**PRELIMINARY NONLINEAR ANALYSES OF POST-TENSIONED TIMBER FRAMED BUILDING WITH DISSIPATIVE BRACING SYSTEMS**

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**ABSTRACT**

Self-centering rocking mechanisms combined with dissipative bracing systems is a passive control technology that can prevent structural and non-structural damages and minimize residual drifts during strong earthquakes. A shaking table campaign has been performed on a 2/3 scaled, 3-dimensional, 3-storey post-tensioned timber framed (Pres-Lam) building with dissipative bracing systems at the structural laboratory of the University of Basilicata. The test structure has been subjected to ground motions scaled to various intensities. In this study a numerical provisional model of the Pres-Lam braced structure has been developed based on lumped plasticity approach. This paper focuses on preliminary results of nonlinear dynamic analyses of a selected ground motion at 25% and 100% of PGA levels that correspond to a Service Level Earthquake (SLE) and a Design Base Earthquake (DBE). Numerical outcomes have been compared with shaking table test results, providing a suitable representation of the global and local seismic response.

*Keywords: Post-tensioned timber framed buildings; Dissipative bracing systems; Shaking table tests; Non-linear dynamic analyses.*

**1. INTRODUCTION**

Modern buildings designed according to the current seismic codes are expected to develop a controlled ductile inelastic response during strong earthquakes, with possibly substantial residual deformations and structural damage. Following recent severe earthquakes worldwide, even modern buildings were extensively damaged and engineers began to realize that the design objectives in current building codes in preserving life safety are no longer sufficient, but it is now the minimum required. Earthquake-resilient structures based on low-damage technologies are developing, such as dissipative rocking mechanisms (Di Cesare et al. 2019a), dissipative bracing systems (Di Cesare et al. 2014, Mazza et al. 2019) and other forms of seismic protection (Ugalde et al. 2019). It not only focuses on preserving life in a strong earthquake, but also on preserving the structural and non-structural elements of the building, without or with easily repairable damages.

In the last decades, there has been renewed worldwide interest in timber as construction material and seismic resistant technologies for multi-storey timber buildings have been developed. Pres-Lam system, originally conceived for use in concrete structures (Priestley et al. 1999), has been applied to timber walled and/or framed buildings (Buchanan et al. 2008). This system uses unbounded steel cables or bars to connect beams to columns or columns/walls to the foundation. In high seismic hazard areas the post-tensioning can be coupled with dissipative systems, such as bracing systems with replaceable and cost efficient dampers (Baird et al. 2014). While the post-tensioning provides the required re-centering capability to mitigate the consequences of residual drifts, the dampers provide additional strength and allow adequate energy dissipation, reducing the inter-storey drifts. During lateral movement, this system exhibits the typical ‘flag-shaped’ behavior (Di Cesare et al. 2017a). Shaking table tests have been recently performed at the structural laboratory of University of Basilicata, on a 3-dimensional, 3-storey, 2/3 scaled post-tensioned timber framed building equipped with dissipative bracing system (see Figure 1). It was part of a collaborative experimental campaign with University of Canterbury (Christchurch, New Zealand), in which the bare specimen was tested in different configurations, both with and without different energy dissipating systems (Di Cesare et al. 2017b).

The main goal of this paper is to set up a simple and robust numerical model, developed with finite element SAP2000 software (CSI 2014) able to provide an accurate prediction of the main seismic parameters. The calibration of the proposed model has been checked by comparing the results of nonlinear time history analyses with preliminary experimental results at PGA levels corresponding to a Service Level Earthquake (SLE) and a Design Base Earthquake (DBE).

**2. EXPERIMENTAL TESTING**

The prototype building was a three-dimensional timber framed structure three-stories tall, two single bays with a footprint of 6 m by 4.5 m and an inter-storey height was 3 m (Di Cesare et al. 2017b). It was post-tensioned in both directions, with steel bars crossing at the beam-column joints, and designed according to current seismic codes (EC8; NTC2018) to represent an office structure (live load of 3kPa for level I and II) with a rooftop garden (2kPa for level III). A scale factor of two-third was applied to the prototype building obtaining an experimental model with inter-storey height of 2m and footprint of 4m by 3m. The section sizes of the frame elements were: 200 x 320mm for columns, 200 x 305mm for primary beams and 200 x 240mm for secondary beams. Beams and columns were made of Glulam grade GL32h (CNR-DT 206 R1/2018). Shaking table tests were performed on the experimental model in different testing sessions (Table 1).

During Session 1 and Session 2 the specimen was tested without (free rocking) and with dissipative rocking mechanisms at beam-column and/or at column-foundation connections (Di Cesare et al. 2019a). During session 3 of testing, the experimental specimen was equipped also with a dissipative bracing system (Figure 1a). In this testing session six dissipative braces, two for each storey, were inserted within the bays of two parallel frames along the test direction. Figure 1b shows a photograph view of the bracing system composed by V-inverted timber rods, with cross section of 160 x 180mm, in series with two U-shaped Flexural Plates (UFP) dampers made of C60 stainless steel. For more information of about UFP dampers (Figure 1c) characteristics refers to (Ponzo et al. 2019).

Table 1. Testing sessions of the experimental campaign.

|  |  |  |  |
| --- | --- | --- | --- |
| Testing  session | Dissipative rocking | | Dissipative |
| Beam-column | Column-foundation | braces |
| 1 | NO | NO | NO |
| 2 | YES | YES | NO |
| 3 | NO | YES | YES |

The dissipative bracing systems were designed to increase the strength, stiffness and damping of the structure and to reduce inter-storey displacements, in order to eliminate or limit damages to structural and non-structural elements. In Table 2 and Figure 1b-c an overview of the main characteristics of the dissipative bracing system, considering a design earthquake level having a PGA of 0.44g and a target drift of 1.3%, is reported.

Table 2. Main characteristics of the dissipative bracing system.

|  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- |
|  | Storey | UFP dampers | | | | V-inverted braces | |
| bu  (mm) | tu  (mm) | Du  (mm) | Steel type | Rod section  (mm) | Glulam type |
| 1 | 60 | 6 | 60 | C60 | 160 x 180 | GL24h |
| 2 | 40 | 6 | 60 | 160 x 180 |
| 3 | 30 | 6 | 60 | 160 x 180 |

|  |  |
| --- | --- |
| (a)C:\Users\Nicla\Documents\UNIBAS\Pres-Lam\foto struttura PRES-LAM\foto Pres-Lam con controventi\20180622_113414.jpg | (b)  (c) |

Figure 1. (a) Experimental model with dissipative bracing systems tested at UNIBAS laboratory; (b) Detail of the bracing system connection; (c) UFP hysteretic dampers used for the bracing system.

In order to evaluate the influence of the bracing system on the seismic response of the experimental model, shaking table testing were carried out considering a set of seven natural earthquakes selected from the *European strong-motion database* at various intensities. The seismic response was recorded by a combination of 54 instruments, including displacement potentiometers, load cells and accelerometers. Local deformations were also recorded across the gap opening of beam-column, column-foundation connections and UFP hysteretic dampers.

In this paper the ground motion selected for test results is the 1992 Ezican earthquake record (ID 535). This motion was scaled to intensities corresponding to a Service Level Earthquake (SLE) and Design Base Earthquake (DBE). Figure 2 shows the response spectra of the selected input for the two intensities (25% and 100% of PGA) compared with the corresponding code spectra.

|  |  |  |
| --- | --- | --- |
| EQ | 535 |  |
| Location | Erzican, Turkey |
| Date | 13/03/1992 |
| MW | 6.6 |
| PGA (g) | 0.769 |
| Scale Factor | 1.5 |
|  |  |

Figure 2. Comparison between acceleration response spectra for SLE and DBE by code and 535 seismic input at 25% and 100% of PGA level.

**3. NUMERICAL MODELLING**

The numerical modelling of the test frame with dissipative bracing systems was developed using a lumped plasticity approach which combines the use of elastic elements with linear or rotational springs representing plastic deformations in the system. The braced frame has been modelled by SAP2000 (CSI 2014), using frame-type 2D finite elements shown in Figure 3a. The structural elements (beams, columns and braces) were modelled as elastic elements with anisotropic glulam timber material. The connection between beam and column was modelled considering a combination of rotational springs to represent the contribution of the post-tensioning and the flexibility of the joint panel. Post-tensioning response was represented using tri-linear-elastic moment–rotation springs and the joint panel was accounted for introducing an additional linear rotational spring (Figure 3b), with stiffness value opportunely arranged on the preliminary shaking table tests (Di Cesare et al. 2019b). The column base connection was modelled with 3 rotational springs in parallel (Figure 3c), which considered the moment resistance given by the contribution of the gravity plus seismic axial load and the additional moment contribution of hysteretic steel elements (Ponzo et al. 2017). Finally, the nonlinear force-displacement hysteretic behavior of the UFP dampers was modeled by using link elements connecting the elastic beam and V-inverted braces (Figure 3d), characterized by the Bouc-Wen cyclic laws (Bouc 1967; Wen 1976). More specifically, the post-yield stiffness ratio was assumed equal to 7%, while the yielding exponent, which regulates the shape of the hysteresis loop, was considered equal to 2. The constitutive laws of rotational springs and nonlinear links are represented in Figure 3e.

|  |  |
| --- | --- |
| (a)    (e) | (b)  (c)  (d) |

Figure 3. (a) 2-D numerical model of the braced frame and details adopted for: (b) beam-column joints, (c) column-foundation joints, (d) dissipative bracing connection and (e) constitutive laws of elements connections.

**4. RESULTS**

The effectiveness of the numerical modelling to describe the seismic response of the structural model is investigated. Computational and experimental studies have been reported for the selected seismic input (ID 535) at different intensities in order to identify and examine key design variables and demonstrate the performance of the system. Figure 4 and 5 show the comparison for two PGA intensity levels of 25% and 100%, corresponding to SLE and DBE, respectively. In Figure 4 the global seismic response of the braced frame is compared in terms of time histories of first floor drift and base shear and of first floor drift vs base shear. In Figure 5 the local behavior is shown in terms of cyclic force vs displacement of UFP dissipative devices at all storey.



Figure 4. Comparison between numerical and experimental results of the braced frame for 535 earthquake input at PGA intensity levels of 25% and 100%.

A good agreement between numerical and experimental results can be observed, only with few discrepancies of the maximum peak values of the base shear. In case of SLE intensity (25% of PGA), it can be clearly observed an elastic global and local behavior preventing the occurrence of yielding of both the framed structure and the dissipative devices. The elastic stiffness of the experimental model is in good agreement with the numerical model. In case of DBE intensity (100% of PGA) the flag-shaped hysteretic behaviour is highlighted. The experimental response is reliably predicted by numerical model, both in terms of energy dissipation capacity and of maximum force and displacements.



Figure 5. Comparison between numerical and experimental force-displacement cyclic laws of UFPs for earthquake output 535 at PGA intensity levels of 25% and 100%.

Figure 6 compares the results of nonlinear static analysis (NLSA) performed applying two distributions of lateral loads, ‘‘modal’’ and ‘‘uniform’’ patterns (EN 1998-1 2003) with the results of tests at 25% and 100% PGA levels in terms of maximum total drift and base shear of the single braced frame. NLSA are capable of predicting adequately the nonlinear dynamic seismic response.

The frame elements and the beam-column joints of the braced model performed as expected with no damage occurring throughout testing, with the application of almost 40 earthquakes, in some cases over the Design Base Earthquake level.



Figure 6. Comparisons between the results of dynamic testing for different PGA levels and the pushover curves obtained from nonlinear static analysis

**5. CONCLUSIONS**

An extensive experimental campaign was carried out on a three dimensional, three storey, 2/3 scaled post-tensioned timber framed building equipped with V-inverted dissipative bracing systems, based on U-shaped Flexural Plate (UFP) hysteretic dampers.

This paper presents the results of nonlinear dynamic analysis compared with seismic response of shaking table testing of one of the stronger spectra compatible earthquake (Erzican input) scaled to intensities of 25% and 100% of peak ground acceleration that correspond to a Service Level Earthquake (SLE) and Design Base Earthquake (DBE). A lumped plasticity approach was used for the numerical model of braced structure, with a suitable combination of elastic elements and nonlinear rotational / linear springs.

The specimen responded within its elastic range at SLE intensity (25% of PGA) preventing the occurrence of yielding of both the framed structure and the dissipative devices. In case of DBE intensity (100% of PGA), the inter-storey drift reaches a maximum value of 1.9%, exceeding the design target drift, without damage to the structural elements. In this case, the UFP dampers were activated at each storey withstanding ductility values of almost 4, with stable cyclic hysteretic behavior for a large number of load cycles. The dissipative and re-centering capability of the structural system was clearly shown by the global flag-shape behavior.

The results of static and dynamic nonlinear analysis are capable of reproducing adequately the global and local seismic response of the experimental model. A satisfactory agreement of in terms of first floor drift and base shear was obtained for all seismic intensities. The cyclic behaviour of the UFPs is well simulated at all storeys. The numerical results described herein are preliminary and further experimental and analytical investigation is forthcoming.

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