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PERFORMANCE BASED SEISMIC DESIGN: CHALLENGES OF APPLICATION FOR A TALL BUILDING IN A HIGH SEISMIC ZONE

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Abstract

This work aims to present the challenges of Performance Based Seismic Design (PBSD) method with an application to a real tall building project in a high seismic area in Beirut, Lebanon. The paper describes the motivations for implementing the PBSD as an alternative method to a code prescriptive approach (i.e. IBC/ASCE 7). Due to the code limitations on common seismic load resisting systems (SLRS) for high seismicity and building height, only a frame or a dual system (frame-wall) was permitted according to the code. The resulting structural design was both uneconomic and not compatible with the architectural aspirations for an iconic structure. As an alternative, a coupled wall system was selected to resist the seismic load and justified by PBSD.

However, a more sophisticated type of approach and analysis is not without it's practical challenges, ranging from the extremely laborious data input procedure within the non-linear model for a complex geometry to actually getting the PBSD approach approved by the local Authority. PBSD has been never adopted for building seismic design within Lebanon. Thus the work required to obtain approval were higher than normal. In order to persuade the authorities having jurisdiction to accept the use of PBSD, extensive documentation through publications, scientific papers and guidelines were required to confirm the validity and reliability of the proposed methodology.

As always the choice of the correct seismic input for performing the analyses was important: the ground motion selection needed the correct identification of the target earthquake scenario for different hazard levels. This task was undertaken by a local seismologist who performed a hazard analysis based on the knowledge of the actual seismicity of the local area, assessing correctly the seismic area source and faults as addressed in recent scientific publications. The 3D mathematical models were developed in Etabs for SLE and Perform3D for MCE. The paper highlights the modelling strategies and implementation, in particular for MCE simulation, inside the software Perform3D. The evolution of the energy dissipated is reported to measure the amount of non-recoverable damage and investigate the compliance with the expected and desired plastic mechanism for the selected coupled-wall system.

Keywords: Tall building, non-prescriptive approach, high seismic zone

1. Introduction

Performance Based Seismic Design (PBSD) has become a very common methodology for seismic analysis and design of high-rise buildings within the last 30 years in the western United States, where the majority of the references and guidelines have been developed. Many other buildings around the world have been also designed through PBSD approaches, especially in high seismicity regions such as Japan, China, Mexico and Chile. However, very few cases of applying this alternative procedure can be found in the Middle East (i.e. Dubai, Istanbul [1]). Beirut is one ME city that has recently renewed its development strategies with the inclusion of several tall buildings in it is central business zone. With the growing development of the large urban areas in high-risk earthquake zones, the probability of a strong earthquake striking a densely populated and economically developed areas increases each day. Therefore design approaches for tall buildings which lead to more reliable and economic development are required.

Traditional design codes were mainly developed to address the majority of the building stock, which are mainly low to medium rise with simple plan forms and framing systems. They deliberately limit the choice of the seismic resisting system with the height of the building where there are limited opportunities for energy dissipation through plasticity and ductility. This prescriptive approach often produces uneconomical solutions, especially in presence of irregularities due to the requirement of higher member sizes to compensate for the introduced irregularity. In the seismic codes it is assumed that certain structural system perform better under a maximum considered earthquake (MCE) if a good seismic detailing is undertaken within the ductile members selected on a basis of capacity design under a reduced seismic force. This reduction in the forces, applied to account for inelastic behavior of the building, is actually quite difficult to be reliably quantified and can be widely affected by the unique structural behavior of different types of tall building. Therefore performing elastic analysis, as prescribed by the code, can lead to inadequate estimations of the demand for MCE. PBSD allows a more realistic estimate of demand to be made, i.e. story and accelerations drifts and allows higher modes effects to be included which are very important in determining tall building response.

2. Lateral seismic force resisting system choice and guidelines framework

The building considered consists of a 32 story commercial office tower with a podium up to level 5 over a 7 level basement. The main block of the building stands approximately 137m tall. The top 12 storeys of the building are offset from the center of the building below.

The choice of lateral stability system is driven by the seismic design category, the most suitable structural form for the scheme, local construction constraints and is informed by the architectural aspirations of the development. The seismic design category is largely related to the size of the seismic design event for the region. The level of design earthquake for Beirut puts the building in seismic design category D.

This automatically restricts the choice of seismic lateral resisting system to certain systems and also restricts the height to which some of these can be designed using the mainly prescriptive provisions in the codes. Reinforced concrete is the predominant structural material in the local market. Available systems for design with reinforced concrete are special reinforced shear walls and dual systems with special reinforced concrete shear walls and special moment frames. At 137m tall, the main tower is above the code limit for special reinforced concrete shear walls alone in this seismic zone. For a dual system to work, the moment frame must take 25% of the seismic base shear. This means down-stand beams that are deep and columns that grow in size from pure gravity columns. This system does not work in terms of net to gross on the floor-plate, the restrictions on servicing or meet the architectural aspirations of the project with a light transparent facade. In order to exceed this limit for a shear wall system alone, the code requires an intensive performance-based design to ensure a ductile and safe response to earthquake loads.

Preliminary design of the building was initially performed for DBE elastic response spectrum analysis and according to capacity design principle and following IBC2009 [2] and ASCE 7-05 [3] prescriptive requirements. The response reduction factor R has been taken equal to 2.5 for the design walls and link-beams to ensure that the desired plastic mechanism will be activated during a nonlinear time-history analysis. This choice was based on experience of previous designs, corresponding to a limited level of system ductility, reducing the amount of plastic hinges in the link beams and activating a plastic hinge at the bottom of ground floor core walls.

The performance was then assessed through nonlinear time history analysis to check the damage level within the region of the structure where plasticity has occurred and their location to verify the expected plastic mechanism has been achieved. The performance-based design procedure was applied with reference to TBI codified guidelines of PEER Center [4] and related referenced reports [5, 6].

3. Site specific seismic hazard assessment

Design requirements and probabilistic hazard assessments for Beirut and Lebanon have been recently updated accounting for new and different interpretations of the seismic sources affecting the hazard [7, 8]. Historical records have documented the occurrence of large earthquake in the region of Beirut with events of Magnitude above 7. The largest event registered is the Mw=7.3 Gulf of Aqaba Earthquake of 1995 which confirmed the high potentiality of high intensity earthquake along the Dead Sea Transform Fault (DSTF). The DSTF is divided into 5 main branches: the Roum, Yammouneh, Seghaya, Rachaiya and Hasbaya fault (Figure 1). Yammouneh fault is considered the main branch of SDTF which crosses the entire country and is more likely to control the hazard in Lebanon, accommodating the majority of the plate motion and overall slip of the DSTF within the Lebanese restraining bend.

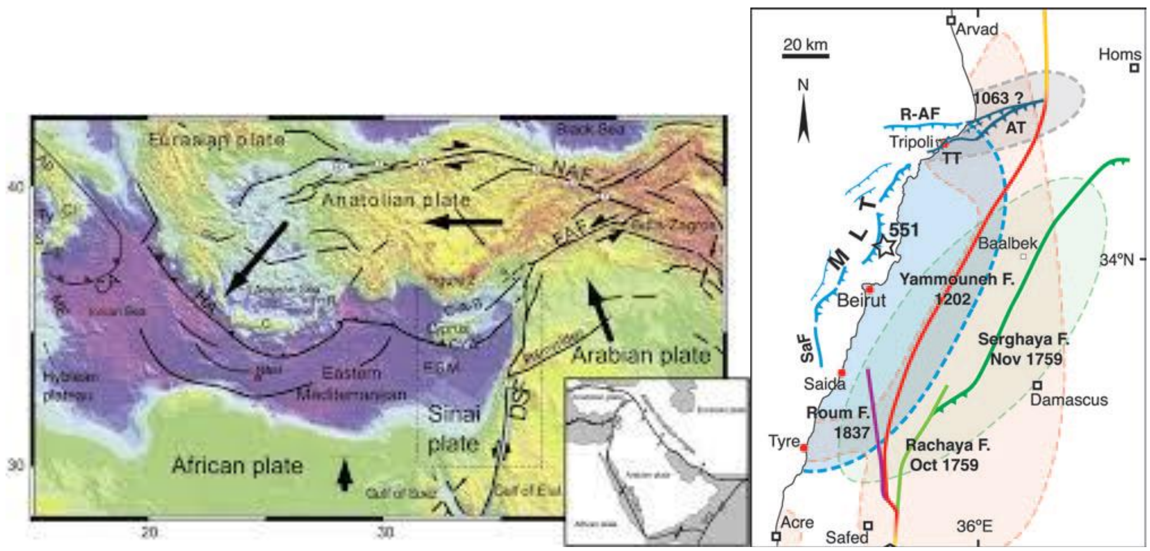

Fig. 1 – Eastern Mediterranean tectonic region (left); Possible sources of large historical Earthquakes in Lebanon [9]

Site-specific probabilistic seismic hazard has been adopted and performed by the local seismologist using the software FRISK [10]. The outcomes of this analysis are the uniform hazard Spectra (UHS) associated with Service and MCE levels; percentage contributions to the ground motion hazard at the key structural periods for each hazard level, evaluated from de-aggregation of the seismic hazard. These contributions depend on the seismic source, earthquake magnitude and source to site distance. The UHS have been provided for return period (Tr) of 43, 500 and 2475 years and 5% damping. The comparison between the spectral acceleration intensity for the different hazard levels and the code spectrum employed for the preliminary RSA analysis and corresponding to two thirds of MCE is reported in Figure 2.

Based on the de-aggregation hazard results given within the probabilistic hazard analysis, the magnitude and distance range that contributes mostly to the hazard are respectively 6.5-7.5 and 5-25km for return period equal to 2475 years and for T=3s and 4s spectral periods.

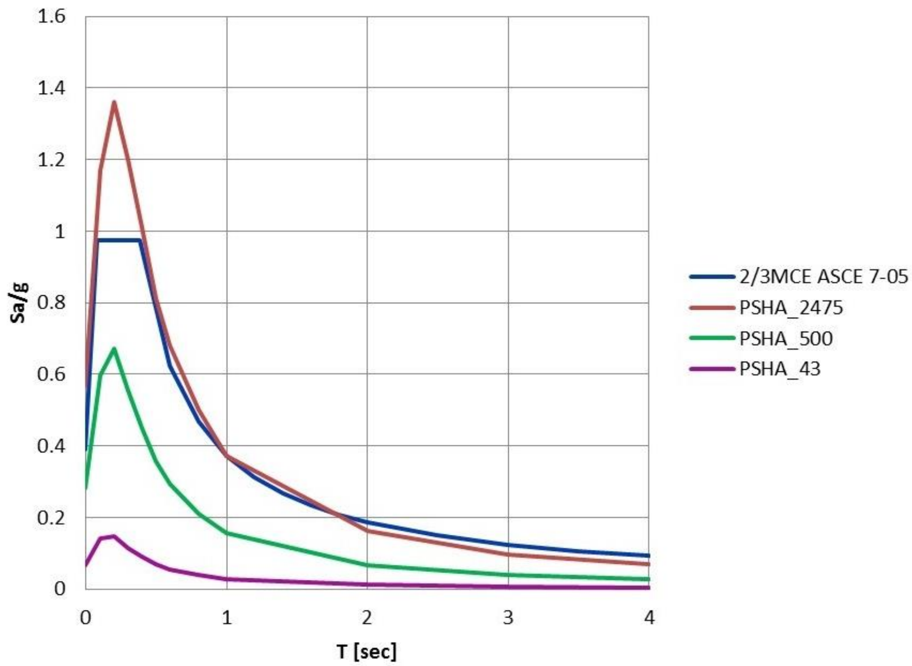

Fig. 2 – Uniform hazard response spectrum for 5% damping: PSHA and ASCE 7-05

4. 3D mathematical modelling

According to TBI guidelines, structural analysis should be based on 3D mathematical models capable of accurate representation of the spatial distribution of mass and stiffness, leading to meaningful estimates of the building dynamic features. Within the proposed performance-based design framework two such 3D models are required: an elastic model for SLE assessments and a nonlinear model for MCE response simulations.

4.1 Elastic model

Elastic models for SLE analysis are justified by the acceptance criteria for this limit state with the requirement of essentially elastic structural response. As the building is expected to remain essentially elastic there is no need to employ computationally demanding nonlinear models. Nevertheless the verification of this assumption is carried out at the end of the SLE analysis stage. In line with generalized seismic design procedures [11], ETABS 2015 [12] was chosen to perform the analysis of the structure subjected to SLE ground motions.

Specific stiffness modifiers are assigned to the relevant structural members in order to account for unavoidable cracking and minor damage within the linear elastic model. The values of the cracking factors are adopted from TBI recommendations [4] and summarized in Table 1. This latter also presents the applicable stiffness modifiers employed within the nonlinear model for the MCE assessment. The stiffness modifiers, with reference to the MCE case, are not considered for core walls and link-beams, as there is already an explicit measure of the damage due the employment of non-linear models.

Another relevant feature to be properly addressed within the performance-based framework is consideration of expected material properties as opposed to nominal ones usually assumed for prescriptive design. In this way the mathematical models provide realistic estimates of building performance. According to TBI recommendations the expected concrete and steel rebar strength might be estimated by multiplying the nominal values by 1.3 and 1.17, respectively. Consequently, the elastic modulus for concrete should be obtained employing the formulae suggested in ACI 318 [13] considering the expected strength value. Moreover a 2.5% equivalent viscous modal damping has been chosen for the SLE elastic analyses.

ELEMENT TYPE	WIND	RSA	SLE	MCE
Core Shear Walls	1.0 E_cI_g	0.7 E_cI_g	0.75 E_cI_g	n.a.
	1.0 G_sA_g	1.0 G_sA_g	1.0 G_sA_g	0.25 G_sA_g
Link Beams	0.5 E_cI_g	0.35 E_cI_g	0.5 E_cI_g	n.a.
	1.0 G_sA_g	1.0 G_sA_g	1.0 G_sA_g	1.0 G_sA_g
RC column	0.7 E_cI_g	0.7 E_cI_g	0.5 E_cI_g	0.5 E_cI_g
	1.0 G_sA_g	1.0 G_sA_g	1.0 G_sA_g	1.0 G_sA_g
PT Slabs	0.1 E_cI_g	0.25 E_cI_g	0.5 E_cI_g	0.5 E_cI_g
	1.0 G_sA_g	1.0 G_sA_g	1.0 G_sA_g	1.0 G_sA_g

Table 1 – Stiffness modifiers for different design objectives

An important outcome of the 3D elastic model is the identification of the inherent dynamic properties of the structure. The modal results are listed in Table 2, where the modal participation factors for the horizontal translations and torsion are presented as cumulative values. A close examination of the values in Table 2 highlights the important contribution of higher modes to the total dynamic response and base excitation for a tall irregular and asymmetric building.

Mode	Period (sec)	Cum PF X	CUM PF Y	CUM PF RZ
1	4.17	0.00	0.35	0.13
2	3.74	0.43	0.35	0.15
3	2.09	0.43	0.47	0.53
4	1.23	0.46	0.58	0.65
5	1.05	0.46	0.68	0.73
6	0.99	0.69	0.68	0.74
7	0.76	0.71	0.68	0.89
8	0.53	0.71	0.80	0.91
9	0.42	0.74	0.85	0.91
10	0.39	0.85	0.86	0.93
11	0.32	0.85	0.87	0.94
12	0.31	0.85	0.88	0.94
13	0.26	0.86	0.92	0.95
14	0.24	0.91	0.92	0.96
15	0.22	0.92	0.93	0.97
16	0.20	0.92	0.93	0.98
17	0.18	0.92	0.93	0.98
18	0.17	0.92	0.93	0.98
19	0.16	0.92	0.94	0.98
20	0.16	0.95	0.94	0.99

Table 2 – Eigenvalue analysis of the main building structure: periods and cumulative mass participation factors using RSA stiffness modifiers

4.2 Nonlinear model

A detailed model including material and geometric nonlinearities was built in CSI Perform-3D [14]. This software is currently used worldwide for performance-based seismic design of high-profile tall buildings in severe earthquake prone areas (see e.g. [15] and [16]). A representative elevation view of the model created in Perform-3D is shown in Figure 3. According to the capacity-design principles supporting the performance-based framework, significant damage is only allowed to occur at the link-beams and specific locations of the core shear walls. For this reason, all other members are modelled with elastic finite elements employing the relevant stiffness modifiers already described in Table 1. Shear walls are modelled with distributed plasticity. Fibre elements are available in Perform-3D in order to capture the coupled bending-axial response. The wall 'compound' has fibers combined with a shear spring to account for shear distortion of the panel. In order to facilitate the data input procedure, which becomes extremely laborious and time consuming for complex geometries such as the current case, each pier was previously subdivided into several segments which correspond to a given reinforcement ratio computed during the schematic design. A sketch of the individual segments is shown in Figure 4, in which the Pier is subdivided in three segments in order to accommodate the two boundary elements for confined concrete and the central web for unconfined concrete.

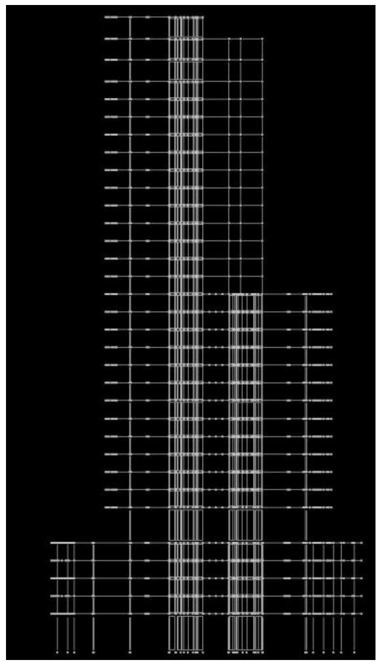

Fig. 3 – Perform3D model – elevation view

Fig. 4 – Modelling strategy for shear wall elements using three fibre-based in Perform-3D

Each single segment of the pier has assigned either a confined or an unconfined concrete material and each individual segment is discretized in six concrete uniaxial fibres and six uniaxial steel fibres. A higher fiber density is chosen for the boundary elements where plasticity is expected and higher ductility is guaranteed by the confinement of the concrete due to the hoops. The adopted constitutive model for concrete is Mander-Priestley-Park model. A schematic plot of the constitutive curves for concrete (a) and steel bars (b) is shown in Figure 5.

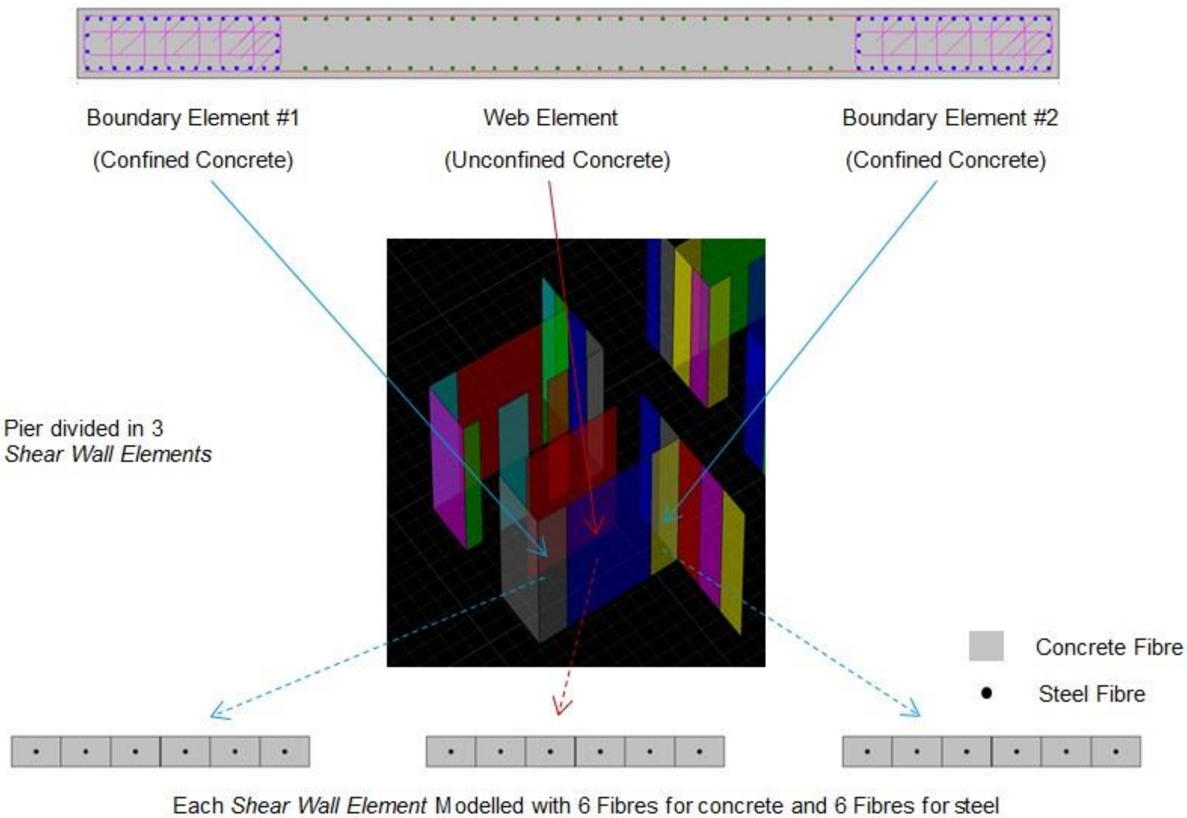

Fig. 5 – Constitutive models for concrete and steel rebar

Link-beams constitute another fundamental structural component governing the nonlinear seismic response, as according to the capacity design criteria, these elements are supposed to dissipate a considerable part of the input energy during the cyclic dynamic deformation. Similarly to the shear walls, a fibre-based approach could have been employed for these beams. Nonetheless, a practical established approach is to model the link beams using lumped plasticity rotational hinges as illustrated in Figure 6, where two nonlinear hinges are located at the element end combined with an elastic beam component to build the link-beam compound component. The plastic hinges are modeled with rigid-plastic moment rotation component based on ASCE 41-06 [6] recommendations.

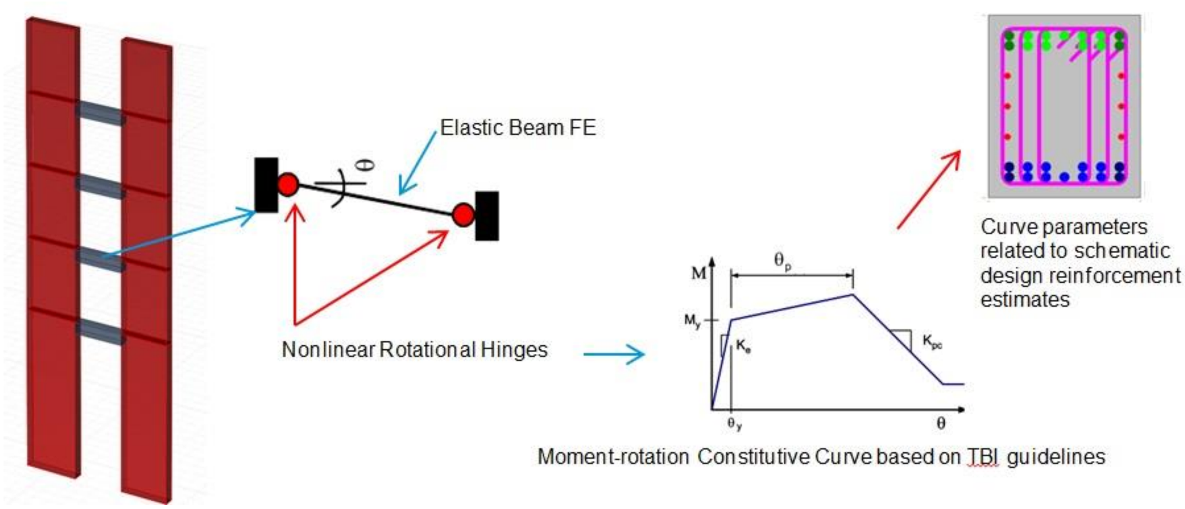

Fig. 6 – modelling strategy for link-beams in Perform-3D

For the nonlinear simulations, choosing the proper amount of viscous damping is not a straightforward task. This is mainly because in nonlinear analysis a considerable portion of the input energy is supposed to be dissipated explicitly at any location where a nonlinear hysteretic loop is activated. In the current case, energy dissipation through hysteretic behaviour is related to core shear walls and link-beams. An example of a hysteretic curve for a single concrete wall is shown in Figure 7 for illustrative purposes. By definition, the area of the graph enclosed by the cyclic force-displacement curve is the energy dissipated through hysteresis. Guidelines to estimate the parameters required to define such hysteretic behaviour in Perform-3D are available in TBI [4], ATC-72 [5] and also in PEER [11] with reference to detailed case studies.

Given this feature of nonlinear seismic simulations, following ATC-72 [11] recommendations a 2.5% equivalent modal damping is considered for MCE assessments. However, such modal damping formulation is intrinsically related to the elastic vibration modes, meaning that upon the activation of nonlinearity a portion of the dynamic deformation is not properly damped [17]. To circumvent this drawback, Powell (2007) [18] suggests the inclusion of a small amount of stiffness proportional damping, this way providing some viscous dissipation to nonlinear modes.

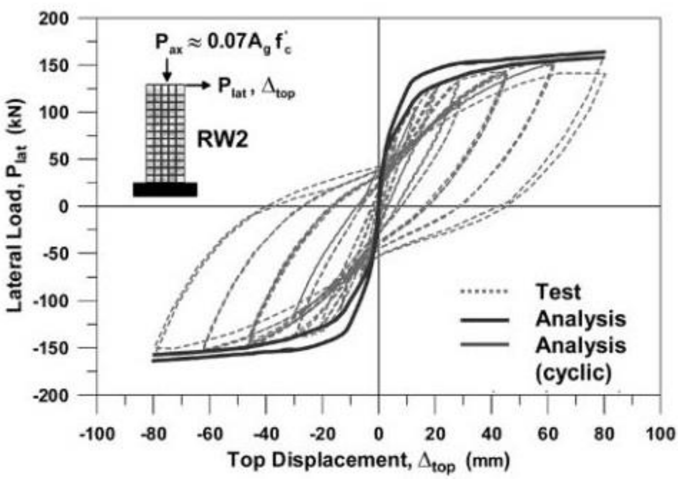

Fig. 7 – Example of a nonlinear hysteretic curve for cyclic lateral displacement (Wallace, 2012)

The presented nonlinear model incorporates the gravity system into the MCE simulation. The adopted TBI guidelines do not clearly state their inclusion within the MCE non-linear analysis. The common approach [11] consists in lumping the mass at each story at the center of the mass in terms of dead load and associated rotational moment of inertia, with inclusion of a rigid diaphragm instead of explicitly modelling the slab members. For the current case, it was decided to keep the gravity system in the Perform-3D model (see Figure 3) allowing for a spatial distribution of mass and gravity loads in the structure, capturing a closer simulation of the dynamic behavior under the ground motion excitation. Moreover the integration of gravity members within the non-linear model allows for a proper description of their demand for deformation compatibility.

5. Non-linear analysis with Perform3D

The nonlinear model created with Perform-3D subjected to one of the MCE level ground motion pairs is reported to show the non-linear behavior of the structural system as a representative scenario. The presented outputs provide evidence that the relevant features of the nonlinear seismic response are properly captured in accordance to the TBI guidelines as it is shown below.

In order to inspect the evolution of damage within the structure, resource is made to the energy plot easily accessible from Perform-3D post-processing features. The total energy plot is shown in Figure 8, visually displaying how the energy imparted by the earthquake is absorbed / dissipated by the mechanical system.

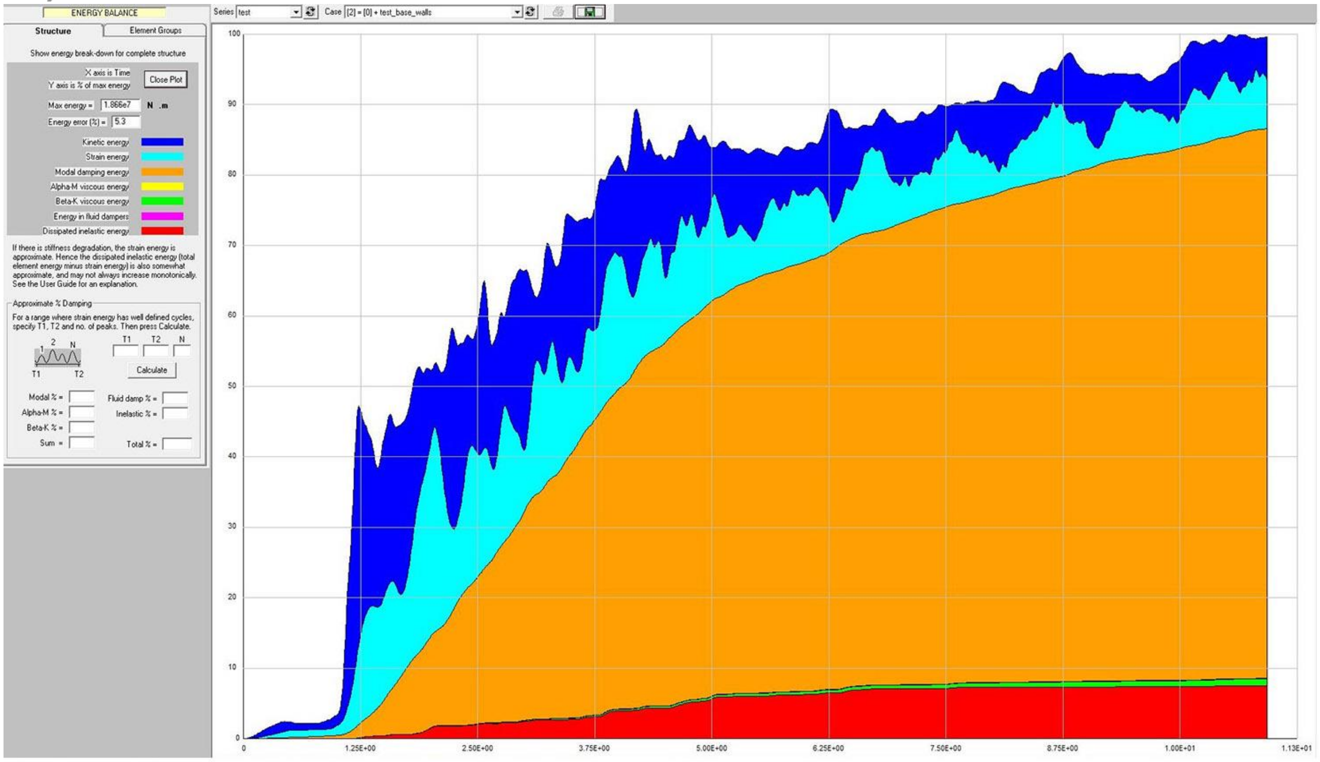

Fig. 8 Energy balance in the structure as function of time

A crucial conclusion from the energy plot in Figure 8 is that the damage experienced by the structure is quite limited, as the amount of energy dissipated through member's plasticity is minimal compared to the total energy in the system. Therefore, viscous damping is responsible for most energy dissipation, emphasising the need to carefully test the sensitivity of the nonlinear dynamic response to this parameter.

Finally, the evolution of energy dissipated through element hysteresis is inspected in Figure 9. It should be noted that the energy dissipated through non recoverable damage is mainly concentrated in the link-beams and only a small portion in the core shear walls, confirming the building is responding with the expected plastic mechanism.

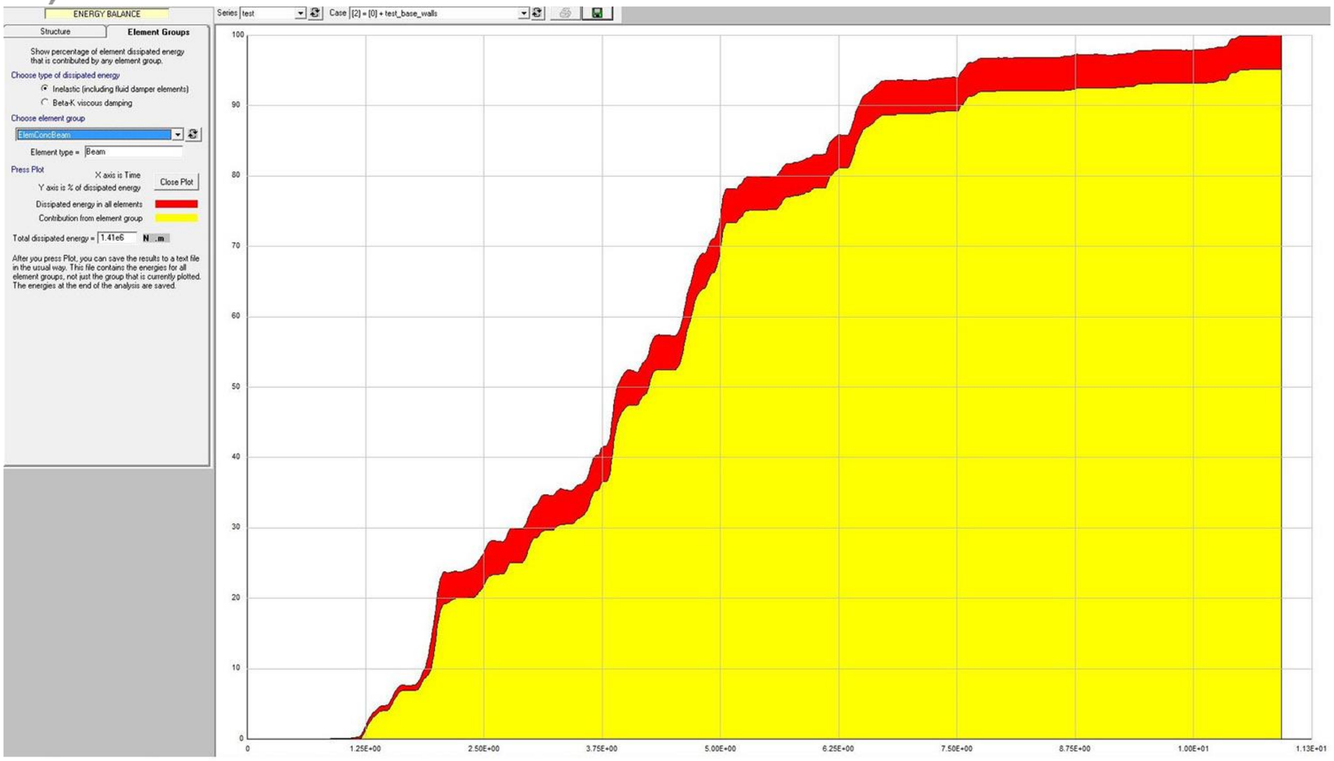

Fig. 9 Energy balance Energy dissipated by hysteretic behaviour of link-beam (yellow) and core walls (red)

6. Conclusions

The performance based design methodology is adopted as an alternative approach respect to the prescriptive code design and has been employed to assess the response of a 32 story irregular tower to show the feasibility of using a coupled shear wall earthquake resisting system.

The energy balance plot in Figure 8 confirms that only a small percentage of the total energy has been dissipated by the link beams and walls, and therefore the damage experienced by the structure is limited. This behavior reflects qualitatively the initial assumption of a response reduction factor R=2.5 chosen to account the system inelastic dissipation into the linear response spectrum analysis.

The PBSD methodology has been particularly beneficial respect to prescriptive approach consequently enabling the design team to achieve a better architectural layout avoiding the presence of deep beams and reducing the size and number of the columns. Furthermore the PBSD increases the reliability and confidence in the structural system by ensuring that the expected plastic mechanism is achieved by clearly identifying plastic regions and confirming that the inelastic energy dissipation distribution occurs within the members designed for a higher ductility.

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