaces and elastic PT bars which clamp beams to the columns and provide self-centering capability. PT connections use yielding-based [1, 2 and 3] or friction-based [4, 5] energy dissipation devices which are activated when gaps open and can be easily replaced if damaged. A new PT connection using web hourglass shape pins (WHPs) has been recently developed and validated both experimentally and numerically in [1, 2 and 3]. Recent work has shown that steel SC-MRFs using PT connections with WHPs have superior collapse resistance compared to conventional steel MRFs [2].

The recent FEMA P-58 report [6] presents a methodology to assess the seismic performance of buildings based on their site, structural, non-structural, and occupancy characteristics. Performance is expressed in terms of the probability of incurring casualties, repair and replacement costs, repair time, and unsafe placarding. In this paper, the FEMA P-58 methodology is applied to a prototype building designed using conventional steel MRFs or SC-MRFs. An immediate comparison in terms of cost is conducted between the two structural systems and the ability of the SC-MRF to eliminate residual drifts and decrease seismic losses is highlighted.

2. SC-MRFs USING PT CONNECTIONS WITH WHPs

Figure 1(a) shows an exterior PT connection with WHPs. Two high strength steel bars located at the mid depth of the beam, one at each side of the beam web, pass through holes drilled on the column flanges. The bars are post-tensioned and anchored to the exterior columns. WHPs are inserted in aligned holes on the beam web and on supporting plates welded to the column flanges. Energy is dissipated through inelastic bending of the WHPs. The beam web and the beam flanges are reinforced with steel plates.



(c)

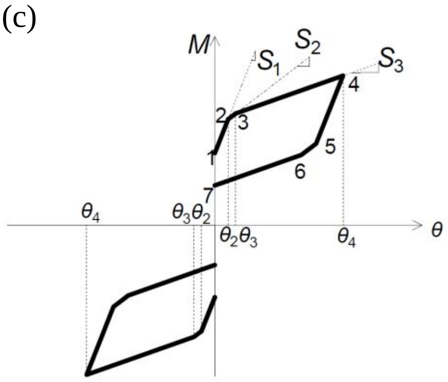


Figure 1. (a) Exterior PT connection with WHPs; (b) Gap opening in beam-column interface; (c) *M*-*θ* behavior of the PT connection with WHP

The connection behavior is characterized by gap opening and closing in the beam-column interface as a result of the re-centering force in the PT bars. Figure 1(b) shows the gap opening mechanism in the connection where d_{1u} and d_{1l} are the distances of the upper and lower WHP from the center of rotation (COR), respectively; d_2 is the distance of the PT bars from the COR; F_{PT} is the total force in both PT bars; $F_{WHP,u}$ and $F_{WHP,l}$ are the forces in the upper and lower WHPs, respectively; and C_F is the compressive force on the beam-column bearing surface. Figure 1(c) shows the theoretical cyclic moment-rotation (*M*-*θ*) behavior of the PT connection with WHPs.

A seismic design process for SC-MRFs using PT connections with WHPs within the framework of Eurocode 8 [7] has been recently proposed in [2]. Performance levels are defined with respect to the interstorey drift ratios (IDR), residual interstorey drift ratio (RIDR) and limit states in the PT connections. The design procedure involves sizing of the connection components (e.g. PT bars, WHPs, reinforcing plates) to achieve a target connection performance.

Models for SC-MRFs using PT connections with WHPs have been proposed in [2, 3]. In this work, a simplified model has been adopted where the *M*-*θ* behaviour of the PT connection is simulated by inserting 2 rotational springs in parallel at the beam ends. These rotational springs simulate the contribution of the WHPs and the PT bars on the overall rotational behavior of the PT connection.

3. LOSS ESTIMATION PROCEDURE

The seismic loss of a building is split into three distinct types according to [6]: (a) structural losses for damage in the load-carrying members of the structure; (b) non-structural loss for damage to non-load carrying components such as partitions, piping systems, etc.; and (c) contents' loss [14]. These types of seismic loss are assessed using component fragility functions parameterized on the engineering demand parameters (EDP) (i.e. IDR, peak floor accelerations, PFA). Following the procedure in [6], at each seismic intensity measure (IM) each component has a certain probability of being in any of its damage states (DS), which is in turn associated with a probabilistic cost function. Summing up such costs over the entire structure yields the total loss.

The probability of collapse is explicitly incorporated according to [8], i.e. collapse is assumed to cause instant loss of the entire building and its contents and dominates vulnerability at higher IM levels. The methodology explicitly incorporates residual deformations by considering the losses resulting from having to demolish the building when excessive RIDR is experienced. The probability of having to demolish the structure conditioned on the peak RIDR, $P(D|RIDR)$, is assumed to be a lognormal distribution with a median of 0.015 and a logarithmic standard deviation of 0.3 according to [9].

Vulnerability functions are developed using a simulation procedure based on the PEER loss analysis framework [10, 11]. In the PEER framework, the mean annual frequency (MAF) of a decision variable (DV), such as the cost or the loss ratio (building loss over the building replacement cost), is estimated as

$$\lambda_{DV}(DV \geq dv) = \int\int\int G(dv|DS)\left|dG(DS|EDP)\right|\left|dG(EDP|IM)\right|\left|\frac{d\lambda(IM)}{dIM}\right| dIM \qquad (1)$$

where $\lambda_{DV}(DV>dv)$ is the MAF of exceeding 'dv' (e.g. value of loss) for the given site and building; $G(dv|DS)$ denotes the probability of exceedance of the dv given a DS (i.e. a damage state associated with a specific repair action); $G(DS|EDP)$ is the probability of exceedance of the damage state given an EDP; $G(EDP|IM)$ is the probability of exceedance of the EDP given an IM; and $\lambda(IM)$ is the MAF of exceedance of the IM. In this work, following the guidelines of FEMA P-58 [6] the spectral acceleration at the fundamental period of vibration, $S_a(T_1)$, is chosen as IM.

In order to assess the performance of the two competing structural design in an objective manner that does not depend on the site, we shall instead employ only a part of eq. (1), using only the integrals of $G(dv|DS)$ over EDP and DS without the final convolution with $\lambda(IM)$. The result is known as the vulnerability function:

$$G(DV|IM) = \int\int G(dv|DS)\left|dG(DS|EDP)\right|\left|dG(EDP|IM)\right| \qquad (2)$$

The vulnerability function computes DV (loss ratio or repair cost of the building) as a function of IM and it is meant to be characteristic of the building and independent of the site (provided a sufficient IM is used). Monte carlo simulation (MCS) is used to evaluate the integrals shown in eq. (2). The MCS approach involves simulating all the random variables in eq. (2) (DV, EDP, DS) and then computing the DV for a wide range of IM. The steps involved in the MCS approach are presented, for example, in [12].

4. PROTOTYPE BUILDING

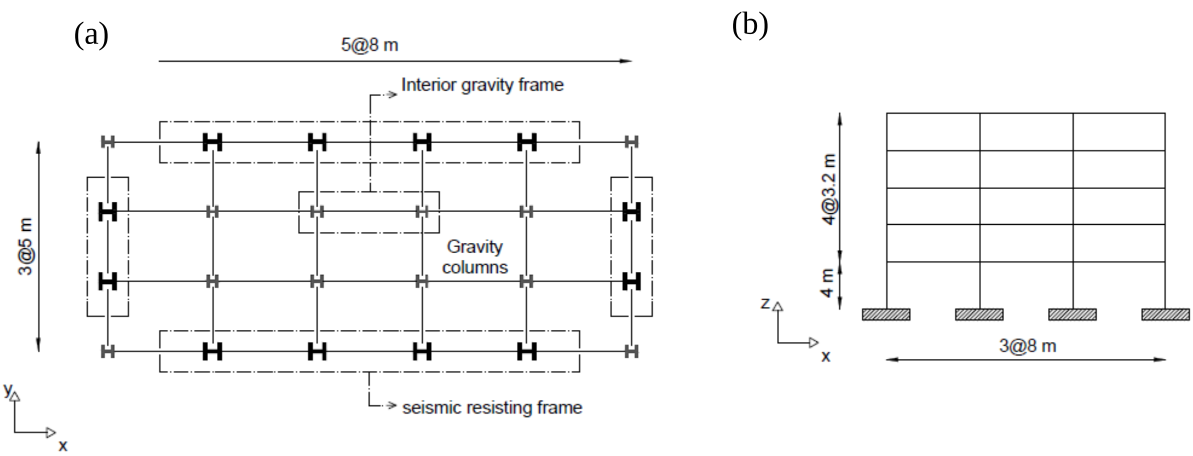


Figure 2. (a) Plan view of the prototype building; (b) Elevation view of the prototype building

Figure 2 shows the plan (a) and elevation view (b) of a 5-storey, 5-bay by 3-bay prototype building having two seismic resisting frames in the 'x' plan direction designed as conventional MRFs or SC-MRFs. Both the MRF and the SC-MRF have been designed to have IDR lower than 0.75% under the frequently occurring earthquake (FOE) [7]. The DBE is expressed by the Type 1 elastic response spectrum of [7] with peak ground acceleration equal to 0.35g and ground type B. The FOE has intensity of 40% (reduction factor v=0.4 in [7]) the intensity of the DBE. The steel yield strength is equal to 355 MPa for the columns and 275 MPa for the beams (characteristic strengths). The steel yield strength of the WHPs is 235 MPa and 275 MPa for the beam reinforcing plates. Design data of the frames are given in Table 1.

Table 1. Design data of the steel MRF and SC-MRF

cross sections		PT connections characteristics						
		PT force	PT bar diameter	WHP ext. diameter	WHP int. diameter	WHP length	Reinf. plate length	Reinf. plate thickness
Beam	Column	T_0 (kN)	d_{PT} (mm)	D_e (mm)	D_i (mm)	L_{whp} (mm)	L_{rp} (mm)	t_{rp} (mm)
IPE550	HEB650	1087	50	43	33	70	1392	35
IPE600	HEB650	1256	60	46	36	70	1660	46
IPE550	HEB650	1087	48	43	33	70	1416	35
IPE500	HEB600	941	38	41	30	70	1092	26
IPE500	HEB600	941	36	39	28	70	743	22

To evaluate the performance of the building in terms of repair and replacement cost, we have assumed that the building includes the structural components, non-structural components and contents listed in Table 2.

Table 2. Prototype building components

MRF components	**FEMA P-58 ID**	**SC-MRF components**	**units**	**EDP**
Steel column base plate	B1031.011b	-//-	8	IDR
Post-Northridge welded steel moment connection, beam one side	B1035.021 / None	PT connection, beam one side	4	IDR
Post-Northridge welded steel moment connection, beams both sides	B1035.031 / None	PT connection, beams both sides	4	IDR
Bolted shear tab gravity connections	B1031.001	-//-	28	IDR
curtain walls	B2022.001	-//-	54	IDR
suspended ceiling	C3032.003a	-//-	26	PFA
cold water piping	D2021.011a	-//-	1	PFA
hot water piping	D2022.012b	-//-	1	PFA
HVAC	D3041.001a	-//-	3	PFA
Modular office work stations	E2022.001	-//-	90	PFA
unsecured fragile objects on shelves	E2022.010	-//-	90	PFA
electronic equipment on wall	E2022.021	-//-	1	PFA
Desktop electronics	E2022.022	-//-	90	PFA
Book case	E2022.102a	-//-	90	PFA

The fragility and cost functions for most of the components of Table 2 are provided in [6]. Market research and engineering judgement were used to determine values for the missing ones (such as the PT connections). Thus, to extract the corresponding cost functions for PT connections, we assume that damage in the PT connections at each damage state is related to the plastic hinge rotation, θ_p, at the end of the reinforcing beam flange plate. θ_p has been associated to IDR on the basis of pushover analysis. An additional DS for the SC-MRF has been defined at the DBE to account for the cost of WHPs replacement. For the definition of fragility functions, equations presented in Chapter 3 in [6] have been used. For the PT connections cost functions, the mean and dispersion values of the corresponding moment resisting connections have been used. The labour and material cost of the WHPs has been used for the definition of the DS associated with WHP replacement. The contents cost functions have been developed based on USA market prices.

Incremental dynamic analysis (IDA) [15] has been performed for both the MRF and the SC-MRF under 11 ground motions developed in [13]. IDA has been performed up to sidesway collapse.

5. RESULTS

Figures 3 and 4 show the vulnerability functions of the MRF and the SC-MRF, respectively. In these Figures, the 16%, 50%, and 84% probabilities of a DV to be exceeded for a wide range of $S_a(T_1)$ are presented. The selected DVs are the repair cost and the loss ratio.

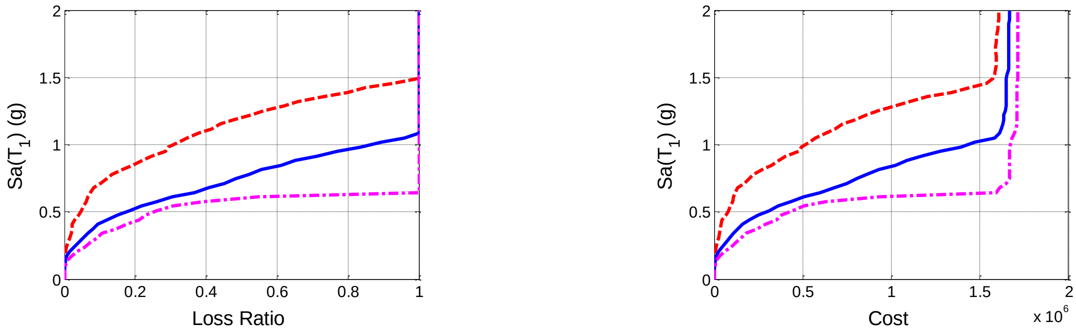


Figure 3. (a) Loss ratio and (b) cost curve of the conventional MRF

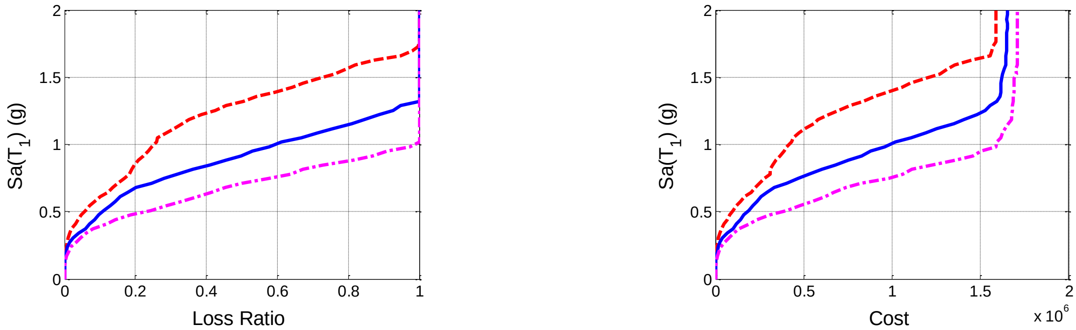


Figure 4. (a) Loss ratio and (b) cost curve of the SC-MRF

Figures (3) and (4) show that the SC-MRF performs better since, for the same $S_a(T_1)$, results in lower cost and loss ratio than the conventional MRF. For example, for $S_a(T_1)$ equal to 1.0 the MRF results to 1.5 million Dollars median loss versus 1 million for the SC-MRF. The main reason behind the better performance of the SC-MRF is the reduction of the RIDR. In particular, the possibility of having to demolish a building as a result of excessive RIDR is reduced, and so, the cost or loss ratio of the building are reduced. At lower intensities, these differences are significantly reduced: It is the loss of contents that drives that total cost. Since

PFAs in these intensities depend mainly on the distribution of stiffness, rather than ductility or strength, the two buildings show nearly the same performance.

6. CONCLUSIONS

In this paper, a prototype 5 storey steel building is designed using SC-MRFs versus conventional MRFs. IDA is performed for both structural systems up to collapse of the building under 11 ground motions. The seismic monetary losses of both structural systems are compared with the aid of the FEMA-P58 methodology. More specifically, vulnerability functions showing the cost and the loss ratio of the building for a wide range of $S_a(T_1)$ values are presented for both structural systems. The results show that the SC-MRF has similar performance to the conventional MRF at FOE levels, while it performs significantly better at DBE levels, leading to consistently lower seismic losses. The higher performance of the SC-MRF at high intensities is attributed to its ability to reduce residual drifts, and so, to avoid the need for demolition due to irreparable damage.

REFERENCES

[1] Vasdravellis G, Karavasilis TL, Uy B. Large-scale experimental validation of steel post-tensioned connections with web hourglass pins. Journal of Structural Engineering 2013; 139(6):1033-1042.

[2] Tzimas AS, Dimopoulos AL, Karavasilis TL. EC8-based seismic design and assessment of self-centering post-tensioned steel frames with viscous dampers. Journal of Constructional Steel Research, under review.

[3] Dimopoulos A, Karavasilis TL, Vasdravellis G, Uy B. Seismic design, modelling and assessment of self-centering steel frames using post-tensioned connections with web hourglass shape pins. Bulletin of Earthquake Engineering 2013; 11:1797–1816.

[4] Rojas P, Ricles JM, Sause R. Seismic performance of post-tensioned steel moment resisting frames with friction devices. Journal of Structural Engineering 2004; 131(4): 529-540.

[5] Kim HJ, Christopoulos C. Friction damped posttensioned self-centering steel moment-resisting frames. Journal of Structural Engineering 2008; 134(11): 1768-1779.

[6] FEMA P58. Seismic Performance Assessment of Buildings. ATC. Applied Technology Council. CA. USA; 2012.

[7] CEN, Eurocode 8: Design of structures for earthquake resistance, EN 1998, European Committee for Standardisation, Brussels, 2005.

[8] Aslani, H. & Miranda E. 2005. Probabilistic earthquake loss estimation and loss disaggregation in buildings, *Report No. 157*, The John A. Blume Earthquake Engineering Research Center, Stanford University, Stanford, CA.

[9] Ramirez CM, Miranda E. Significance of residual drifts in building earthquake loss estimation. Earthquake Engineering and Structural Dynamics 2012; 41:1477-1493.

[10] Cornell CA, Krawinkler H. Progress and challenges in seismic performance assessment. PEER Center News 2000; 3(2):1–4.

[11] Deierlein GG. Overview of a comprehensive framework for earthquake performance assessment. Technical report, International Workshop on Performance-Based Seismic Design Concepts and Implementation, Bled, Slovenia, 2004.

[12] Jayaram N, Shome N, Rahnama M. Development of earthquake vulnerability functions for tall buildings. Earthquake Engineering and Structural Dynamics 2012; 41:1495-1514.

[13] FEMA P695. Quantification of building seismic performance factors. ATC-63 Project. Applied Technology Council. CA. USA; 2008.

[14] Vamvatsikos D, Porter K. Tall Building Analytical Seismic Vulnerability Functions for the Global Earthquake Model. Safety, Reliability, Risk and Life-Cycle Performance of Structures and Infrastructure 2014; Edited by George Deodatis, Bruce R Ellingwood and Dan M. Frangopol, Pages: 911–918

[15] Vamvatsikos D, Cornell CA. Incremental dynamic analysis. Earthquake Engineering and Structural Dynamics 2002; 31(3): 491-514.

ΣΥΓΚΡΙΣΗ ΟΙΚΟΝΟΜΙΚΩΝ ΑΠΩΛΕΙΩΝ ΛΟΓΩ ΣΕΙΣΜΟΥ ΣΕ ΜΕΤΑΛΛΙΚΑ ΚΤΙΡΙΑ ΠΟΥ ΕΧΟΥΝ ΠΛΑΙΣΙΑ ΜΕ ΣΥΜΒΑΤΙΚΕΣ ΣΥΝΔΕΣΕΙΣ ΡΟΠΗΣ Ή ΣΥΝΔΕΣΕΙΣ ΠΟΥ ΕΧΟΥΝ ΤΗΝ ΙΚΑΝΟΤΗΤΑ ΝΑ ΕΠΑΝΕΡΧΟΝΤΑΙ ΣΤΗΝ ΑΡΧΙΚΗ ΤΟΥΣ ΘΕΣΗ

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ΠΕΡΙΛΗΨΗ

Τα μεταλλικα πλαίσια με συνδέσεις που εχουν την ικανοτητα να επανερχονται στην αρχική τους θέση συνιστούν κατασκευές που αποφεύγουν τη βλάβη στις δοκούς και τις παραμένουσες μετατοπίσεις υπό το σεισμό σχεδιασμού. Σε αυτήν την εργασία ένα κτίριο σχεδιάζεται χρησιμοποιώντας πλάισια με συνδέσεις που έχουν την ικανότητα να επανέρχονται στην αρχική τους θέση ή με συμβατικά πλαίσια ροπής και οι σεισμικές οικονομικές απώλειες των δυο περιπτώσεων σχεδιασμού συγκρίνονται με τη βοήθεια της μεθοδολογίας FEMA-P58. Η μεθοδολογία FEMA-P58 είναι με διαδικασία σεισμικής αποτίμησης με βάσει την επιτελεστικότητα η οποία αποτιμά τις σεισμικές απώλειες (κόστος) με πιθανοτικό τρόπο. Τα αποτελέσματα της παρούσας εργασίας δείχνουν ότι τα μεταλλικα πλαίσια με συνδέσεις που εχουν την ικανοτητα να επανερχονται στην αρχική τους θέση έχουν σημαντικά βελτιωμένη συμπεριφορά συγκρινόμενα με τα συμβατικά πλαίσια ροπής και μικρότερες οικονομικές απώλειες. Τα αποτελέσματα επίσης υποδεικνείουν πως είναι σημαντικό να λαμβάνονται υπ’ οψιν οι παραμένουσες μετατοπίσεις ως παράμετρος σεισμικής αποτίμησης διότι καθορίζουν άμεσα εάν το κτίριο είναι επιδιορθώσιμο ή χρειάζεται να κατεδαφιστεί μετά απο έναν ισχυρό σεισμό.