Supplemental Damping for the seismic retrofit of 8-storey RC Hotel building in the Mexican Pacific using Yielding Restrained Braces – part B, Methodology

Oscar Galindo[1], Richard Frazao[2] Carlos Pastor[3], Guillermo Coronado[3], Daniel Gonzales[3]

[1] Ph.D. Student, Department of Civil Engineering, Concordia University - Montreal, QC, Canada.
[2]R&D Director, Quaketek Inc. - Montreal, QC, Canada.
[3]Senior Engineer, Consultores en Ingeniería Estructural S.A. de C.V. - Guadalajara, Jalisco, México.

ABSTRACT

Simplified method for seismic friction damper application for the seismic retrofit of a concrete building in a high seismic zone. The system is designed using a conventional force-based method and reduction factor. The Yielding Restrained Brace (YRB) concept is presented, relying on limiting the forces in braces by using Ten-Co seismic brakes. Effectiveness of the simplified method and system is then confirmed by a NLTH. Results show high seismic performance of the proposed system with little to no ductile damage in the structure. Complete comparison with other alternatives, shear walls and BRBs, is presented in a separate document, **part A**.

Keywords: seismic dampers, friction damper, seismic retrofit, Yielding Restrained Brace (YRB)

INTRODUCTION

This document is the continuation of **part A** [1] and explains the methodology used for the retrofit of a major hotel project in the Mexican state of Nayarit. This is the highest seismic zone of the Mexican pacific and is categorized as such for the local regulation. The project consisted of a series of more than ten buildings from which three were existing buildings in reinforced concrete built in 2000, before the latest update of the Mexican code [1]. Engineers performed a linear analysis based on a force-based reduction factor that permitted a very accurate though straightforward implementation of the Yielding Restrained Brace (YRB) System. A YRB is an element composed of a common brace connected to a Ten-Co [10]. As its names indicates, it's a in-line Tension-Compression seismic brake (commonly labeled a damper), that when activated, either in tension or in compression, and even under cyclical loading will have the same behavior when the relationship between force and displacement is plotted. The concept of a YRB has been thoroughly explained in the **part A** [1], along with a complete description of the building structure, existing status, and the comparison between retrofitting alternatives where one proposal with shear walls and another with Buckling Restrained Braced Frames are compared with the YRB one.

This document also presents a detail description of the a NLTH analysis that was carried out to evaluate the predicted extent of damage in the structure and the accuracy of results obtained form the linear method applied.

Existing structure and seismic demand

The existing moment resisting frame structure (see Image 2) was 21.7m height with inter storey height of 3.4m and 4.3 in the last floor. Notable was the finding that during construction too many perforations had been done to some beams and slabs. After carrying out material testing, they decided to limit concrete capacity to 200kg/cm^2 and to consider the structure as moderately ductile. Existing foundation was formed by a single 100cm pile per column with 20m depth. With 15m width in the short sense, the structure was excessively flexible: expected spectral accelerations of 1.63g and a drift of approximately 0.03h, double the code allowance for such structures. For more details on the existing structure please refer to **part A** [1]

METHODOLOGY

For the structural analysis, a conventional force-based method was used. In this linear analysis, a damping-based reduction factor is used to reduce earthquake forces.

A R_d = 5 and R_o = 1.1 factor, had been calibrated and subsequently tested at full scale showing that the system was effective for the enhanced seismic response of multi-storey building applications in high seismic regions [9]. Researchers performed a comprehensive parametric study with extensive nonlinear response history analyses of SDOF composed of a Friction Braced

Frame acting in parallel with an elastic moment frame (MRF) under firm and soft soil conditions and representing buildings with periods between 0.5 to 2.5sec. Results of this analysis were later tested in the linear design and detailing of complete 3D models where a combined $R_d \times R_o = 5.5$ was used along with a MRF of 0.25x the capacity of the main system. Numeric results showed that the factor kept median building peak drift ratios between 1 and 1.5% for both soil conditions, being thus considered reasonable to limit structural and no-structural damage. Finally, numerical results were validated by the researchers on an extensive test program at full scale and sub-assembly configurations.

Criteria for the use of the methodology

To ensure that the assumptions of this methodology hold, engineers established the following criteria:

1. The seismic brake must have a stable hysteretic loop and a constant force throughout the entire displacement. Variations in the dynamic slip force should be within 10% at any point. This could only be achieved by a Ten-Co.

2. The system containing the YRB (SFRS in red in Image 1), must be detailed at 100% of the design shear and as the local code requires for Special Concentric Braced Frames.

3. In the building, the SFRS, must be accompanied by a back-up system (in blue in Image 13), detailed to resist independently at least 25% of the design shear, this system can be detailed as an Ordinary Moment Frame

4. Characteristics of each and every of the Ten-Co for the YRBs, particularly its force vs displacement relationship shall be confirmed by a rigorous testing program. This will help provide assurance of the design hypothesis. This program must at a minimum, test each single Ten-Co at 100% of the force and 100% real scale MCE displacements. In addition, at least 10% of randomly selected units must be re-tested in the presence of the Engineer of Record. The manufacturer should present results according to point 1 of this list.

5. Connections, braces, gusset plates, bolts and welding connections from the Ten-Co to the brace and the frame, and connecting the brace to the frame, shall be detailed as per local code requirement for Special or High Ductility at 1.3x the Ten-Co slip load. Slip load should be modified to at least 1.3x service load forces when necessary (gravity, wind, machinery vibration etc.)

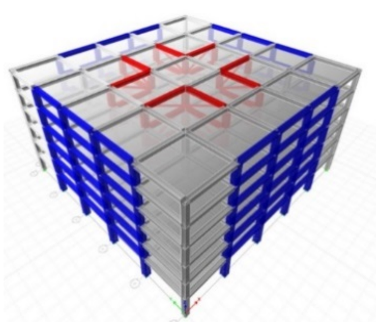


Image 1 Schematic representation of a complete model. SFRS as SCBF (in red interior) and OMF (in blue exterior) as a back-up system

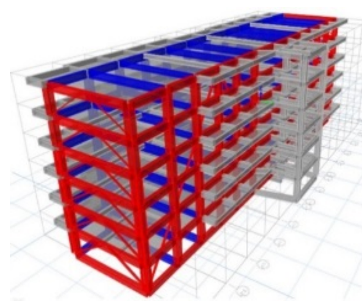


Image 2 Analysis models for the project. Complete system, (in red interior) and OMF (in blue exterior) as a back-up system

Methodology steps

Engineers followed the following steps trying to mirror [9]. These steps are applicable for a wide range of buildings if the five above criteria are met:

1. **Design forces:**
   With a seismic weight of W = 5678Tonf and the spectral acceleration value of 1.37g coming from the 5% critical damping response spectra for a period of 0.9sec elastic shear was calculated. Then, this force was reduced by $R_d \times R_o = 5 \times 1.1 = 5.5$, in addition $5.5 \times 0.8 = 4.4$ revised due to irregularity. According to local code [1] the building shape was categorized as irregular. The static reference base shear was $V = S_a(T)W / (R_d \times R_o) = 1.37 \times 5768\text{Tonf} / 4.4 = 1796\text{Tonf}$ (1). From there, a minimum base shear for design was computed at $0.8V = V_D = 1431\text{Tonf}$ (2), since response spectrum was being used for the analysis [1]. In that order, design forces will be $V_D = 1431\text{Tonf}$ (3) for design of the SCBF representing the YRB system in the model (red in Image 14) and $0.25V_D = 358\text{Tonf}$ (4), for the back-up system (blue in Image 14)

2. Analysis and Design of the SFRS

The system with YRBs is modelled as a Specially Concentrically Braced Frame system (SCBF) using the reduction factor applied in (3) including an accidental eccentricity of 10% in the horizontal plane. Inter-storey drift limits revision was performed so that the SCBF alone (red in Image 14) was able to limit them up to 0.015h. Detailing and selection of the braces in the analytical model followed provisions of the [18] [19].

3. Analysis and design of the Back-up system

This system (in blue in Image 14) was designed for the effects of (4) as an Ordinary Moment Frame (OMF) with 10% accidental eccentricity in the horizontal plane. Inter-storey drift limits revision was performed so that by itself was able to limit them up to 0.015h. Drifts found by the result (4) where scaled up by $R_d \times R_o = 5.5 \times 0.8 = 4.4$ for the revision. This was indicated as such for an OMF by the local code [1].

4. YRB design

a. Ten-Co slip loads and strokes

For the concept of the YRB to work the Ten-Co must 1) activate before that the brace reaches its yielding/buckling point and 2) provide as a minimum the same effective stiffness the brace provided in the analytical model in step 2 (red in Image 14). Therefore, slip loads were selected as the design force of the braces in step 2 rounded down to the nearest multiple of 50kN. The manufacturer recommended this approximation to find economies of scale.

For the stroke, engineers used as preliminary reference the inter-storey drift limits they have used for design, 0.015h. A simple kinematics formula, $\cos(\theta) \times 0.015h$, where θ was the angle of the brace with the horizontal plane the displacement/deformation (stroke) in the Ten-Co could be estimated. For an inter-storey height of 3.5m (4.3m in the last floor), the short side (Y) presented the largest displacements in the linear analysis of step 2. The angle of the YRB with the slab was 32 degrees. Using that floor as reference, $\cos(32°) \times 0.015h = \cos(32°) \times 0.015 \times 3.5m = 0.0441m = 4.41cm$ (5)

The manufacturer suggested a safety margin of 1.3x displacement producing a value of 5.73cm. A total of 48 Ten-Co, between 26Tonf and 376Tonf of slip load and with strokes available rounded to 6cm were selected to form the YRB to be installed.

b. Braces and connections

Actual braces forming the YRB, were Specially detailed following [18] at a force 1.3x the Ten-Co slip load. The same criteria were used for elements (gusset plates, bolts, welding, etc.) connecting the Ten-Co with the brace and the whole YRB with the frame. In the case of these later elements, they were also designed for the actual expected buckling of the brace and not the nominal [18] capacity. Connecting elements are therefore designed to fail last. This safety factor tries to account for variables affecting the brace yielding point as it happens for any steel elements, one of them being strain rate [4] [3].

5. Stiffness relationship

To find better efficiencies, manufacturer's specialists recommended, based in observation and past research, to keep stiffness relationship approximately constant at every floor. This is the relationship between the lateral stiffness of the SFRS and the lateral stiffness of the back-up system or other redundant, or existing systems already in place, referred to as the "bare" frame. It has been found that values between 2.33 and 5 can affect the system such that the optimum slip load can be within reach [20]. In other words, efficiencies in the integration of the system are more likely to exists when the braces' lateral stiffness in a floor are 0.6-0.9 of the total floor stiffness. Braces' stiffness can be easy calculated by $(k=AE/L)\ \cos^2\theta \times \#$ of braces in one direction; where θ is the angle of the YRB with the slab, L is the total length of the YRB (including the Ten-Co length), A and E are the cross section of the actual brace and the Young axial modulus of steel respectively. The total floor stiffness is the lateral stiffness of all elements participating in lateral resistance in that direction, including the YRBs. Since it's natural that this number not to be perfectly constant due to practical purposes (e.g. standard brace sizes) , the specialists recommended to be always higher in the floor above when differences exists to encourage TenCo activation from top to bottom in the case they activated at different instants (see Image 15 for an example). This was easily controlled with the selection of the braces in step 4b.

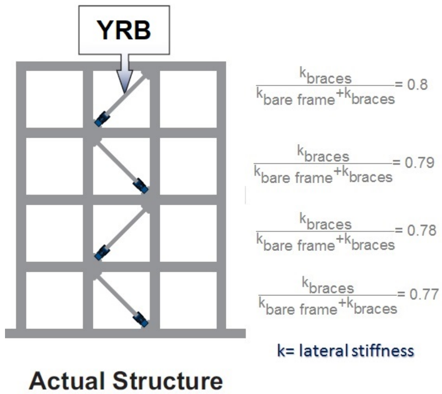
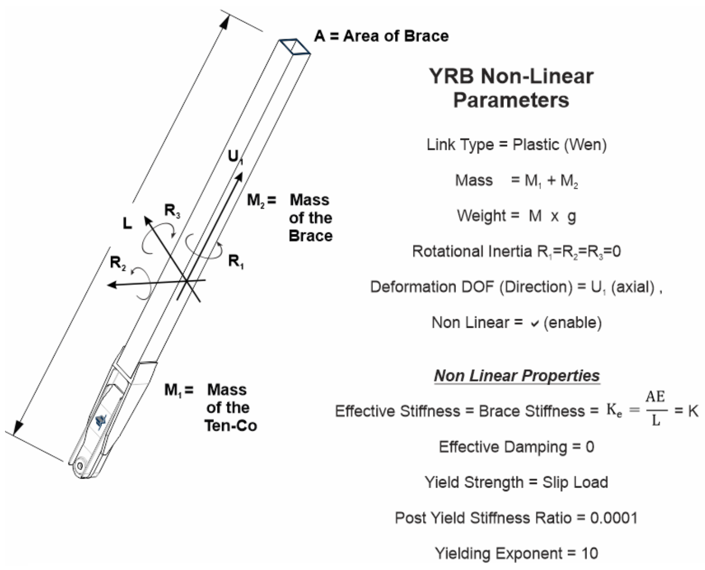
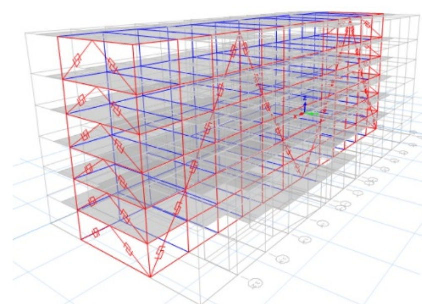


Image 3. Example of stiffness relationship. Values indicative only

Methodology's numeric validation

To test the performance of the designed structure using the methodology, engineers performed a NLTH analysis. Braces were replaced in the model by nonlinear links that would have a bilinear behavior, particularly represented by the [21] model, recommended by the manufacturer and available in most popular design softwares. At some fictitious yield force, the link will have a stiffness slope close to zero representing the rectangular hysteretic loop of the YRB (see Image 10 right). The slip load chosen for the Ten-Co in step 4a is the vertex. Please refer to Image 16 (left) to see general linear and non-linear parameter for the links.





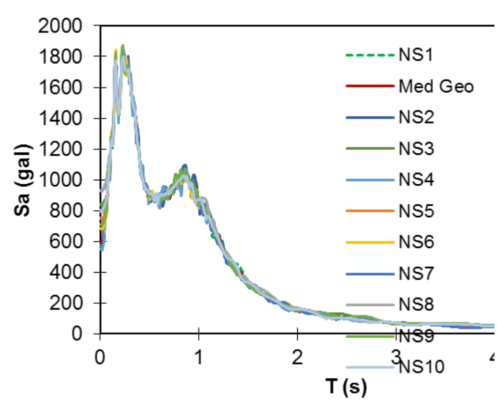


Image 4. Parameters for a link representing the YRB (left). Model with links incorporated (right top). Ground motions used (right bottom)

Plastic hinges were also created for beams and columns based on the detailing performed in steps 2 and 3 of the methodology but also establishing control points for plastic deformation based on tables 6-7 and 6-8 of [22]. Image 17 shows hinge modelling parameters for a column of the SFRS. More information about the detailing of reinforced elements is found the **part A** [1]

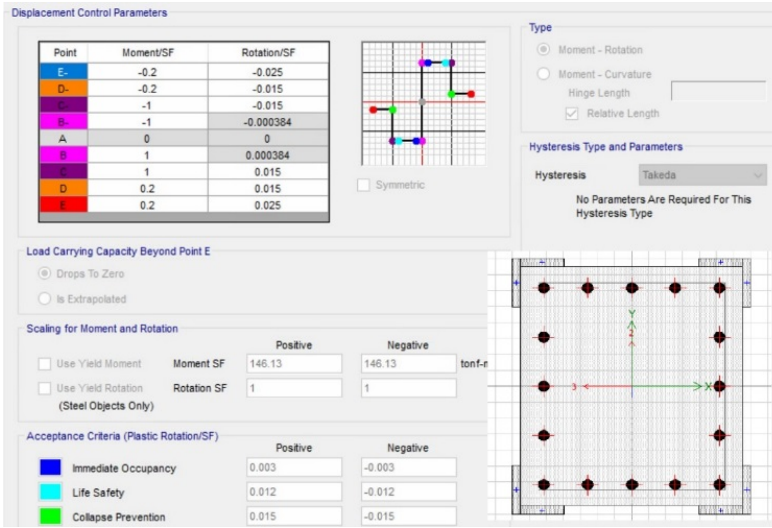


Image 5. Plastic hinge parameters of a column of the SFRS

A set of 10 specific site ground motions was used in each direction to create a more realistic response spectra at 5% of damping (see Image 16 right bottom).

RESULTS

Design forces

Maximum shear forces were in line with design forces used in (3) with 5NS and 3EW being the ground motions that demanded the most in the structure with 1342Tonf and 1567Tonf for the short side (Y) and long side (X) respectively. 3EW was the only ground motion that touched the limit of design capacity of the structure (see Image 18)

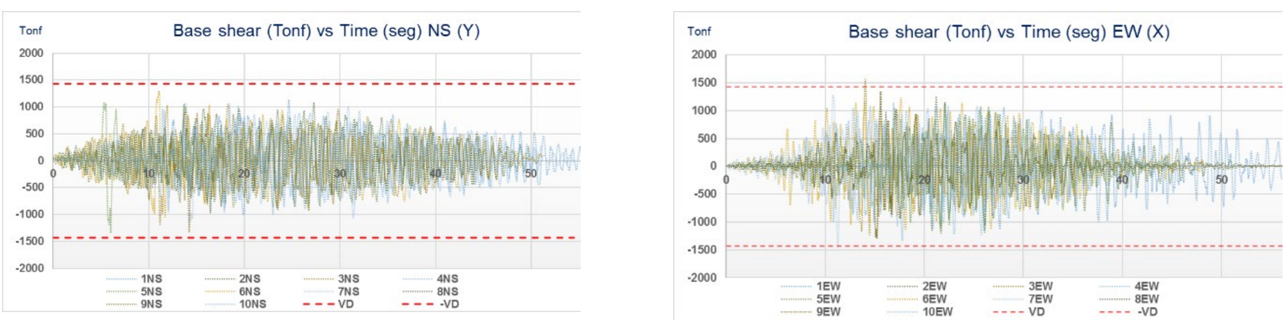


Image 6. Max shear forces at the base for all ground motions, Y sense (top), X sense (bottom)

Inelastic Drift

As assumed in the Methodology for linear analysis, inelastic drift was below 0.015h in the short sense (Y) with 0.0143h from the ground motion 6NS and was significantly lower for the long side (X) at 0.083h with ground motion 3EW (see Image 19). Consequently, permanent drift was below the assumed limit of 0.005h. For example, for the ground motion that caused the highest drift, 6NS (load case THY6-1), in the short sense (Y), permanent displacement was 0.55cm with respect to the base or 0.34cm relative to the storey below, being around 0.001h. This measure was taken from the floor with the highest flexibility and the joint with the worst plastic deformation. Additionally, residual displacement at the roof was 1.87cm with respect to the base or 0.35cm relative to the storey below, being around 0.0008h (see Image 20)

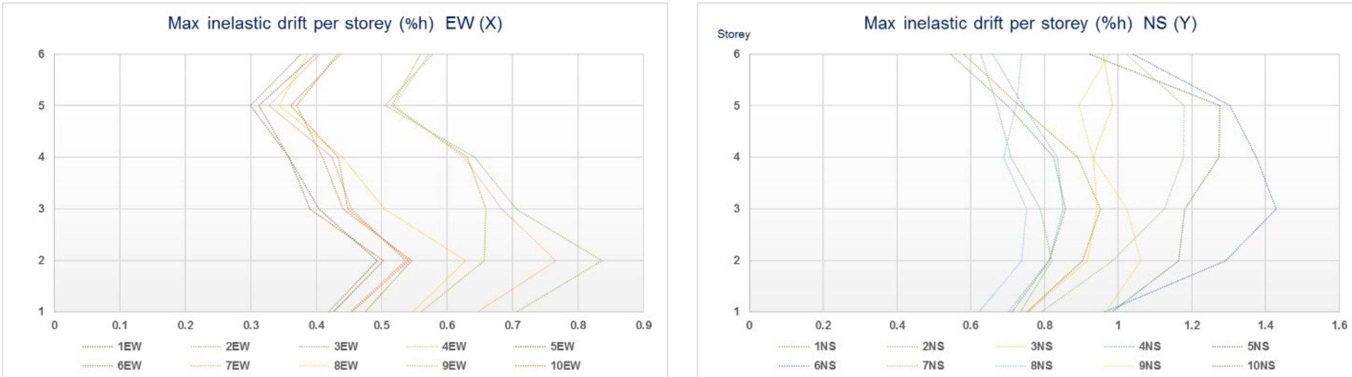


Image 7. Max inelastic drift short (Y) side (bottom), long side (X) (top).

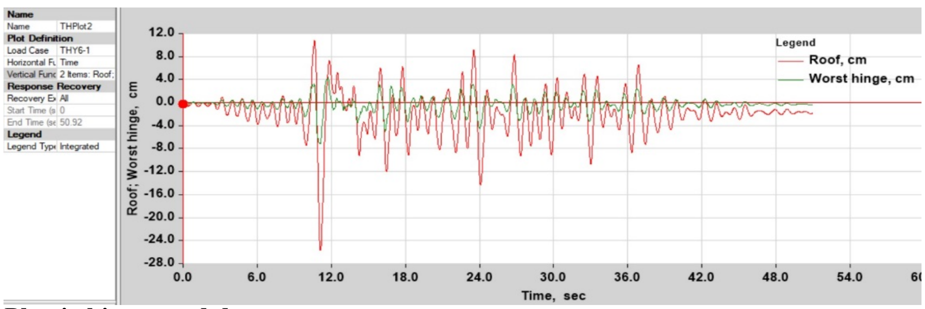


Image 8 Displacements at the roof and storey 2, ground motion 6NS (Y)

Plastic hinges and damage

Most of highest rotations in elements were within the Immediate Occupation (IO) performance and before of control point C (0.015rad for columns and 0.02rad for beams) where capacity starts decreasing (see Image 21). In line with inelastic displacements, highest rotations appear in the short sense (Y) at the same floor maximum displacements were registered with ground motion 6NS (see Image 19). One beam hinge in this floor just barely entered the Life Safety (LF) to Collapse Prevention (CP) state with -0.01154rad . Moreover, this hinge is capable to finish the most demanding ground motion with a plastic rotation of 0.005837rad consistent with permanent drifts above.

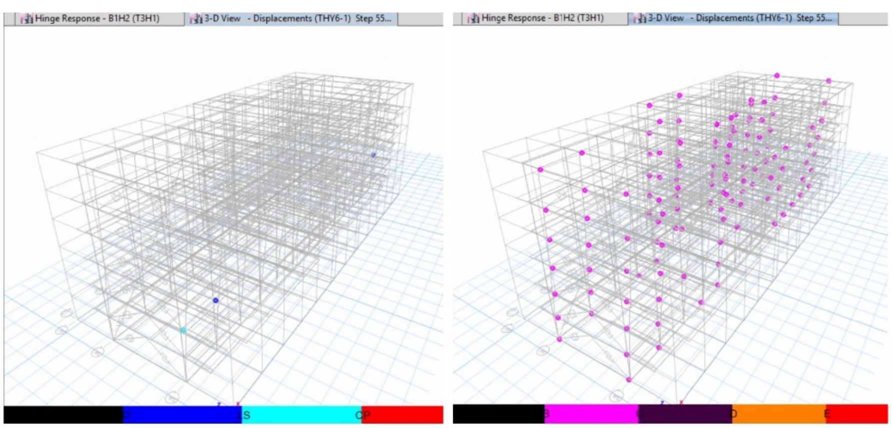


Image 9 Hinge states (left) and Hinge status (right) during 6NS ground motion

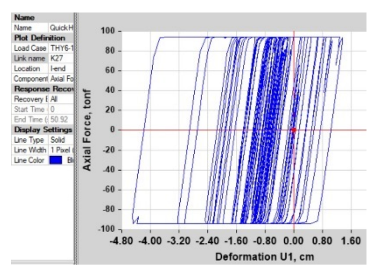


Image 10 Ten-Co with max displacement, 4.37cm

Energy Dissipation

Work done by the YRB system was around 60% of the input, which was a known quality of the system and in line with observed past results [15]. However, this performance had not yet been shown after applying this simplified linear method. The fact that the structure only has 40% of the input to deal with can be seen in the results shown above for inelastic drifts and damage states.

It's very important to highlight that, energy dissipation is very similar, even when two very different ground motions are compared. For example, ground motion 6NS (load case THY6-1) caused the highest displacements in the system (see Image

19) and 3EW (load case THX3-1) caused the highest shear (see Image 18) but with half of the displacement. In both cases, with very dissimilar displacement input in the structure, the energy dissipated is alike (see Image 23).

The fact that friction-based mechanisms have been categorized as displacement dependant [23] has helped create the misconception that significant displacement is needed in order to dissipate energy. Although, an efficient friction device is certainly independent from velocity and rotation angle, they are not necessary dependent on high displacements since the slip load can be set at any force depending on required displacement. For such a device, the force vs displacement relationship is not proportional. This is the main advantage of the YRB, decoupling stiffness from yielding point. In contrast, yielding dependant mechanisms, such as BRBs or TADAS, are absolutely dependant on displacement since they cannot modify the yielding point, inevitably increasing the stiffness of the structure, attracting more accelerations and not being able to dissipate input energy in low displacement structures, such as those with masonry walls.

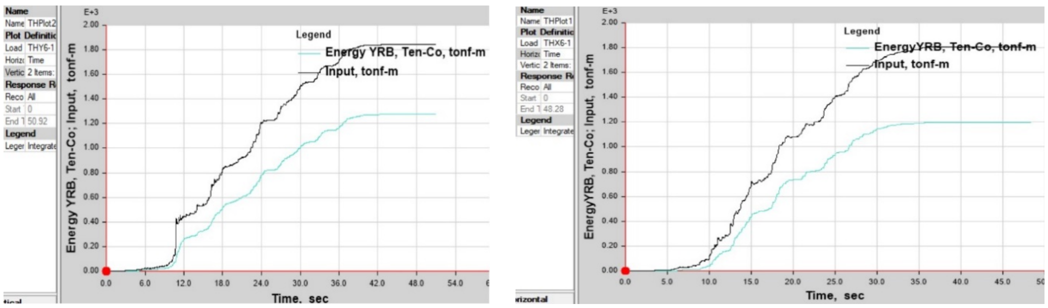


Image 11. Energy dissipated ground motion 6NS (Y) at the left, and 3EW (X) at the right

Had this project needed to protect non-isolated masonry walls, the long side (X) would be in decent state with more than 80% of displacement being below of 0.006h drift in the event of the Maximum Credible Earthquake (MCE) (2% probability in 50 years). In the short sense (Y) though, adjustments to the slip load would be needed to reduce max displacements to acceptable limits while still dissipating an important amount of the input.

Displacements in the Ten-Co

Displacements in the YRB that would be recorded on the Ten-Co were very well within the limits of what was calculate with the simplistic formula (5). This process, described in step 4a of the Methodology, showed to be very conservative with he maximum Ten-Co displacement at 4.37cm for the ground motion 6NS in the short (Y) sense. This device was located, again, in the frame at the floor with max displacement, identified as link 27 (see Image 22) . In the event of the MCE, the device would still have stroke available until 6cm.

CONCLUSIONS

The implementation of the force-based linear method presented in the Methodology with an $R_d \times R_o = 5.5$ presented very decent results regarding the calculated damage in the structure after the MCE event (2% probability in 50 years). The method can be followed with common local codes guidelines. However, criteria for the use of this methodology (see page 2) must be respected to ensure that the analysis and design hypotheses hold in the actual structure's behavior.

Regarding the methodology used, engineers concluded that:

1. If criteria for using the methodology are respected, the structure will likely have maximum inelastic drifts and residual drift below 0.015h and 0.005h respectively therefore showing considerably better post-earthquake condition than structures using similar guidelines with ductility-based reduction factors

2. For the YRB and its corresponding reduction factor to produce expected results, it should be equipped with a Ten-Co so that the energy dissipation assumed by the factor is available. Therefore, conditions enabling the YRB concept should be strictly met (see page 5 of the **part A** [1]), otherwise there may be excessive variation in the design forces assumed

3. The use of the YRB was able to dissipate a considerable amount of energy (60%) in the structure with very little damage induced. It was able to do so, at the same levels, under different ground motions characteristics (see page 6) such as long and short seismic displacements and under different accelerations

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