PRECAST BUILDINGS EQUIPPED WITH SLB SEISMIC DEVICES

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Abstract

This work initially describes Shear Link Bozzo or SLB devices. These devices provide more than 104 single dissipators that combined together in parallel result in a very large set of potential structural solutions. All devices are stiff but ductile start yielding at displacements as low as 0.20mm. The paper includes modelling aspects and two automated selection procedures called: (1) direct iterative procedure and (2) inverse or fixed force iterative procedure. Both procedures are implemented in an automated selection program in a plug-in for ETABS commercial software. Under the premise that a large number of devices are required in order to assess a significant structural benefit, their unit cost is very cheap.

This technique is applied to a five-story precast reinforced concrete building frame, called SLB Building. The building is made up of eleven 40cmx40cm section columns and it has hinged beams. Consequently, the structure without dissipators is very flexible but adding these devices the period shortens drastically. The interior floors are diaphanous so the structural configuration may have diverse occupancy such as housing, offices or hospitals. This building was equipped with 80 small SLB devices showing its performance for the maximum earthquake of the Peruvian seismic code. The analysis was nonlinear for the ten time-history records compatible with the S1 soil spectrum in Zone 4.

All seismic energy dissipation was concentrated in the dissipation devices so there would be no structural damage. In addition, the levels of non-structural damage were controlled with initial stiffness of these devices since lateral displacements were reduced to levels below 0.007, for the maximum Peruvian design spectra. Two structural configurations were studied for the columns: fixed or hinged at their bases. The performance of both solutions is compared in terms of drifts, floors accelerations, and base shear. In all cases, the accelerations show a clear reduction with height achieved by the energy dissipated by the devices. The story shear forces divided by the structural weight is calculated for each signal resulting in values between 0.13 and 0.27 and consequently the force reduction factor (R) is between 5 and 7 without any structural and non-structural damage.

It is concluded that this precast solution equipped with SLB devices is optimal for homes, schools and hospitals. Its structural performance is comparable to base isolation but with its significant simplicity, low cost and minimum or no special maintenance required using affordable technology for a widespread type of conventional buildings. Finally, the proposed general system has less uncertainty than base isolation since over strength is controlled by simple previous tensile tests for the steel plates, adjusting final device dimensions before their manufacturing and without very large base displacements.

Keywords: precast buildings; energy dissipators; SLB devices; seismic precast buildings.

1. Introduction

In recent years, a good part of the research for structures in seismic zones has focused on the development of seismic-resistant control systems, both for new constructions and for the rehabilitation of structures damaged by earthquakes. On the other hand, prefabricated structures have been developed for economic and speed of construction, among other factors, being a constantly evolving technology. These two concepts, seismic protection (which for this investigation are energy dissipators) and prefabrication, can be oriented to obtain safe (using dissipators) and economic structures (using precast construction), which is one of the main objectives of this research.

Earthquake engineering has developed new technologies and solutions for seismic protection of structures with a philosophy different from that of the traditional designs. The new seismic protection systems not only prevent the collapse of structures in the event of severe earthquakes, but also can minimize damage to non-structural elements by protecting the contents of the structure [1, 2]. There is a large classification of seismic protection systems than designers can choose according to specific demands [3]. However, this article focussed on Shear Link Bozzo (SLB) seismic protection system because they have more than 104 single devices to choose from and because they are all initially all very stiff but ductile (start yielding at 0.20mm displacement). These characteristics make SLBs suitable for bare very flexible structure such as precast structures [4]. Prior manufacturing any device the steel base plate is tested in order to adjust final dimensions according to real stress-strain material curves thus minimizing over strength problems.

The use of prefabricated buildings in high seismicity areas has not been as widespread worldwide as passive seismic protection systems, among other reasons, because there is some professional rejection for considering them "unsafe". However, their structural seismic supposedly weakness cannot be generalized since there are satisfactory experiences of prefabricated structures after a seismic event. However, particular solutions for precast construction have been development. Among others, the development of the Technical University of Istanbul, together with the Consultant in Engineering PROEMER [5] considered friction dissipators in prefabricated industrial facilities. On the other hand, studies at the Politecnico di Milano showed the advantages of this combination of systems from the numerical point of view [6]. Previously, they investigated the use of dissipators in structures, emphasizing their general behaviour. Research has also been carried out on prefabrication with dissipative systems: dissipation in a prestressed non-adherent connection [7]. Later, there are other investigations that have focused on the behaviour of the connections in prefabricated structures [8] and precast panels with friction connections [9]. A significant difference in this research is that the precast structure, without devices, is intentionally very flexible (all the beams are hinged connected) with the final objective of taking maximum advantage of stiff dissipators under seismic loads [4].

Investigating the use of prefabricated structures incorporating dissipators bring benefits both in the part of the seismic behaviour and in construction procedures. The possible savings in construction time and better final quality compared to conventional “on site” structures make them appealing for different structures such as social housing, schools, among others [10]. Furthermore, it is particularly interesting in Peru given its complex topography and inherent difficulties in local construction, in addition to the huge need for economic social protection homes. Building social housing and schools, but with the vanguard of structural security, is the final objective of this research.

2. SLB system

2.1 Description of the SLB system

SLB is a passive seismic protection system made of commercial steel. The steel hysterical behaviour is used to deform in their inelastic range yielding from very small displacements such as 0.20mm. Unlike other metallic systems the SLB have a double deformation mode to dissipate energy, as shown in Fig 1. The main deformation corresponds to those caused by shear stresses at the “dissipative windows” but after their breaking point they subsequently dissipate due to bending stresses at the stiffener frame.

There are currently two design tables for SLB devices: a) steel A36 (SLB) and b) steel Grade 50 (ESLB). Each of the tables has 52 dissipators where the dimensions and geometry of the devices vary according to the yielding forces [11]. The vertical dimension of the devices is set at 235mm, including the connection, see Fig. 2. These devices are designated as SLB (ed) X_Y where (ed) depends of the thickness of the dissipator, X indicates the width of the device in cm and Y indicates the thickness of the dissipative windows in mm. The X parameter varies between 60 and 500 mm and the Y parameter can be 2, 3, 4, 5 or 6 mm for each given value of X. Another significant feature is that the *battlement* connection avoids transferring axial force to the device. This no-axial-force-transferred characteristic has the advantage that the walls or supporting braces do not necessarily have to be aligned in height so they could be added where the architectonical project allows. Besides under vertical seismic movements they dissipate energy.

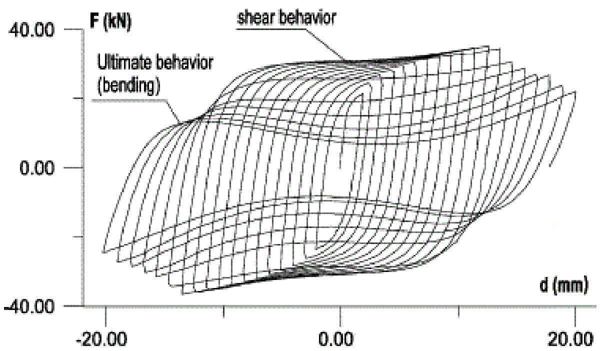


Fig. 1– Hysterical curve of the dissipater SLB with its double deformation mode.

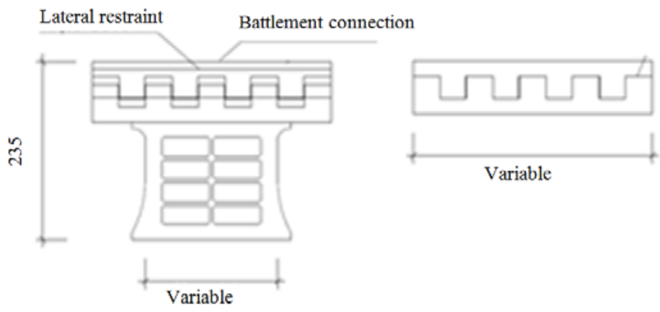


Fig. 2–Geometry of SLB devices

2.2. Procedure for the Design of SLB Seismic Dissipators

There are two methods for the selection of SLB devices actually incorporated in an ETABS application. Both are based on elastic modal analysis, which replace procedures that make use of nonlinear time history analysis, thus achieving significant savings in computational time for the solution (see Figure 3). This is particularly useful in preliminary design where most important decisions are taken.

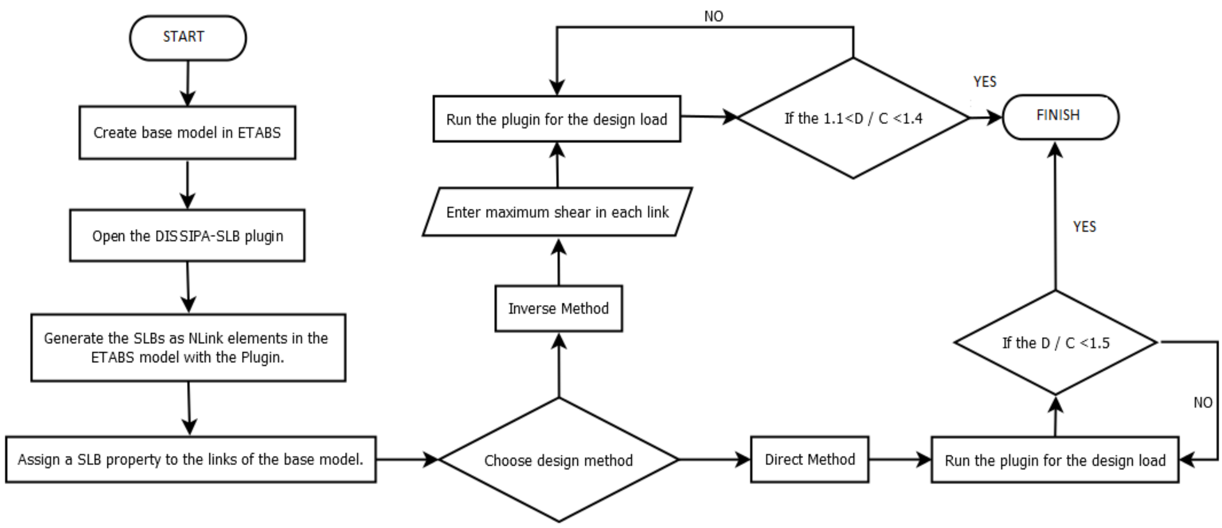


Fig. 3–Flowchart for the design of SLB dissipators using plugin for ETABS Software

These two processes have been automated in DISSIPA SLB plug-in [12]. This application requires the results of the seismic analysis process iterating in ETABS model by accurate information transfer between models. The plug-in currently supports two automated selection procedures: (1) Direct and (2) Reverse iteration methods, as well as the automatic loading of single, double, triple and quadruple dissipators from the two aforementioned design tables (section 2.1).

2.2.1. Direct Iteration Method

The direct method consists in iterating a group of devices previously defined by means of a series of seismic analysis, of the modal response spectrum type, until reaching a shear demand compatible with the capacity of the device. It is required that the ratio between the acting shear and the yield force of the device be less than a certain demand/capacity ratio typically assumed as 1.5. This ratio is considered by various cumulative factors such as the kinematic hardening of the steel or its greater resistance to dynamic loads as well as for the linear modal analysis procedure. These factors can only be considered precisely through a nonlinear time history analysis, which is recommended, as verification, at the end of the procedure. The direct method is performed automatically for several devices using DISSIPA SLB plug-in.

2.2.2. Inverse Iteration Method

The "fixed force" procedure or "reverse" iterative procedure serves to limit the dimensions of the supporting device elements such as uncouple concrete walls or steel braces, as well as limiting their size. Thicknesses of uncoupled concrete walls greater than 300 mm are usually regarded as excessive and their cost, as well as the buckling of steel supporting diagonal bracing, can be determining factors for setting a "maximum force". For example, according to ACI code, the shear capacity of a structural wall (obtained considering a certain value of f'c, length and thickness) is fixed and according to this capacity, the maximum force value that could act on the devices. Due to the fact that the special battlement connection does not transfer axial force there is a direct isostatic equilibrium relation between the dissipators shear forced and the shear at the supporting concrete wall or axial force at brace system. Precisely due to this fact the walls are called “uncoupled”. In the inverse method the shear force is fixed and, therefore, the iteration consists typically in reducing the size of the device in the numerical model (and not in reality) in order to calibrate such transferred shear force. Take into account that the analysis is linear elastic so the shear force transferred by the devices in the numerical model is not limited. Consequently, the shear force at the device could be much larger than the real capacity of the actual chosen devices conducting to an unsafe preliminary design.

2.3. SLB device modelling

To represent steel yielding dissipators such as SLBs, many programs have included nonlinear properties such as Multi-Linear Plastic and Plastic (Wen Model) simulating the behaviour of such devices with great precision [13].The plastic (Wen Model) property that makes use of Bouc-Wen's hysterical model represents the transition of linearity and non-linearity in a more real way and resembles the hysterical behaviour of steel dissipators using a relatively simple procedure.

For the correct application of the DISSIPA SLB plug-in the local axis 1 of the NLink (Nonlinear Link) must be in the direction of the location of the “battlement” connection. In the case of the uncoupled walls, the battlement connection is located on the beam because the wall could embed the base of the dissipater so the Nlink will be modelled bottom up. However, in the case of a chevron braces to support the battlement connection, it will rest on the diagonals (where the “battlement” element or zero moment point is) and the Nlink will be modelled top bottom.

3. Description of the five-story precast SLB building

3.1. Architecture

The analysed structure is a 5-story building with typical floors consisting of 4 apartments per level. The total area is 300 m^2 per floor, as shows in Fig 4, so each apartment has $70m^2$. It has been proposed as a technological, economic and functional solution for low income families in Peru. The interior floors are diaphanous, so the structural configuration could have a diverse occupation, such as schools, offices or hospitals. The minimum number of precast structural elements to move is desirable because it reduces construction time. The maximum weight for any precast element is less than 50kN, so they can be transported with simple moving truck cranes.

3.2. Structural System

The configuration is a frame systems based on concrete precast columns and beams which also incorporates semi- precast slabs. The connections of the beams with columns have been proposed articulated at their ends, so the structural design of these elements is governed by gravity loads.

It is commonly assumed that precast concrete buildings have less ductility capacity than conventional concrete structures. In fact, one of the most widespread structural schemes for prefabricated buildings is characterized by all beams simply supported by columns, resulting in a static scheme with a low number of static indeterminacies [14]. This means that plastic hinges can only be formed at the bottom of the columns, while for a similar but conventional reinforced concrete structure the number of plastic hinges would be much higher (they can be developed at the ends of all beams).

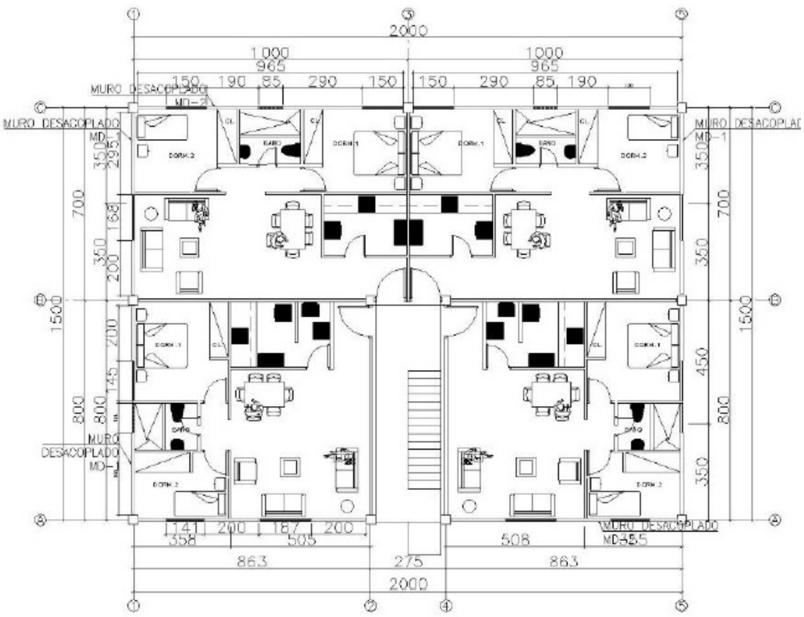


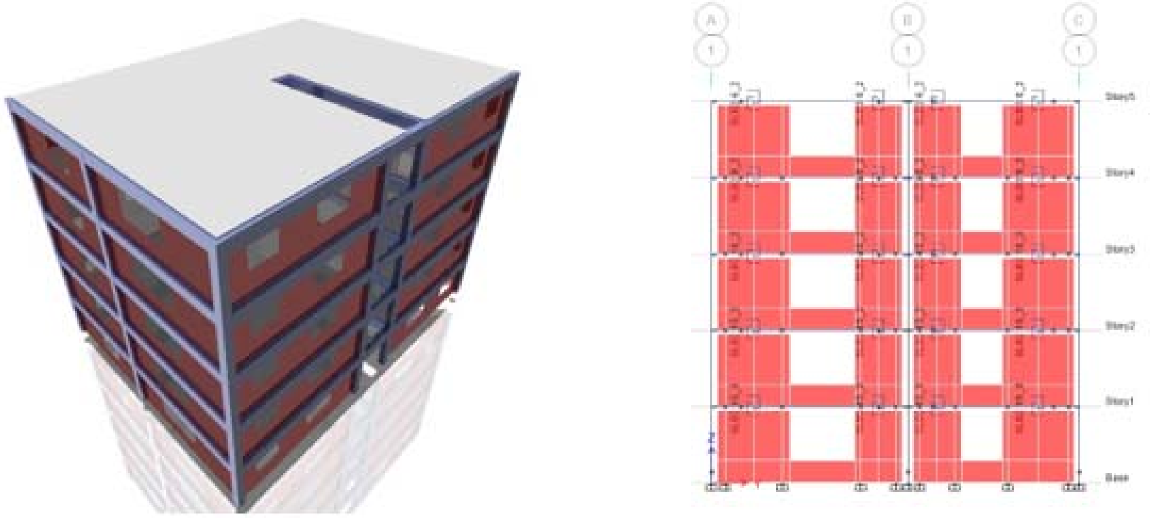
Fig. 4–Typical architectural distribution of the precast building

Structural analysis was performed using ETABS software with the ability to evaluate the non-linearity of SLB devices. To control the lateral displacements of earthquake in both directions, a wall system with dissipators has been considered (Fig. 5). The walls are typically 12cm thickness and they are connected to the beams by steel rods and plated. The vertical loads are assigned to beams, columns and slabs, while the seismic actions are completely transferred by the dissipative system. Regarding the connection at the base of the columns, two cases are compared: fixed and hinged.

Description of the structural elements:

- Beams: There are 15 beams per level and their connection with the columns is hinged supported on corbels. Consequently they are mainly designed for gravity loads since they do not transfer significant seismic action. The maximum length is approximately 9m with a total depth of 70cm (55cm + 25cm). The beams incorporate prestressed active steel for a better control of vertical deformation and depth reduction. The beams are built in two stages: initially, their depth is 55cm, leaving 25cm free on top to place the slab and, at the same time, they are self-supporting for faster construction. Their maximum unit weight is less than 50kN.

- Columns: There are 11 columns with a constant square section of 40cmx40cm placed according to the architectural design in a frame system. Their concrete has a compressive strength of 35 MPa while the steel reinforcement has a yielding strength of 420 MPa. Their maximum unit weight is also less than 50kN.



a) b)

Fig. 5–3D View and elevation of precast SLB building

- Slabs: The slabs are semi-prefabricated, lightened and unidirectional with a total thickness of 25cm (5cm+15cm+5cm) for spans of 8m. This system consists of a prefabricated 100cm width pre-slab with a thickness of 5cm in its lower part incorporating concrete on the ribs (in order to be also self supported). The system has the advantages of reducing on site formwork and controlling vertical deformation. There are 39 slabs per level resulting in a relatively low number of crane movements. Furthermore the system allows incorporating directly line services and the facilities. Finally, the on top 5cm cast in place concrete completes the structure generating a rigid diaphragm.

- Uncoupled walls: These are slender concrete walls of 12cm thickness separated from the main structure by 30mm seismic joints. They are supported by the beams connected in the bottom part, while in the upper part they are linked to SLB devices. The interior steel reinforcement consists of diagonal bars also used for the anchorage of the seismic devices. The walls allow increasing significantly the lateral stiffness of the building providing large ductility at the devices. These walls have been placed mainly along the perimeter of the building. In this way, the walls and SLB devices are the only elements which resist the seismic force.

- SLB Dissipators: This building was equipped 16 devices per level, so there are, in total, 80 small SLB devices (see Table 1).

Table 1 – SLB devices Distribution per level

Level	Fixed at the base of columns	Hinged at the base of columns
1st	SLB2 15_4	SLB2 15_3
2nd	SLB2 15_2	SLB2 15_2
3rd	SLB2 10_3	SLB2 10_2
4th	SLB2 8_4	SLB2 8_2
5th	SLB2 6_2	SLB2 6_2

4. Nonlinear Analysis Time History

The performance assessments are made through non-linear analysis with ten time history records compatible with S1 soil spectrum in Zone 4 (Fig.6) without any ductility reduction. The signals are applied in two horizontal directions and according to current Peruvian code. The SLB devices were initially selected to satisfy interstory drift requirements (<0.00875) and subsequently the columns were designed according to the Peruvian reinforced concrete code E.060 [15]. The mathematical model has a specified damping of 2% in relation to the periods of the structure. The first period was in the Y direction (0.483s as shown in Table 2) and the second one in the X direction (0.461s). The structure is quite symmetric and clearly the first mode in each direction controls the response with almost 80% of the participative mass. The time-history nonlinear analysis was solved with the time integration parameters of Hilber Hughes Taylor using ETABS software.

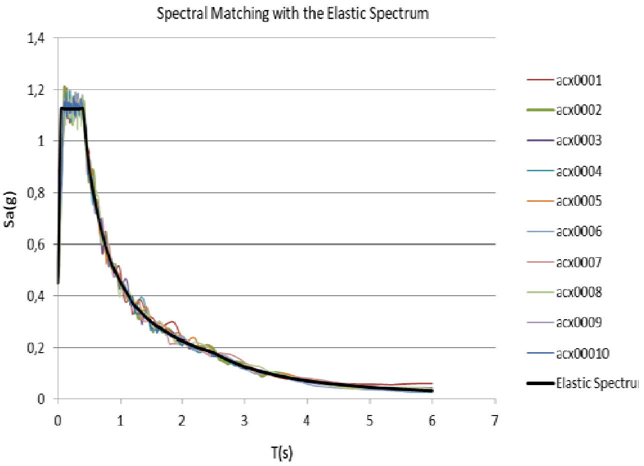


Fig. 6 – Ten seismic signals (Peruvian Spectrum)

Table 2 – Modal properties base-fixed building

Mode	Period sec	UX	UY	Sum UX	Sum UY
1	0.483	0.0001	0.7888	0.0001	0.7888
2	0.461	0.774	0.000	0.774	0.789
3	0.333	0.009	0.000	0.783	0.789
4	0.189	0.000	0.125	0.783	0.914
5	0.182	0.128	0.000	0.911	0.914
6	0.132	0.000	0.000	0.911	0.914
7	0.122	0.000	0.042	0.912	0.956
8	0.118	0.045	0.000	0.956	0.956
9	0.114	0.000	0.000	0.956	0.956
10	0.11	0.000	0.000	0.956	0.956
11	0.099	0.001	0.000	0.957	0.956
12	0.099	0.000	0.000	0.957	0.956
13	0.099	0.001	0.000	0.958	0.956
14	0.099	0.000	0.000	0.959	0.956
15	0.098	0.000	0.000	0.959	0.957

5. Numerical performance of the SLB building

The performance for ten seismic signals and for each direction is reported in terms of (1) diaphragm accelerations; (2) maximum interstory drift; (3) maximum deformation of the devices and (4) maximum base shear. The response for two basic global models is compared: columns fixed or hinged at their bases.

5.1. Precast SLB Building with fixed connection at the base

This is the most standard precast solution consisting in leaving holes at the foundation in order to accommodate vertical steel column reinforcement and casting in place expansive concrete at the joint. Table 3 shows peak ground acceleration (PGA) values and time history response analysis for different parameters along each horizontal direction. The average values of the maximum base shear in each direction is Vx,max=2456.98kN and Vy,max=2426.98kN. The seismic coefficient are obtained dividing these shear values with the seismic weight of the structure which is Pz=14075.72kN. The resulting maximum average value results in 0,17 which is a particularly low value, taking into account no structural damage is allowed. The maximum device displacement is 41.32 mm and its average value 25.61mm which is smaller than the 30 mm standard actual device limit. The table also includes maximum drifts for each signal and it can be observed that the acx0006 signal generates most critical results. Although the signal is short with duration of only 10.23s, it generates a larger response than, for example, signal acx0002 with a twice duration of 20.47s. This result is particularly interesting because it shows that the proposed precast building is robust under a variety of frequency contents. It also shows the need of nonlinear time history analysis compared to over simplify analysis procedures, such as linear equivalent damping.

Figure 7 shows the average drifts for the signals with values between 0.00441 and 0.00739. The line that stands out corresponds to the average drift and the vertical straight line is the Peruvian code limit which for this analysis type is 0.00875 [16]. It can be observed that the maximum drifts take place in the third level and all values are lower than the maximum code limit. Figure 8 shows the diaphragm accelerations which clearly reduce with height and their average values range between 0.35g and 0.56g in both directions.

Table 3 - Results corresponding to model with fixed connection at the base

Signal	PGA (g)	Duration (s)	Vx,max (kN)	Vy,max (kN)	Pz (KN)	Vx/Pz	Vy/Pz	Disp. Link mm	Drift, Xmax	Drift, Ymax
acx0001	0.891	10.23	2531.13	2507.604	14075.72	0.180	0.178	28.86	0.00870	0.00874
acx0002	0.506	20.47	2190.33	2209.365	14075.72	0.156	0.157	16.46	0.00473	0.00490
acx0003	0.546	10.23	2559.84	2504.170	14075.72	0.182	0.178	23.21	0.00733	0.00744
acx0004	0.551	10.23	2432.79	2398.521	14075.72	0.173	0.170	26.04	0.00781	0.00787
acx0005	0.477	20.47	2458.44	2440.194	14075.72	0.175	0.173	25.79	0.00759	0.00781
acx0006	0.555	10.23	2746.44	2741.413	14075.72	0.195	0.195	41.32	0.01258	0.01249
acx0007	0.496	10.23	2309.72	2255.313	14075.72	0.164	0.160	26.46	0.00766	0.00800
acx0008	0.476	20.47	2365.41	2315.635	14075.72	0.168	0.165	24.89	0.00753	0.00718
acx0009	0.492	40.95	2431.22	2361.507	14075.72	0.173	0.168	20.63	0.00639	0.00628
acx0010	0.597	40.95	2544.52	2536.079	14075.72	0.181	0.180	22.47	0.00662	0.00698
Average values			2456.98	2426.98	14075.72	0.17	14075.72	25.61	0.00769	0.00777

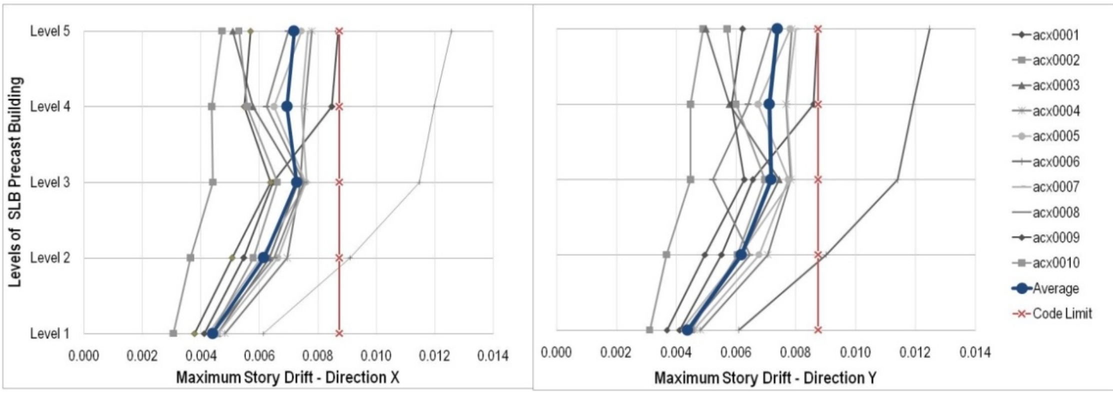


Fig. 7 – Maximum X, Y direction drifts per level and seismic signal. Building with base hinged connection

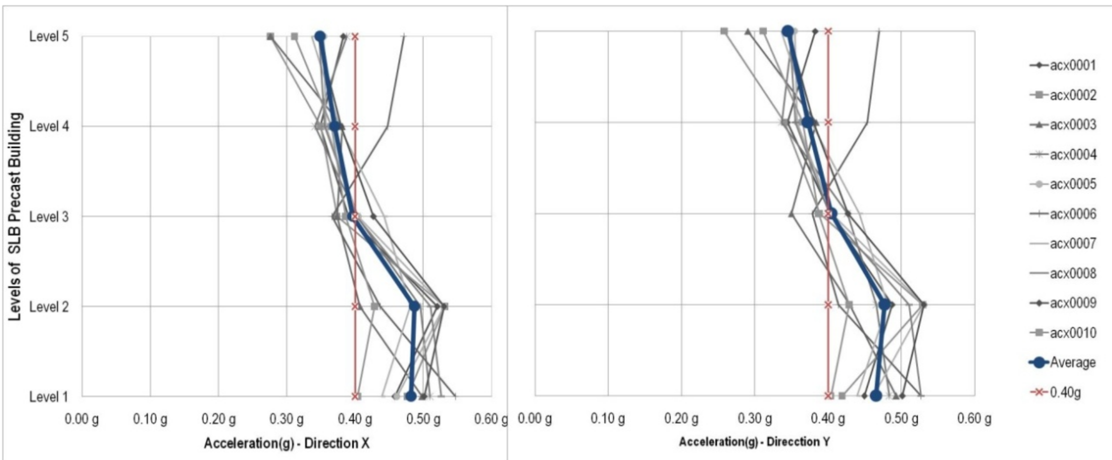


Fig. 8 – Maximum X, Y accelerations per level and seismic signal. Building with base fixed connection

5.2. Precast SLB Building with hinged connection at the base

This solution, with hinged columns at their base, is not standard since the system, without the devices, would be globally unstable (take into account all the beams are pinned end). However, due to the presence of the devices, the system is stable and furthermore the reduction of seismic forces is even larger than compared the previously presented fixed based solution.

Table 4 shows peak ground acceleration values and time history response analysis for different parameters along each horizontal direction. The average values of the maximum base shear in each direction is Vx,max=1828.22 kN and Vy,max=1800.26 kN. In both directions, it is observed a 26% reduction in base shear coefficient compared to the fixed base one. The resulting maximum average value for base shear coefficient results in 0.13 which is a particularly low value, taking into account all input energy is dissipated in the devices. The maximum device displacement is 39.97 mm and its average value 25.21 mm similar to previously reported values for the fixed base model. The table also includes maximum drifts for each signal and it can be observed that again acx0006 signal generates most critical results but the maximum values are similar than for the fixed base model. These results indicate that in the case of selecting the fixed base model in the event of a very strong unexpected input signal that originate plastic hinges at the columns base, the base shear would tend to reduce while the drift would remain controlled resulting in a robust structure.

Table 4 -Results corresponding to model with hinged connection at the base

Signal	PGA (g)	Duration (s)	Vx,max (kN)	Vy,max (kN)	Pz (KN)	Vx/Pz	Vy/Pz	Disp Link (mm)	Drift, Xmax	Drift, Ymax
acx0001	0.891	10.23	1735.82	1720.020	14075.72	0.123	0.122	29.36	0.00882	0.00890
acx0002	0.506	20.47	1527.45	1492.214	14075.72	0.109	0.106	16.32	0.00483	0.00497
acx0003	0.546	10.23	1935.12	1891.000	14075.72	0.137	0.134	24.13	0.00729	0.00744
acx0004	0.551	10.23	1952.43	1937.850	14075.72	0.139	0.138	26.49	0.00823	0.00835
acx0005	0.477	20.47	1863.79	1832.837	14075.72	0.132	0.130	25.92	0.00761	0.00784
acx0006	0.555	10.23	2244.40	2218.440	14075.72	0.159	0.158	39.97	0.01226	0.01222
acx0007	0.496	10.23	1693.11	1694.700	14075.72	0.120	0.120	23.60	0.00706	0.00729
acx0008	0.476	20.47	1746.48	1716.067	14075.72	0.124	0.122	23.38	0.00701	0.00714
acx0009	0.492	40.95	1725.01	1689.527	14075.72	0.123	0.120	21.58	0.00655	0.00658
acx0010	0.597	40.95	1858.62	1809.952	14075.72	0.132	0.129	21.36	0.00687	0.00693
Average values			1828.22	1800.26	14,075.72	0.13	14075.72	25.21	0.00765	0.00776

Figure 9 shows the average drifts for the signals with values between 0.0063 and 0.00758, in all cases, these are smaller than the limit of the Peruvian code (0.00875). The drift values are similar to the building with columns fixed at the base and again the maximum value is reported at the third level. Figure 10 shows the diaphragm accelerations which clearly reduce with height and their average values range between 0.28g and 0.49g in both directions. For the floor acceleration, it is observed a clear reduction varying from 13 % to 20 % less value compared to the fixed base model.

Figure 11 compares the maximum average drifts and floor accelerations per level and X,Y directions for the fixed and hinged columns at the base structural proposals. In terms of drifts it is observed that maximum values are similar, although at the base and for the hinged solution are significantly larger than for the fixed one. However for the hinged base model, the drifts are quite uniform with height with an almost 0.007 constant value. In terms of accelerations, the values are similar in the lower levels although they are clearly smaller on top levels for the hinged base structure. A clear trend observed in all cases is that the floor acceleration is reduced with height which would be an interesting subject for future research proposing a similar precast structure but for taller buildings.

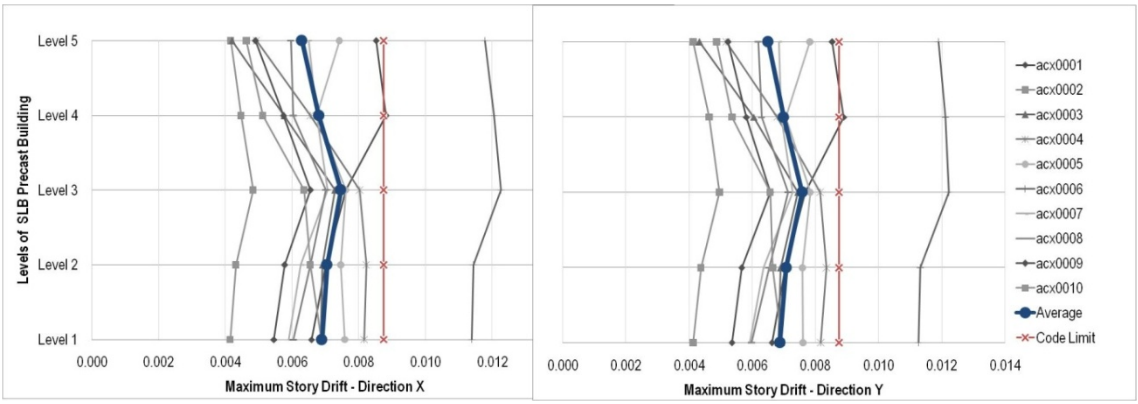


Fig. 9 – Maximum X, Y direction drifts per level and seismic signal. Building with hinged base connection

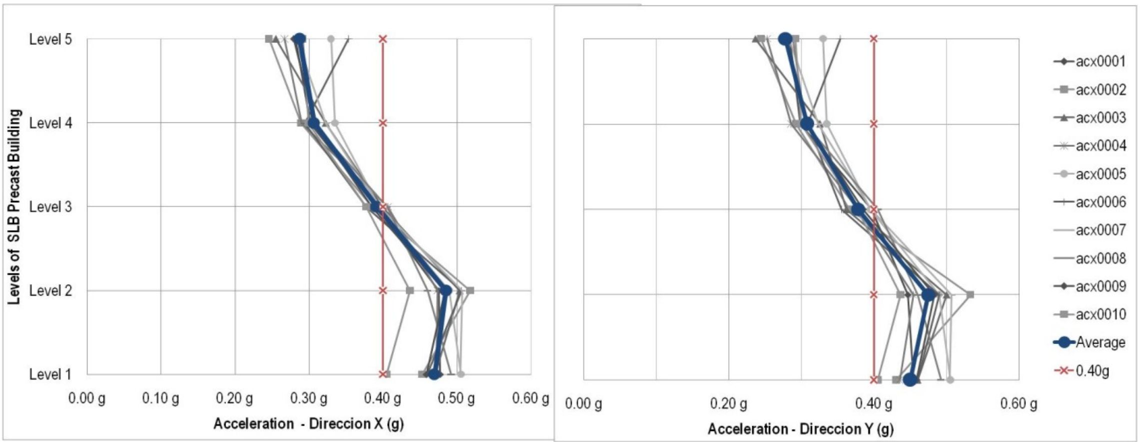


Fig. 10 – Maximum X, Y accelerations per level and seismic signal. Building with base hinged connection.

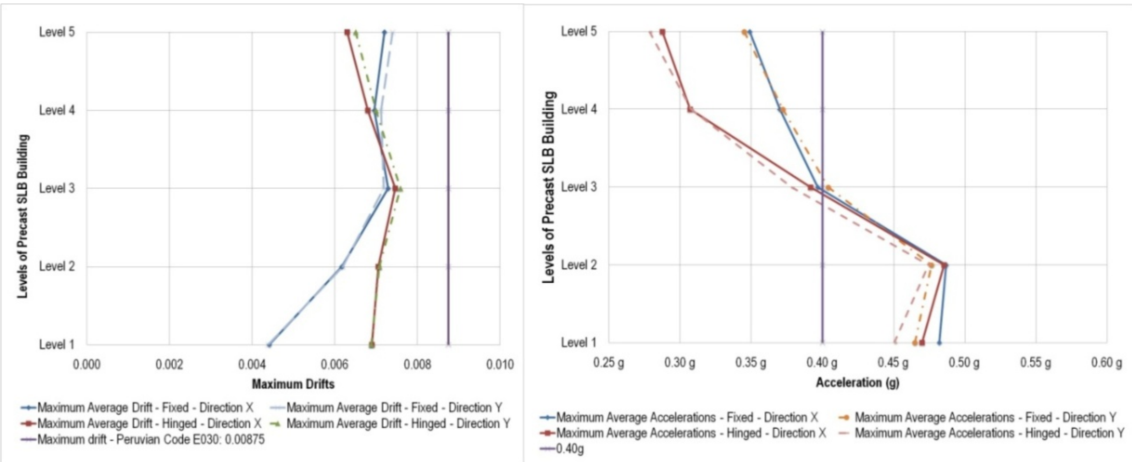


Fig. 11 – Maximum Average drift and diaphragm accelerations per level. Hinged and fixed models.

Finally, figure 12 shows the seismic coefficients for each of the ten signals and for the fixed and hinged base models. It can be clearly observed that in all cases the hinged structural system results in smaller values than for the fixed base structure. The average reduction in base shear coefficient is almost 25%. This implies that for the fixed base configuration structure and in the event of a very strong earthquake signal that would originate plastic hinges at the columns base the structural response would improve. In fact, it would tend to reduce base shear forces and floor accelerations but maintaining similar levels of maximum drift.

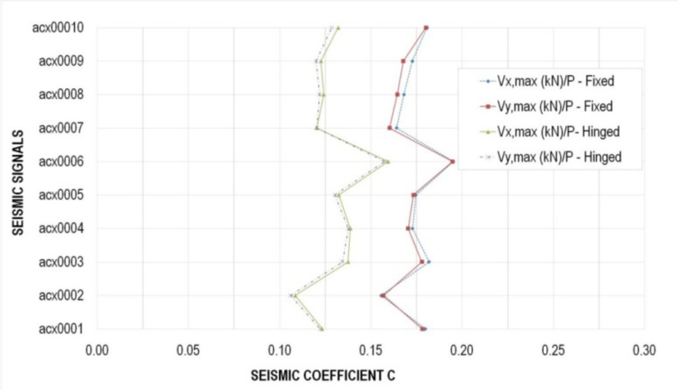


Fig. 12 – Base shear coefficient per signal and for the fixed and hinged models.

6. Conclusions and future research

The SLB energy dissipators provide 52+52 standard devices that combined in parallel, result in a very large potential set of alternatives for seismic protection design. All devices are very stiff and start yielding at displacement as small as 0.20 mm with a low post yielding stiffness. Before manufacturing any device their dimensions are adjusted according to standard unidirectional steel plate tests. The unit cost of these dissipators is very cheap, compared with alternative devices, so they are adequate for a massive use in buildings, to significantly affect structural response. This advantage has the drawback that each set of devices selected provides different structural performance levels resulting in analysis/design complexity. Larger devices may result in less structural damaged controlled by inter-story drift but increasing their cost and story accelerations. Therefore, it is convenient to automate as much as possible the selection process in order to optimize global structural response taking into account usual strict architectonic limitations in their position or theirs supports elements.

Consequently, an application that automatically loads all the standard simple and combined SLB devices modelled as NLINK elements has been developed and implement in the ETABS program. Using this database, two iterative selection procedures were implemented in a plug-in DISSIPA SLB. This application allows automatic selection of a given set of device locations based on architectonic restrictions. Since most important decisions in the design process are in preliminary design, the proposed analysis and selection procedures are linear elastic modal spectral analysis method. This allows fast trial configurations studies and a subsequent nonlinear time history analysis for the selected set of devices is required just as a final validation of the design. The two automatic selection procedures are: (1) direct iterative and (2) inverse iteration or fixed force procedure. In the first one, given a supposed set of devices, the application iterates forward according to the linear shear force determined for each device. In the inverse procedure an "objective" force is searched for, iterating with fictitious devices in the model until reaching that force.

This article studies a 5-story precast building incorporating 80 SLB simple small devices. The building has full height columns but hinged beams for simple fast precast construction. The building is precast in order to provide a potential solution for low income families around Peru. There are 4 apartments per level without interior columns or walls. The full seismic demand is concentrated on the devices so there is no structural damaged expected. The analysis performed is nonlinear using 10 time-history seismic signals compatible with the Peruvian spectrum in a S1 soil condition in Zone 4. Two structural configurations were studied for the columns: (1) fixed or (2) hinged at their bases. The performances of both solutions are compared, showing similar drift results, in the range of 0.003 to 0.007. In terms of floors accelerations, the range is between 0.30g to 0.50g with smaller values for the hinged column base solution and in all cases with a clearly reduction with height achieved by the energy dissipated by the devices. Taking into account that, for the R=1 spectrum and for a fundamental period T_1=0,5s the seismic coefficient is around 0.95g a clear reduction is achieved particularly for top stories with a ratio of 3 (0.95g /0.30g).

Similar conclusions are obtained for the seismic coefficient with average values around only 0.13 and 0.17. This result implies an “R” reduction factor between 5 and 7 for the precast building without any structural damage since all the input energy is dissipated by the devices. At the same time, the solution achieves low displacement levels compatible with immediate occupancy. It is also concluded that if the finally adopted solution is the fixed base column and in the event of an extreme earthquake that overpasses the flexural capacity at the base of the columns, the overall structural response would tend to improve reducing floor accelerations but maintaining maximum displacements. This implies the structural system is robust. Also it is observed that floor acceleration clearly reduced with height.

Finally, two interesting future research topics related to this work are: (1) apply the system for taller precast buildings where it is expected even better performance and (2) apply the special battlement connection for seismic vertical vibration control, since it may be a source of frictional or viscoelastic damping. In our opinion, nowadays vertical seismic input is not clearly controlled in seismic design.

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