Dynamic behaviour of fully precast rocking bridge

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ABSTRACT:

This paper describes the dynamic behaviour of multiple segmented bridge pier and deck systems. Furthermore, the feasibility of the implementation of a combined prefabricated post-tensioned bridge system with a rocking dissipative segment situated between the pier and deck is studied. Our work includes an experimental setup of a scaled bridge structure (1:30) in a dynamic test on a shake table. The bridge is composed of multi-segmental pier and deck parts and completely post-tensioned. During the experiment different kind of pier post-tension levels are examined. A sinusoidal input with frequency range similar to typical earthquakes is used to carry out the experiments, which were recorded using a high-speed camera.

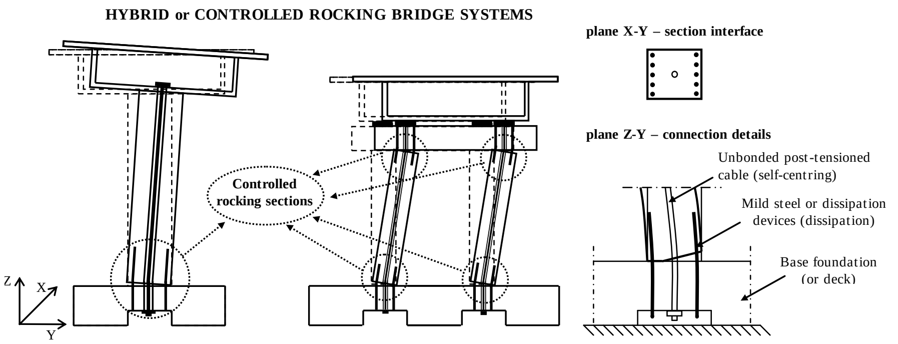
Experimental results reveal dominant gap openings between the foundation and the first segment, in agreement with the shape of the bending moment. The final discussion includes the feasibility to implement dissipater devices at the right locations in accordance to the new findings.

1 INTRODUCTION

Traditional seismic design methodologies require structures to respond inelastically by detailing members to accommodate significant plasticity (“plastic hinge zones”) which implies accepted post-earthquake damage. This widespread design philosophy in the world of seismic structural design is called “Capacity Design”, a concept developed in New Zealand around 30 years ago. Over the past 20 years prefabrication of buildings and increasingly bridges (sub- and superstructure) has been promoted in low seismicity regions. The development of an alternative solution for precast concrete structures, based on lumped ductile connection, began in the 1990s as part of PRESSS (PREcast Seismic Structural System) programme. The major intent is to limit the post-earthquake damage in the plastic hinge regions by creating a “rocking dissipative mechanism” occurring in one single section. The solution provides an alternative to the emulation of the cast in place approach (Priestly 1991; Priestley et al. 1999), typical of current prefabrication. Building designer are already moving toward post-earthquake low-damage and precast system technology (Englerkirk 2002) and several buildings have been constructed already. The bridge engineering community has to inherit those enhanced concept and technologies to satisfy the growing demand for high-performing structures. Currently in the United States, there is a strong momentum to develop Accelerated Bridge Construction techniques (ABC) which implies high prefabrication. In 2002 the American Association of State Highway Transportation Officials (AASHTO) began a strong collaboration with several research institutes and universities with the aim to develop a strategic plan, for the next years, to improve the bridge performance under different aspects (ASSHTO 2005; AASHTO 2005; AASHTO 2004). The precasting of bridge components in the past has been intended primarily for superstructures elements for bridge with short and moderate spans. However, construction time, un-skilled labour can drastically impact on construction time-frame, causing further disruption and alteration to the business. A significant accelerated progress, in terms of construction, could be given by using precast column and bend. Although the application of ABC for bridge pier is becoming more popular in the region with low seis-

micity, it is still limited in the region with high seismicity due to the concerns about their seismic response (Freyermuth 1999). Due to this scepticism and consequently to the success obtained with precast building researches and applications, in the last decades precast segmental concrete bridge construction has done substantial improvements (Billington et al. 2001). Previous experiment work specifically on post-tensioned precast concrete segmented bridge column with controlled rocking or hybrid response under simulated earthquake loading (Hewes & Priestley 2001). Afterwards finite elements analyses were proposed on a monotonic unbounded concrete pier with the aim to study a monotonic a cyclic behaviour (Kwan & Billington 2003; Ou et al. 2007). Palermo and Pampanin (Palermo & Pampanin 2008; Palermo et al. 2007), designed, modelled and quasi-statically tested a 1:3 scaled hybrid concrete bridge pier and one benchmark specimen representing a typical cast-in-place solution to enhanced the differ behaviour between the two different solution, Fig. 1. In the hybrid solution the energy dissipation capacity was provided by internal mild steel dissipaters. The experimental results confirmed the advantages in terms of performance of jointed ductile connections compared to the traditional monolithic solutions. Another, more recent relevant experiment was carried out by the University of Buffalo in 2010. Where a half-scale fully precast bridge with sliding and rocking dry joint was tested with an earthquake of magnitude 7.0 Richter. After the test, the bridge remained functional with no structural damage (Sideris et al. 2011).
While a significant amount of analytical and experimental research has been carried out to quantify the performance of a single multi-segmental rocking bridge pier, the response of a fully precast rocking bridge has to be confirmed through additional dynamic test. Furthermore, issues relating to the quantification of the gaps opening in the pier and deck can only be addressed through such test methods. This paper discussed the dynamic behaviour of multi-segmented bridge system which has one multi-block bridge pier and a prefabricated deck. Different levels of post-tensioning in the deck simulate the different restraining effect provided by the deck-to-abutment connection.

a)



b)

Figure 1: a) hybrid or “controlled rocking” solutions for bridge pier (Palermo & Pampanin 2008); b) hybrid bridge pier connection details

1.1 EXPERIMENTAL PROGRAMME

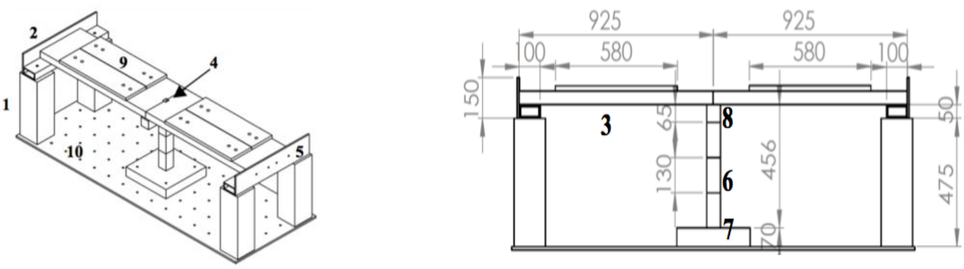
We designed, built and performed dynamic tests on a uni-axial shake table of a scaled two span bridge with single pier scale 1:30. The bridge is composed by a multi-segmental pier and the deck allows to rock, while both, the pier and the deck, are completely post-tensioned. The different number of post-tension bars in the deck replicates different types of connections between abutment and deck. The tests are divided in three macro categories, as show in Table 1. Those categories are differentiated by the typology of restrains between the deck and the abutment. The different connections are obtained by different level of initial post-tensioning and number of bars accommodated in to the deck. All four tensioned bars represent a fully fixed connection; no bars tensioned represent the pin connection and only the two bars locate in the middle represent a hybrid connection between pin and fully fixed connection. The nominal level of the deck tension given in each category of tests is 15 kN. The level of post tension of the pier tendon is equal to 40 kN, which represents the 0.45 $f_{y,pt}$. A sinusoidal input with a frequency of 3 Hz and peak of acceleration equal to 0.85 g was selected. The total displacement stroke given to the shake table to achieve the acceleration needed with a frequency of 3 Hz was ±20mm.

TABLE 1 – Experimental programme

	Deck connection		
	Fully fixed	Pin connection	Free connection
	Nominal pier post-tensioning level [kN]	Nominal pier post-tensioning level [kN]	Nominal pier post-tensioning level [kN]
	40	40	40
Frequency[Hz]	3	3	3
Acceleration[g]	0.85	0.85	0.85

1.2 SCALING AND CONSTRUCTION DETAIL OF THE POST-TENSIONED BRIDGE

The selection of the bridge type was based on maximizing the dimension of the scaled bridge on the shake table, without losing sight of the bridge reproducibility in reality. Consequently the adopted bridge dimensions are a pier 12 m tall with spans 28 m in length. In accordance with New Zealand Transport Agency and their report published in 2008 (NZTA 2008), a Super-T Bridge precast concrete deck was considered. The design load to design the precast beam and the design spectra with a 1000 years return period are considered in accordance with the NZTA Bridge Manual (Transit_NZ 2004). The base pier section was designed by using CUMBIA programme (Kowalsky & Montejo 2007), which resulted in a section equal to 1.80 × 1.80 m^2. The global dimensions of the structure are length: 54 m, each span: 28 m, width: 10.35 m, and height 14 m. The main aim is to scale the prototype bridge in to the specimen scale 1:30. During the dynamic test not only the geometry scaling is important, but more significant is achieving the same dynamics effect. It is possible trying to obtain the same inertial force. Similitude rules comprise a quantitative relationship between the behaviour of the model tested (scaled) and that of the prototype structure. The Cauchy-Froude similitude law not only keeps stress-strain constant, but also maintains accelerations, which is an advantage point to be considered during the shake-table test. After the scaling was necessary adds to the specimen 730 kg to try to obtain the same inertial force. It was possible by adding 44 steel plate anchored on and under the decks. The specimen is composed by many individual part and all components are assembled together through post-tensioned bars. Fig. 2 shows the components which compose the specimen.



1) Abutment Column; 2) Abutment; 3) Deck; 4) Post tension hole pier; 5) Post tension hole abutment; 6) Segmented pier; 7) Foundation; 8) Cap;9) Load steel plate; 10) Steel plate fixed on Shake-table

Figure 2: CAD assembly and dimensions of the multi segmental rocking bridge

The abutment columns are 150 × 150 × 475 mm^3. The aim of this element is only to provide an adequate rigid support to the abutment. The four abutment columns are designed with the purpose to be fully fixed on the shake table plane, even under the sinusoidal input. Due to the increase in stiffness and for the sake of preventing the structure from any damage and bending of the element, as well as simplifying the construction procedures, the external section is composed by a steel Square Hollow Section (SHS) and filled with concrete inside. Those columns are equipped with a D22 Threaded Bar 8.8 Grade. The abutment in this simplified model is composed by a beam and by a steel plate welded with the horizontal element. The abutment in this simplified model is composed by a beam and a steel plate welded with the horizontal element. The beam has geometrical dimensions equal to 50 × 100 × 640 mm^3. Due to the lack of room to insert longitudinal and transversal bars to provide to the element enough shear and moment resistance RHS was adopted. The steel plate connected with the beam has the purpose to allow the anchoring of the deck post-tensioned bars and provide a stiff deck rocking base. The deck has the dimension of 925 × 350 × 50 mm^3 and it was cast in to a timber mould, where ducts for the post-tensioned bars and mild steel were been inserted, see Fig. 3. The longitudinal reinforcement are composed by five D10 mild steel bars in the lower section and three D10 mild steel bar in the upper section. The pier was composed by three segments with the dimension of 65 × 65 × 130 mm^3. Equal to the others structural members not enough room was present for the classical reinforcement. With the aim to prevent the smashing of the edge during the rocking motion and simplify the construction procedures, steel SHS were adopted. The cap is 65 × 65 × 70 mm^3. Both pier segments and cap are cast in inside a SHS. The foundation is 350 × 350 × 70 mm^3. It is not scaled from the bridge prototype because is not involved in the dynamic motion. The Fig. 2 shows the CAD assembly and dimensions of the scaled bridge. To calculate the parameter to design the characteristics and the tension level of the post-tensioned bars, a quick design procedure was adopted (Pampanin et al. 2010). No dissipater devices are inserted.

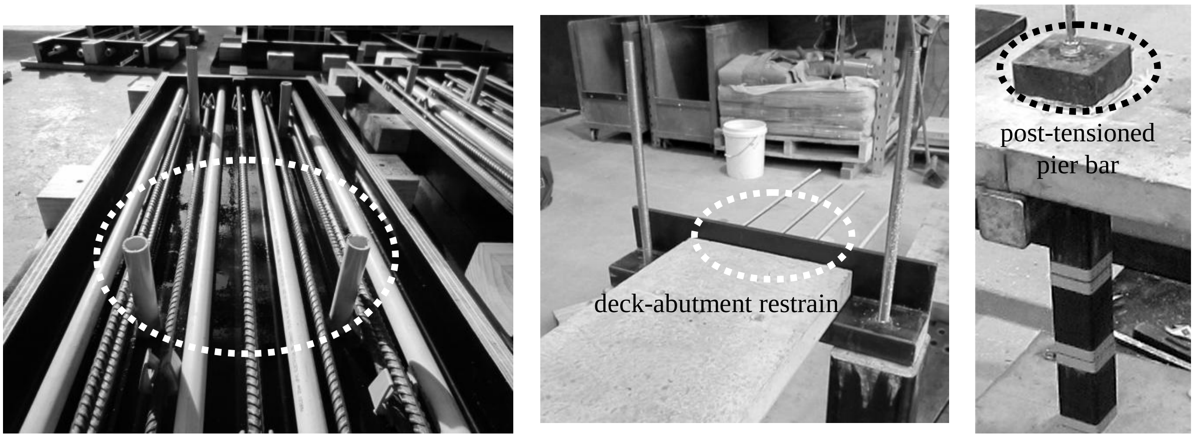


Figure 3: – Left: Close up of deck mould with longitudinal reinforcement horizontal and vertical ducts; Centre and Right: close up of the deck and pier assembly.

1.3 TEST SETUP AND IMAGE DATA ACQUISITION

The general layout of the laboratory shake table set-up is illustrated in Fig. 4. The model is positioned perpendicularly to the direction of shake table motion. The data are collected by two high speed cameras, marked with letters A and B in Fig. 4. The first camera set above the deck captures the gap between the two spans. Another high-speed camera is set on the side of the bridge; it quantifies the gap in the pier section. The lens was set to focus on the gap between the bottom segment and the middle segment. Figure 5 shows a close up view of the pier and deck. Figure 5 also shows some light sources (marked with letter C) that are supporting the video/picture quality. The camera above the specimen was set to record with 300 fps, while the camera on the side recorded using 600 fps. All images are analysed using the “Digital Image Correlating and Tracking” (DIC) tool in MatLab. DIC is an optical method that employs tracking and image registration techniques for accurate 2D and 3D measurements (Sutton et al. 1983,Tiwari et al. 2009). DIC is using the maximization of a correlation coefficient which is determined by examining pixel intensity array subsets on two or more corresponding images and extracting the deformation function that relates the images. The output data of this analysis are absolute displacements, which then were turned into rocking angular variations.

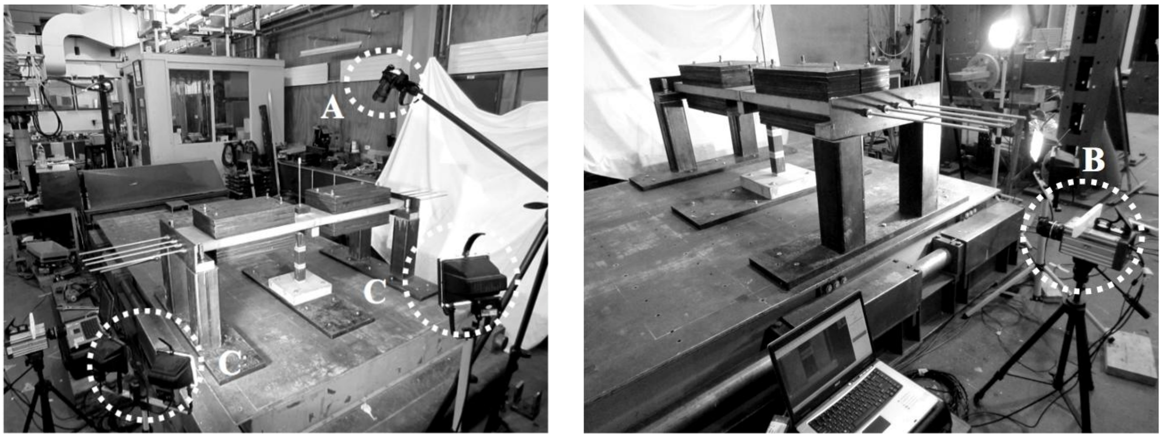


Figure 4: Photograph of the specimen and shake table test set-up

Figure 5: Zoom in of the cameras view: pier view with in evidence the rocking section analysed on the left and deck view on the right

1.4 RESULTS

The results are focused on the second rocking section, Fig. 5. This choice was made according to the following reasons: 1) the highest rocking partner (pier-deck) is covered by additional steel plates (thus inaccessible for the side camera) which are located underneath the deck to prevent the deck from undergoing torsional motion; 2) the bottom section is the interface that is widely studied in the monolithic configuration; 3) the section between the second and third pier segments shows a much lesser gap opening. However, we also observed the deck gap opening, but for reasons of clarity in the results only one test run will be shown. The plots (Figs. 6-7) below represent the temporal evolution of the x and y displacements, respectively. The phase portrait (Figs. 6b) is particularly useful in identifying the system behaviour as it consolidates to remove the distracting effect of the time. Figure 6b shows a range of orbits corresponding to the time-history results (Fig. 6c). The quasi-elliptic traces are given by the simultaneous displacements in x direction combined with the vertical y displacements due to the gap opening. The plots reveal a “not symmetric” system behaviour. This non symmetric behaviour is due to both, construction imperfections and non-symmetric shake table displacements. Although the focus is on the gap opening, it is interesting to observe that the segment undergoes a compression phase, underlined by the negative values in y direction. The rocking motion of the gap partners (represented by the maxima of x or y displacements of the elliptic-like trajectories) is of anti-phase nature; while the blue trace points experience the maximum uplift, the black trace points show a compression phase and vice versa, Fig. 6a. This behaviour is as expected regarding the position corresponding to ±20 mm in x direction. Figure 6b depicts a phase portrait between angular velocity and rocking angle with respect to the horizontal line. Also in this plot a non-symmetric behaviour is observed, indicating a favored rocking direction. Figure 6c shows the time response of the rocking motion. Negative values refer to clock-wise motion.

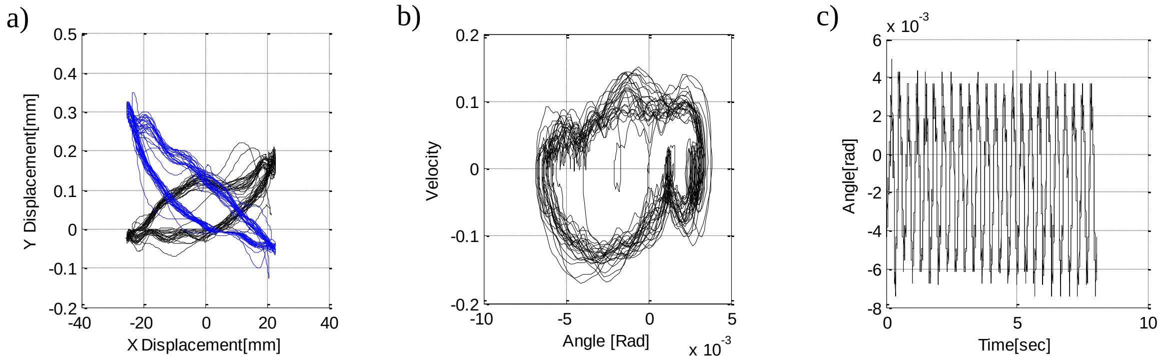


Figure 6: a) Temporal evolution of the x displacement and y displacecement; b) angular velocity vs angular variations in the time domain; c) rocking angle variation in the time domain of the free deck connection test-run

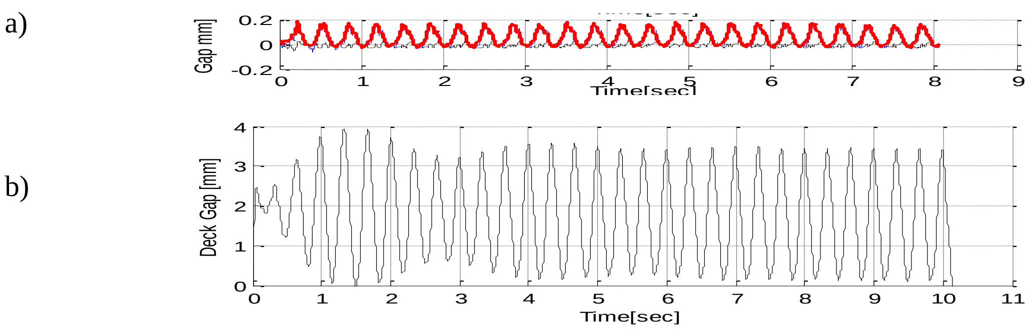


Figure 7: a) Gap opening between the first and the second pier segment; b) deck gap opening phase of free deck connection test-run

After changing the restraint of the system from a pin connection to a fully fixed connection, further reduction of the vertical displacements of the block is expected. Thus, an increase of the stiffness of the system results in a decrease of the vertical y displacements as well as a decrease of the rocking motion, Figs. 8-10. The increase of stiffness can be detected in the phase Fig. 9. A reduction of the non-symmetric behaviour of the entire system is readily observed a reduction and the answer of the model to the sinusoidal input became closer to the pure theoretical behaviour.

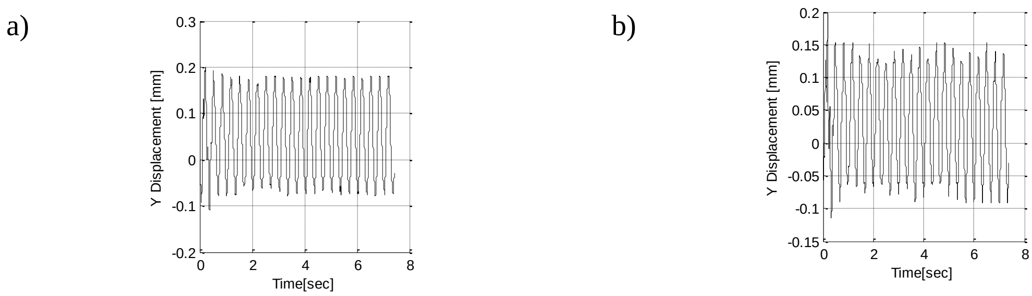


Figure 8: Angular variation with angular velocity in time domain of (a) pin deck connection and (b) fully fixed deck connection

a) b)

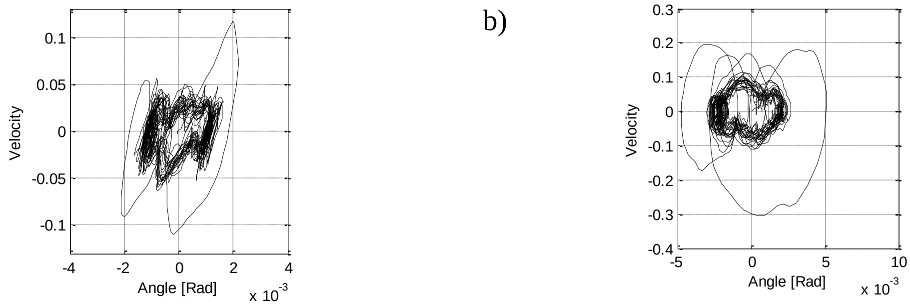


Figure 9: Angular variation with angular velocity in time domain of (a) pin deck connection and (b) fully fixed deck connection

a) b)

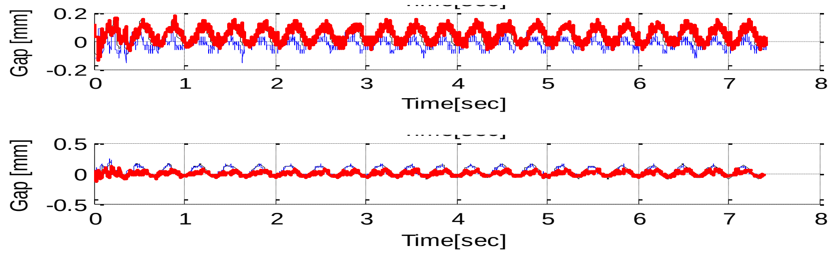


Figure 10: Gap opening phase between the first and the second segment of (a) pin deck connection and (b) fully fix deck connection test-run

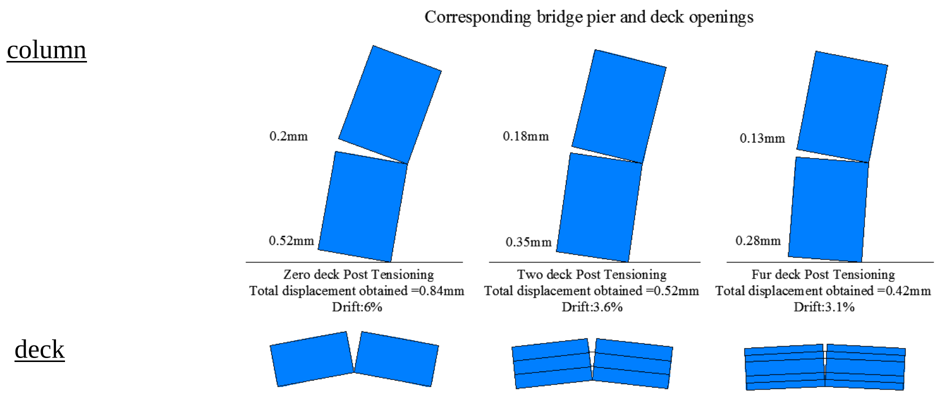


Figure 11: Summary of the main results obtained during the three configurations test

1.5 CONCLUSION

The behaviour of the scaled bridge subjected to harmonic vibrations confirms the hypothesis that the values of the gap openings are in analogy to the shape of the bending moment and stiffness of the system. No damage on the structure and no sliding between the segments are detected. The main gap was recorded on the bottom interface between the last segment and the foundation surface. Moreover a second relevant gap opening was recorded between the bottom segment with the middle block. By incrementing the restrains of the system all the gaps decrease. Due to the small scale the recorded gap openings are very small, in the order of 0.15-0.4 mm (3.1-6 % drift comparison). Due to the small scale, the amplification of some effects such as the torsion of the deck is significant, which, in a full scale, is not present during a dynamic solicitation. These preliminary results are valuable starting points for a deeper understanding of the effects of the geometrical/mechanical boundary conditions on the seismic design of dissipative rocking bridge piers. Further steps will see the inclusion of energy dissipation devices in the lower part of the pier, skewed and/or curved deck, bi-directional response and possibly soil structure interaction.

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