EXPERIMENTAL EVALUATION OF THE CYCLIC BEHAVIOR OF A DOUBLE-COLUMN BRIDGE PIER WITH A SHORT SPAN COUPLING BEAM

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SUMMARY:
Within the framework of High-Speed Railway Line (HSRL) bridges, a common design problem is achieving a balance between the structural stiffness required by the restricting horizontal deformation code limits and a ductile response regarding seismic events. On that subject, an experimental campaign was devised at Faculdade de Engenharia da Universidade do Porto (FEUP) regarding reduced scale models of the RC bent-type frame structures of the now cancelled Portuguese Poceirão – Caia HSRL project, consisting of massive vertical piers connected by tall and short-spanned beams.
On the scope of common response spectrum based design methods and the geometrical conditioning of such configurations, it is difficult to choose an adequate behaviour factor due to the uncertainty regarding the location of the energy dissipation mechanisms. Therefore, the main objectives supporting the program were evaluating the cyclic response and ductility exhibited by such systems, as well as possible improvements using shear-wall coupling beams reinforcement layouts.

Keywords: Experimental Campaign, RC Bridge Columns, Bent-type frames, Cyclic behaviour, Coupling Beams

1. INTRODUCTION

The development of multiple High Speed Railway Line (HSRL) projects throughout the world has been responsible for the forthcoming of many important contributions to bridge engineering. The harsh track geometry conditioning combined with an expectancy of high-quality travels frequently leads to a need of achieving those goals with the aid of several bridges, viaducts and tunnels. On that regard, while for some countries it might not be the most relevant design field for this type of structures, the development of HSRL on seismic prone areas such as Spain and Taiwan, and also the Portuguese venture, brings the subject to mind. However, the applicable horizontal deformability criteria are severe, to a point where a good balance between low displacements for serviceability limit states and high ductility regarding structural collapse might be difficult to achieve.

In this framework, the Portuguese Poceirão – Caia HSRL development project (now recently cancelled) was analysed, wherein a bent-type frame structure can be found for the smaller spanned bridges. It consists of robust vertical columns, connected by a short-spanned beam, as represented in figure 1, and due to the potential of this solution, the authors focused on analysing its behaviour regarding transverse seismic motions. Typical bridge configurations with single piers develop a simple behaviour when subjected to earthquake motions, characterized by an inverted pendulum deformation pattern. The non-linear incursion of those structures is mostly dependent upon factors such as the regularity of the bridge and the stiffness relationship between the deck and the substructure. Plastic hinges are usually expected to develop at the base of the columns, serving as energy dissipation mechanisms in order to provide the structure the required ductility. However, in bent-type frame structures it is not clear whether the critical zone is to be designed in the piers or the beam. Particularly, with a small shear span-to-depth ratio (α_s=1.0) and the robust column cross sections to which it is connected to, the beam can be expected to develop a deformation pattern where shear distortions must be duly considered. Therefore, if the frame is expected to develop plastic hinge mechanisms on the column bases irrespective of the beam's influence, still the latter should be designed to resist high shear demands, since the lateral deformation of each column will mobilize a large differential movement between each of the beam ends. Considering that the columns' non-linear incursion would only increase such difference, the frame could end up not benefiting of the plastic deformation capacity of the columns and exhibit a brittle shear failure.

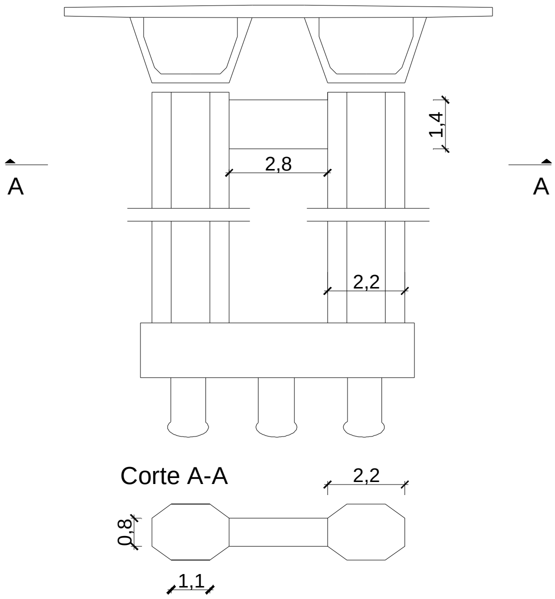
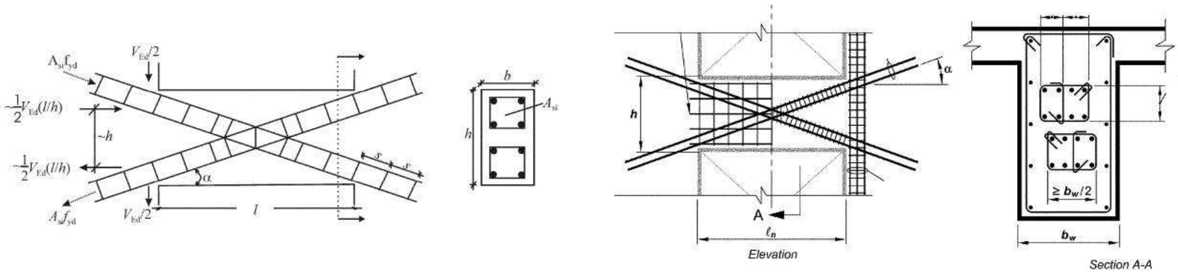


Figure 1. Geometrical configuration of the studied bent-type frame

On another hand, the beam can also be designed taking into account more severe ductility requirements, by adopting special reinforcement layouts that can provide both the necessary shear strength and higher energy dissipation capacity. This problem is, thus, similar to coupling beams in shear-walls, although the columns in the present solution are not so stiff and there is no confinement provided by each floor slab.

2. SEISMIC DUCTILITY IN SHEAR-WALL SYSTEMS

The problem of the seismic performance of shear-wall systems and its coupling beams was first studied in the 70's. Experimental studies by Paulay (1971) showed that coupling beams built with conventional double-side reinforcement layouts evidence severe shear vulnerabilities, prone to diagonal cracking or sliding shear failure near the beam/wall interface, which is found more important for smaller α_s ratios. Other authors such as Subedi (1991) or Hindi and Hassan (2004) also focused on the subject, suggesting improvements to the analysis method. Most noticeably, several proposals, such as Paulay and Binney (1974), Tegos and Penelis (1988), Tassios *et al.* (1996), Galano and Vignoli (2000), were supported by thorough experimental research activities regarding the improvement of the conventional reinforcement layout. In particular, the most widely accepted proposal consisted on forming a cross-shaped steel reinforcement layout, with both diagonals intertwined, and bearing individual confinement reinforcement. It is the configuration that is foreseen in both Eurocode 8 and ACI318-08, as shown in figure 2.



a) Eurocode 8 (NP EN1998-1 (2010)) b) ACI318-08 (2008)

Figure 2. Diagonal reinforcement layout for coupling beams

The above represented layout was shown to exhibit the best results for the lowest α_s ratios. However, it was also apparent that such a detailed configuration involves severe construction difficulties, particularly due to the large diameters usually required in the diagonals and the increased anchorage lengths that are needed for this type of reinforcement. Because of that, some authors have also investigated alternative ways to keep the ductility and energy dissipation capabilities of the diagonal layout, while alleviating the construction, e.g. Canbolat *et al.* (2005) or Parra-Montesinos *et al.* (2010).

Other suggestions were also made, with less constricting features, such as the rhombic truss reinforcement layout proposed by Tegos and Penelis (1988), shown in figure 3, that was developed both for coupling beams and for short columns, or conventionally reinforced beams with embedded dowels in the beam-column interfaces (Tassios *et al.* (1996)).

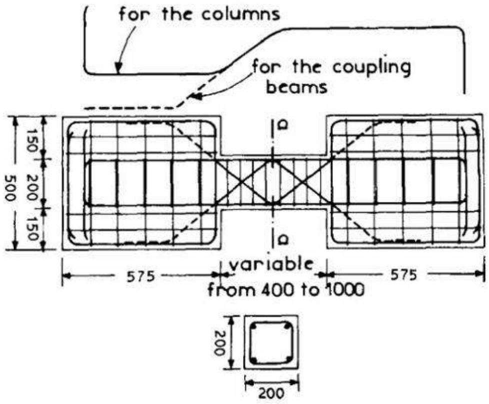


Figure 3. Rhombic truss reinforcement layout (Tegos and Penelis (1988))

Ultimately, experimental evidence showed that diagonal reinforcement was very effective for α_s values lower than 3/4, while conventional detailing could be used for values higher than 4/3. In between those ratios the diagonal layout was not so effective, while other, more construction-friendly detailing schemes could be used to more or less the same effect.

3. EXPERIMENTAL CAMPAIGN

The experimental campaign referred in this paper is currently developing at the Laboratory of Earthquake and Structural Engineering (LESE) of FEUP and is part of a larger research project, named SIPAV (Inovative Systems for Precast Systems in High-Speed Railway Lines). With respect to the seismic performance research activities, an experimental campaign was planned, focused on testing several bent-type reduced scale frame specimens, with the geometry shown in figure 1.

3.1. Test setup

Considering the objective of simulating models representative of a possible real situation in HSRL structures, an average target height of around 15meters was considered for real bridge piers. Since the laboratory height constraints enforced the specimens to be designed up to nearly 4.0 meters high, 1:4 scale models were adopted. Additionally, taking advantage of the approximate anti-simmetry of the bending moment diagrams for lateral actions, only the top half of the frame was considered, and the setup developed around the constraints needed to materialize those conditions. Therefore, several key issues had to be taken into account while designing the experimental setup represented in Figure 4:

- Application of constant vertical loads on a structure that exhibits internal variation of axial forces in piers under lateral loading;
- The cyclic load application according to how it is actually transmitted during a seismic event;
- Free rotation in both columns' bases.

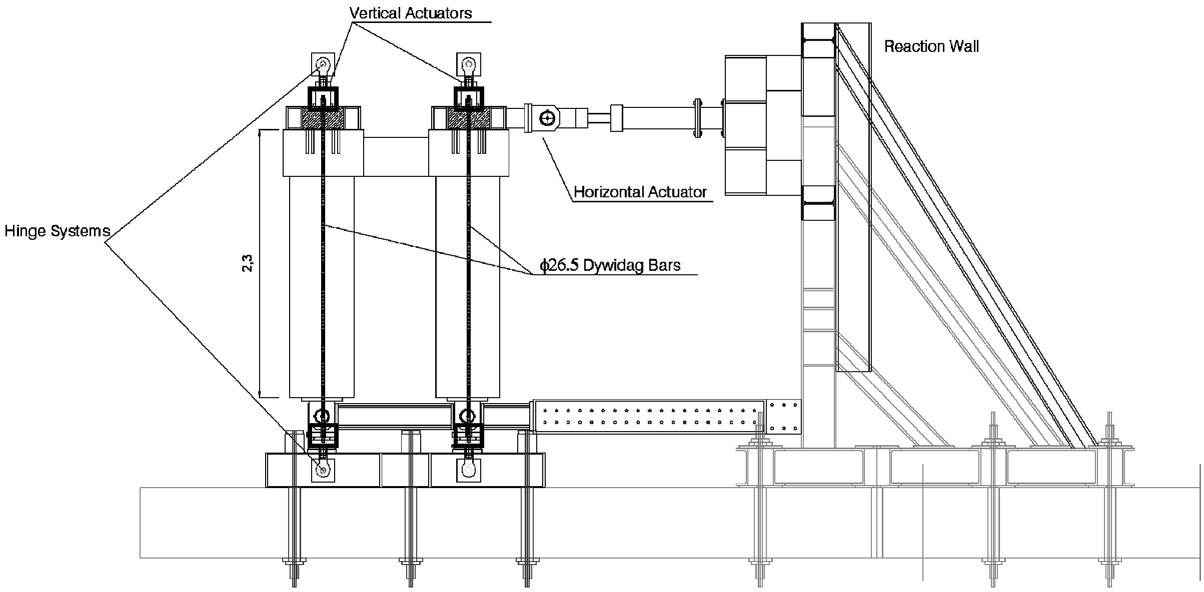


Figure 4. Test Setup at LESE

3.1.1. Vertical Load

The application of a constant vertical load is a real problem on such a system, particularly because, as mentioned before, the cyclic behavior of this frame structure mobilizes high shear forces that interact with the vertical force provided by the test setup, thus alternately increasing and decreasing the axial force in both columns. In order to be able to monitor and continually adjust the load transmitted to the columns, it was decided to apply the load via pre-stressing of ϕ26.5mm Dywidag rods with both ends attached to rotation-free hinges. The top system, located above each column (see Figure 5), is connected to an upper steel beam providing reaction to a vertical double-effect ENERPAC 500kN jack, which is force-controlled in order to keep constant tensile forces in the rods.

Figure 5. Constant axial load control system (jack, hinges and Dywidag rods)

The adopted Dywidag bars were also prepared to allow monitoring the load actually being transmitted to the frame structure, which was nominally set to 300 kN.

3.1.2. Horizontal Load

A seismic event in a real bridge mobilizes the inertia of the structure, which is mainly located in the bridge deck. Thus, the corresponding application of the seismic action in a testing setup should reproduce the transmission of that inertia force through the interface of the columns with the bearings, which means it should be as close to the top surface as possible. In common sized structures (e.g. building frame structures), this issue is meaningless because the dimensions of the testing apparatus do not conflict with the stress distribution in the structure (the node’s stress state is not influenced by the point of load application). In this case, however, the beam-column node is of considerable size (when compared to the beam length) and, therefore, the load application point should be as closely simulated as possible in order to accurately replicate the stress state in a real structure during an earthquake.

Thus, the application of the cyclic horizontal load is made using a 500 kN horizontal hydraulic actuator, connected to the structure through a free-rotation hinge attached to a steel cap (consisting of pairs of HEB200 reinforced shapes) which also supports the vertical jack; therefore, this cap distributes the vertical load through the whole cross-section of the corresponding pier and transmits the horizontal forces to the node upper surface by means of 8 M20 shear connectors.

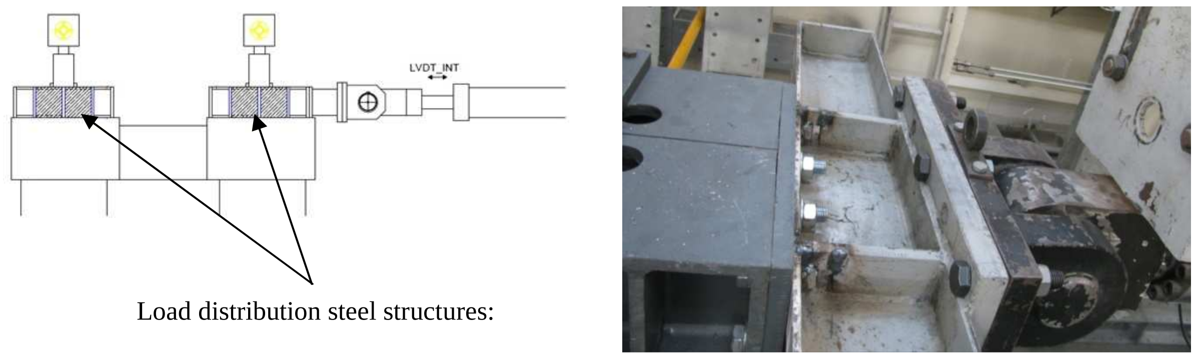


Figure 6. Load distribution system

Additionally, taking into account the possibility of mobilizing some out-of-plane deformations, a sliding system was also provided, with sliding bearings attached to a reaction structure positioned alongside the frame plane (see Figure 7).

Figure 7. Horizontal lateral sliding system

3.1.2. Rotation System in the Column's Base,

The system specifically designed for these tests implied the use of a hinged mechanism on the columns' base, capable of resisting complex states of multi-directional loading combined with rotation freedom. As such, two mechanical hinges were specially fabricated for this effect, taking into account the combined action of the horizontal and vertical load. Both hinges were connected by a rigid steel beam (which is attached to the reaction frame) and were provided with load cells in order to allow recording of the pier axial load, caused by both the beam shear forces and the vertically applied force. This system is represented in Figure 8.

Figure 8. Column base hinge mechanism

3.2. Test Specimens

As mentioned before, this work (under development) focuses on the cyclic behavior evaluation of bent-type piers for HSRL bridges. For this purpose, reduced scale specimens were designed to comply with target maximum base shear strength around 2500 kN in real full-scale piers up to 15m high and with moderate energy dissipation (associated to a behavior factor about 2.5). Therefore, the first phase of the experimental activity includes one specimen designed according to each layout of the four types mentioned in section 2 and described in table 3.1. For all specimens the column capacity was kept the same while different design options were adopted only for the beams.

At the moment, only SP_M01 was tested, while the remaining specimens are in production stage with testing foreseen for July 2012.

Table 3.1. Test Specimens

Label	Reinforcement layout type
SP_M01	Bi-diagonal reinforcement, following EC8 provisions for coupling beams of shear-walls.
SP_M02	Rhombic truss type reinforcement, designed according to Tegos and Penelis (1988)
SP_M03	Bent-up bars only in the beam-column interface (adjusted rhombic truss detailing)
SP_M04	Conventional reinforcement with vertical stirrups.

3.2.1. SP_M01 Detailing

The specimen SP_M01 was designed according to Eurocode 8 guidelines for coupling beams of shear walls. Thus, the main shear resisting mechanism relies on the two cross-shape steel diagonals, designed according to equation (3.1), where A_{si} is the total steel bars area per diagonal, f_{yd} is the steel strength and α is the angle between each diagonal and the horizontal direction.

$$V_{Ed} \leq 2.A_{si}.f_{yd}.sen\alpha \qquad (3.1)$$

Besides individual confinement stirrups for each diagonal, additional minimum transverse and longitudinal reinforcement is also required to control the expected intense cracking effects. At the end, as mentioned in section 2, these provisions lead to a very dense layout as shown in figure 9.

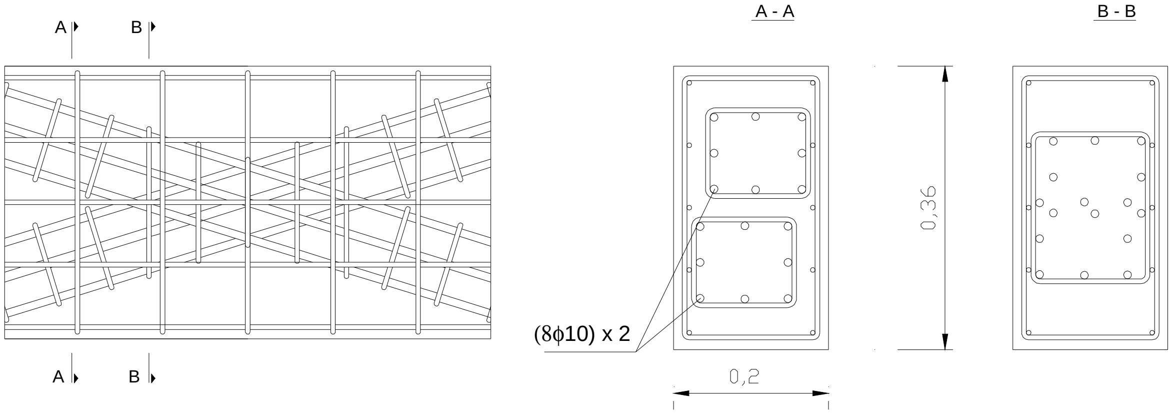


Figure 9. SP_M01 beam detailing and construction photograph

4. PRELIMINARY EXPERIMENTAL RESULTS

The first specimen of this campaign was very recently tested, using a cyclic imposed displacements controlled evolution represented in Table 4.1. The force – displacements test results are depicted in Figure 10 (horizontal actuator force *vs.* horizontal drift measured at the beam axis level), although the setup was prepared to go further than 3.5% drift. Besides testing the SP_M01 solution, this first test served also to check the system efficiency and to detect any possible corrections to be implemented.

Since the test was completed little before this paper submission, results are not yet processed and analyzed. However, an overall observation of the force-drift plot allows pointing out the non-symmetry of strength between positive and negative loading senses.

Although not yet fully explained, this effect is most likely due to loading control system which relies on the internal displacement transducer of the actuator. Since the control displacement is applied only at South side of the structure, once strong damage is imparted to the beam, a given displacement imposed by actuator, equal in the positive (South-North, S-N) and negative (North-South, N-S) senses, does not correspond to the same drift at the beam axis level in the two opposite senses. Therefore, for S-N motion, both columns are similarly mobilized, while for the N-S sense, the North column is less pushed than the South one, which is also related with the larger residual drift observed in the positive sense than in the negative one. In fact, when the system displaces in the negative sense, because this residual drift has to be first recovered (which is not "seen" by the internal actuator transducer), it follows that the N-S actuator displacement induces less negative drift in the structure than it does for S-N displacement and, therefore, the whole bent-type pier is not equally strained for the two motion senses.

Anyhow, still it can be observed that, until the onset of yielding (i.e. in the post-cracking phase, up to approximately 0.75% drift, when yielding can be considered to have started), the response is essentially symmetric, with reduced energy dissipation, which is consistent with the cracking pattern shown in Figure 11-a); further pictures in the Figure 11 show the main issues of damage evolution.

Table 4.1. Load evolution

Drift level (%)	Number of Cycles	Load Rate (mm/s)
0.05	1	0.2
0.1	3	0.2
0.2	1	0.5
0.3	3	0.5
0.4	1	0.5
0.5	3	0.5
0.75	3	1.0
1.0	3	1.0
1.5	3	1.0
2.0	3	1.0
2.5	3	1.0
3.0	3	2.0
3.5	3	2.0

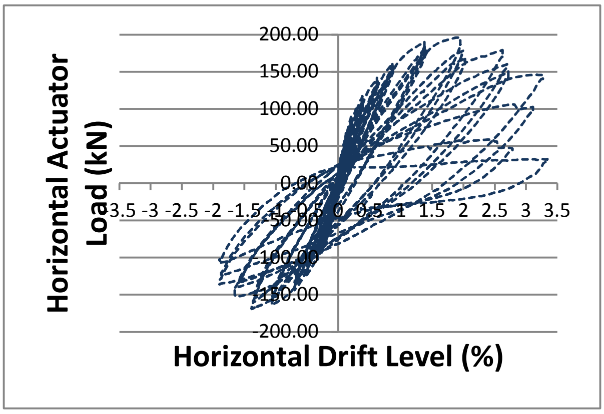
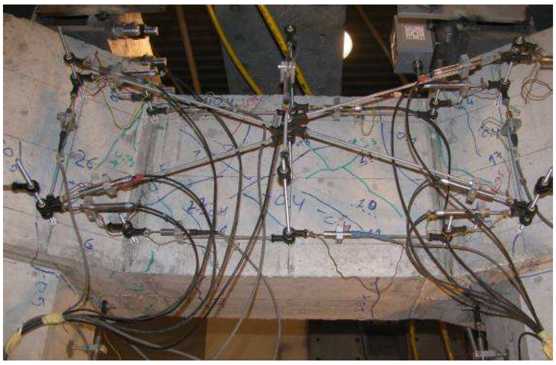
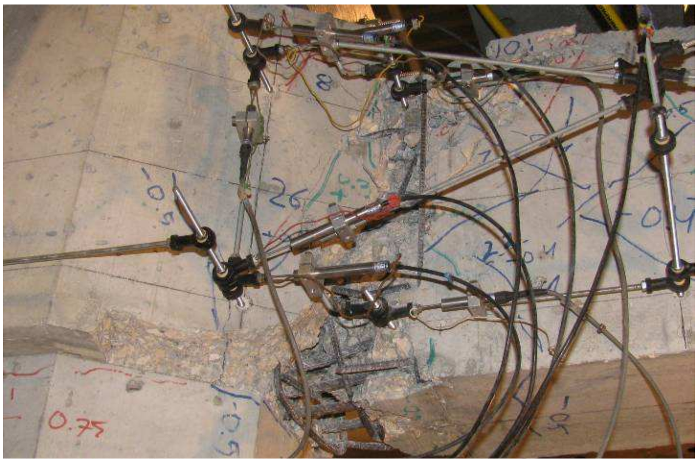


Figure 10. Force – drift curve for SP_M01 test.



a) Cracking pattern – 0.75%

b) Formation of the large crack



c) Development into failure mode

Figure 11. SP_M01 test photographs

The first crack occurred in the beam at about 0.2% drift, further developing to largely pronounced shear cracking that started appearing in the beam at 0.4% drift and evolved as illustrated in Figure 11-

a) for 0.75% drift, until which no visible cracks were found in the columns. By 1.5% drift, one of the beam cracks developed into a heavy crack near the North beam/node interface shown in Figure 11-b), in accordance with the finally obtained yielding pattern.
The large crack illustrated in Figure 11-c) highlights the failure mode exhibited by this specimen, corresponding to sliding shear rupture at 2.5% drift. Actually, although the global behavior seemed to sustain a ductile response, conveyed by low pinching effect and some energy dissipation (Figure 10), as soon as that shear mechanism was activated, the structure quickly evolved into a failure-like state, which is clearly an issue to be corrected in order to achieve a good behavior under cyclic loading.
The remaining specimens, already in production, have taken this effect into account and, therefore, are expected to provide improved behavior and to help widening the scope of this study. Since the project is still developing, no additional information can be provided herein.
As for the testing system, it performed well, although some issues still need to be improved, namely the loading control system to allow exploring similar deflections in both loading senses and the reacting system at the column base that raised a few doubts about real flow of the reaction force.

5. CONCLUSIONS

An under development experimental campaign was presented regarding bent-type frames useable in HSRL bridges. Bent-type frames are one of the possible ways to provide lateral stiffness to high-demanding HSRL structures and have been chosen for this study in order to assess its suitability for pre-cast construction. However, since the nature of those structures is very dependent upon the geometry of the railway track, sometimes the relative positioning of the frame relative to the bridge deck is very conditioned and enforces the use of short-span frames.
Concerning the seismic performance of such structures, common design methods require the conception of adequate energy dissipation mechanisms which, in the context of bridges, are commonly located in the columns base. However, the lateral frame deformation can be the cause of large shear deformations and, possibly, of brittle failure. Therefore, special attention must be given to the design of these short-span beams by providing suitable forms of energy dissipation and ductile response.

Therefore, several specimens were designed taking into account other authors' proposals for the similar problem of coupling beams in shear walls. The first test was carried out, showing possibilities of energy dissipation but finally exhibiting an undesired sliding shear failure. Other specimens are already in production, in order to provide more data for this still ongoing project. Therefore, more thorough conclusions should be provided after the next tests. Nevertheless, the specially designed testing system performed well but slight improvements still might be required.

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