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NUMERICAL SEISMIC ASSESSMENT OF AN EXISTING BRIDGE WITH DIFFERENT SUPPORT CONFIGURATIONS

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Abstract. *The assessment of existing infrastructures is a critical issue in the seismic protection of the modern societies. Field surveys in the aftermath of major seismic events have demonstrated the high vulnerability of highway bridges. This led to an increasing interest in the seismic performance assessment of existing bridges. The present paper deals with the seismic assessment of a bridge substructure reproducing typical existing Italian infrastructures. The reference structural system is a reinforced concrete (RC) 1:3 scale single span bridge that will be tested on the shaking table facilities at the University of Naples, Italy. The details of the numerical models adopted in the nonlinear dynamic analysis and the seismic performance prediction required for the test preparation are discussed in details. The record selection, the development of a step-by step validated numerical model are presented for the case study comprising an existing RC bridge designed primarily for gravity loads . The advanced numerical models and the assumptions made to account for the shear behavior of the piers are also illustrated. The preliminary results of the incremental dynamic analysis are presented and analyzed with reference to the effectiveness of the use of base isolation strategy commonly adopted for the seismic retrofit of the bridge deck.*

1 INTRODUCTION

The damage and the collapse experienced by several highway RC bridges worldwide after moderate-to-major earthquakes are caused by the high vulnerability of bridges (e.g. [1]–[5], among many others). A number of existing RC bridges constructed in 1980s or earlier are designed primarily for gravity loads and require major structural retrofitting. Thus, there is an urgent need to assess the seismic performance of existing bridges, considering also the ageing effects that may detrimentally affect the performance of such structures. The reliable evaluation of the support conditions of highway bridges is essential related to the structural dynamic response. Retrofitting measure may either employ traditional local and/or global interventions or use vibration control techniques, such as base isolation systems and supplemental damping. Adequate structural response parameters, such as pier chord rotations, lateral drift thresholds, shear for circular and hollow sections with smooth bars require experimental and numerical studies.

Towards this aim, a recent comprehensive research program was initiated in the laboratory of structures and materials of University of Naples, Federico II. Such program, funded by the Italian Ministry of Research through the Project PON-FESR 2007-2013 (PON01_02366, STRIT) “Tools and Technologies for the Management of the Transportation Infrastructures” focuses on the assessment of the structural response analysis of as-built and retrofitted RC bridges. This research includes a series of shaking table tests to be carried on substructures reproducing typical existing simply supported highway bridges in Italy, as further discussed in the next paragraphs. A representative bridge system, which is assumed to be located in a region of medium-to-high seismicity in the South of Italy, is first assessed in its as-built configuration with simple supports at both end of the deck and then retrofitted with two types of base isolators that can be installed at the base of the deck. The design of such bridge configuration is based on displacement-based approaches [6].

This paper deals with the numerical seismic assessment procedure carried out in preparation for a set of shaking table tests on 1:3 scale bridge subassemblies. Incremental dynamic analysis are performed in order to characterize the seismic performance of the bridge considering the variability of input motions. A detailed report on the step-by-step validation of the proposed numerical model is presented along with the assumption made to account for the shear behavior. In compliance with the scope of the experimental program, the influence of the pier type (i.e. circular pier with full and hollow cross section) and the support conditions (i.e. with or without isolators) are investigated by means of incremental dynamic analysis.

2 SUBASSEMBLY NUMERICAL MODEL

2.1 Subassembly description

The selected bridge subassembly is a 1:3 scaled specimen representing existing structural systems typical of the Mediterranean area designed with obsolete non-seismic code provisions [7]. The subassembly is a single span bridge with two piers supporting the bridge deck (see Figure 1). Because the objective of the study is to evaluate the response of the bridge piers and supports, the deck was designed to remain in elastic range during the tests. Thus, the deck is used to simulate the inertial force resulting from the structural mass only. Two different RC pier types are investigated; the variable is the member cross-section with its geometry and internal reinforcements. A bridge pier with a circular cross section and a hollow-circular cross sections have been designed according to typical 70-80s Italian practice. The reduced design seismic intensity and the absence of adequate seismic details led to structural systems with insufficient lateral capacity. Such response is caused primarily by insufficient transverse and longitudinal reinforcements. Furthermore, smooth bars have been adopted for internal longitudinal and transverse reinforcements, according to old design practice.

To improve the evaluation of the seismic capacity of the single span bridge, different seismic isolation devices, along with the simply supported configuration, have been properly designed and manufactured for the scaled subassembly. Specific construction details have been installed in the pier cap to allocate the different supports to be used during the laboratory tests. The specimen geometry and reinforcements details are reported in Figure 1.

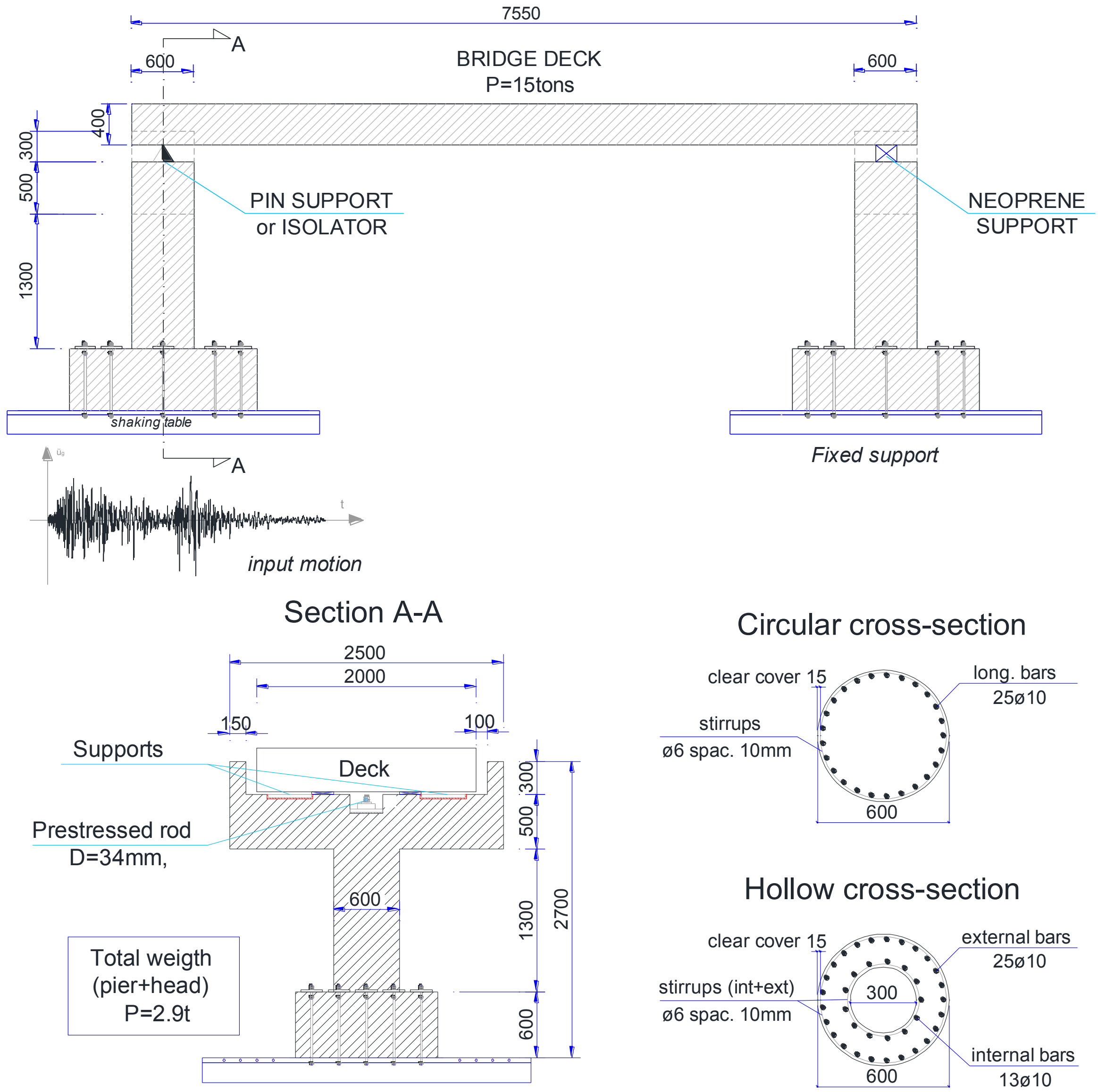


Figure 1: Details of the 1:3 scale bridge subassemblies (dimensions in mm).

The experimental shaking table tests, will be carried out in different configurations, varying the pier and support type and the input motions. However only four configurations, derived from the combinations of two piers and two different support configurations (simply supported or with Friction Pendulum Systems, FPS, isolators) have been analyzed in this study. In compliance with experimental tests, the input motion is applied to the left pier only (see Figure 1). Furthermore, to be representative of the real axial load available on typical bridge piers, further axial force have been introduced in the piers by means of internal prestressed rods. A total axial load about 0.05 (in terms of normalized axial load $\nu=N/(A_c f_c)$) will be adopted for all the tests. The deck supports will be subjected to the deck self-weight.

The specimens have been designed using materials properties commonly adopted in the old construction practice. In particular, a mean concrete compressive strength about 24 MPa have been adopted in the calculation. The AQ50 steel (f_{ym}=370 MPa) has been adopted for internal reinforcements. Preliminary tests on reference coupons confirmed the assumed mechanical properties.

The adopted friction pendulum (FP) isolators have been experimentally characterized through dynamic tests at different frequencies and displacement amplitudes. Further details on the device specifics are reported in the next paragraphs.

2.2 Numerical models

One of the objectives of present study is to built a reliable and efficient numerical model of the selected bridge subassembly able to represent the real mechanical characteristic of the system until the collapse. Thus, a complete characterization of the specimen behavior with particular care of the seismic relevant response characteristics and failure mode is needed. This aspect has been found to be very sensitive for the seismic assessment of RC structural systems [8], [9]. The low amount of transverse reinforcements strongly reduce the lateral capacity of the piers, resulting in low ductility capacity and potential shear failure. To account for these failure mechanisms, proper numerical models should be adopted for the specimen response prediction. Available literature studies [8]–[10] pointed out that, recently, specific computer programs have been developed to reliably predict the hysteretic behavior of shear critical members. New mechanical models and solution algorithms have been developed and great effort has been made to combine them with the state-of-the-art knowledge in refined tools suitable for the use in the practical applications (e.g., OpenSees [11], VecTor programs [12], among others). The numerical models considered in this study are based on the Modified Compression Field Theory (MCFT) [15] and Disturbed Stress Field Model (DSFM)[13]. Both these theories have been developed at University of Toronto. In this paper, the response of the selected bridge piers will be predicted by mean of two different computer software (Response 2000, R2k [14], and VecTor2, VT2 [18]). The latter software presents an increasing level of difficulty in the modeling and, in turn, higher accuracy in the response prediction.

As described in the previous sections, the two piers of the single span bridge present structural details that could strongly affect the structural response. As well as these details are typical of existing structural viaducts, they can be difficult accounted in the numerical modelling. Furthermore, the dynamic response of the sample columns has been not completely characterized. Thus, due to the peculiarities of these structural members, the simplified numerical models may be approximated and present many sources of uncertainties in the response prediction. To develop a reliable numerical model suitable for shaking table response prediction and parametric studies, a step-by-step validation against experimental tests on specimens with the same structural details is adopted. Once that the proposed VecTor2 numerical model is validated, it is thus adopted to predict the hysteretic response of the two bridge piers. Later, the hysteretic response will be used to calibrate a more efficient OpenSees model of the piers. This model will be included in the dynamic model of the bridge subassembly along with the different support configurations. To move from a static model to a dynamic one a further step of validation against a shaking table test is needed in order to proper understand how to manage the input motion, masses, damping and solution algorithms.

2.3 Step-by-step modelling

Although the shear behavior of RC column subjected to cyclic loads can accurately captured by the MCFT/DSFM [8], [10], difficulties arise in the model definition in the case of circular members, especially if they have an hollow core. In fact, the behavior of a circular cross section can be accurately reproduced in a 2D model by linearizing the curve shape and preserve the area and the inertia [9], [15], the reduced confinement pressure of the hollow cross section is difficult to be accounted in a 2D model. This phenomena was investigated by Ranzo and Priestley [16] that suggested for practical purpose to reduce the confinement pressure calculated for a circular cross section of the 30% to account of the hollow core. To validate the proposed VecTor2 numerical model for this particular case, the analytical results were compared against experimental evidence for the columns HS2 and HS3 (Ranzo and Priestley, 2000) exhibiting a shear failure. The consolidated modelling procedure [8]–[10] has been adopted to define the member mesh, longitudinal reinforcements, percentage of in-plane and out of-plane transverse reinforcements, applied load and displacement. Also in this case, VecTor2 default material properties and analysis options have been adopted except for the concrete constitutive model by Popovic and Mander [17], [18] is adopted to reproduce concrete compressive behavior.

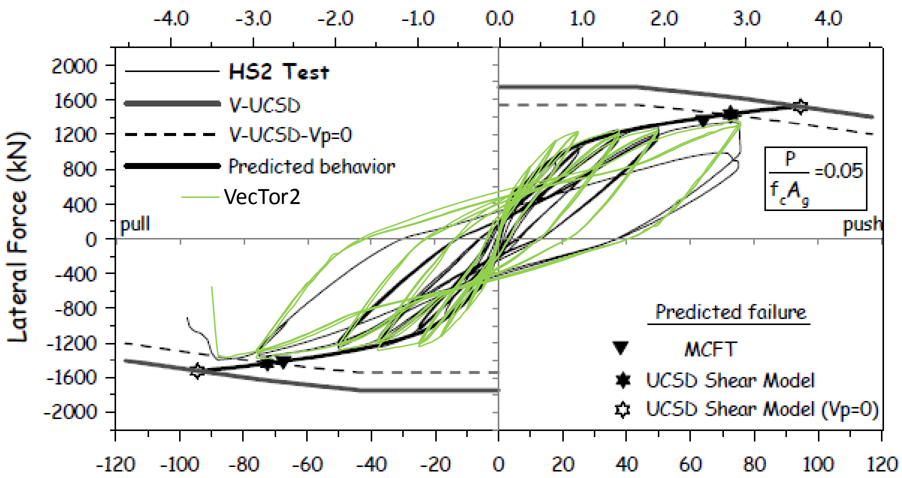
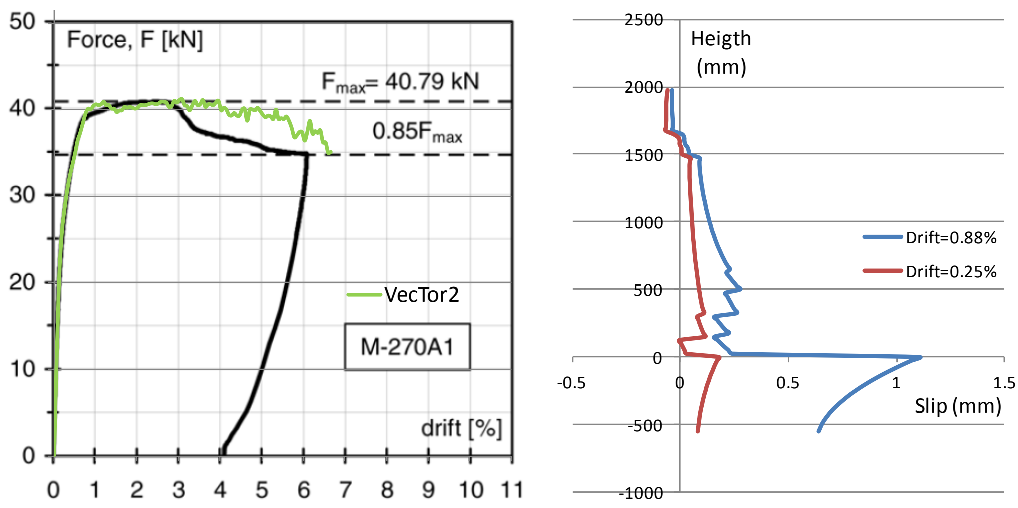


Figure 2: Comparison of experimental and analytical response for HS2 column [16].

The VecTor2 numerical model show a good match with experimental results in terms of strength and deformability subjected to monotonic and cyclic loads. Thus, because no calibrations have been made in order to fit the experimental results, it can be adopted for the prediction of the structural response of hollow circular column that failed in shear. Response 2000 is also adopted to estimate the column shear strength, as a reference value. Although a good match with experimental results has been achieved for the selected hollow circular columns, it is less accurate than VecTor2 in the estimation of the member deformations. This is due to the assumption at the base of the MCFT that assumes no slip on the cracks and could not work properly in the disturbed stress field regions as at the base of the column in which there is a significant flexural-shear interaction.

Another aspect that should be considered in modelling the bridge piers is the presence of smooth longitudinal reinforcements that strongly affects the structural behavior of reinforced concrete columns [19]–[22]. The significant slip at the interface of concrete and reinforcements commonly resulted in a characteristic failure mode, with a large crack at the column base and a significant rocking mechanism. The numerical modelling of the structural members with smooth bars is quite a challenging task. The significant slip between concrete and internal reinforcements fight against the basic assumption of perfect bond of the classic theories of RC. VecTor2 can use a piecewise linear bond slip model in terms of τ-slip to account for the effective slip between concrete and reinforcements at each node of the 2D mesh. The adopted numerical model was validated against experimental results on a shear critical frame tested by Paolacci and Giannini (2012) (Del Vecchio et al. 2015). However, a more specific validation at the column level is required to evaluate the accuracy of the model predicting the strong nonlinear behavior at high level of ductility characterized by marked rocking mechanism. For this purpose the experimental test on underdesigned RC column tested by Verderame et al. [19], [20] are adopted for the validation. The selected test (M270-A1) allows also to account of the overlap length of the longitudinal reinforcements at the column base. This is a typical construction detail that can be properly accounted only with an accurate modelling of the bond-slip behavior. Furthermore, the base overlapping may strongly affect the structural response due to the stress concentrations at the hooked ends detrimental for other local phenomena such as bar buckling and bond degradation.

The smooth internal reinforcements have been modeled by using link elements in VecTor2. A typical bond-slip behavior in terms of τ-slip directly derived from pull-out tests have been adopted to characterize the link elements along the bar and in correspondence of hooked anchorages [21]. These elements have been inserted in each node of the mesh between the concrete and the truss elements adopted for the longitudinal reinforcements.



(a) (b)

Figure 3: (a) Comparison between experimental results (Verderame et al. 2008) and adopted numerical models (VecTor2); (b) Analytical slip distributions along member height.

The comparison with experimental results in Figure 3a, demonstrated the accuracy of the proposed analytical model reproducing the column monotonic behavior and the failure mode characterized by a significant rocking followed by concrete crushing. The slip distribution in Figure 3b shows that before the first cracking (red line) the slip is uniformly distributed along the specimen height. On the other side, at the yielding of longitudinal reinforcements (blue line) the slip are concentrated at the column base. This allows the use of simplified numerical models (i.e. OpenSees model with strain penetration model at the column base, Bond_SP01 command) to reproduce the nonlinear behavior of columns with smooth internal reinforcements [23]. However, this model can lead to initial stiffness overestimation because of slip concentration at the column base instead of a more realistic uniform distribution along the height in the elastic range. Thus, a proper calibration of the slip at the significant point of the longitudinal reinforcement stress is needed.

2.4 Pier response prediction

A monotonic push-over analysis was carried out with VecTor2 nonlinear FEM model in order to define the nonlinear behavior of the two bridge piers. The member dimensions and the cross section details are those reported in Figure 1. In compliance with the design assumption and preliminary tests, the material properties adopted in the analysis are: concrete compressive strength f_{cm}=24MPa; the steel adopted for the ϕ10 longitudinal reinforcements is characterized by f_{ym}=370MPa and an ultimate stress f_{su}=480MPa. The elastic modulus is about 195000MPa, the strain at the hardening ε_{sh}=0.0032 and ultimate strain ε_{su} = 0.022. The steel adopted for ϕ6 transverse reinforcement is characterized by a f_{ym}=400MPa and an ultimate stress fsu=480MPa. The elastic modulus is about 195000MPa, the strain at the hardening ε_{sh}=0.0035 and ultimate strain ε_{su} = 0.017. The adopted mechanical properties are in compliance with mechanical characterizations of the steel AQ50 reported in available literature studies ([20], [22], [24]).

The column axial load is computed considering a normalized axial ν load equal to 0.05 (where ν=N/($A_c f_{cm}$)). Because of the differences in the effective column cross section, axial force about 254kN and 340kN loads the hollow and the circular column, respectively.

The Vector2 modelling procedure described in the previous sections (including bar bond-slip, circular-hollow cross section, longitudinal reinforcement overlapping) has been adopted. The pier model includes the foundation block and the pier cap. The imposed external force and displacement have been applied at the pier cap in compliance with test setup for shaking table tests. The response prediction for both the piers is reported in Figure 4.

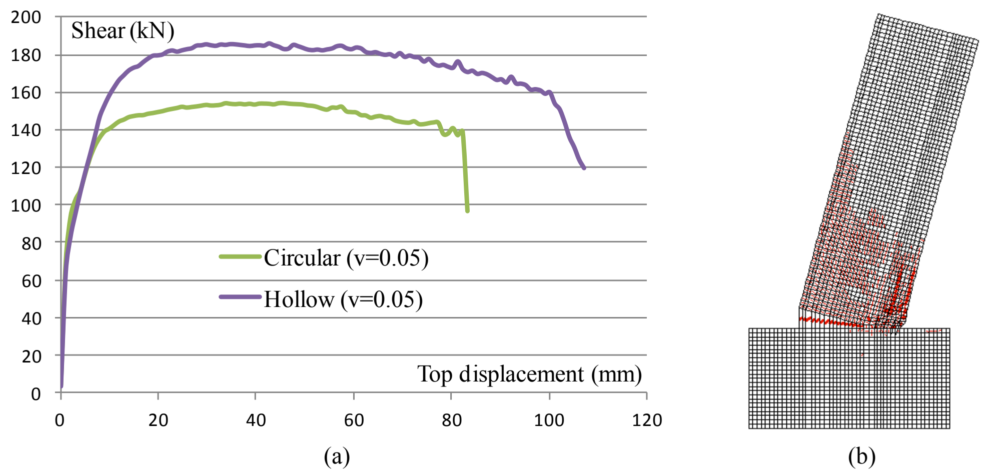


Figure 4: VecTor2 response prediction for both bridge piers: (a) monotonic response; (b) detected failure mode.

The monotonic response of the two piers shows a typical nonlinear behavior of RC column with smooth internal reinforcements. The high initial stiffness depends by the low member aspect ratio (L/h≈2), typical of squat columns. Although a low amount of transverse reinforcement has been adopted, a ductile failure mode occurs. This is related to the significant slip of the longitudinal reinforcements at the column base which allows the concentration of inelastic deformation at the column base. Once that the reinforcement yielding has been achieved, a marked nonlinear behavior without significant hardening phenomena occurs. For large displacement demands, significant rocking phenomena can be detected. The concrete crushing and bar buckling limit the member ductility. The crack pattern analysis at the failure mode, confirmed this trend, also pointed out in other experimental tests [19], [20], [22], [23].

Both sample piers, i.e. the hollow and the circular piers, have a structural behavior characterized by a marked rocking mechanism. Due to the high strength and stiffness degrading phenomena of the pier response, a more accurate characterization can be achieved performing a quasi-static cyclic analysis with increasing top displacement.

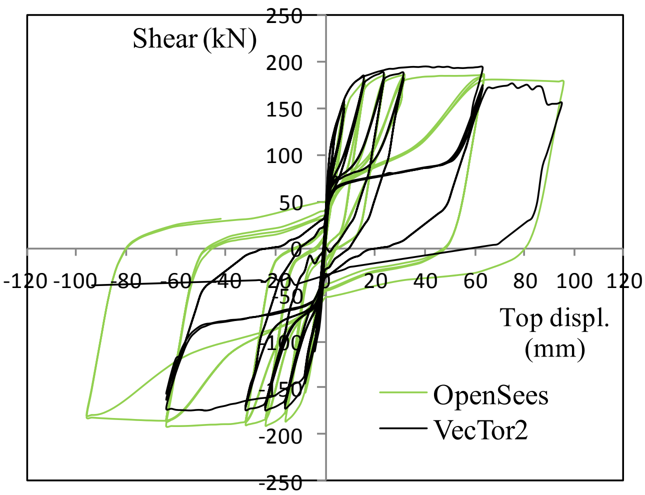


Figure 5: Comparison of the two proposed analytical models for the hollow bridge pier.

The cyclic response is similar to the monotonic response in terms of maximum shear strength both for the circular and hollow piers (see Figure 5). Significant differences can be observed in terms of the ultimate displacement capacity. The cyclic response is characterized by a displacement capacity lower than the monotonic response and this is related to the concrete and bond degradation. The ultimate displacement capacity, conventionally measured in correspondence of a strength reduction of the 20% respect to the peak strength, is about 72 mm (5.3% Drift) for the hollow pier and 58 mm (3.9% Drift) for the circular one. Both the drift and top displacement are measured in correspondence of the pier top end. The cyclic response shows marked pinching effects, typical of rocking mechanism related to significant slips at the column base.

2.5 OpenSees models

With the scope to achieve a wide seismic characterization of the bridge subassemblies by means of nonlinear time-history analysis (NLTH) a more efficient numerical models need to be created. The latter requirement is due to the large computational demand associated with VecTor2 software [8], [9]. In compliance with consolidated modelling procedures for RC columns [8]–[10], an OpenSees non-linear fiber model has been created including the foundation block and the pier cap. In the fiber model, three different materials are used to represent the cyclic behavior of column cross sections: the confined and unconfined concrete is simulated with the nonlinear *Concrete01* material; the longitudinal reinforcement is modelled with the *Reinforcing Steel* uniaxial material with isotropic strain hardening. This command is strongly suggested for dynamic model of RC elements subjected to input motion, in order to accurately reproduce energy dissipation and residual displacements [25]. The confining effects of transverse reinforcement on the mechanical properties of the concrete core are calculated according to Mander et al. [18]. The ultimate compressive strain of confined concrete is calculated according to Fardis [26]. The uniform radial discretization scheme is employed, and each section is divided into 50 fibers in circumferential direction, 8 fibers in the radial direction in the core, and 2 fiber in the radial direction in the cover. The *Bond_SP01* command is used to in-

clude in a *zeroLengthSection* element the effect of bar slip and strain penetration at the column base as suggested by Arani et al. [23] and with opportune calibration of the slip parameters. Five integration points along the finite elements have been adopted, thus allowing the spreading of inelasticity to be accurately described. Applied axial load (*P*) and cyclic displacement (Δ) are determined in accordance with the VecTor analysis. The P-Δ effects are accounted by means of the geometric transformation tool *PDelta Transformation*. The Newton-Raphson solution algorithm is adopted to solve model nonlinear equations.
The OpenSees model provides a close match to the the VecTor 2 prediction except for the strength loss at the ultimate displacement (see Figure 5). For this reason the ultimate displacements pointed out by the VecTor2 analysis should be adopted to identify the collapse.

In order to validate the OpenSees modelling procedure also for dynamic loads, the proposed analytical model has been adopted to predict the dynamic response of shaking table tests performed by Hachem et al. [27]. The effects of added mass are accounted both in the lateral and rotational displacements. Furthermore additional viscous damping proportional to the current stiffness has been included in the model by means of the *Rayleigh Damping* command with the initial value of 2% as suggested by Petrini et al. [28]. The input motion has been defined by means of the *UniformExcitation* pattern in terms of acceleration history. The analysis has been performed with the *AccelSeries* command with a time step of 0.01s. The results of the comparison is reported in the Figure 6.

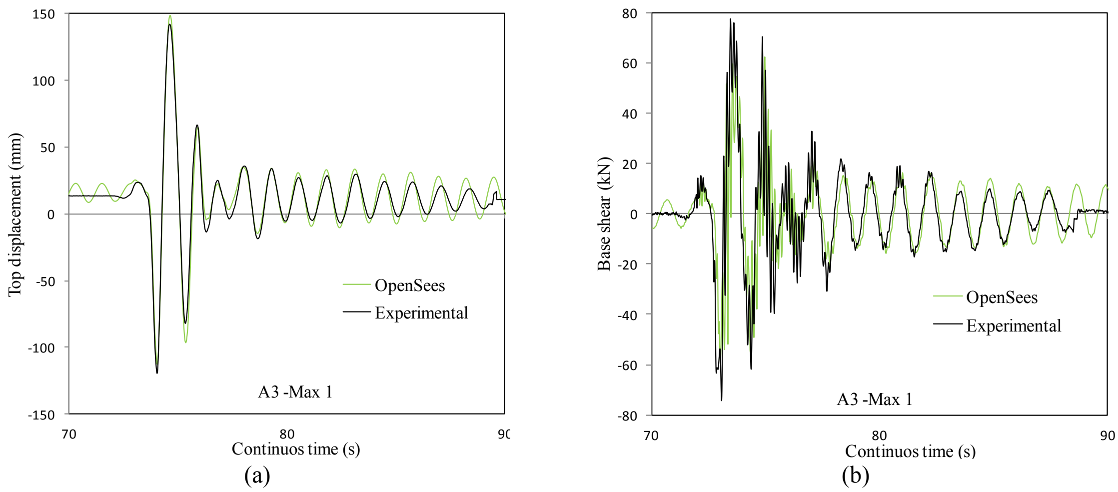


Figure 6: Comparison of experimental test and analytical results in terms of top displacement history; (a) run A3-Max1 (peak displacement); (b) base shear comparison at run A3-Max1.

The results presented in Figure 6 show the accuracy of the numerical modeling in predicting the dynamic behavior of the bridge pier. Comparing the displacements and the base shear history of the A3-Max1 run (Figure 6a and b), a close match with experimental results has been achieved. A low scatter of about 10% has been achieved in the estimation of maximum displacement.

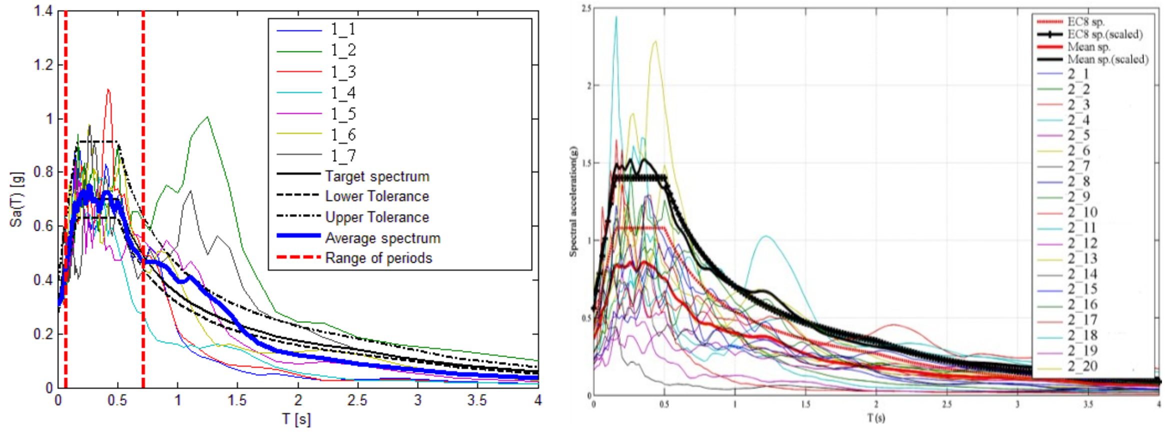
Once that the proposed modelling procedure has been validated for dynamic loads, the numerical model of the entire bridge subassembly has been created. The OpenSees model is composed of two bridge piers modeled in order to reproduce the VecTor2 hysteretic response predictions and including dynamic properties and input motion as proposed for the Hachem et al. [27] pier. In order to include the FP isolator on the top of the left pier, a *zeroLength* element has been introduced between node 7 and 8 not included in the Rayleigh damping calculation. The FP hysteretic behavior has been reproduced by means of the Steel01 uniaxial material in the horizontal direction. In order to match the experimental behavior of tested FP and considering that two FP devices support the bridge deck on the left pier, the following parameters have been set: yielding force F_y=5000N, initial elastic modulus E=10000MPa, hardening ratio b=0.0036. The authors want to specify that this model is properly built to have preliminary results to make a subassembly characterization for shaking table tests. Thus, the analyzed support configurations (pin-roll or isolated-roll) may not reflect the real ones.

The eigenvalue analysis and the analysis of the response to a white-noise signal carried out before the record motion analysis showed a fundamental period (T_1) ranging between 0.10 and 0.12s for the simply supported configuration. Using higher amplitude signal the period evolves until 0.4s at the ultimate displacement. A period significant higher has been detected for the isolated bridge. It ranges between 2.2 and 4s.

3 INCREMENTAL DYNAMIC ANALYSIS

3.1 Record selection

Two sets of record motions have been selected for this study. The records have been selected basing on the hazard disaggregation for the site selected for this study. In particular, the sets consists of far-field records with a magnitude in the range 5.2-7.0 and epicentral distance in the range of 10–30 km. The first set of records has been selected respecting the spectrum compatibility with the design response spectrum built in compliance with Italian building code [29] at the life safety limit state for a soil type B by means of the REXEL software [30]. As reported in Figure 7a, the spectrum compatibility is guaranteed in the entire range of variation of the structural period for the as-built configuration. However, it can be observed that by scaling the selected records, the average response spectrum matches the target spectrum also for period larger than 2.0s. Thus, the same set can be used in the seismic assessment of the isolated system. Because few records matching these requirements can be found in the European strong motion database, a new set of records has been selected from the PEER strong motion database of worldwide records. The record selection has been performed with the software ISSARS [31] by matching the EC8-pt2 Bridges spectrum [32]. The target spectrum for a soil type B and reference PGA 0.36 has been adopted to match the average response spectrum. All the records are on a C/D soil type, according to the National Earthquake Hazards Reduction Program (NEHRP) classification. As depicted in Figure 7b, an acceptable spectral matching has been achieved at all the periods and this allow to use the same records both for as-built and isolated configuration. The damage factor (ID) [33] was computed for all records, and only records with a reduced damage were selected. This specific rule was included in the selection of records because it allows the consideration of ground motion properties (e.g., number of plastic cycles and energy content of earthquake) that can strongly affect the dynamic response of SSD systems. More details about the selected records are reported in Table 1and Table 2.



(a) (b)

Figure 7: Spectrum compatibility of the selected records: (a) set 1; (b) set 2.

N°	Event	Location	Station ID	Year	M	EC8 Site Class	Epic. Dist.	PGA	PGV	I_A	I_D
[-]	[-]	[-]	[-]	[-]	[-]	[-]	[km]	[g]	[cm/s]	[cm/s]	[-]
1_1	Patras	Greece	ST178	1993	5.6	B	27	0.05	4.7	2.7	7.3
1_2	Campano-Luc.	Italy	ST276	1980	6.9	B	16	0.16	26.8	104.9	16.0
1_3	Patras	Greece	ST178	1993	5.6	B	27	0.04	2.3	2.1	13.8
1_4	Potenza	Italy	ST103	1990	5.8	B	24	0.04	1.9	1.5	14.4
1_5	Almiros-aftersh.	Greece	ST1300	1980	5.2	B	14	0.07	5.5	4.7	7.6
1_6	aftershock	Montenegro	ST77	1979	6.2	B	20	0.06	4.3	4.1	11.0
1_7	aftershock	Montenegro	ST63	1979	6.9	B	24	0.30	38.5	185.1	10.3

M, earthquake magnitude; EC8, Eurocode 8; Epic. Dist., Epicentral distance; PGA, peak ground acceleration; PGV, peak ground velocity; I_A, Arias intensity; I_D, damage factor.

Table 1: Set 1 of earthquake records selected for the study.

3.2 Scaling procedure

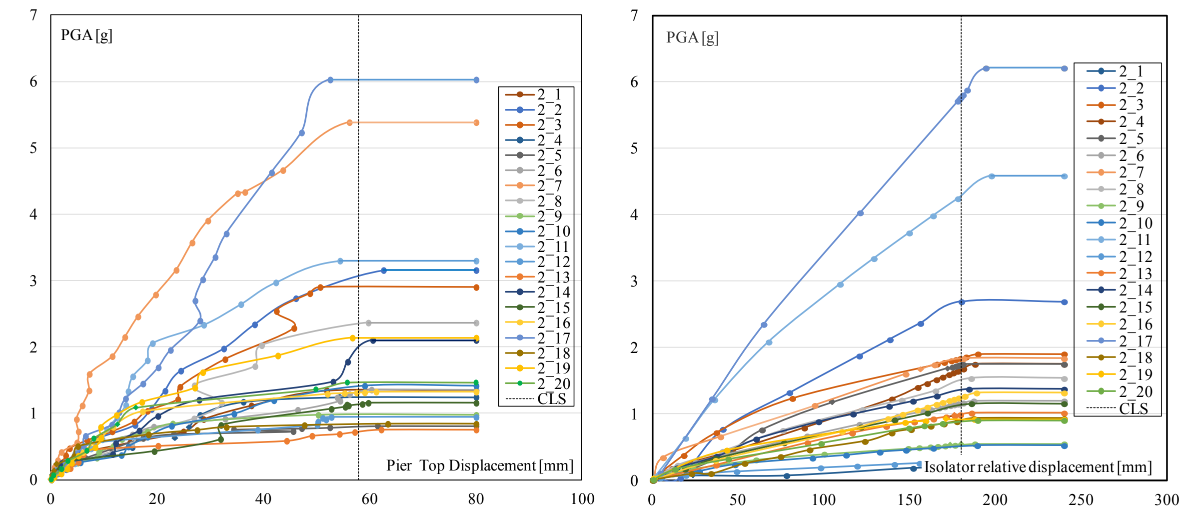
The proposed advanced numerical models have been used to run NLTHs. The intensity measure (IM) is represented by the spectral acceleration for the elastic period Sa(T_1, 5%), while the pier top displacement, measured in correspondence of the pier top end, is adopted as damage index (ID). The ultimate capacity is fixed at the displacement identifying the collapse of the piers or the FP devices. However, due to the high variability of the nonlinear dynamic response, a proper estimation of the motion intensity at the collapse is not easy. As a result, the nonlinear dynamic analyses have been carried out according to the IDA procedure [17]. The Hunt and Fill optimized scaling procedure has been adopted to scale the record to incremental IMs, optimizing the number of runs to identify the collapse with an accepted level of confidence (i.e. 10%).

A total of 1300 NLTH analyses have been carried out for the four studied structural systems. Twelve runs at different IMs for the two sets of record motions (7 and 20 records, respectively) were required to properly assess the seismic capacity. The results of the analysis in the form of IDA curves are reported in Figure 8. Although the spectral acceleration has been used as IM to scale the selected records, the results are presented in terms of PGA in order to allow the comparison between the as-built and isolated configuration.

N°	Event	Location	Station	Year	M	NEHRP Soil	Epic. Dist.	PGA	PGV	I_A	I_D
[-]	[-]	[-]	[-]	[-]	[-]	[-]	[km]	[g]	[cm/s]	[cm/s]	[-]
2_1	Imp. Vall.-02	California	El Centro Arr #9	1940	7.0	D	13	0.26	29.7	170.4	14.2
2_2	Friuli-01	Italy	Tolmezzo	1976	6.5	C	20	0.35	22.0	78.0	6.5
2_3	Tabas	Iran	Dayhook	1978	7.4	C	21	0.35	20.5	142.3	12.6
2_4	Imp. Vall.-06	California	El Centro Arr #10	1979	6.5	D	26	0.21	47.5	56.8	3.7
2_5	Irpinia-01	Italy	Calitri	1980	6.9	C	15	0.15	16.4	57.8	14.7
2_6	Corinth	Greece	Corinth	1981	6.6	D	20	0.26	23.3	69.3	7.2
2_7	Nahanni	Canada	Site 3	1985	6.8	C	22	0.15	6.1	28.3	19.7
2_8	New Zeal-02	New Zeal.	Matahina Dam	1987	6.6	C	24	0.29	21.7	65.2	6.5
2_9	Super.H.-02	California	Westm. Fire Sta	1987	6.5	D	20	0.21	23.5	80.3	10.4
2_10	Super.H.-02	California	Wildl. Liquef. Arr	1987	6.5	D	29	0.19	29.9	69.4	7.7
2_11	Loma Prieta	California	UCSC Lick Obs.	1989	6.9	C	16	0.46	18.7	266.1	19.9
2_12	Cape Mend.	California	Fortuna-Blvd	1992	7.0	C	30	0.12	29.9	26.1	4.7
2_13	Cape Mend.	California	Rio Dell Over-FF	1992	7.0	D	23	0.42	43.8	152.3	5.2
2_14	Landers	California	Desert Hot Sprin.	1992	7.3	D	27	0.14	20.0	70.7	15.9
2_15	Landers	California	Morongo Valley	1992	7.3	D	21	0.16	16.6	95.8	22.8
2_16	Northrid.-01	California	Arleta-Nord Fire St	1994	6.7	D	11	0.33	40.4	152.3	7.3
2_17	Northrid.-01	California	Glendal -Las Palm.	1994	6.7	C	30	0.26	12.2	117.2	23.8
2_18	Kobe	Japan	Kobe University	1995	6.9	C	25	0.30	54.8	122.3	4.7
2_19	Duzce	Turkey	Lamont 1059	1999	7.1	C	24	0.13	12.0	38.2	15.5
2_20	Hector Mine	California	Hector	1999	7.1	C	27	0.31	28.6	83.0	6.1

M, earthquake magnitude; NEHRP, National Earthquake Hazards Reduction Program; Epic. Dist., Epicentral distance; PGA, peak ground acceleration; PGV, peak ground velocity; I_A, Arias intensity; I_D, damage factor.

Table 2: Set 2 of earthquake records selected for the study.



(a) (b)

Figure 8: IDA curves for bridge subassembly with circular piers: (a) Set 2 “as-built” configuration; (b) Set 2 isolated.

The results of NLTHs can be useful to make important considerations on the variability of the seismic performances with the input motion, pier type and on the effectiveness of the FP isolators. The median PGA at the collapse limit state are reported in Table 3 for the eight different test configurations. The collapse limit state it has been identified with the ultimate placement of column or bearings.

Median PGA [g] at the collapse				
	as-built		isolated	
Set	Circular (58mm)	Hollow (72mm)	Circular (180mm)	Hollow (180mm)
1 (7 records)	1.253	1.708	3.504	3.304
2 (20 records)	1.403	1.606	1.261	1.103

Table 3: Summary of the Median values in terms of PGA at the collapse of the piers or the FP for the 8 analyzed configurations.

Because the safety check on the seismic demand requirements for the selected bridge is out of the scope of this paper, only some considerations on the effects on the seismic response of pier type, input motion and support configuration are made. The NLTHs confirm the higher seismic capacity of the bridge pier with hollow cross section, due to the displacement capacity higher than the circular pier. The Median PGAs at the collapse of the two piers are almost in the same ratio as for the displacement at the collapse. This because the initial stiffness is very similar and only significant differences in the ultimate displacement and yielding strength can be appreciated (see Figure 5). No significant differences can be observed in the results respect to the input motion variability. The use of FP isolators allow to move the failure from the piers to the devices. In fact, FP devices allow to strongly reduce the acceleration transmitted to the piers, which response becomes essentially elastic not exceeding the flexural cracking. The comparisons in terms of median PGA between the as-built and isolated configuration shows a slight reduction in maximum seismic demand for the isolated systems. This results can be related to the variability of the input, but also to the configuration of the isolators, placed on the left pier only, while they have been designed to be applied on both piers. Further analysis are required to characterize the response of the bridge in the other configurations.

The high variability in the seismic response of the isolated systems respect to the set of input motions can be related to the particular spectral shape in terms of displacements of the records of the set 1. This evidence point out that further analysis, with displacement spectra matching the target displacement spectrum, are needed in order to accurately characterize the seismic response of the isolated systems.

The NLTHs can be used also to select the input motion for shaking table tests. For this purpose, one of the record of the set 2 has been selected. Although, the records of the set 1 are all selected from the European strong motion database, and, thus more representative of the Italian earthquakes, they have a spectral shape in term of displacement irregular with a descending slope for long periods. The selected record should be close, as much as possible, to the median value of the PGA both for the as-built and isolated configuration.

4 CONCLUSIONS

The present paper deals with the numerical seismic assessment of a bridge subassembly which will be tested on shaking tables. A total of 1300 NLTHs have been carried out investigating the variability of pier type (i.e. circular or circular-hollow cross-section), input motion variability and support configuration (with or without FP devices). To be representative of the real behavior of the structural system and provide reliable information to design a shaking table test program, an advanced numerical model has been created. The model was built in the

OpenSees environment, however to reliable predict the pier hysteretic response, preliminary analysis with VecTor2 program have been conducted. In fact, the presence of smooth longitudinal reinforcements and the lack of proper transverse reinforcements may allow the occurrence of premature shear failure or other nonlinear phenomena which is typical of old-type members and difficult to be accounted for in the classical fiber-based program. The model has been validated at different levels against experimental results.

Preliminary results of NLTHs show the influence of pier ductility on the seismic response and the effectiveness of the adopted seismic isolation solution is demonstrated. The use of FP devices as supports for the bridge deck limits the inertial force transmitted to the piers. The piers remained mostly in elastic range. Further considerations can be made on the effectiveness of the proposed record selection for seismic isolated structures. In fact, the spectral matching with the target acceleration spectrum does not allow to control the variability in the response of displacement sensitive systems. High variability in the seismic response is pointed out comparing the response of isolated bridge subjected to the records of set 1 and set 2. This emphasizes that a more appropriate record selection considering the displacement spectra is needed. The final results of this study can be useful to determine the support configurations and the earthquake signal for shaking table tests.

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