COMPDYN 2023
9th ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, M. Fragiadakis (eds.)
Athens, Greece, 12-14 June 2023

INELASTIC TIME-HISTORY ANALYSES OF FRP-RETROFITTED BRIDGE COLUMNS SUBJECTED TO NEAR-FIELD GROUND MOTIONS

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Abstract

The material model proposed by Megalooikonomou et al. (2012) was added to the source code of OpenSees as a uniaxial material. In order to evaluate the performance of this material model, it was implemented in the simulation of a series of shaking table tests performed by Kumar and Mosalam (2015). The two test specimens, having identical geometry and reinforcement details but different spacing of the transverse bars (one column had closely spaced stirrups satisfying code requirements, and the other had larger spacing, representing a shear-critical column), were subjected to a series of horizontal and vertical excitations on a shaking table and experienced moderate to high damage. The damaged columns were subsequently repaired with unidirectional CFRP and GFRP composite laminates and subjected to the same set of near-field earthquake excitations as the one tested in as-built configuration. Both specimens were simulated using nonlinear beam with hinge elements, in which the FRP-confined concrete was modelled using the aforementioned material model. Comparison between the numerical and experimental time history response of the column is indicative of the effectiveness of the implemented modelling. The numerical model confirmed that the repaired columns were able to resist higher shear and developed fewer peaks of tensile forces due to the vertical component of the ground motion. Therefore, the effect of changes of axial load from compression to tension due to the vertical component which can result in significant decreases in column shear strength, was diminished through the contribution of the FRP jacket. Finally, after performing all time history dynamic analyses, the residual displacements were found to be much lower for both repaired specimens in comparison to their bare counterparts. This is indicative of the effectiveness of the investigated FRP repair technique to produce resilient bridge columns when subjected to earthquake loading with strong vertical component.

Keywords: Bridge Pier, FRP jacket, Vertical Excitation, Nonlinear Dynamic Analysis, OpenSees.

1 INTRODUCTION

The library of materials, elements and analysis commands makes OpenSees [1] a powerful tool for numerical simulation of nonlinear structural and geotechnical systems. The OpenSees library of components is ever-growing and at the leading edge of numerical-simulation models. Its interface is based on a command-driven scripting language which enables the user to create more-versatile input files. OpenSees is not a black box, making it a useful educational tool for numerical modeling. Material, element or analysis tools can be incorporated into OpenSees.

Addition of a new uniaxial material module by the developer is achieved by providing a new C++ subclass of the UniaxialMaterial class, along with an interface function which is used to parse the input and create the new material. Unlike the C and Fortran modules, no information about the state of the model is provided as argument to the material routine. Retaining the required information and rejection of the unnecessary one is performed within the material model. This information includes simultaneously (a) parameters, i.e., information needed to define the material, and (b) state variables or history variables, i.e., information needed in order to define its current state and, consequently, compute the applied stress and tangent.

The present work provides information on the implementation of a recently developed material model for Fiber Reinforced Polymer (FRP)- and Steel – confined concrete, proposed by Megalooikonomou et. al. [2], in OpenSees under the name '*FRPConfinedConcrete*'. To date, the model has no tensile strength and uses the degraded linear unloading/reloading stiffness in the case of cyclic loadings based on the work of Karsan and Jirsa [3].

In order to evaluate the performance of this material model, it was implemented in the simulation of a series of shaking table tests performed by Kumar and Mosalam (2015) [4]. The two ¼-scale test specimens, having identical geometry and reinforcement details but different spacing of the transverse bars (one column had closely spaced stirrups satisfying code requirements, and the other had larger spacing, representing a shear-critical column), were subjected to a series of horizontal and vertical excitations on a shaking table and experienced moderate to high damage. The damaged columns were subsequently repaired with unidirectional CFRP and GFRP composite laminates and subjected to the same set of near-field earthquake excitations as the one tested in as-built configuration.

Strong vertical shaking is usually considered in near-field regions and may lead to severe impacts on structures such as buildings and bridges. To be more specific, the Northridge Earthquake (January 17, 1994) whose generated ground motions recorded at Pacioma Dam Station are employed in the current study, was magnitude 6.7 on a previously unidentified thrust fault, creating strong horizontal and vertical ground motion lasting up to 10 seconds. The area of strongest shaking in the earthquake was about 50 km in diameter from Northridge, California, encompassing southern Ventura and northern Los Angeles counties. Earthquake activity was felt as far away as Las Vegas and San Diego. Strong forward directivity was observed also during this seismic event. Directivity is related to the rupture direction. Forward directivity is the case in which the rupture is toward the site while backward directivity is the case in which the rupture is away from the site. For forward directivity, if the slip direction and the rupture direction are aligned, then the SH waves generated between the epicenter and the site will arrive at about the same time at the site and at long periods the SH waves will constructively interfere. For most of the rupture, the SH waves will be primarily on the fault normal component. This leads to a large 2-sided velocity pulse on the fault normal component. Seven major freeway bridges in the above-described area collapsed, and 212 were damaged, disrupting traffic in the Ventura-Los Angeles region for weeks following the earthquake.

Among all the components of a bridge, columns are the most critical from a structural point of view and therefore require precise seismic assessment and retrofit. Confining wraps or jackets to rehabilitate and strengthen existing bridge columns has proven to be an efficient technique for seismic retrofit of structures. However, most of the compressive strength models of confined concrete only consider the increased strength and ductility provided by fiber reinforced polymers (FRPs), neglecting the contribution of the existing steel reinforcement inside the column's section. Even if the existing steel stirrups in a reinforced concrete column are not sufficient to confine the concrete core they must also contribute, along with the FRP jacket, in confining the section. In this context, the Fiber Reinforced Polymer (FRP)-confined concrete model contained in a well-known Bulletin by the International Federation for Structural Concrete (*fib*) [5] has been enhanced to take into account the superposition of the confining effects of the already existing steel reinforcement with that of the FRP jacketing applied when retrofitting reinforced concrete (RC) columns (i.e., '*FRPConfinedConcrete*' implemented in OpenSees).

2 FRP-RETROFITTED BRIDGE COLUMNS' MODELING IN OPENSEES

The OpenSees command used in order to construct the uniaxial '*FRPConfinedConcrete*' is provided in the following syntax:

uniaxialMaterial FRPConfinedConcrete $tag $fpc1 $fpc2 $epsc0 $D $c $Ej $Sj $tj $eju $S $fyl $fyh $dlong $dtrans $Es $vo $k $useBuck

Each input parameter defined above corresponds to the mechanical and geometrical properties of the FRP&Steel-confined element which affect its overall performance. Their description is provided in Table 1.

Table 1: '*FRPConfinedConcrete*' input parameters.

1	***tag***	*Material Tag*
2	***fpc1***	*Concrete Core Compressive Strength*
3	***fpc2***	*Concrete Cover Compressive Strength*
4	***epsc0***	*Strain Corresponding to Unconfined Concrete Strength*
5	***D***	*Diameter of the Circular Section*
6	***c***	*Dimension of Concrete Cover*
7	***Ej***	*Elastic Modulus of the Jacket*
8	***Sj***	*Clear Spacing of the FRP Strips - zero if it's continuous*
9	***tj***	*Total Thickness of the FRP Jacket*
10	***eju***	*Rupture Strain of the Jacket*
11	***S***	*Spacing of the Stirrups*
12	***fyl***	*Yielding Strength of Longitudinal Steel Bars*
13	***fyh***	*Yielding Strength of the Hoops*
14	***dlong***	*Diameter of the Longitudinal Bars*
15	***dtrans***	*Diameter of the Stirrups*

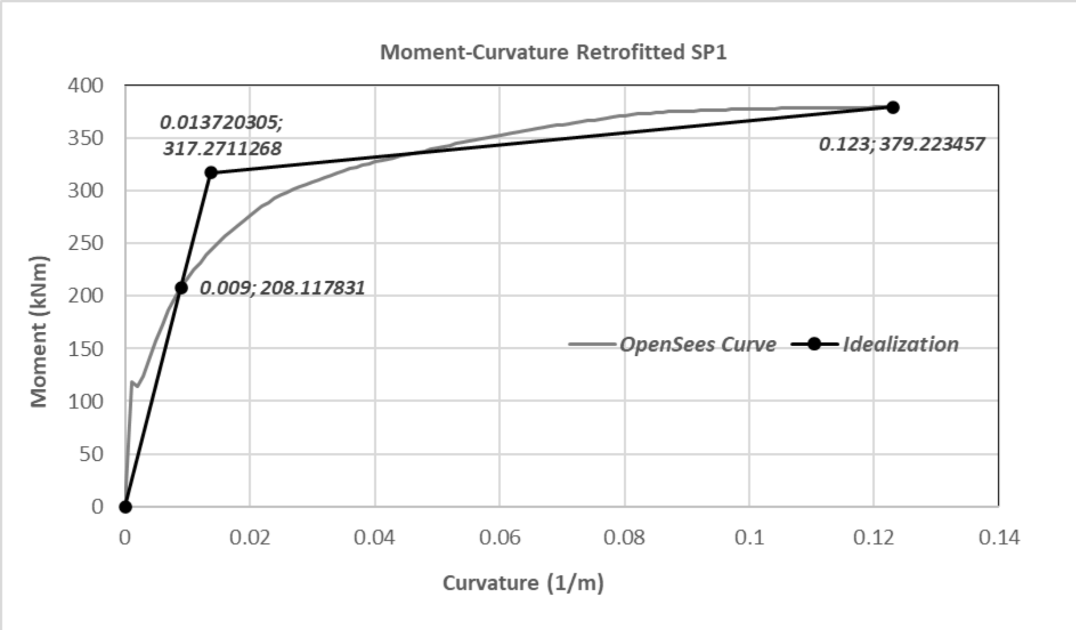
16	*Es*	*Steel's Elastic Modulus*
17	*vo*	*Initial Poisson's Coefficient for Concrete*
18	*k*	*Reduction Factor (0.5-0.8) for the Rupture Strain of the FRP Jacket*
19	*useBuck*	*FRP Jacket Failure Criterion due to Buckling of Longitudinal Compressive Steel Bars (0 = not include it, 1= to include it)*

The experimental specimens were SP1 and SP2 and were repaired with continuous unidirectional GFRP and CFRP composite laminates with four layers and with two layers, respectively with both having 1 mm thickness of each layer. The specimens were laterally confined by 6.35 mm deformed hoop bars having a yield strength of 435.3 MPa. The tie reinforcement ratio was 0.545% (50.8 mm spacing) for SP1 and 0.363% (76.2 mm spacing) for SP2. All specimens were reinforced in the longitudinal direction by 16 - 15.875 mm deformed bars having a nominal yield strength of 533.7 MPa. Unconfined concrete compressive strength was 27.6 MPa. Rupture strain of GFRP was 2.3% and of CFRP was 1.2%. Finally, the columns' clear height was 1778 mm and cross section diameter was 508 mm with a clear concrete cover of 25.4 mm. The total load on the top of the column specimens was 380.8 kN, axial load ratio of 6.8%.

Moment curvature analysis was firstly performed in OpenSees of the RC sections of the above two specimens where FRP and Steel confined concrete was modelled with the aforementioned implemented uniaxial material i.e., *FRPConfinedConcrete* and the constitutive model by Menegotto and Pinto (1973) [6] is used to model the longitudinal steel behavior.

A Zero Length element with the fiber discretization of the cross section is used. For the zero-length element, a section discretized by concrete and steel is created to represent the behavior. UniaxialMaterial objects are created which define the fiber stress-strain relationship's. According to *FRPConfinedConcrete* model, the averaged response of the two different regions - concrete core (confined by both the FRP & the existing reinforcement) and concrete cover (confined only with the FRP wrap) - in the cross-section allows the assignment of a unique stress-strain law (*FRPConfinedConcrete*) to all the concrete fibers/layers of the circular section. Reinforcing steel material is modeled with Steel02 [6] -as mentioned before - uniaxial material. The procedure takes as input the tag of the section to be analyzed, the axial load, P, to be applied, the max curvature to be evaluated and the number of iterations to achieve this max curvature. The procedure first creates the model which consists of two nodes, boundary conditions, and a Zero-Length-Section element. After the model has been created, the analysis is performed. A single load step is first performed for the axial load, then the integrator is changed to DisplacementControl to impose nodal displacements, which map directly to section deformations. Figure 1 depicts the results of the above analyses for specimens SP1 and SP2 along with the bilinear idealization of the produced curves to be later employed in the performed time-history analyses and overall modeling.

The experimental specimens consist of a footing, a column and a top block. Steel beams and mass blocks are placed on top of the test specimens and four load cells connect the specimen to the shake table below the footing. These features are expected to affect the dynamic and nonlinear responses of the test columns. Hence, the whole setup above the shake table is



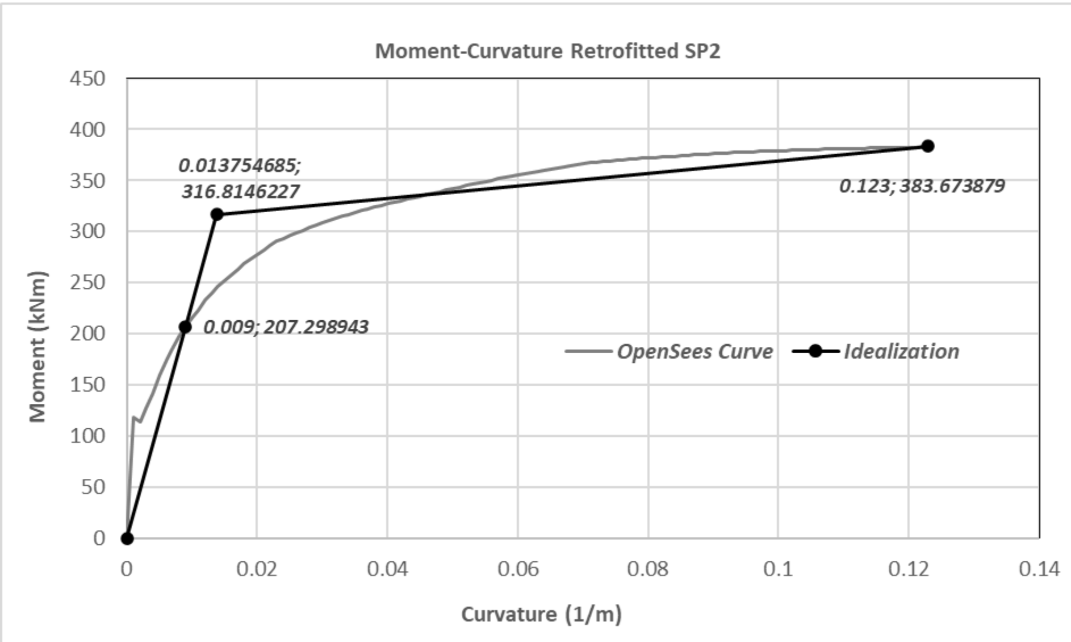


Figure 1: Moment-Curvature Analysis of RC Sections SP1 and SP2 produced in OpenSees along with their bi-linear idealization.

modeled in this computational investigation.

The 'Beam with Hinges' (BWH) element [7] is a commonly used force-based element to examine the nonlinear response of frame structures. It has localized plasticity at the ends i.e., hinges, and the remaining part is kept linear elastic. The length of each hinge is defined by the user. In this study this plastic hinge length is defined according the equations developed by Megalooikonomou et al. (2018) [8], Megalooikonomou (2019) [9]. According to the latter study the definition of plastic hinge length was reassessed through consideration of yield penetration effects. The required confined zone in critical regions of columns and piers undergoing lateral sway during earthquakes is related to the plastic hinge length where inelastic deformation and damage develops. The exact definition of the plastic hinge length stumbles upon several uncertainties, the most critical being that the extent of the inelastic region evolves and spreads with the intensity of lateral displacements. Design codes quantify a reference value for the plastic hinge length, through calibrated empirical relationships that account primarily for the length of the shear span and the diameter of primary reinforcing bars. The latter term reflects the effects of bar yielding penetration in the support of columns. In the aforementioned studies a consistent definition of plastic hinge length was pursued analytically with reference to the actual strain state of the reinforcement.

Figure 2 presents the test specimen model using BWH element to represent the column. Two rigid elements at the top and the base are used for the top block and the footing, respectively. A rotational spring is added below the rigid element at the base, because the specimen was placed on four load cells which were connected to the shaking table and they are not perfectly rigid. A hysteretic material that follows the bilinear moment-curvatures responses of Figure 1 is employed for the section aggregator object in OpenSees which aggregates previously-defined UniaxialMaterial objects into a single section force-deformation model inside the plastic hinge length of the BWH element. Each UniaxialMaterial object represents the section force-deformation response for a particular section degree-of-freedom (*dof*). There is no interaction between responses in different *dof* directions.

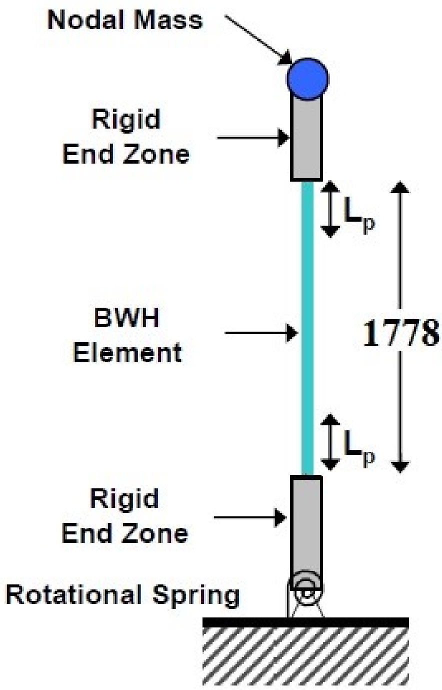


Figure 2: Specimen Modeling

3 NONLINEAR TIME - HISTORY BRIDGE PIERS' ANALYSES AND CORRELATION WITH EXPERIMENTAL RESULTS

For the nonlinear dynamic simulation of the two FRP-retrofitted specimens SP1 and SP2 in the horizontal and vertical direction, the same set of horizontal and vertical ground motions were selected as used in testing of as-built specimens. Table 2 gives the details of loading protocol. As it is mentioned before the ground motion recorded at the Pacoima Dam station of

1994 Northridge earthquake was employed for the nonlinear time-history analyses of the specimen model of Figure 2 in OpenSees.

Table 2: Ground motions for shaking table testing

Test	Earthquake	Station	Scale Factor	Components
EQ1	Northridge	Pacoima Dam	0.05	Horizontal and vertical
EQ2	Northridge	Pacoima Dam	0.125	Horizontal and vertical
EQ3	Northridge	Pacoima Dam	0.250	Horizontal and vertical
EQ4	Northridge	Pacoima Dam	0.250	Horizontal only
EQ5	Northridge	Pacoima Dam	0.500	Horizontal and vertical
EQ6	Northridge	Pacoima Dam	0.500	Horizontal only
EQ7	Northridge	Pacoima Dam	0.700	Horizontal and vertical
EQ8	Northridge	Pacoima Dam	0.950	Horizontal and vertical
EQ9	Northridge	Pacoima Dam	1.250	Horizontal and vertical
EQ10	Northridge	Pacoima Dam	1.250	Horizontal only
EQ11	Northridge	Pacoima Dam	1.250	Horizontal and vertical

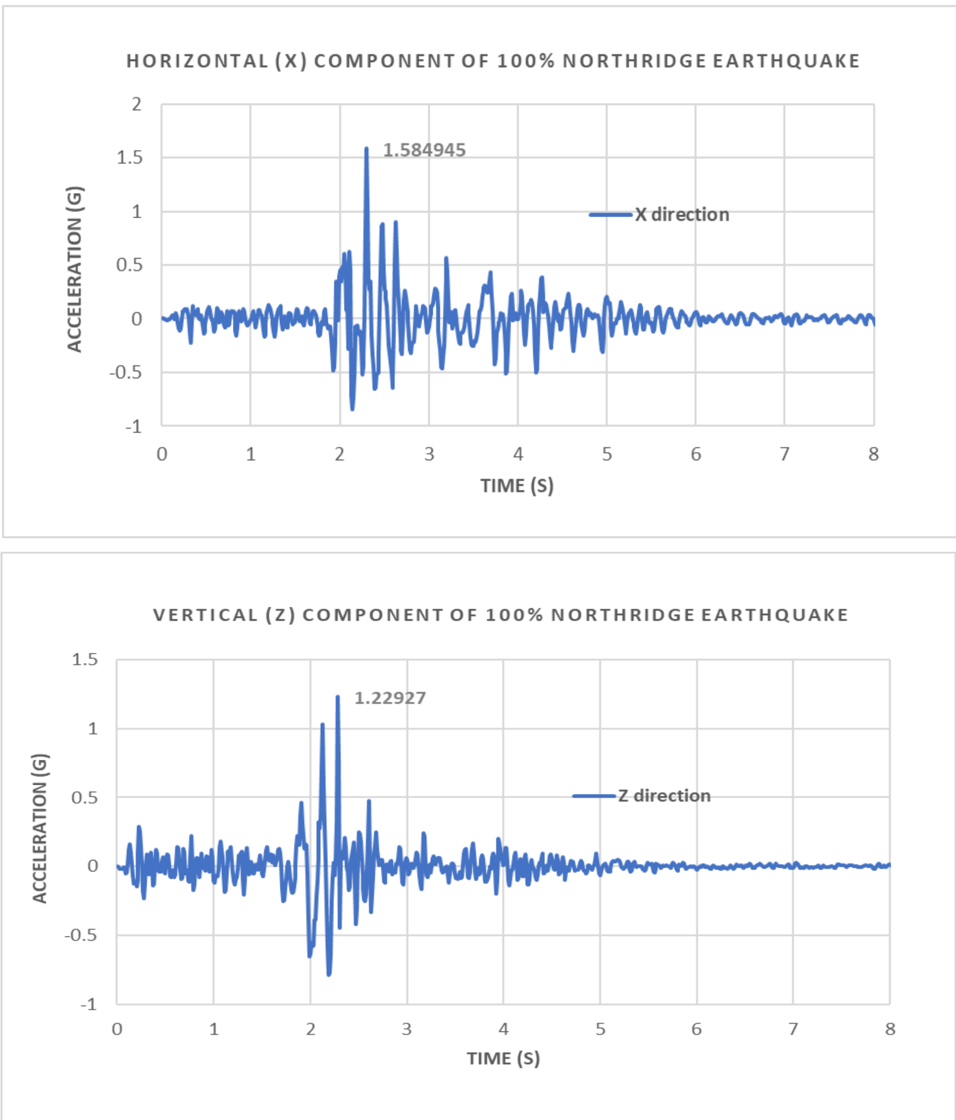


Figure 3: Horizontal (X) and Vertical Components (Z) of 100% Northridge earthquake

Of the two horizontal components, the X component was selected because it produces larger shear strength reduction than the Y component. Since the geometrical scale of the specimen corresponds to the ¼-scale, each component of the ground motion was time-compressed by a factor of 2 as shown in Figure 3. It should be noted that the acceleration history depicted in the latter Figure is 100% unfiltered input ground motion obtained from the PEER NGA database [10].

As it is mentioned already the shaking table is not perfectly rigid. Its flexibility affects the response of the test specimen, especially in the vertical direction. The vertical natural period of the column is much shorter than that of the shaking table and the vertical period of the shaking table is dominant in the whole system (combined test specimen and shaking table as one system). If the shaking table is ignored and the vertical acceleration recorded on the shaking table is directly used as the input to the analytical model (as it is performed in this analytical study), acceleration history with higher frequencies is obtained at the top of the column [11]. However, these high frequencies are not present in the test data because of the dominant shaking table period in the vertical direction. Based on the latter issues, modeling the complex table response with several sources of uncertainties and assumptions is not feasible and should be kept in mind in the correlation with the experimental results as it can be seen below especially for the model in the vertical direction.

3.1 Correlation with SP1 shake table test

Figure 4 depicts the results of the inelastic time-history analysis of specimen SP1 for the last ground motion (125% scale) of the ground motions' sequences applied in the already described loading protocol of Table 2. The comparison in Figure 4 is in terms of drift at the top of the column. Overall, the correlation is satisfactory under the view of the complex behavior of the combined test specimen and shaking table system and its complex response. As it can be seen in the response, the residual displacements were found to be much lower for the repaired specimen SP1 in comparison to its bare counterpart [11].

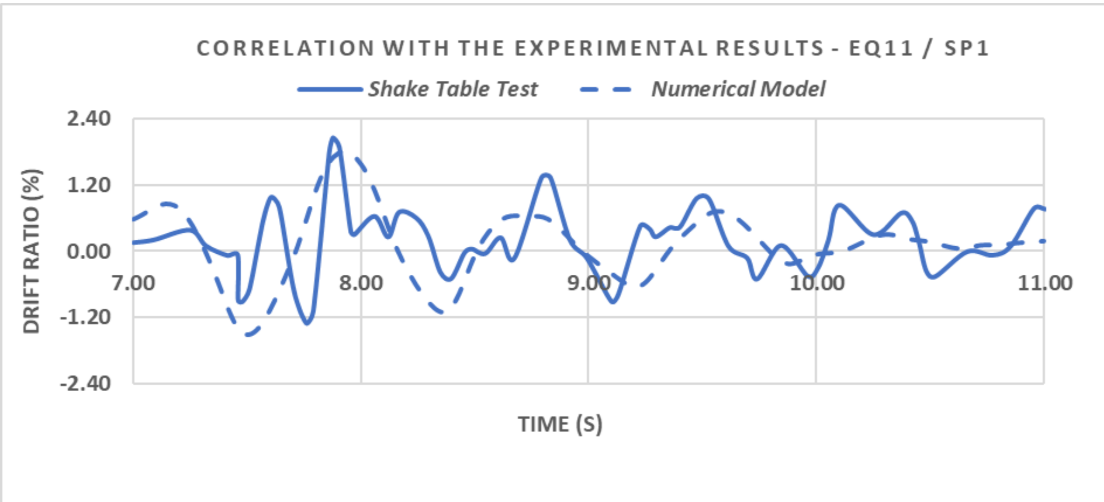


Figure 4: Correlation with experimental test of SP1 specimen in terms of specimen's drift for the last ground motion of the loading protocol

Figure 5, presents the comparison of the recorded axial and shear forces in the analytical bridge column model in OpenSees with the corresponding ones reported in the experimental study of this test specimen. Shear forces are satisfactorily correlated with the experimental results while the axial forces deviate a lot due to the shaking table effect which is not feasible to be properly modeled in the analytical model of OpenSees. However, it can be stated that the effect of changes of axial load from compression to tension due to the vertical component which can result in significant decreases in column shear strength, was diminished through the contribution of the GFRP jacket.

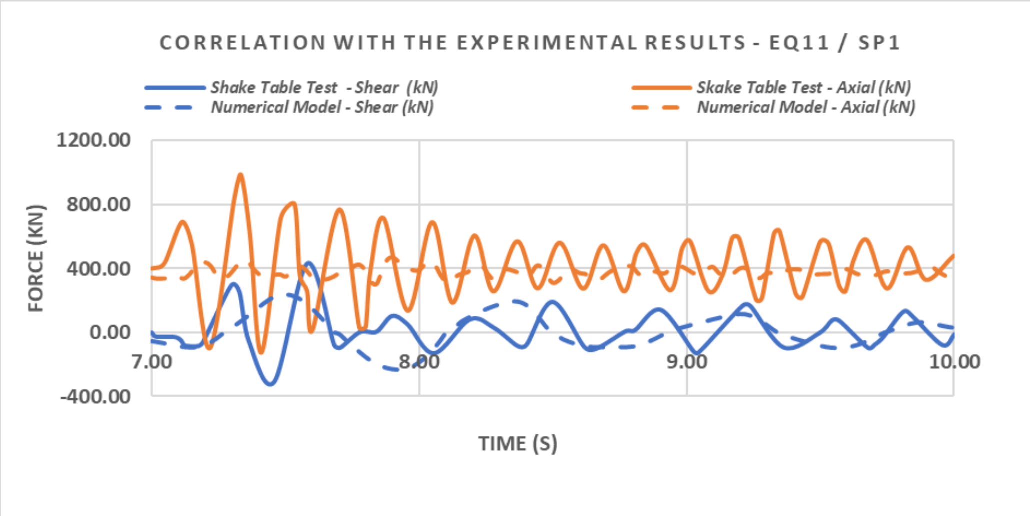


Figure 5: Correlation with experimental test of SP1 specimen in terms of specimen's axial and shear forces for the last ground motion of the loading protocol

3.2 Correlation with SP2 shake table test

Figure 6 depicts the corresponding results of the inelastic time-history analysis of specimen SP2 for the last ground motion (125% scale) of the ground motions sequences' applied in the already described loading protocol of Table 2.

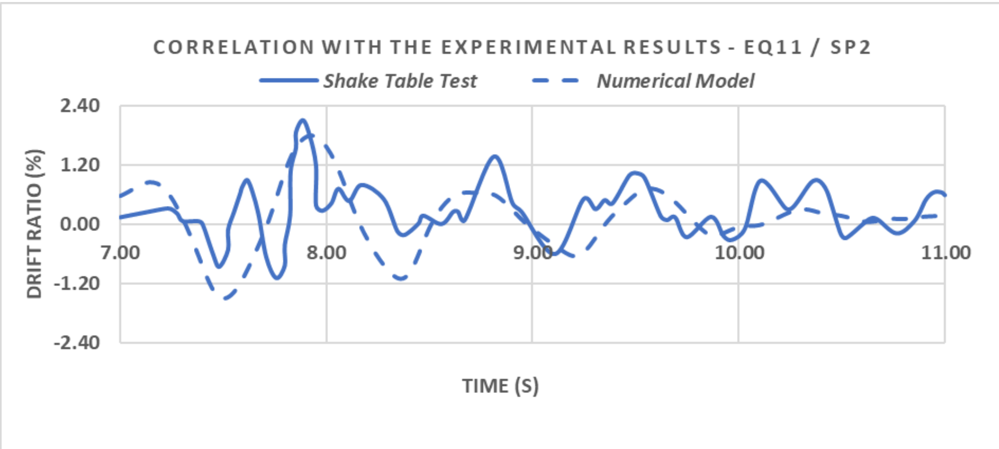


Figure 6: Correlation with experimental test of SP2 specimen in terms of specimen's drift for the last ground motion of the loading protocol

The comparison in Figure 6 is again in terms of drift at the top of the column. Overall, also here the correlation is satisfactory under the view of the complex behavior of the combined test specimen and shaking table system and its complex response. As it can be seen in this response too, the residual displacements were found to be much lower for the repaired specimen SP2 in comparison to its bare counterpart [11].

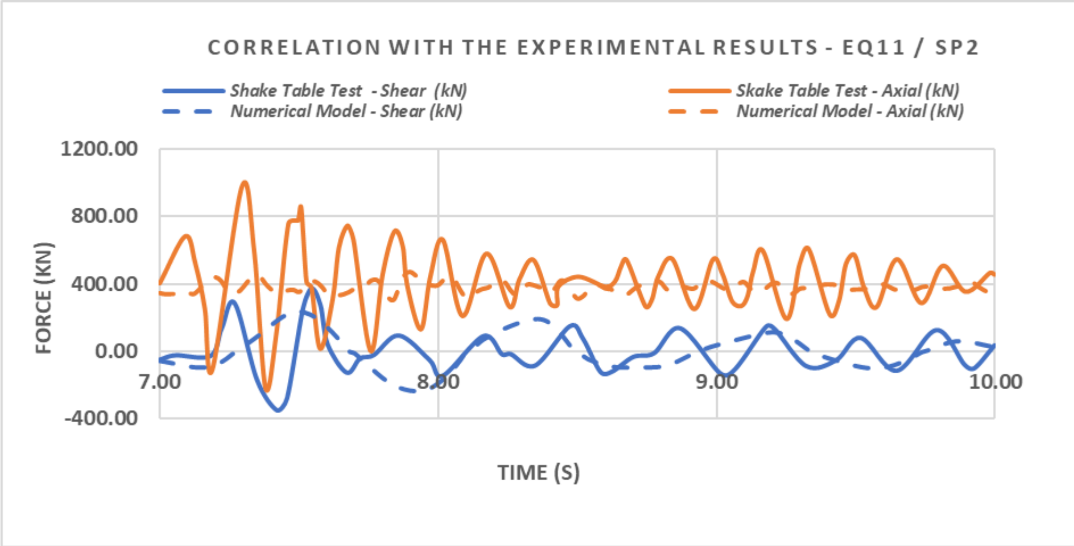


Figure 7: Correlation with experimental test of SP2 specimen in terms of specimen's axial and shear forces for the last ground motion of the loading protocol

Figure 7, presents the comparison again of the recorded axial and shear forces in the analytical bridge column model in OpenSees with the corresponding ones reported in the experimental study of this test specimen. Shear forces are satisfactorily correlated with the experimental results while the axial forces deviate a lot due to the shaking table effect which is not feasible to be properly modeled in the analytical model of OpenSees. However, it can be stated here too that the effect of changes of axial load from compression to tension due to the vertical component which can result in significant decreases in column shear strength, was diminished through the contribution of the CFRP jacket.

One way to overcome the deviation of the analytical results from the experimental ones in the vertical direction is to apply directly to the analytical bridge pier the recorded axial force history (from the load cells installed underneath the test specimen footing and above the shaking table) to the column as an external force excitation in the conducted analyses. Moreover, in this case in order to equate the restoring forces to the external forces, model mass in the vertical direction should be set to zero. However, this data from the shaking table test is not available to the author and therefore the inelastic time-history analyses were performed the way it is described already previously.

4 CONCLUSIONS

Based on the analytical study presented in this paper the following useful conclusions can be drawn:

- Comparison between the numerical and experimental time history response of the columns is indicative of the effectiveness of the implemented modelling based on the uniaxial material '*FRPConfinedConcrete*' implemented in OpenSees independently of the type of the FRP jacket applied as a retrofit to the columns.
- The numerical model confirmed that the repaired columns were able to resist higher shear and developed fewer peaks of tensile forces due to the vertical component of the ground motion. Therefore, the effect of changes of axial load from compression to tension due to the vertical component which can result in significant decreases in column shear strength, was diminished through the contribution of the FRP jacket.

- Finally, after performing all time history dynamic analyses, the residual displacements were found to be much lower for both repaired specimens in comparison to their bare counterparts.
- The modeling approach described in this analytical study is indicative of the effectiveness of the investigated FRP repair technique to produce resilient bridge columns when subjected to earthquake loading with strong vertical component.

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