Seismic analysis and retrofitting of an existing R.C. highway bridge: investigation through pseudo-dynamic

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SUMMARY:
The “Retro” TA project funded by the European commission within the Series-project aims at studying numerically and experimentally the seismic behaviour of an old existing reinforced concrete bridge with portal frame piers and the effectiveness of different isolation systems. In particular, an experimental test campaign will be performed at ELSA Laboratory of JRC (Ispra, Italy). Two piers (scale 1:2.5) will be built and tested using the PsD technique with sub-structuring; the modelling of the entire viaduct is considered along with the non-linear behaviour of each pier, due to bending, shear on the transverse beams and strain penetration effect at the column bases. The comprehensive numerical investigations have shown the high vulnerability of the sample bridge. Consequently two isolation systems (yielding-based and friction-based bearings) have been currently designed and characterized. Because the test will start after the summer 2012, in this paper the relevant issues will be here addressed and discussed.

Keywords: PsD test, sub-structuring, non-linear analysis, R.C. frame piers, plain steel bars

1. INTRODUCTION

The seismic vulnerability assessment of existing and new lifeline systems, especially transportation systems, is becoming of paramount importance in resilient social communities. The Italian transportation systems were mainly built in the late 60s and early 70s and were designed primarily for gravity loads. As a results most of the bridges do not employ seismic details and hence their structural performance are generally inadequate under earthquake ground motions. Recently, a comprehensive research program funded by Italian Reluis consortium was initiated to formulate pre-normative European guidelines for the assessment of existing bridges (Pinto and Mancini, 2009). This research program was motivated by the urgent needs to assess the seismic vulnerability and retrofit existing bridge structures. The implementation of comprehensive guidelines for the seismic assessment and retrofit of existing bridges requires the thorough understating of complex local and global response mechanisms. At this purpose, a full scale testing program was initiated within the European project “RETRO”, a research program of the Seismic Engineering Research Infrastructures for European Synergies (SERIES), financially supported by the Seventh Framework Programme of the European Commission (Taucer, 2011). The experimental test program aims at studying the seismic behaviour of an old reinforced concrete viaduct with frame piers and at investigating the effectiveness of seismic isolation systems. For this purpose, an experimental test campaign will be performed at ELSA Laboratory of JRC (Ispra, Italy). Two piers (scale 1:2.5) will be built and tested using the PsD technique with sub-structuring. Despite the fact that the experimental activity are not yet carried out and will be ended at the end of 2012, many interesting aspects of the problem have been already addressed. For this reason in this paper the on-going activities, currently carried out within the RETRO project, are described and the main issues are analysed and discussed.

2. DESCRIPTION OF THE CASE STUDY

The case study bridge comprises an old RC viaduct, built at the beginning of 1960s, consisting of a thirteen-span deck with two independent roadways, supported by 12 couples of portal frame piers (Fig. 2.1), each composed of two solid or hollow circular columns of variable diameter (120-160 cm), connected at the top by a cap-beam and, at various heights, by one or more transverse beams of rectangular section. All the members were reinforced using plain steel bars. The deck consists of two Π reinforced concrete beams 2.75m high, which are interrupted by some Gerber saddles placed at the second, seventh and twelfth bay respectively (Fig. 2.1). The deck is connected to the piers by two steel bars inserted in the concrete, whereas, at the abutments it is simple rested. The linear distributed weight of the deck is approximately 170kN/m for each road-way.

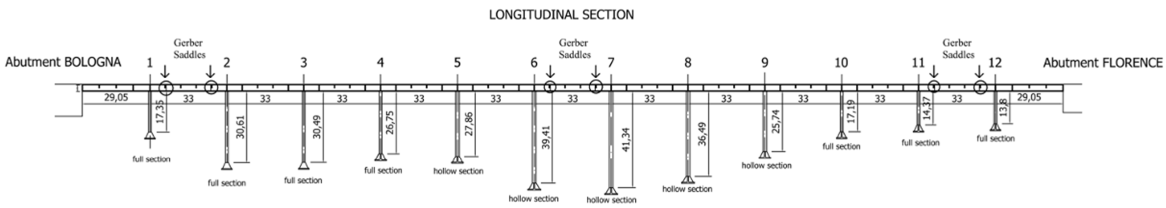

Figure 2.1. Longitudinal view of the viaduct Rio-Torto

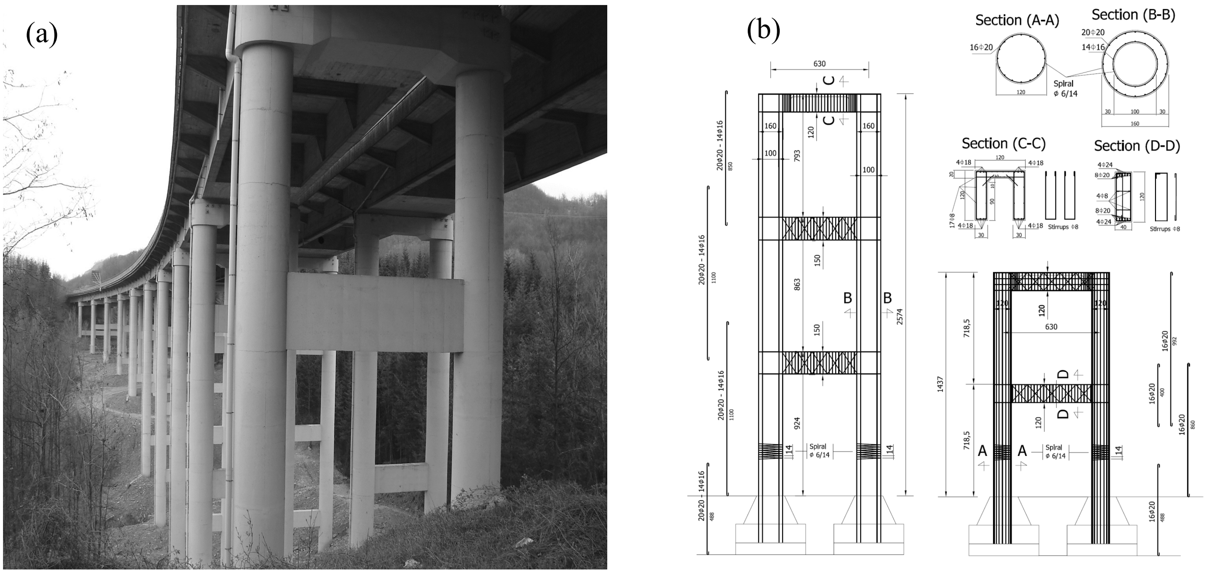

Figure 2.2. (a) Frontal view of the bridge, (b) Reinforcement details of pier 9 and 11

The columns have two types of cross-sections: a solid circular one with diameter of 120 cm and an hollow section with external and internal diameters equal to 160 cm and 100 cm respectively. Some details of the longitudinal steel bars in these two sections are illustrated in Fig. 2.2b, where the reinforcement layout, of the pier 9 and 11 is shown. More details on geometrical and mechanical characteristics of the viaduct can be found in (Paolacci and Giannini 2012).

3. PSD TEST DESIGN

3.1 Test rig configuration

The test rig for testing the two piers in the non-isolated configuration is shown in Fig. 3.1a. The base of each of the two models is fixed to the reaction floor by means of 16-36 mm diameter Dywidag bars pre-stressed to a load of 500 kN in the vertical direction. To prevent cracking and excessive deformation of the base during testing, 16- and 10-36 mm diameter Dywidag bars are pre-stressed to a load of 500 kN in the tangent and normal directions of testing, respectively. The two pier models are positioned in front of the reaction wall one next to each other, with horizontal and vertical actuators connected to the cap beam by means of a steel rig as shown in Fig. 3.1b. The steel rig is made of HEB and C S235 steel sections, with base plates of different dimensions adapting to the different geometry of the cap beam in each pier and connected by means of 6-M27 class 8.7 bolts at each support. Two 500 kN actuators with a displacement capacity of ± 50 cm and spaced at 1.34 m in the horizontal direction are connected to the steel rig, with a lever arm of 0.8 m with respect to the top of the cap beam (equal to the distance from the centre of mass of the deck to the top of the cap beam of the prototype scaled by a factor of 1:2.5). The horizontal actuators are displacement controlled so that during testing there is no rotation along the horizontal place. A system for restraining out-of-plane displacements of the piers with not interaction in the direction of testing will be designed by the ELSA staff. Vertical gravity loads are imposed on the model by means of two 500 kN capacity vertical pistons, controlled and maintained constant during testing. The vertical piston is connected to a 36 mm Dywidag bar running through the centre of the columns (made hollow for the short pier) and connected to the base by means of a nut (the vertical load is self-equilibrated). For the taller pier couplers are used to allow incremental extension of the Dywidags during construction.

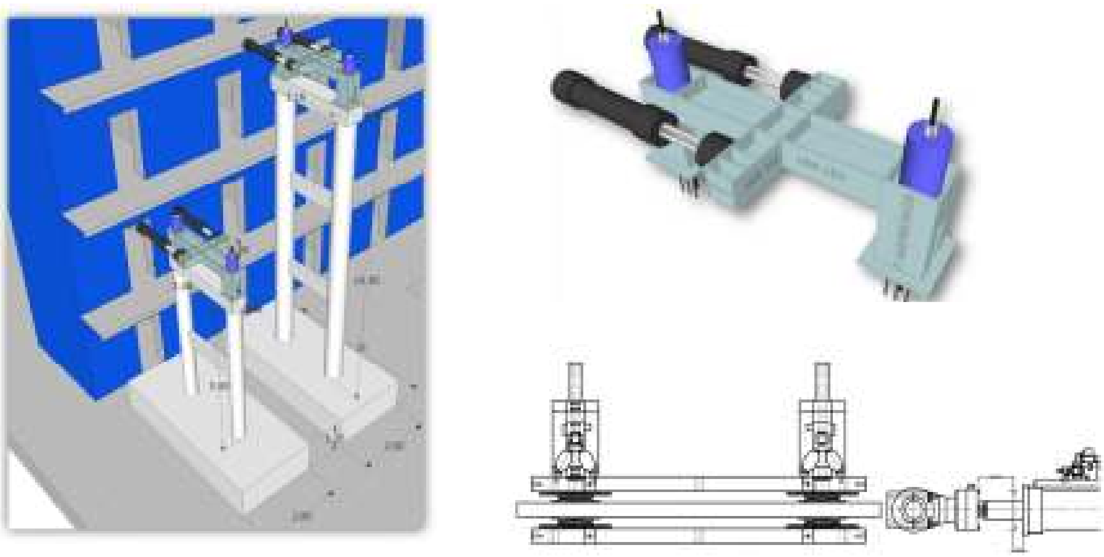

Figure 3.1 (a) The two piers to be tested, (b) Horizontal and vertical load systems (c) Seismic Isolation setup

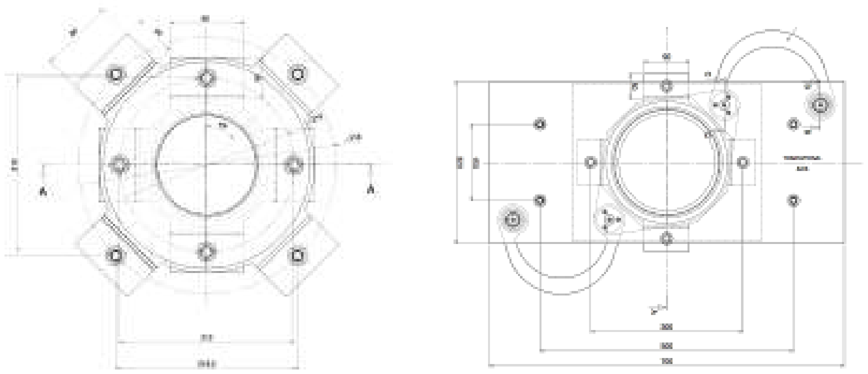

Figure 3.2 (a) Sliding bearing with EP damper, (b) Friction pendulum bearing

For the isolated case, because the sub-structuring technique will be used, the isolation system, composed by 4 isolators and placed at ground, according to the scheme shown in Fig. 3.1c, will be used. The only condition to be imposed is the continuity of displacements at the interface between the isolation system and the pier, under the condition of having the same vertical load. In this way the test configuration of both the piers will remain unvaried both in the isolated and non-isolated case. A sketch of the isolators, realized by ALGA S.p.a, Milan (Italy) is shown in Fig. 3.2

3.2 Numerical model of the viaduct

In the present section the numerical model of one of the two road-ways of the viaduct is outlined. The bridge is modelled by using the non-linear code OpenSees (McKenna et al 2007). The finite element scheme is illustrated in Figure 3.3c. The finite element scheme here adopted for all the piers, has been already used for the simulation of the cyclic behaviour of three 1:4 scale mockups of pier #12, tested at the University Roma Tre (Paolacci and Giannini 2012). The structural elements are modeled by nonlinear beam elements with flexibility formulation. It takes into account for: 1) non-linear flexural behaviour of the element using a fiber discretization of the sections, 2) non-linear shear behaviour of transverse beams using a global model, calibrated by using experimental results and analytical models [Priestly, Vecchio and Collins] 3) the strain-penetration effect of the reinforcing bars at the columns-foundation joints using the Zhao and Shitaran model [Zhao e Shritaran]. This latter has been calibrated using the results of a pull-out test campaign (Paolacci and Giannini 2012). The P-delta effect has been also considered. For the behavior of concrete the Kent-Scott-Park model has been adopted. Moreover, the contribution of the tensile strength was neglected. According to the results in the literature, especially from experimental tests, the contribution of concrete tensile strength in modeling structures with plain steel bars and poor seismic details may be neglected (Marefat et al. 2009). The rebars were modeled with the Menegotto-Pinto law. A yield stress equal to 350 MPa is assumed, along with a modulus of elasticity equal to 205000 MPa and a hardening parameter equal to 0.025. The translational mass has been defined on small pieces (6 m) along the longitudinal, transversal and vertical directions (m_x, m_y and m_z), while the rotational mass has been defined only around longitudinal direction (global y-axis). The supports of the piers has been considered fully fixed in all directions while the abutments at both the sides of the bridge were assumed as simply rested in the longitudinal direction (global y) but restrained in the x and z directions. The Gerber saddles have been modelled using rigid elements with gap in the longitudinal direction and rigid rotational gap elements around the vertical direction. In addition, the relative displacements along x-direction were considered restrained, thus including the possibility to transfer shear in the transversal direction (Figure 3.3b). Because the pier-deck connection has been realized using two steel bars (dowels) of diameters 34 mm for each column, they have been modeled using elasto-plastic elements with shear strength and elastic stiffness of the pairs of steel bars (Figure 3.3a). Moreover, the vertical relative displacements are considered restrained, whereas all the rotation components are permitted. In order to correctly simulate the behaviour of the deck, it has been modelled using elastic elements placed at the centre of gravity of the deck, connected to each pier using a couple of rigid beams.

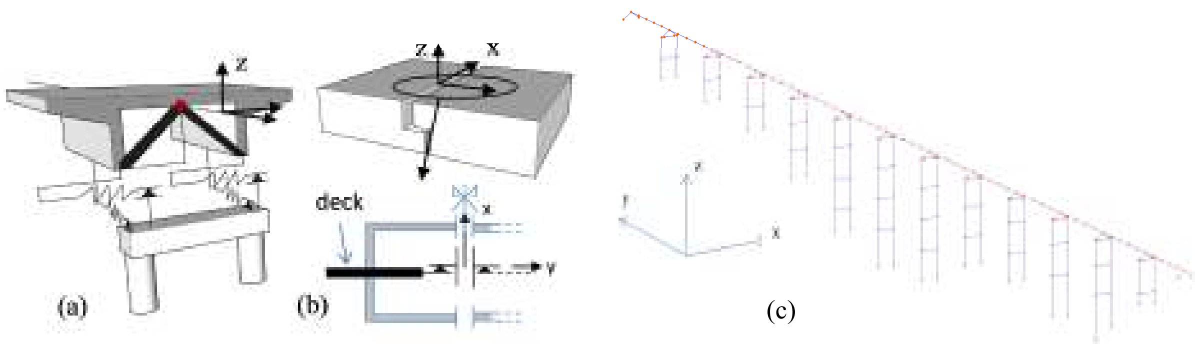

Figure 3.3. (a) Pier-deck connection model (b) Gerber Saddle model (c) The OpenSees model of the viaduct

3.3 Selection of input signals

In order to obtain reliable results on the assessment of existing bridges selection of input is another key point. In fact, a careful ground motion selection can achieve the reduction in bias and variance of structural response (Baker and Cornell, 2006). At this end, seismic hazard assessment (SHA) is one of the key step that has to be conducted. Regarding to this aforementioned reason, earthquake record selection is being done on the basis of both SHA and UHS of Italian code (NTC08). The reference seismic hazard maps of Italy were developed by the Instituto Nazionale di Geofisica e Vulcanologia (INGV) and the current Italian, seismic code is based on the work of the INGV, which computed uniform hazard spectra over a grid of more than 10,000 points for 9 return periods from 30 to 2475 years, and 10 spectral ordinates from 0.1 to 2s (http://esse1.mi.ingv.it/). The Rio Torto viaduct is located in Emilia Romagna region of the Italy and geological investigations indicate that it is constructed on an extensive zone of argillite calcareous together with sandstones. The INGV maps indicates that the bridge is constructed in a moderate-to-high seismic source zone (913). The return period for the life safety condition is about 2000 years. The expected PGA ranges between 0.23g and 0.25g, whereas for the collapse prevention condition (probability of 2% in 50 years) PGA ranges between 0.30g and 0.35g. For the current Italian seismic code and Reluis guidelines (Reluis, 2009), assuming soil conditions B, nominal life=100 years and class of construction=IV, the maximum PGA for several limit state is PGA=0.174g for damage limit conditions, PGA=0.308g for Life safety and PGA=0.352g for collapse prevention. This is consistent with the indications of INGV shaking map (Figure 5b). The corresponding response spectra are illustrated in Figure 6.

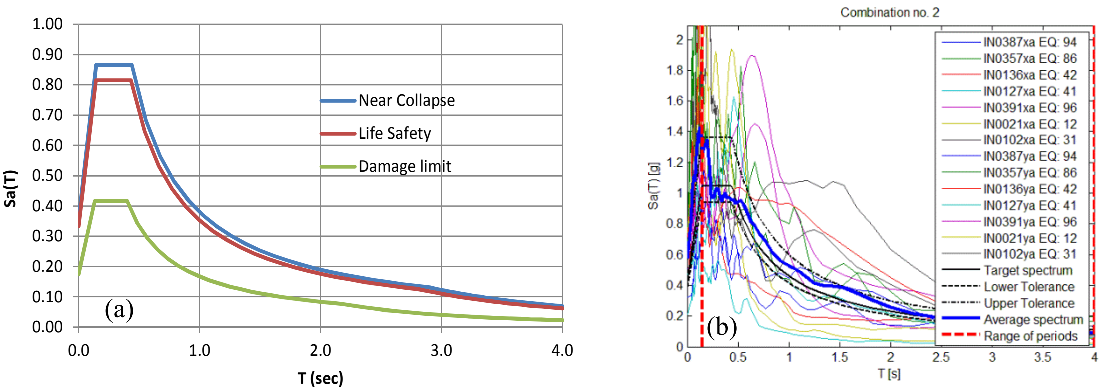

Figure 3.4. (a) Response spectra of the viaduct (Italian Code), (b) Response spectra of selected accelerograms

On the basis of SHA studies, selected earthquake ground motions will be compatible with the developed Target Spectra. Then, nonlinear dynamic time-history analyses will be performed to assess the performance of the bridge under different earthquake levels. Among the selected earthquake recording 5 sets of 3 records will be used as input in PseudoDynamic Testing. At this end, preliminary non-linear analysis of the entire bridge have been executed using a set of 7 pairs of accelerograms have been selected on the basis of near collapse conditions by using the REXEL software (Iervolino et al 2010). The corresponding response spectra are shown in Figure 3.4b.

3.3 Integration scheme for PsD test

At ELSA they have been using the so-called Continuous PsD scheme for many years (Magonette 2001). This scheme allows to load the experimental structure continuously (elapse time 2ms) by performing in the same loop the time integration of the PsD model of the structure and the digital control of the actuators loading the structure. By avoiding the hold period associated with standard PsD implementation, the continuous method avoids load relaxation problems, optimizes the ratio signal/noise associated with the experimental errors, works with a constant time dilation and thus globally improves the quality of the results. The test of the viaduct is substructured, in the sense that a part of the structure is in the laboratory (experimental structure), and the other part is modeled numerically (numerical structure). The substructured test will be performed using an inter-field version (Pegon 2008, Bonelli & al. 2008) of the time partitioned scheme proposed by (Gravouil & Combescure, 2001). This scheme works with different time steps for the two substructures. Almost the same continuous PsD time integration is applied to the experimental structure. The PsD scheme is in fact slightly modified using additional constrain steps, which allows introducing integration sub-stepping with respect to the time integration of the numerical structure for which potentially complex and non-linear models should be operated, requiring more computational time than the 2ms needed by the PsD scheme. The time integration schemes for both substructures just need to exchange information at the time step of the numerical structure, and, in normal condition, does not need to wait one-another, thus maintaining the smooth character of the continuous loading of the experimental structure and at the same time the low impact of experimental errors on the overall integration scheme.

3.3 Identification of the non-linear behaviour of specimens

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4 . PRELIMINARY NUMERICAL INVESTIGATION ON TH SEISMIC RESPONSE OF THE VIADUCT

4.1 Cyclic behaviour of the piers

A preliminary investigation aiming at studying the cyclic behaviour of the piers has been carried out both numerically and experimentally. The test campaign, performed at the structural laboratory of University Roma Tre, consists of quasi-static cyclic displacements imposed to three 1:4 scale specimens of the pier #12. Details on the experimental results can be found in (Paolacci and Giannini 2012). The results confirmed that the response of the pier is highly affected by the behaviours of local details as non-linear shear deformability of the transverse beam or strain-penetration of the plain steel bars. In Fig. 4.1a the numerical and experimental cyclic force-deflection responses of pier 12 are shown, whereas in Figure 4.1b the experimental shear crack pattern of a transverse beam is shown.

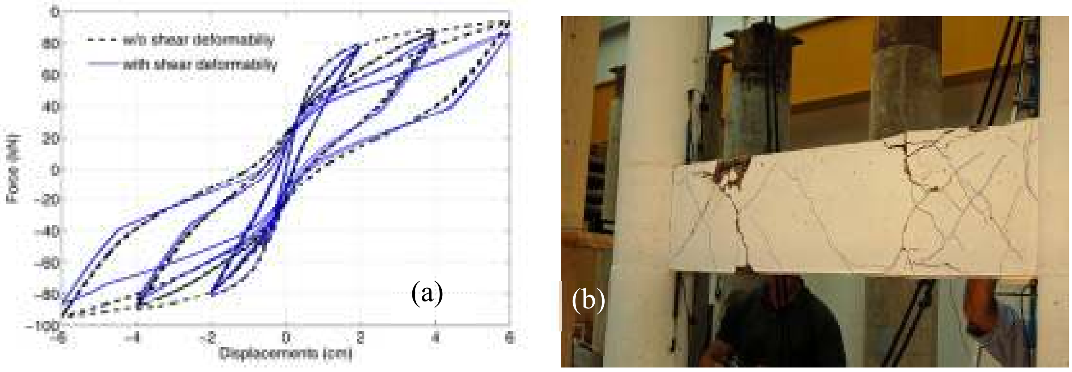

Figure 4.1. (a) Force-deflection cycle of Pier 12, (b) Shear Damage in the transverse beam of pier 12

4.2 Preliminary results of non-linear seismic analysis of the entire viaduct

Figure 4.2 shows the maximum displacement at top of each pier and the corresponding maximum base shear. These quantities are useful to identify the most stressed piers and their damage condition. The results are presented considering both the model with flexural behaviour only and the complete model, which includes non-linear shear-deformation of the transverse beams and the strain-penetration effects at columns bottom. The piers placed at the two edges of the viaduct present the maximum lateral displacement. Piers #1, #2, #11 and #12 exhibit maximum interstory drift greater than 1%. The collapse limit of concrete structures is generally of the order of 4% as indicated for example in (FEMA 2000). In the present case the maximum mean values of drift is greater than 1.5%. This means that the bridge could suffer a relevant damage, but the probability of collapse should be limited. In any case, taking into account the results in the previous section, the expected level of plastic deformation in the pier 9 and 11 can be considered high.

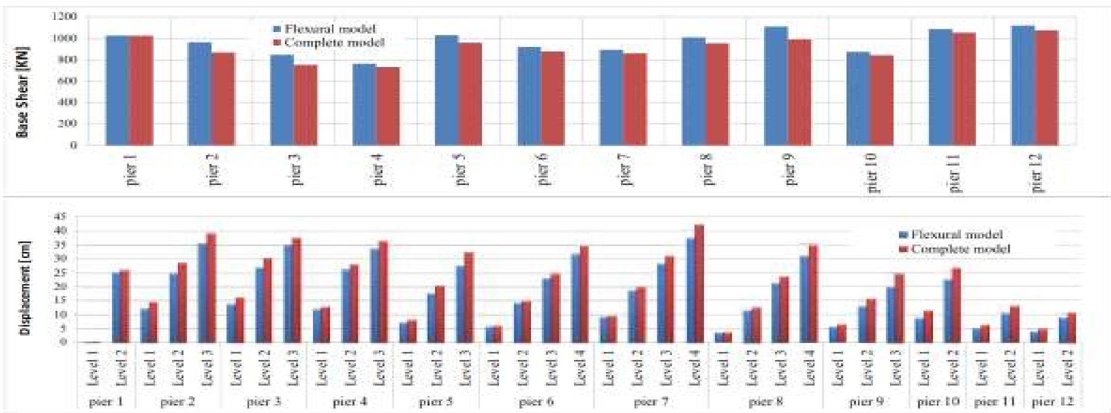

Figure 4.2. Seismic response of the viaduct (a) Top displacement at each pier (b) Base shear at each pier

In addition, because the drift corresponding to the shear failure of transverse beam or to the failure of beam-column joints, is lower than 1% (Giannni and Paolacci 2012), and this value is exceeded (on average) in all the piers, the seismic action could induced a generalized failure of the transverse beams and beam-column joints. The failure of the transverse beams may not cause the collapse of the piers, but it can be enough for inducing the authorities to close the bridge, limiting the viability on such an important thoroughfare. This means that the original structure does not comply with the requirements of the modern seismic codes for the collapse limit state, particularly because of the potential for brittle shear failure in some beams of the bridge piers. Therefore seismic isolation of the bridge deck has been examined as possible protection systems. In particular two possible devices have been designed and will be tested during the PsD test: sliding devices with elastoplastic dampers and Friction Pendulum bearings. In the next section, design and preliminary numerical results on the dynamic response of the isolated viaduct are presented.

5. DEISGN OF DECK SEISMIC ISOLATION

5.1 Design method for FP bearings

A viable retrofitting scheme is the isolation of the bridge deck. The isolation system illustrated herein includes the friction pendulum system (FPS) (Zayas *et al.*, 1990; Mokha *et al.*, 1991). The design of the FPS devices was aimed at: (a) keeping the piers in the (quasi-) elastic range and (b) minimizing the displacement demand on the expansion joints located at the abutments. The FP isolation system has been designed in compliance with the method presented by Della Corte *et al.* (2012). The method, which is derived from the direct displacement based procedure proposed by Priestley *et al.* (2007), proposes a few modifications to the general method; specific design charts have also been developed for the case of isolation by means of FPS.
The adopted methodology focuses on the design of FP systems; nevertheless it can be easily extended to other systems exhibiting a bilinear hysteretic behaviour. According to the design method, assuming a dynamic friction coefficient equal to 0.04 and a maximum sliding displacement equal to 0.10 m, the required radius of curvature of the FP device is equal to 3 m. The maximum isolator sliding displacement of 10 cm corresponds to a global deck drift smaller than 0.03% and corresponds to a drift smaller than 0.05% due to global mechanical behaviour. This small value corresponds to limited rotations at the abutment joints. Fig. 5.1 shows the design results in terms of deck transverse displacements and in terms of assumed piers displacement at the starting of sliding in the FP devices. For design purposes, an average displacement shape (shown with a dashed line in Fig.5.1 has been adopted.

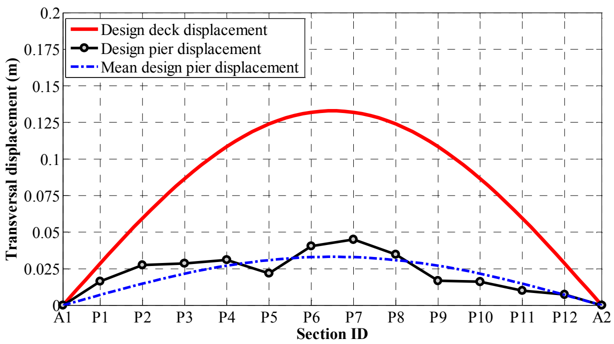

Figure 5.1. Design transverse displacements.

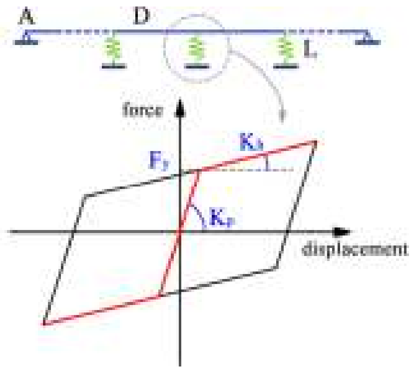

Figure 5.2. a) Simplified model

The strength capacity of each pier must also be checked against the peak forces transmitted by devices at the breakaway of the motion. Neglecting the pier inertia forces, the maximum acceptable friction coefficient can be obtained by the equation μ_{max} = Vy/N, where V_y and N are respectively the yielding base shear force and the total axial load of the pier. For the case study, this coefficient is equal to about 8%. Friction coefficients depend on the contact pressure at the sliding surfaces, on the sliding velocity and on temperature. Experimental studies (Quaglini *et al.* 2009) showed that that for a contact pressure of 20 MPa, a temperature of 21°C and a sliding velocity equal or larger than 10 mm/s, the friction coefficient is less than 7%. The initial friction coefficient is not larger than about 9%. This latter value appears to be slightly larger than the maximum acceptable value coming out from calculation (8%). Consequently, either some damage to piers should be accepted or some pier strengthening should be carried out.

4.2 Modelling and numerical results

Two numerical models of the isolated bridge have been implemented in OpenSees in order to check the seismic response: (a) a simplified model and (b) a refined model. In both numerical models the deck has been simulated with an elastic beam element. The system composed by a frame pier and the corresponding isolators placed on the cap beam has been represented in the simplified model by means of a single spring. Fig.5.2 shows the simplified model for the viaduct. In this latter, 5% stiffness and mass proportional Rayleigh viscous damping has been introduced, by controlling the viscous damping ratio corresponding to the first and third vibration modes. In the refined numerical model the bridge piers are represented as non-linear fiber beam-column elements. For design purpose shear-failure of transverse beams and strain-penetration effect have been neglected. Isolators are modeled through a Coulomb friction model, considering the "hardening" effect due to the radius of curvature. The mass distributions of the simplified and refined model are different. In the simplified model, the mass is distributed only onto the deck while in the refined model the piers nodes have been assigned the corresponding mass.

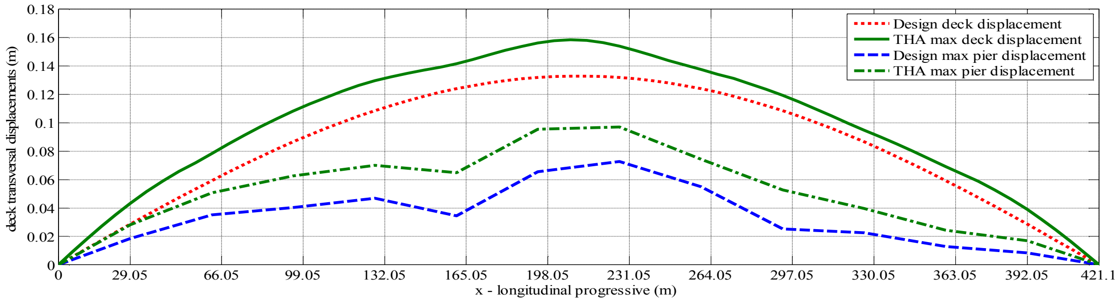

Figure 5.3. Refined model numerical results

The design of the isolated bridge has been validated through dynamic non-linear analyses both on refined and simplified numerical models. The average of the maximum results from a selected quite of 7 acceleration strong motion records (di Sarno *et al* 2011) is reported in the following for each of the investigated response parameter Fig. 5.3 show the deck transverse displacements and corresponding top pier displacements. The design values of peak displacements, simply termed design displacements, are also shown in the Figures. Such displacements are compared with the average of peak displacements obtained from the refined model. Simplified model, here omitted for brevity, has shown similar results. The comparison shows that the design method is sound. The larger displacements obtained from the numerical analyses may be due to the larger average spectral displacements of the selected acceleration records for periods longer than about 2s. Therefore, the results seem to be consistent with the code-specified record selection procedure and can be considered on the safe-side and thus acceptable. However, since a number of assumptions is needed in the design process according to a direct displacement based design procedure, further numerical analyses are on-going using a larger set of records and more stringent spectrum-compatibility conditions.

5. CONCLUSIONS

In the present paper the on-going activities of RETRO project funded within the SERIES project (7th European Framework Program) have been presented and the main relevant aspects have been addressed. The research project aims at studying the seismic behaviour of existing R.C. bridges together with the analysis of the effectiveness of isolation systems. The research program focuses on the assessment of an old R.C. viaduct with frame piers through PsD test. The experimental program will be performed at the European Laboratory for the Assessment of Structures of Joint Research Center at Ispra (Italy). In particular, two of the twelve piers will be built in scale 1:2.5 whereas the remaining part of the viaduct will be numerically simulated. The viaduct, built during the 1960s, has been realized with hooked plain steel bars. Under this conditions, a correct simulation of non-linear local behaviours plays a key role in assessing the seismic behaviour of the bridge. For this reason a refined numerical model has been used for preliminary simulation of the seismic response of the entire viaduct; the model has been calibrated using literature results and experimental data coming from a test campaign carried out at the University Roma Tre on R.C. frame piers. This allowed to select a couple of piers to be physically tested during the experimentation (Pier #9 and #11). All the key aspects of the problem have been here addressed: the most suitable test rig configuration, the integration scheme to be adopted during the PsD test, the selection of input, the numerical model for both isolated and non-isolated case. For the test the Continuous PsD scheme will be adopted, which allow maintaining the smooth character of the continuous loading of the experimental structure and at the same time the low impact of experimental errors on the overall integration scheme. To ensure reliable results, the non-linear behaviour of the tested piers evaluated during the PsD test, will be also identified using non-linear identification techniques. The test will be concluded at the end of 2012. The final results will be object of further publications.

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