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Seismic response of continuous span bridges through fiber-based finite element analysis

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Abstract: It is widely recognized that nonlinear time-history analysis constitutes the most accurate way to simulate the response of structures subjected to strong levels of seismic excitation. This analytical method is based on sound underlying principles and has the capability to reproduce the intrinsic inelastic dynamic behavior of structures. Nonetheless, comparisons with experimental results from large-scale testing of structures are still needed, in order to ensure adequate levels of confidence in this numerical methodology. The fiber modelling approach employed in the current endeavor inherently accounts for geometric nonlinearities and material inelasticity, without a need for calibration of plastic hinges mechanisms, typical in concentrated plasticity models. The resulting combination of analysis accuracy and modelling simplicity, allows thus to overcome the perhaps not fully justifiable sense of complexity associated to nonlinear dynamic analysis. The fiber-based modelling approach is employed in the framework of a finite element program downloaded from the Internet for seismic response analysis of framed structures. The reliability and accuracy of the program are demonstrated by numerically reproducing pseudo-dynamic tests on a four span continuous deck concrete bridge. Modelling assumptions are discussed, together with their implications on numerical results of the nonlinear time-history analyses, which were found to be in good agreement with experimental results.

Key words: bridges; seismic response; pseudo-dynamic testing; nonlinear dynamic analysis; fiber modelling

1 Introduction

Older design codes based on equivalent elastic force approaches proved to be ineffective in preventing damage caused by destructive earthquakes. After recent major earthquakes (e.g. Northridge 1994, Kobe 1995, and Kocaeli 1999 etc.), the necessity for using more accurate methods, which explicitly account for geometrical nonlinearities and material inelasticity, to evaluate seismic demand on structures, became evident. Within this framework, two analysis tools are currently offered with different levels of complexity and required computational effort; nonlinear static analysis (pushover) and nonlinear dynamic analysis (time-history). Even if the latter is commonly considered to be complex and not yet mature enough for widespread professional use, it constitutes the most powerful and accurate tool for seismic assessment; in the latest generation of seismic regulations, dynamic analysis of three-dimensional structural models is indeed recommended for the assessment of existing critical structures in zones of high seismic risk, as well in the planning and design of appropriate retrofitting strategies.

Given the increased computational effort and analytical complexity of nonlinear finite element (FE) methods in design applications, a clear demonstration of its accuracy and reliability is first required. Within this scope, the current work tries to establish the ability of a fiber-modelling approach in predicting the seismic response of continuous span reinforced concrete bridges, by reproducing pseudo-dynamic tests carried out at the Joint Research Centre of Ispra (Pinto *et al.*, 1996; Guedes, 1997). The numerical algorithm used allows for automatic accounting of both local (beam-column effect) and global (large displacements/rotations effects) sources of geometric nonlinearity, together with proper modelling of material cyclic inelasticity.

Further, the nonlinear analysis software package used in this endeavor is freely downloadable from the Internet, thus giving readers the opportunity to try the proposed numerical scheme in a graphical-interfaced software package adequate for general use, and in this way overcome, at least partially, the sense of complexity associated to nonlinear dynamic analysis.

2 Modelling approaches for inelastic analysis

Nonlinear time-history analyses are a very powerful tool, provided they are supported by proper approximations and modelling. The analysis is

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inherently complex and may be very time-consuming, depending on the choice of the integration time-step, the integration scheme, the nonlinear incremental iterative algorithm strategy, and the size of the mesh; an optimum balance among all these features will enable accurate solutions with reduced computational effort.

As described by Spacone (2001), a suitable classification of the different modelling strategies available may be based on the objective of the numerical study: (i) Global Models (or Lumped Parameters Models) where the nonlinear response of a structure is represented at selected degrees of freedom; (ii) Discrete FE Models (also called Member Models, Structural Elements Models, or Frame Models) where the structure is characterized as an assembly of interconnected frame elements with either lumped or distributed nonlinearities; and (iii) Microscopic Finite Element Models, which use the FE general method of structural analysis, in which the solution of a problem in continuum mechanics is approximated by the analysis of an assemblage of two or three-dimensional FEs that are interconnected at a finite number of nodal points and represent the solution domain of the problem.

The level of refinement of the model depends on the required accuracy and available computational resources. While refined FE models might be suitable for the detailed study of small parts of the structure, such as beam-column joints, frame models are currently the only economical solution for the nonlinear seismic analysis of structures with several hundred members. In other words, member FE models are the best compromise between simplicity and accuracy, as they represent the simplest class of models that nonetheless manage to provide reasonable insight into both the seismic response of members and of the structure as a whole.

Assumptions and simplifications on the model with respect to the real structure are necessary, but need careful consideration because of their influence on results, which must be critically analyzed accordingly. In the particular case of bridges, the structural subsystems that may be potentially affected by intense seismic action are the deck, the bearing structure and the foundation system. Due to the cost and technical difficulties in repair, foundations are usually protected from damage, while for reasons of life safety, the deck is kept elastic (though some cracking is allowed). The most common trend in earthquake-resistant design of bridges therefore assigns a key role in dissipating the energy introduced by the earthquake loads to the bearing structure by means of inelastic deformation mechanisms, thus these are the elements that require the most accurate modelling.

2.1 Representation of inelasticity

Two different modelling philosophies are commonly employed in analytically reproducing the inelastic response of structures subjected to seismic action: the 'concentrated plasticity' and the 'distributed inelasticity' modelling approaches.

As stated by Spacone (2001), due to the typical concentration of inelasticity of RC frames at the extremities of its structural elements, "an early approach to modelling this behavior was by means of nonlinear springs at the member ends (Clough and Johnston, 1966; Giberson, 1967; Takizawa, 1976). Among the lumped plasticity constitutive models proposed, some include stiffness degradation in flexure and shear (Clough and Benuska, 1967; Takeda *et al.,* 1970; Brancaleoni *et al.,* 1983), 'pinching' under load reversal (Banon *et al.*, 1981; Brancaleoni *et al.*, 1983), and fixed end rotations at the beam-column joint interface to simulate the effect of bar pull-out (Otani, 1974; Filippou and Issa, 1988)." Such a concentrated plasticity approach should be used with care, since accuracy of the analysis may be compromised whenever users are not highly experienced in the calibration of the available response curves needed to characterise the lumped plasticity elements. The limitations of lumped models are discussed in several studies, such as Charney and Bertero (1982) and Bertero *et al.* (1984), amongst others.

The 'distributed inelasticity' model more accurately describes the continuous structural characteristics of reinforced concrete members, requiring simply geometrical and material characteristics as input data. The constitutive behavior of the cross section can either be formulated according to the classical plasticity theory in terms of stress and strain resultants, or explicitly derived by discretizing the cross section into fibers. The latter approach, known as "fiber modelling," represents the spread of material inelasticity along both the member length and across the section area, thus allowing an accurate estimation of the structural damage distribution even in the highly inelastic range.

Quoting Spacone (2001) again, "the first elements with distributed nonlinearity were formulated with the classical stiffness method using cubic hermitian polynomials to approximate the deformations along the element (Hellesland and Scordelis, 1981; Mari and Scordelis, 1984). Menegotto and Pinto (1973) interpolated both section deformations and section flexibilities and accounted for the axial force-bending moment interaction. Shear effects were first included in the model proposed by Bazant and Bhat (1977)." More recently, alternative flexibility-based formulations have been developed by Mahasuverachai and Powell (1982), Kaba and Mahin (1984), Zeris and Mahin (1988, 1991), however these posed difficulties with regard to their implementation in FE programs. To overcome such complications, Ciampi and Carlesimo (1986) proposed a consistent flexibility-based method for formulating frame member models, later applied by Spacone (1994) to the formulation of a fiber beam-column element. A detailed discussion on the differences between stiffness-based and flexibility-based approaches may be found in Papaioannou *et al.* (2005), for instance.

For the purpose of the current work, a classical stiffness-based formulation, as developed by Izzuddin (2001), has been adopted.

3 Fiber modelling approach

In fiber modelling, the sectional stress-strain state of the elements is obtained through the integration of the nonlinear uniaxial stress-strain response of the individual fibers in which the section is subdivided, distinguishing steel, confined and unconfined concrete, as illustrated in Fig. 1. The adopted stiffness-based element cubic formulation then allows both the representation of the spread of inelasticity along the member length as well as the implicit incorporation of interaction between the axial force and transverse deformation of the element. The use of a sufficient number of elements per structural member permits the reproduction of a plastic hinge (in their full length), typical of members subjected to high levels of material inelasticity. The spread of inelasticity across the section and along the member length is thus achieved without requiring expert calibration of any lumped plasticity elements.

Structural members are represented by means of frame elements, with finite length and assigned cross-sections. Structural and nonstructural inertia mass may also be introduced, in either lumped or distributed fashion, while joint/link elements, defined as spring-type elements joining coincident locations, can be used to model discontinuous connections. By means of such element types, a number of different element classes (columns, beams, walls, beam-column joints, etc.), nonstructural components (energy dissipating devices, inertia masses, etc.) and different boundary conditions (flexible foundations, seismic isolation or structural gapping and pounding) can be represented.

Fiber-discretization enables realistic modelling of different materials that make up a given member cross-section and their distribution possible. The employable material models may feature different levels of accuracy/complexity in their definition; the bilinear, the Menegotto and Pinto (1973) and the Monti and Nuti (1992) models are among the most popular for steel, while concrete may be characterized by tri-linear or nonlinear with constant or variable confinement constitutive laws (e.g. Scott *et al.*, 1982; Mander *et al.*, 1988). Many other material constitutive laws are available in the literature.

4 Assembling a FE model for dynamic analyses for reinforced concrete structures

Since the structural details of an actual structure are complex, simplification is needed to develop a FE bridge model, in order to obtain predictions accurate enough with a reasonable computational effort; a proper balance is required to avoid numerical instability and, on the other hand, to obtain results with a sufficient level of accuracy.

Among the most common simplifications/assumptions requiring thoughtful consideration are:

(i) The structural mass is generally concentrated at the top of the pier, representing one single translational DOF. However, this may not be sufficiently accurate when the transversal size of the deck section is large with respect to the pier height, thus requiring an additional rotational DOF of the deck.

(ii) Assuming a top concentrated mass may not be acceptable when piers are massive with respect to the deck.

(iii) The soil-structure interaction can be neglected or modelled with different levels of complexity.

(iv) The influence of the shear deformation needs adequate analytical characterization on squat members, for which shear collapse modes and flexure-shear interaction are relevant.

(v) The penetration of plasticization at the base of piers may be modelled by extending the actual pier length.

(vi) The influence of the spatial variability and/or loss of coherence of the ground motion may be represented by means of asynchronous input definition.

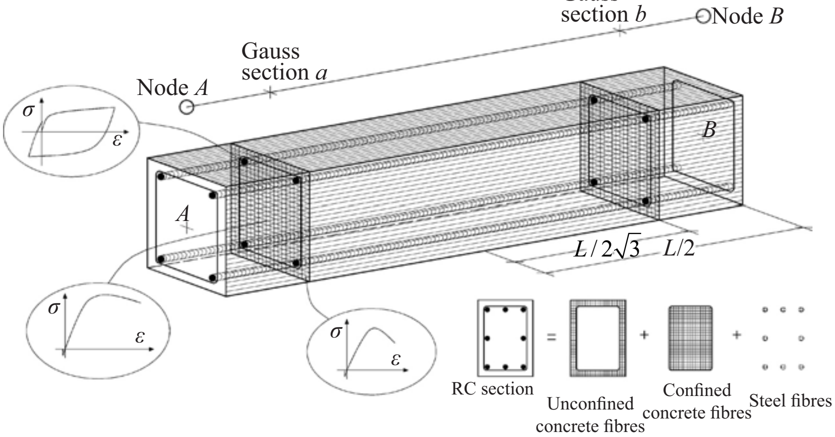

Fig. 1. Discretization of a typical reinforced concrete cross-section

(vii) Connections among foundations, piers and deck can be modelled with different levels of complexity and detail.

Some recommendations for nonlinear analyses of reinforced concrete models are summarized as follows:

• At the onset of the development of a FE model, a sensitivity study will allow reliable models to be created. A too fine mesh may cause numerical instabilities, while on the other hand, if the mesh is exaggeratedly coarse, the analysis will not be sufficiently accurate. The meshing of the structure can be optimally carried out by refining critical structural locations, such as the zones where high inelasticity is expected or where abrupt changes in the stiffness of joined elements are present, such as plastic hinges locations, element connections, and structural boundaries.

• The modelling of each structural element should be based on its expected behavior; some examples are the linear behavior of the deck or the modelling of plastic hinge length to account for the flexibility of the foundations. In this case, the use of inelastic fiber elements allows the explicit consideration of the spread of inelasticity.

• Model accuracy can be improved by using realistic materials property values and by properly defining boundary conditions. Considering soil-structure interaction allows a more realistic prediction of the seismic response of the model. In its simplest form, this may be implemented through boundary springs to which reasonable stiffness values, obtained from site investigations, are assigned.

• The use of preliminary eigenvalue analysis will assist in the verification of the correct assemblage of the model, in terms of stiffness allocation to structural members and mass distribution. In addition, the frequency characterization of the structure is also commonly used to calibrate viscous damping, if the latter is deemed necessary, and will also provide some preliminary insight into the expected response of the structure to a particular input motion.

4.1 Dynamic analysis features

As stated earlier, dynamic analysis is used to predict the nonlinear inelastic response of a structure subjected to earthquake loading; the seismic action may be introduced by means of acceleration loading curves at the supports, which may also be different at each support to represent asynchronous ground excitation. Mass and damping elements must be defined.

Dynamic analysis involves the direct integration of the equations of motion, which may be accomplished using the numerically dissipative-integration algorithm (Hilber *et al.*, 1977) or, as a special case of the latter, the well-known Newmark scheme (Newmark, 1959). The nonlinearity of the analysis scheme calls for the use of an incremental iterative solution procedure. This means that loads are applied in predefined increments, equilibrated through an iterative scheme, whereby the internal forces corresponding to a displacement increment are computed until either convergence is achieved or the maximum number of iterations is reached. At the completion of each incremental solution, before proceeding to the next load increment, the stiffness matrix of the model is updated to reflect nonlinear changes in structural stiffness. The solution algorithm may feature a hybrid incremental algorithm, obtained from a combination of the Newton-Raphson and the modified Newton-Raphson procedures, whereby the stiffness matrix is updated only in the first few iterations of a load step, thus obtaining an acceptable compromise between velocity in achieving convergence and required computational effort. The reader is referred to the work of Cook *et al.* (1989) and Crisfield (1997) for further discussion on this topic.

In nonlinear analysis, automatic time-step adjustment controls load step sizes for optimum accuracy and efficiency. If the convergence is achieved easily, automatic time stepping will increase the load increment up to a selected maximum load step size, while if convergence is hard to achieve, automatic time stepping will bisect the load increment until a selected minimum load step size is obtained.

Different convergence check schemes, which may make use of three distinct criteria (displacement/rotation, force/moment, energy based), can be employed to check the convergence of a solution at the end of each iteration. The displacement/rotation criterion provides direct local control over the precision obtained in the solution of the problem, usually ensuring overall accuracy. The force/moment criteria are suggested if a displacement convergence check is not sufficient for the internal forces of the elements to be adequately balanced. The maximum accuracy and solution control is obtained by combining the displacement and force convergence check criteria, while the maximum analysis stability is obtained if convergence is achieved for one of the two criteria checked, sacrificing analytical precision. Tolerances in convergence criteria should be carefully defined.

5 Case study

In the framework of an integrated European program of pre-normative research in support of Eurocode 8 (CEN, 2002), six bridge prototypes, representative of typical multi-span continuous deck motorway bridges, have been designed (Pinto *et al.*, 1996) with different procedures for a PGA of 0.35g, in medium soil conditions (soil type B), applying the EC8 provisions. Corresponding large-scale (1:2.5) bridge models were then constructed and tested in pseudo-dynamic(PsD) fashion at the Joint Research Centre at Ispra (Italy).

A PsD test, despite being carried out quasi-statically, employed on-line computer calculations and control together with experimental measurement of the properties of the actual structure, to provide a realistic simulation of its dynamic response. Inertial and viscous damping forces were modelled analytically, and an earthquake ground acceleration history was given as input data to the computer running the pseudo-dynamic algorithm. The horizontal displacements of the controlled degrees of freedom were calculated and then applied to the test structure by servo-controlled hydraulic actuators fixed to the reaction wall. The PsD testing of the bridge was performed using the substructuring technique, in which the piers were physically tested and the deck was numerically simulated on-line. Further details and references can be found in Pinto *et al.* (1996), Pinho (2000), and Sullivan *et al.* (2004), amougst others.

5.1 The bridge model

The tested bridge model labelled B213C consisted of three piers 5.6, 2.8 and 8.4 m high, respectively, and a continuous deck with four identical 20m spans. To model the boundary conditions, the deck was considered to end at the abutments with shear-keys, with the extremities free to rotate, as shown in Fig. 2. The deck-pier connections were assumed to be hinged (no transmission of moments), transmitting lateral forces caused by engaging of the sub- and superstructure in the transversal direction by means of the aforementioned shear keys.

The piers have rectangular hollow sections with 160 mm wall thickness (Fig. 2). The minimum diameter of the longitudinal rebars and stirrups was 8 mm and 6 mm, respectively. The reinforcement layout of the pier models are shown in Fig. 3. The mechanical characteristics of materials (B500 Tempcore steel with E = 206 GPa for longitudinal rebars and C25/30 concrete) and of the pier cross-sections are shown in Table 1 and 2.

The deck is a hollow-core pre-stressed concrete girder 5.6 m wide, as depicted in Fig. 2. In the PsD test, it was simulated numerically with 32 linear elastic Timoshenko eccentric beam elements, whose mechanical characteristics are presented in Table 3, where A is the cross-section area, I_2 and I_3 are the two moments of inertia with respect to the local principal axes, J is the torsional constant and E is the Young Modulus of 25 GPa. The inertia characteristics of the deck are based on a specific weight of 25 kN/m^3. As the deck was assumed to behave elastically, the sub-structured part included a Rayleigh damping matrix, featuring a damping ratio ξ=0.016 associated to the two lower transversal natural frequencies of the complete bridge.

At the top of each pier, an axial force N = 1700 kN was applied by means of actuators, to simulate the vertical load that is transmitted from the deck. The input ground motion was represented by an adequately scaled accelerogram with duration of 4 seconds and a nominal peak acceleration of 0.875g, as shown in Fig. 4. Two pseudo-dynamic tests were performed on the structure:

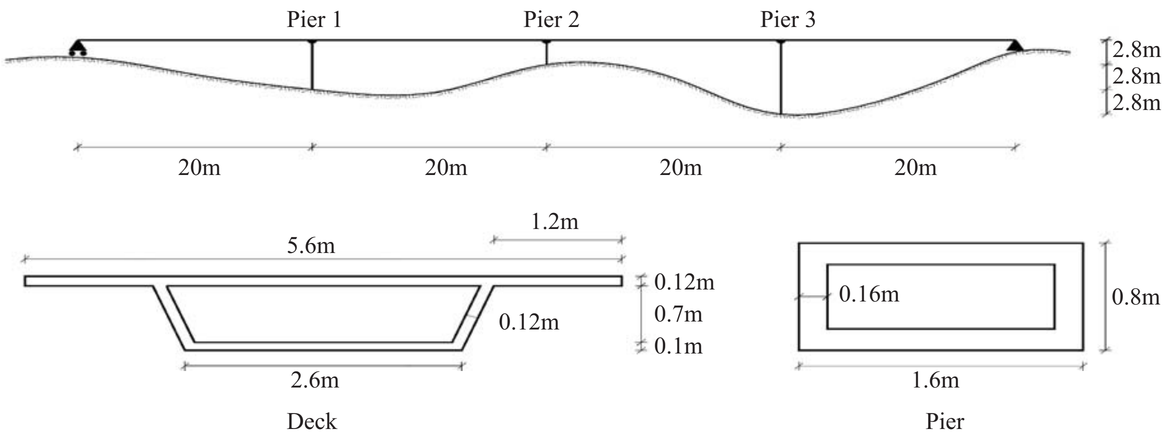

Fig. 2 Bridge configuration and member cross sections

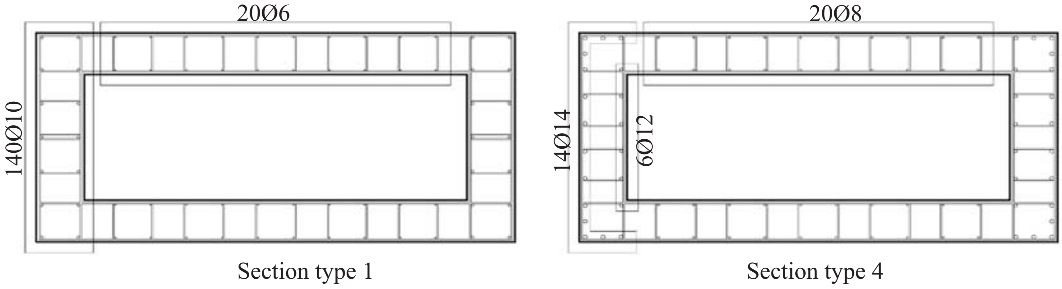

Fig. 3 Reinforcement layout

Table 1 Steel mechanical properties (Guedes, 1997)

Diameter (mm)	Yield strength (MPa)	Ultimate strength (MPa)	Yield strain (%)	Ultimate strain (%)	Hardening
6	363.7	430.4	0.177	15.10	0.0022
8	503.4	563.0	0.244	12.30	0.0024
10	489.3	572.3	0.238	14.50	0.0028
12	558.2	646.8	0.271	12.80	0.0034
14	477.2	577.7	0.232	13.00	0.0038

Table 2 Summary of the pier cross section characteristics of the bridge (Guedes, 1997)

Pier	Section type	Height (m)	Longitudinal steel (%)	Cubic concrete strength (MPa) (Compressive/tensile)
1	4	14	1.15	37.0 / 3.1
2	1	7	0.50	14.2 / 3.1
3	4	21	1.15	50.5 / 3.1

Table 3 Deck cross section geometrical and mechanical characteristics (Guedes, 1997)

EA (10^7kN)	EI_2 (10^7kN·m^2)	EI_3 (10^7kN·m^2)	GJ (10^7kN·m^2)
2.7837	1.3544	5.6517	2.8017

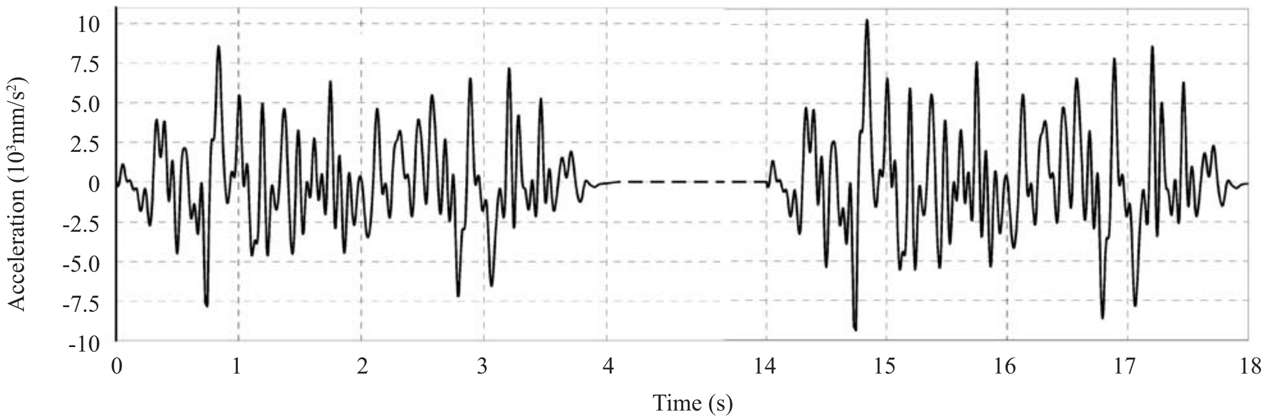

Fig. 4 Input ground motion, design earthquake (Guedes, 1997)

one with the input motion corresponding to the design earthquake and another defined on the basis of the estimated ultimate capacity of the bridges, and thus equal to 1.2 times the design earthquake.

6 Modelling of the structure in the FE program

The program employed herein, SeismoStruct (SeismoSoft, 2005), is a fiber–modelling FE package for seismic analysis of framed structures and can be downloaded at no charge from the Internet. The program is capable of predicting the large displacement behavior and collapse load of any framed type of structural configuration under static or dynamic loading, accounting for geometric nonlinearities and material inelasticity.

Each structural element has been defined through an element type with assigned section and materials, described by their characterizing parameters. In the following discussion, a description of geometry and discretization of the model, its element connections, boundary conditions and loading state, is given. Adopted nonlinear analysis procedures and convergence criteria are also explained in some detail.

6.1 Modelling of the bridge piers

As discussed previously, the piers are the elements where inelastic deformation will be concentrated, therefore, a high level of accuracy in the characterization of materials and in the discretization of the mesh should be ensured. The piers have thus been modelled through a 3D inelastic beam-column element capable of capturing geometric and material nonlinearities. The number of fibers used in section equilibrium computations was set to 400; the selection of this number guarantees an adequate reproduction of the stress-strain distribution across the element cross-section, considering the shape and material characteristics of the latter, and the degree of material inelasticity that it is likely to reach.

The pier cross-section has thus been defined through an RC rectangular hollow section of 0.8 m × 1.6 m, with a wall width of 0.16 m, a concrete cover of 8 mm, and a steel layout reproducing the test specimen of Fig.3. The specimen sections contains steel rebars with different mechanical properties, as illustrated in Table 1. However, given the possibility of specifying only one steel material per section in the computer code used, an equivalent steel has been defined for each section (see Table 4), weighting its properties, i.e. yielding strength f_y and strain hardening parameters, proportionally to the distance from the sectional center of gravity and to the area of each rebar.

Table 4 Equivalent steel properties per section

Section	Yield strength(MPa)	Hardening
1	468	0.0027
4	496	0.0036

The stress-strain behavior of the reinforcing steel (see Fig. 5, left) was described by the nonlinear model of Menegotto and Pinto (1973), as modified by Filippou *et al.* (1983) to include isotropic strain hardening. This is an accurate and convenient model, due to its computational efficiency and its very good agreement with experimental results. It utilizes a damage modulus to more accurately represent the unloading stiffness, and has been modified and improved by Fragiadakis *et al.* (2006) to attain better stability and accuracy. The concrete has been represented through a nonlinear constant confinement concrete model (Fig. 5, right), as a good compromise between simplicity and accuracy: it is an uniaxial nonlinear model following the constitutive relationship proposed by Mander *et al.* (1988), later modified by Martinez-Rueda and Elnashai (1997) for numerical stability reasons under large deformations. The constant confinement factor was defined as the ratio between the confined and unconfined compressive stress of the concrete. The model calibrating parameters, fully describing the mechanical properties of steel and concrete, have been set as shown in Table 5 and 6, where the concrete cylinder strength have been estimated as being equal to 85% of the cubic resistances listed in Table 2.

After the PsD testing of the bridges, the tall and the medium piers were tested cyclically until failure (Pinto *et al.*, 1996; Guedes, 1997), up to 230 and 150 mm, respectively, of top displacement under the imposed displacement history shown in Fig. 6. Additional cyclic

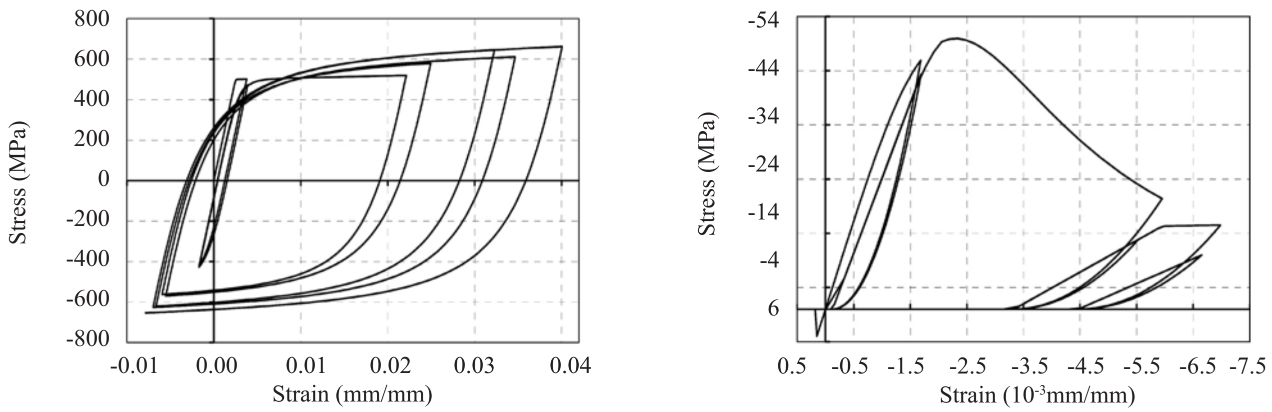

Fig. 5 Menegotto-Pinto steel model, with Filippou isotropic hardening (left), and nonlinear constant confinement concrete model (right)

Table 5 Parameters for the Menegotto-Pinto steel model, with Filippou isotropic hardening

Parameter	Sec 1	Sec 4
Modulus of elasticity (MPa)	203000	203000
Yield strength (MPa)	468	496
Strain hardening parameter	0.0027	0.0036
Transition curve initial shape parameter (default value)	20	20
1st transition curve shape coefficient (default value)	18.5	18.5
2nd transition curve shape coefficient (default value)	0.15	0.15
1st isotropic hardening coefficient (default value)	0.025	0.025
2nd isotropic hardening coefficient (default value)	2	2

tests (up to 72 mm at the top of the pier) were carried out on a short pier similar to the one tested pseudo-dynamically. These cyclic tests on the piers were numerically reproduced herein through a static time-history analysis, to enable a first check on the accuracy of the model being assembled. The numerical reproduction of the cyclic test has been performed by imposing the displacement history resulting from the PsD test (Fig. 6) on the piers. In addition, the steel young modulus of the medium-height pier was halved, as suggested by Pinto *et al.* (1996), in order to reproduce the reduction in the stiffness due to the shear damage that this pier suffered prior to this cyclic test (recall that these cyclic tests were carried out after the PsD testing); no reduction in the steel properties was applied to the tall pier, as it was not damaged during the PsD test.

Figures 7, 8 and 9 show a very good match between experimental and numerical results for all the piers; only the reduction in member strength at the very last cycle, when failure occurs, is not perfectly captured.

6.2 Modelling of the bridge deck

Normally, the deck can be modelled as linear elastic, since this is typically the behavior of actual bridges under seismic actions. The deck in fact is generally pre- and/or post-stressed, which means that no damage nor plastic deformations are allowed to occur. Moreover, in the case of isolated bridges, the deck is protected by the isolating system, and damage is negligible.

The deck was modelled with a 3D elastic nonlinear beam-column element, still capable of modelling local geometric nonlinearities. This type of element is fully described by the sectional properties values, based on

Table 6 Parameters for the nonlinear constant confinement concrete model

Parameter	Pier 1	Pier 2	Pier 3
Cylinder compressive strength (MPa)	31.5	35.0	42.9
Tensile strength (MPa)	3.1	3.1	3.1
Strain at unconfined peak stress (m/m)	0.002	0.002	0.002
Constant confinement factor	1.2	1.2	1.2

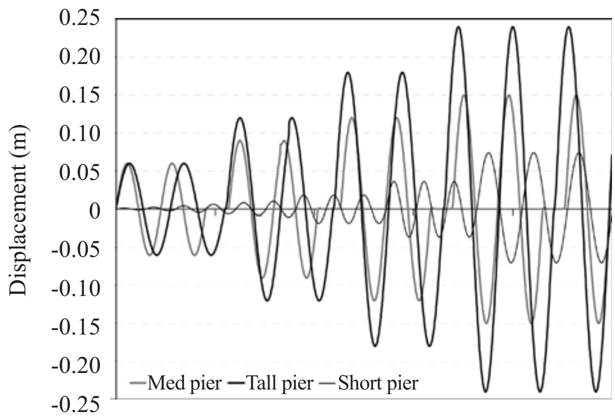

Fig. 6 Cyclic test displacement histories

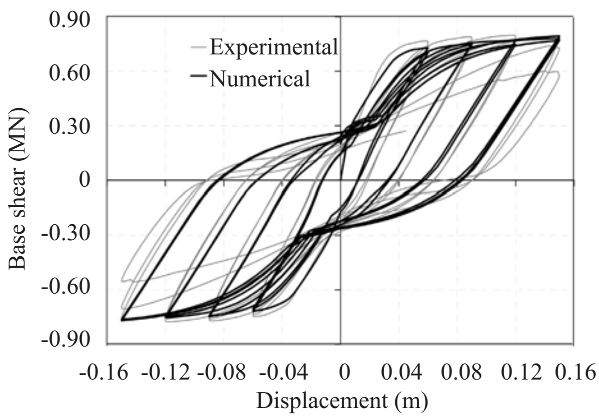

Fig. 8 Cyclic test results for the medium pier

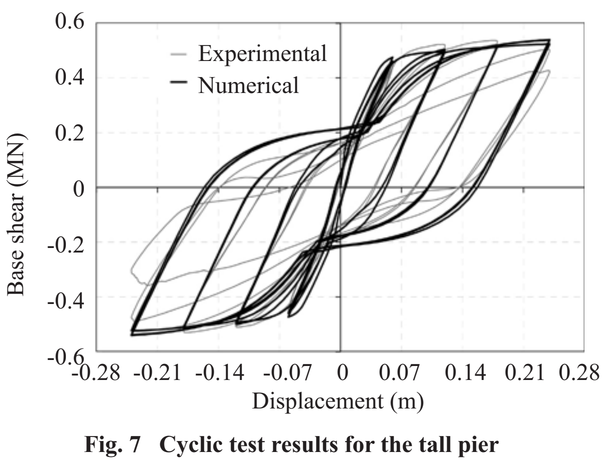

Fig. 7 Cyclic test results for the tall pier

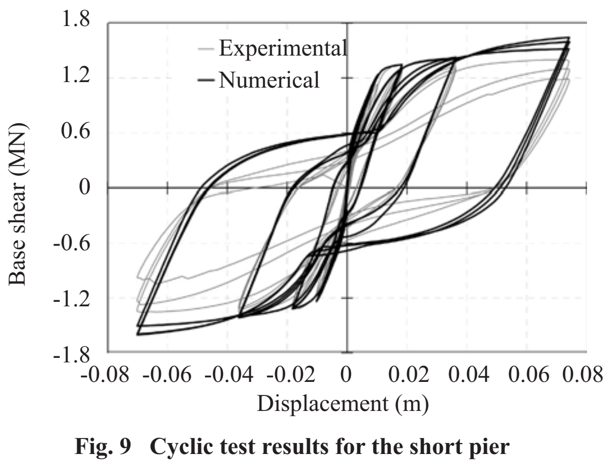

Fig. 9 Cyclic test results for the short pier

geometric and mechanical characteristics. In the current application, the Young and shear modules have been taken, respectively, as 25 and 10 GPa, and the element parameters were set as listed in Table 3.

The deck was located at the height of its center of gravity, 0.602 m above the pier top, and connected by a rigid element. The deck can be either modelled as described, or located right at the top of the pier, provided that the moment of inertia with respect to the horizontal axis is translated to that location. In the analyzed case, similar results are achieved, but the authors opinion is that the first choice is preferable, in order to more accurately model the deck displacement in case of non-rigid connections to the pier; e.g., when a relative rotation of the connection contributes to the drift proportionally to its vertical location.

Before carrying out nonlinear analyses, eigenvalue analysis was run to compare the first transversal modes of the numerical structure (Table 7) with the initial dynamic characteristics of the bridge specimen. The match between the test and numerical results was precise for the first mode (T = 0.183 s), and fairly good for the other two transversal modes as well.

6.3 Element discretization of the FE model

Each span of the deck was discretized with four elements, with length equal to 10%, 40%, 40% and 10% of the span. The linear elastic behavior of the element does not strictly call for this fine subdivision, but it was nonetheless preferred, for accuracy, to refine the mesh near the connections with the piers, where the change of stiffness and properties of the mesh are important. The extremities of the pier constitute the locations for potential plastic hinges, which may be assumed to extend for one twentieth to one tenth of the member length (Priestley *et al.*, 1996), depending on the boundary conditions. As the discretization of the pier pursues the most accurate capture of this phenomenon, each pier was subdivided into six elements of length equal to 5%, 10%, 30%, 40%, 10% and 5% of the structural member.

FE analysis requires careful meshing of the model. The adequate mesh density was achieved when an increase in the number of elements has a negligible effect on the global results. In order to check the effectiveness of the meshing, a second finer discretization was applied, verifying the matching of the results in the two cases. Note that localization phenomena (i.e., dependence of obtained results on element size) was not as relevant here, since the objective was the modelling of the overall response of the bridge.

6.4 Other modelling details

Rigid connections have been modelled either through elastic frame or link elements to represent an “infinitely” stiff connection, thus avoiding numerical difficulties. The stiffness of these elements was set 100 to 1000 times that of the adjacent elements. The rigid arm connecting the top of the pier and the center of gravity of the deck was constituted by an elastic element, while the connection between the base of the rigid arm and the top of the pier was modelled as a hinge. The link element representing the latter connects two initially coincident structural nodes and requires the definition of an independent force-displacement (or moment-rotation) response curve for each of its local six DOF. In order to model the engaging of the sub- and superstructure of the pier-deck connection in the transversal direction by mean of shear keys, the link element was set as a spring with infinite stiffness in the vertical and transversal directions, and fully flexible in all the other four DOF. Boundary conditions were defined as restraints in global coordinates; the deck was simply supported at the first end and hinged at the other, while piers were fully fixed at the base. Figure 10 shows these connection details.

Modal and dynamic analyses require as a matter of necessity the definition of the masses, which can be either lumped nodal masses or distributed. The piers were characterized with a distributed mass element of 1.664 t/m, while the deck mass was concentrated at the top of each pier for an amount corresponding to the tributary deck length (56 t per pier), and distributed at the two half span deck extremities with an amount of 2.784 t/m.

Equivalent viscous damping, with values typically ranging around 1%-2%, is customarily introduced to separate minor energy dissipation mechanisms from the hysteretic ones (e.g. friction across cracks, radiation through foundations, and so on). This usually involves the use of Rayleigh damping matrices, whereby damping is defined as proportional to the mass and stiffness of the

Table 7 First three modal shapes in the transversal direction of the deck

Mode	Period (s)		Modal shapes
	Experimental	Numerical	
1st	0.183	0.183	
2nd	0.146	0.148	
3rd	0.085	0.076	

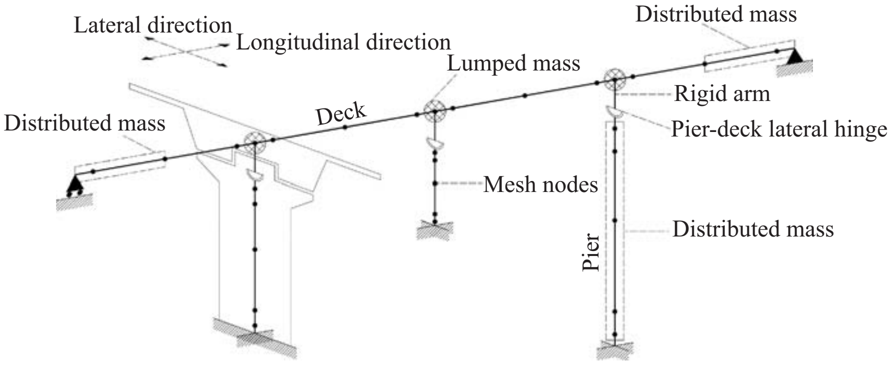

Fig. 10 Details of the FE model of the bridge

structural members (e.g. Clough and Penzien, 1994). However, given the uncertainties associated with the quantification of such equivalent viscous damping, and also considering the recent doubts raised with regards to the use of Rayleigh damping in nonlinear dynamic analyses (Hall 2005; Priestley and Grant 2005), this minor source of dissipation has been conservatively neglected in the numerical simulations.

A static load of 1700 kN was applied at the top of each pier, representing the deck weight according to the test setup, while the time history described in Fig. 4, including the 10 s interval with no acceleration (needed to damp out the structure motion after the first earthquake run), was imposed at the pier bases and at the abutments. In this manner, the cumulative damage effects caused by the testing of the same structure under successive earthquake input motions were adequately modelled.

The time step for the dynamic analysis was selected as 0.004 s, coincident with the input record sampling time step (hence the input motion is accurately considered), and sufficiently small with respect to the dominant vibration period of the structure (0.4 s) to guarantee numerical stability. For what concerns the nonlinear solution algorithm, the Hilber-Hughes-Taylor integration scheme (Hilber *et al.*, 1977) was used, associated to a displacement and force based convergence criterion.

7 Comparisons between numerical and experimental results

Numerical and experimental results, in terms of displacements and forces at the top of the short, medium and tall piers observed when the bridge was subjected to the second and stronger earthquake input motion, are discussed in this section. Figures 11, 12 and 13 show the results of the top displacement (left) and the top shear (right), and good agreement between both the amplitude and the frequency content of the response was observed. Table 8 presents the ratios of the maximum absolute response obtained from the numerical calculation to those obtained from the test. Note that the force response of the squat pier was not reproduced with full accuracy, whereas displacements were instead very well predicted. The numerical overestimation of the action at the top of the short pier can be explained by the fact that the fiber-based element formulation used did not feature the possibility of modelling shear flexibility (or section torsion/warping); thus, the stiffness of this pier was not reduced as it would be due to the shear damage.

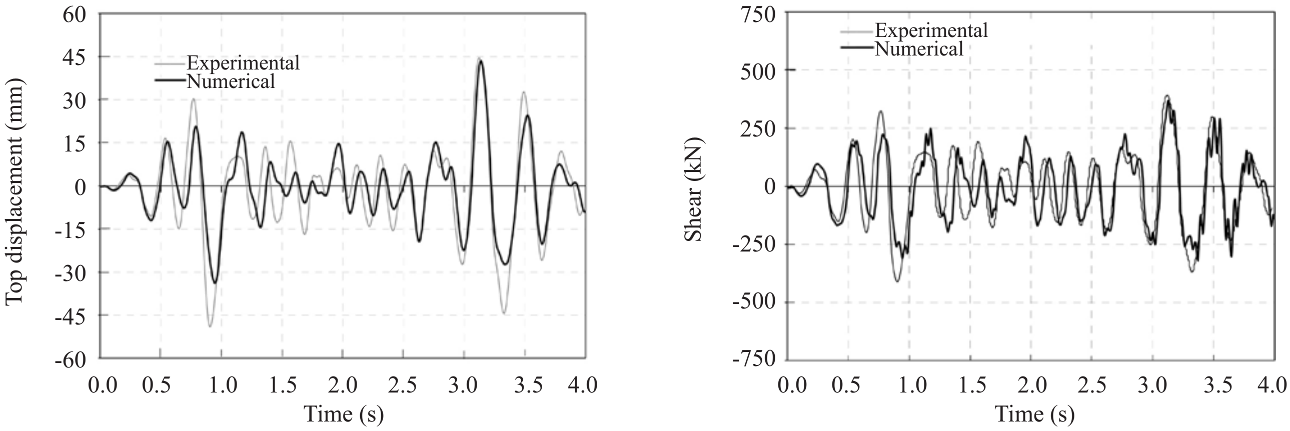

Fig. 11 Tall pier top displacements (left) and top shear (right)

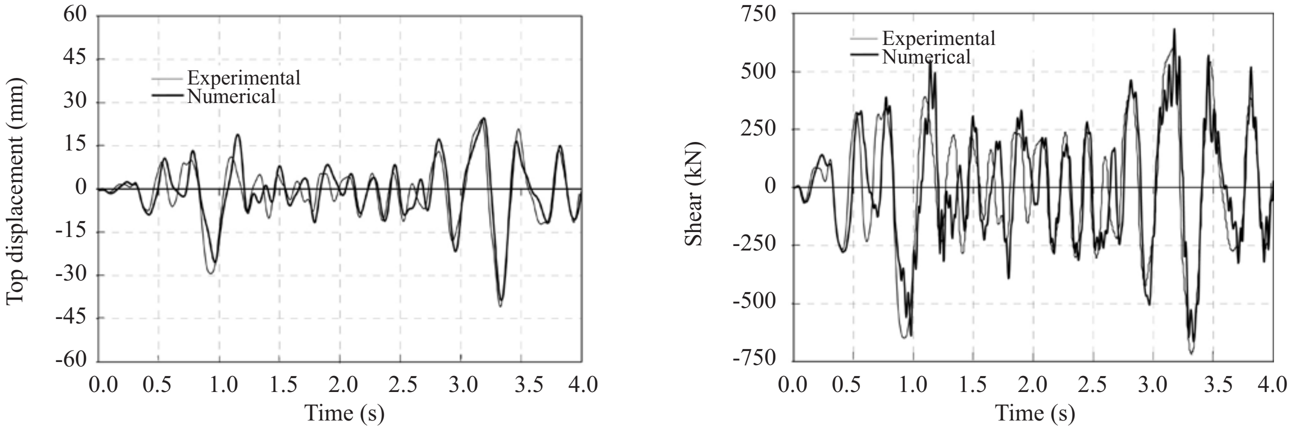

Fig. 12 Medium pier top displacements (left) and top shear (right)

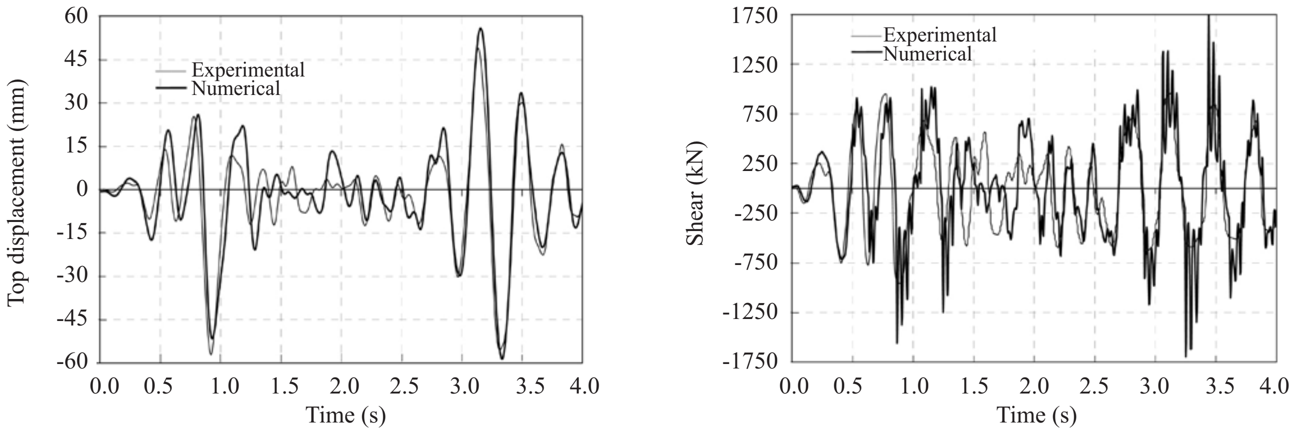

Fig. 13 Short pier top displacements (left) and top shear (right)

Table 8 Ratios of the absolute maximum response obtained from numerical calculation to that from tests

	Tall pier	Med pier	Short pier
Displacement	0.88	0.94	1.02
Top Shear	0.90	0.95	1.88

8 Conclusions

Structural behavior is inherently nonlinear, particularly in the presence of large displacements or material nonlinearities. The structural response can only be accurately modelled through nonlinear dynamic analyses. The fiber modelling approach used herein was shown to be simple to use, even for inexperienced users. Moreover, its ability to simulate the nonlinear dynamic response of reinforced concrete bridges to seismic loads was proven by simulating large-scale experimental pseudo-dynamic tests. Results of the dynamic and modal analyses reveal a good agreement between the pseudo-dynamic tests and the numerical simulation, both in terms of displacements and forces at the top of the tall and medium-height piers. At present, shear strains across the element cross-section are not included in the fiber-element formulation adopted, i.e. the strain state of a section is fully represented by the curvature at centroidal axial strains alone. This approach is not accurate enough to represent the squat pier deformation state, where shear deformations are relevant. In this case, despite the relevance of the shear response, the prediction of the deformation of the squat member was still fairly good.

This paper illustrated how the use of simple-to-calibrate fiber structural models can be used to reproduce the nonlinear structural response of continuous span bridge structures with an adequate level of accuracy. In other words, it is believed that such an advanced analytical tool can be readily used in a professional engineering environment, provided a basic level of awareness of the decisions that the designer must make, as discussed in this paper, is available.

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