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SEISMIC BEHAVIOUR OF R/C BRIDGE PIERS: NUMERICAL SIMULATION AND EXPERIMENTAL VALIDATION

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SUMMARY

This paper presents a constitutive model suitable for reproducing the seismic behaviour of hollow section bridge piers, and resumes the contribution from the Faculty of Engineering of the University of Porto to the European Project "*Advanced Methods for Assessing the Seismic Vulnerability of Existing Motorway Bridges*". The paper describes a numerical model suitable for simulating the non-linear behaviour of reinforced concrete under seismic loading. Basically a recent constitutive model founded on Damage Mechanics is used for the concrete itself, incorporating two independent scalar damage variables to reproduce degradation produced under tensile or compressive stress conditions. 2D finite element discretizations are adopted for the concrete volume, in order to fit geometric peculiarities of hollow section bridge piers. Steel reinforcement is discretized via truss elements and the corresponding cyclic behaviour simulated in accordance to the Giuffré-Menegotto-Pinto model. Calibration of the numerical model is performed on the basis of the experimental results obtained from a set of reduced scale piers tested pseudodynamically. Strategy pursued for the numerical simulation of those cyclic tests and comparison between numerical and experimental results are focused in detail. Finally a prospective assessment of the seismic behaviour of the Austrian 'Talübergang Warth' bridge is presented.

INTRODUCTION

Important deficiencies in the behaviour of bridges were brought into evidence during recent Los Angeles and Kobe earthquakes. Most of this unexpected vulnerability was due to considerable damages in their reinforced concrete piers, which implies that the non-linear behaviour of these structural elements during intense earthquakes remains an important issue, both for designers and researchers. Numerical models for reproducing the non-linear behaviour of reinforced concrete bridge piers are therefore a priority, either for design, a posteriori interpretation of case studies, or even for retrofitting purposes.

Within this scope, in the European Project "*Advanced Methods for Assessing the Seismic Vulnerability of Existing Motorway Bridges*" the Faculty of Engineering of the University of Porto is encharged of a specific task termed "*Numerical Models to Predict the Non-Linear Behaviour of Bridge Piers Under Severe Earthquake Loading*", concerned with the modelling of the cyclic behaviour of reinforced concrete (R/C) hollow section bridge piers. Concrete is reproduced via 2D plane-stress finite elements, and modelled by a constitutive model founded on Damage Mechanics which accounts for the unilateral effect of concrete [Faria *et al.* 1998], since it incorporates two independent scalar damage variables, one for tension and the other for compression. In what concerns the modelling of the steel rebars, 2-noded truss elements are adopted in association with the Giuffré-Menegotto-Pinto model, which account for the non-linear behaviour of the steel reinforcement under cyclic loops.

A validation of the model devised for the analysis of R/C bridge piers under cyclic loading will be based on a 'case study' pier, whose performance during pseudodynamic tests is reported in [Guedes 1997]. This pier was

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firstly submitted to a monotonic loading in order to evaluate the static horizontal collapse load, and afterwards to a cyclic loading, in order to assess its dissipative behaviour.

After this preliminary validation stage the R/C constitutive model will be used to produce refined predictions of the real behaviour of the rectangular hollow section piers of Talübergang Warth Austrian bridge. In the future such predictions will be validated through cyclic tests to be performed under pseudodynamic conditions, but for the present they can substitute such experimental results, providing refined response curves which are useful for calibration of the much more simplified numerical models usually adopted within the context of vulnerability analyses, which involve intensive and heavy computations of the whole bridge and piers.

CONCRETE DAMAGE MODEL

The constitutive model adopted for the numerical simulation of concrete's behaviour is based on the Continuum Damage Mechanics, with a strain-driven formalism, suitable for reproducing the distinct degradation phenomena which occur under tension and compression [Faria *et al.* 1993, Faria *et al.* 1998]. A basic entity in the model is the 'effective stress tensor' $\bar{\boldsymbol{\sigma}}$, which for many applications it is acceptable to assume to coincide with the elastic stress one. In order to clearly distinguish stress contributions due to tension or to compression, a split of the effective stress tensor $\bar{\boldsymbol{\sigma}}$ into tensile and compressive components ($\bar{\boldsymbol{\sigma}}^+$, $\bar{\boldsymbol{\sigma}}^-$) is introduced, and performed according to

$$\bar{\boldsymbol{\sigma}}^+ \;=\; \sum_i < \bar{\sigma}_i > \; \boldsymbol{p}_i \otimes \boldsymbol{p}_i \tag{1}$$

$$\bar{\boldsymbol{\sigma}}^- \;=\; \bar{\boldsymbol{\sigma}} \;-\; \bar{\boldsymbol{\sigma}}^+ \tag{2}$$

where $\bar{\sigma}_i$ denotes the *i*-th principal stress extracted from tensor $\bar{\boldsymbol{\sigma}}$ and $\boldsymbol{p}_i$ corresponds to the unit vector for the associated principal direction. Symbols $<\cdot>$ are the McAuley brackets, which return the value of the enclosed expression if positive, and set a zero value if negative. Indices (+) and (–) will be extensively used hereafter, to point out tensile and compressive entities, respectively.

Two scalar damage variables, d^+ and d^-, with independent evolutions, are adopted to reproduce concrete degradation under tension and compression. Following the necessary thermodynamic requirements, these damage variables have to be non-decreasing quantities, which is accomplished by adopting appropriate evolution laws, which will be described latter. According to Coleman's relations the final constitutive law results [Faria *et al.* 1998] in

$$\bar{\boldsymbol{\sigma}} \;=\; (1-d^+)\,\bar{\boldsymbol{\sigma}}^+ \;+\; (1-d^-)\,\bar{\boldsymbol{\sigma}}^- \tag{3}$$

In order to clearly define concepts such as 'loading' or 'unloading' two damage criteria are introduced, one for tension and the other for compression, based on two scalar positive quantities [Faria *et al.* 1993]

$$\bar{\tau}^+ \;=\; \sqrt{\bar{\boldsymbol{\sigma}}^+ : \boldsymbol{D}^{-1} : \bar{\boldsymbol{\sigma}}^+} \tag{4}$$

$$\bar{\tau}^- \;=\; \sqrt{\sqrt{3}\,(K\,\bar{\sigma}^-_{oct} \;+\; \bar{\tau}^-_{oct})} \tag{5}$$

termed 'equivalent stresses'. In the last equation $\bar{\sigma}^-_{oct}$ and $\bar{\tau}^-_{oct}$ are the octahedral normal stress and the octahedral shear stress obtained from $\bar{\boldsymbol{\sigma}}^-$. K is a material property, devised so that predicted 2D and 1D compressive strengths could match the usual 1.16-1.2 ratios reported for concrete in the experimental tests. Inspired in reference [Simo *et al.* 1987] the damage criteria assume the following forms:

$$\bar{\tau}^+ \;-\; r^+ \;\leq\; 0 \tag{6}$$

$$\bar{\tau}^- \;-\; r^- \;\leq\; 0 \tag{7}$$

Variables r^+ and r^- are current damage thresholds, which control the size of the expanding damage surfaces. Previously to the application of any loading the r^+ damage threshold must be set to r_o^+, assumed a material property, which bounds the linear-elastic domain; a similar reasoning applies for compression, and therefore the onset of damage in compression will occur at $\overline{\tau}^- = r_o^-$. Defining as $f_o^{+(-)}$ the stresses beyond which nonlinearity becomes visible under 1D tensile or compressive tests, according to equations (4-5) the elastic thresholds $r_o^{+(-)}$ can be established as

$$r_o^+ \;=\; f_o^+ \big/ \sqrt{E} \tag{8}$$

$$r_o^- \;=\; \sqrt{\sqrt{3}\big/3\,(K-\sqrt{2})\; f_o^-} \tag{9}$$

From the damage consistency conditions, and either for tension or for compression, it results $\dot{r}^{+(-)} = \dot{\overline{\tau}}^{+(-)}$ and consequently for a generic instant *t* it occurs:

$$r_t^{+(-)} \;=\; \max\left\{ r_o^{+(-)},\;\; \max_{s\in[0,\,t]}\,[\overline{\tau}_s^{+(-)}] \right\} \tag{10}$$

In the present concrete model the evolution laws for the damage variables are explicit in terms of the damage thresholds, that is $d^+ = d^+(r^+)$ and $d^- = d^-(r^-)$. As possible evolutions for these damage variables the following definitions can be postulated:

$$d^+ \;=\; 1 \;-\; r_o^+ \big/ r^+\, e^{A\,(1-r^+/r_o^+)} \qquad ,\text{ if } \quad r^+ \;\geq\; r_o^+ \tag{11}$$

$$d^- \;=\; 1 \;-\; r_o^- \big/ r^-\,(1-B) - B\, e^{C\,(1-r^-/r_o^-)} \qquad ,\text{ if } \quad r^- \;\geq\; r_o^- \tag{12}$$

Anyway, and without loss of generality, for the present work some modifications were introduced to the evolution laws (11-12), in order to cope with some particular features of the intended 1D concrete behaviour [Vila Pouca *et al.* 1998], which are reproduced in Figure 1: *(i)* for tension a linear law for the σ–ε softening branch substituted the exponential one inherent to equation (11), and *(ii)* for compression a combination of a parabolic branch up to the peak stress, followed by a linear descending branch, replaced equation (12). The σ–ε definitions for each branch in compression are

$$\textit{Parabolic:}\quad \sigma \;=\; f_{cm}\,\left[2\,\varepsilon/\varepsilon_{cm} - (\varepsilon/\varepsilon_{cm})^2\right] \tag{13}$$

$$\textit{Linear:}\quad \sigma \;=\; f_{cm}\,\left[1 - Z\,(\varepsilon-\varepsilon_{cm})\right] \tag{14}$$

in which the peak strength f_{cm}, the peak strain ε_{cm} and parameter *Z* are established taking into account the degree of concrete confinement *k* due to transversal reinforcement. A possible definition for *k* is

$$k \;=\; 1 \;+\; \rho_v\, f_{syt} \big/ f_{co} \tag{15}$$

where f_{syt} characterises the yielding stress of transversal reinforcement, $\rho_v = A_{sw}\, l_w / (b_c\, h_c\, s)$ stands for the volumetric confinement ratio, defined as a function of A_{sw}, the cross sectional area of the stirrups (with perimeter l_w and separation *s*), and of $b_c \times h_c$, the area of the concrete core effectively confined.

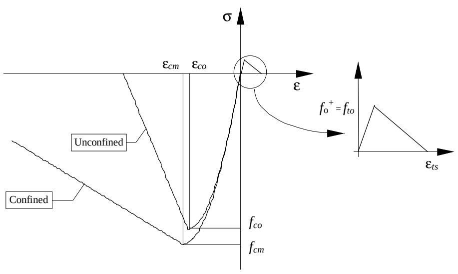


Figure 1. 1D behaviour for confined and unconfined concrete.

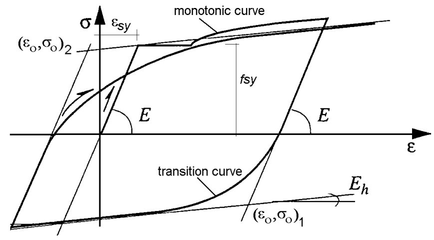


Figure 2. Steel cyclic model.

Denoting by f_{co} and ε_{co} the compressive peak stress and strain obtained from standard 1D unconfined tests, the effect of confinement may lead to the following increments on concrete strength and peak strain

$$f_{cm} = k\ f_{co} \tag{16}$$

$$\varepsilon_{cm} = k^2\ \varepsilon_{co} \tag{17}$$

As for parameter *Z*, an estimation of the form

$$Z = 0.5 \Big/ \left[(3 + 0.29\ f_{co}) / (145\ f_{co} - 1000) + 3/4\, \rho_v \sqrt{h_c/s} - \varepsilon_{cm} \right] \qquad (\, f_{co} \text{ in MPa}) \tag{18}$$

was assumed.

STEEL CYCLIC MODEL

The explicit formulation proposed by Giuffré and Pinto and implemented in reference [Menegotto *et al.* 1973] was chosen to model reinforcement cyclic behaviour. As illustrated in Figure 2, a family of transition curves between two asymptotes intersecting at point $(\varepsilon_o, \sigma_o)$ and with slopes *E* and E_h (the elastic and the hardening modulus) is defined according to equation

$$\sigma^* = b\,\varepsilon^* + (1-b)\,\varepsilon^* \big/ [1 + (\varepsilon^*)^R]^{1/R} \tag{19}$$

where

$$\sigma^* = (\sigma - \sigma_r)/(\sigma_o - \sigma_r) \qquad \varepsilon^* = (\varepsilon - \varepsilon_r)/(\varepsilon_o - \varepsilon_r) \tag{20}$$

$$b = E_h / E \qquad R = R_o - a_1 \xi / (a_2 + \xi) \tag{21}$$

$$\xi = (\varepsilon_{r\max} - \varepsilon_o)/(\varepsilon_o - \varepsilon_r) \tag{22}$$

As for $(\varepsilon_r, \sigma_r)$, they represent the co-ordinates of the last reversal point ($\varepsilon_{r\max}$ is the maximum ε_r ever reached) and *R* is the parameter which tunes Bauschinger's effect. Parameters a_1, a_2 and R_o should be established on the basis of experimental results.

VALIDATION: PSEUDODYNAMIC TESTS OF BRIDGE PIERS

The application that follows documents the numerical simulation of a complex experimental, which concerns to a quasi-static cyclic test of a reduced scale bridge pier, reported in Guedes (1997). Pier's height is 8.4 m, and the cross section is a 0.8×1.6m^2 hollowed rectangle, with walls 0.16m thick, as depicted in Figure 3. Steel reinforcement is constituted by longitudinal rebars with three diameters, that is, 28ϕ14, 12ϕ12 and 40ϕ8, as well as by ϕ5 stirrups with 60 mm spacing, as depicted in the same figure. Concerning the finite element mesh adopted in the numerical simulation, a 2D plane stress discretization was assumed for the concrete (as illustrated in Figure 7 for the next application), whereas the steel bars were reproduced with 2-noded truss elements, reproducing the exact positions of the rebars in the cross section depicted in Figure 3. A constant axial force of 1700kN was firstly applied to the pier's head, accounting for the vertical dead load transmitted by the deck. Afterwards a cyclic horizontal displacement was prescribed at the top of the pier, forcing it to move along the strong axis of the cross section. An infinitely rigid foundation was assumed on pier's footing. Owing to geometric and load symmetries plane stress conditions were assumed. Concerning to the material characterisation, two types of concrete were considered: *(i)* the unconfined one which recovers the steel bars and *(ii)* the confined one, interior to the stirrups. Table 1 resumes the basic material properties which characterise the 1D behaviour of these two types of concrete. Table 2 resumes the material properties assumed for the steel reinforcement.

Comparison between the numerical and the experimental results is depicted in Figure 4, from which it can be inferred that under cyclic loading the pier experiences strong incursion into nonlinearity, both for the concrete or for the steel reinforcement. The force-displacement diagram obtained at the top of the pier with the proposed model exhibits a fairly good agreement with the experimental one. Although the behaviour of the pier is greatly influenced by the nonlinearity installed in the reinforcement [Delgado *et al.* 1999], the agreement for the entire loading history demonstrates that the behaviour of concrete is realistically captured as well, since collapse was obtained in the experimental test during the last cycles. It becomes clear that the model is able to reproduce the continuous change in the structural stiffness, namely the cracking of concrete, the 'pinching' effect dictated by the crack-closing, or even the nonlinearity in compression.

Table 1. Concrete properties (E = 36 GPa).

Concrete	f_{co} (MPa)	ε_{co}	f_{to} (MPa)	f_{cm} (MPa)	ε_{cm}
Confined	50.5	2.5‰	3.8	59.6	3.0‰
Unconfined	50.5	2.5‰	3.8	–	–

Table 2. Steel properties (E = 200 GPa).

Steel	ε_{sy}	ε_{su}	E_h/E	f_{sy} (MPa)	f_{su} (MPa)
Longitudinal	2.50‰	100‰	0.0075	500	650
Stirrups	3.50‰	16‰	0.0116	700	730

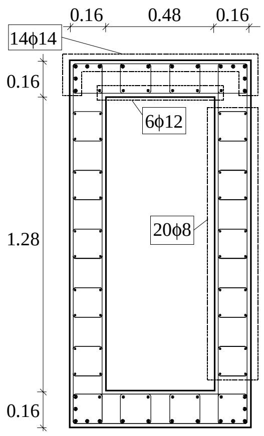


Figure 3. Pier's hollow section.

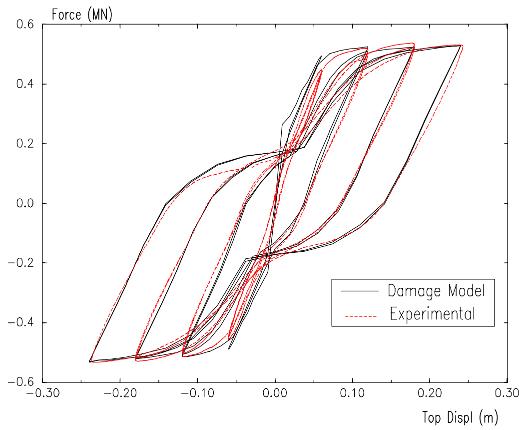


Figure 4. Force-displacement diagrams.

APPLICATION TO WARTH BRIDGE PIER

Talübergang Warth bridge, depicted in Figure 5, is situated about 63 km south of Vienna. A detailed seismic vulnerability assessment is currently under way, and consequently relevant information concerning the ductility and the energy dissipation of their piers is required. The application to be presented herein refers to pier P3, following the notation of Figure 5. The pier has a rectangular hollow cross section, as depicted in Figure 6. For the concrete discretization plane stress finite elements were adopted, according to the mesh reproduced in Figure 7. The longitudinal steel reinforcement varies along the pier's height, but essentially two regions are of relevance: *(i)* a first one from the pier's foundation up to 7m and *(ii)* a second one respecting the remainder uppermost 30m of the pier. Linked to notation from Figure 6, Table 3 resumes the layout of such longitudinal steel reinforcement. Transversal reinforcement is executed with a single rectangular stirrup on each wall of the hollow section, constituted by ϕ12//0.20m on a 1m extension at pier's bottom, and ϕ8//0.20m on the rest of the height. Owing to the poor confinement provided by such stirrup arrangement no favourable confinement effect was assumed for the concrete, and consequently the material properties concern solely the unconfined situation: E = 33.5 GPa, f_{co} = 43MPa, ε_{co} = 2‰ and f_{to} = 3.1MPa. Table 4 resumes the material properties assumed for the longitudinal and transversal steel reinforcement and the parameters adopted for the Menegotto Pinto model were $R_o = 30$, $a_1 = 27.0$ and $a_2 = 0.15$, the same as for the previous pseudodynamic validation test.

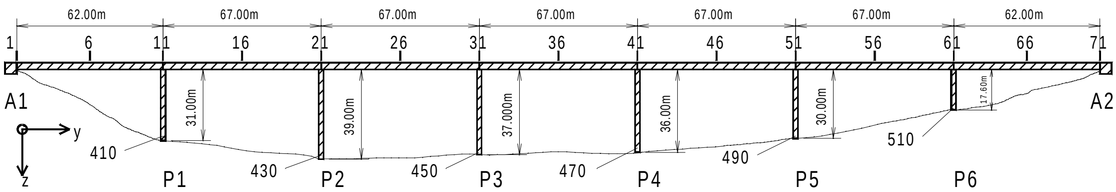


Figure 5. Talübergang Warth bridge.

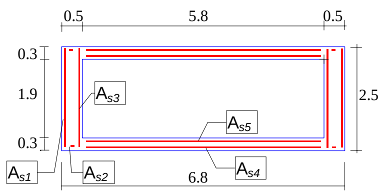


Figure 6. Cross section of pier P3.

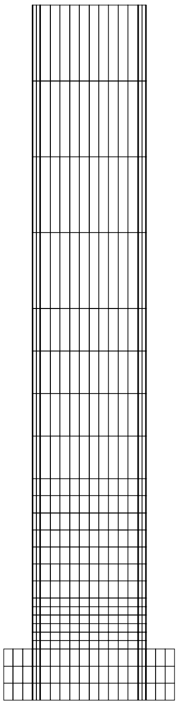
Table 3. Longitudinal steel reinforcement in pier P3.

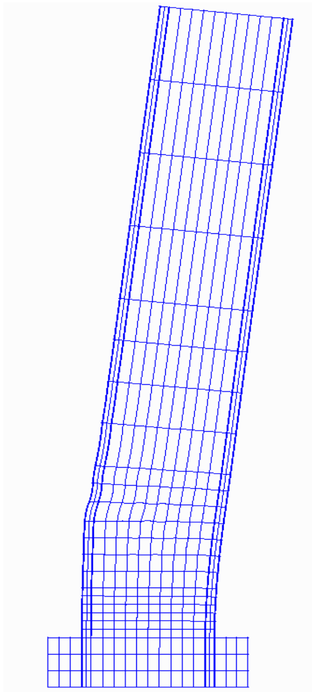
	A_{s1}	A_{s2}	A_{s3}	A_{s4}	A_{s5}	A_{sTotal}
Region *(i)*	19ϕ16	2ϕ16	17ϕ16	47ϕ16	47ϕ16	268ϕ16
Region *(ii)*	10ϕ16	1ϕ16	8ϕ16	24ϕ16	24ϕ16	136ϕ16

Table 4. Steel properties (*E* = 200 GPa).

Steel	ε_{sy}	ε_{su}	E_h/E	f_{sy} (MPa)	f_{su} (MPa)
	2.725‰	100‰	0.0034	545	611

A vertical force of about 23MN was firstly applied to the head of the pier, so as to account for the dead load which arises from the deck. Subsequently an increasing horizontal displacement was imposed to the top of the pier, forcing it to move monotonically along the strong axis of the cross section. The base of the pier's foundation was assumed perfectly fixed. Figure 8 reproduces the deformed configuration close to collapse, which occurs due to failure of the reinforcement, while Figure 9 provides a detail of the concrete compressive stresses which occur on the pier's bottom. The occurrence of a dominant diagonal crack at this location emphasises that the failure of the pier leads to a concentration of deformations on the steel reinforcement at level 7m, where the longitudinal reinforcement is reduced to 50%.





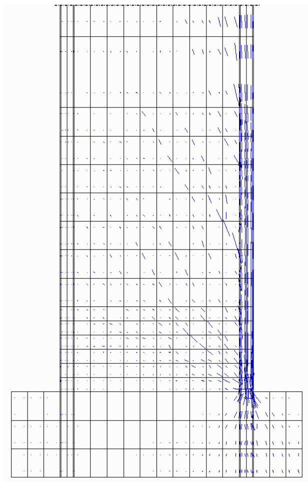


Figure 7. Concrete mesh. **Figure 8. Deformed configuration.** **Figure 9. Compressive stresses.**

Obviously this strut-and-tie mechanism is due to the influence of shear, fully captured by the proposed R/C model, but which can not be detected by classic fibre models. Owing to this limitation of the fibre model, and since the collapse load is much more influenced by the reinforcement concerning region *(ii)* in Table 3 than by the maximum one occurring in region *(i)*, for the failure force to be captured in such a model the corresponding numerical simulations should preferably be performed not with the real reinforcement layout reproduced in Table 3, but with the reinforcement concerning the region *(ii)*. Reproducing the envelope curve concerning the horizontal force *versus* displacement registered at the head of the pier for the entire monotonic test, Figure 10 allows to compare the R/C model proposed here with a fibber model described in [Guedes 1997]. The predictions from the fibre model were obtained for both of the longitudinal reinforcement layouts above referred: the real one expressed in Table 3 (case A in Figure 10) and the one assuming constant reinforcement of region *(ii)* (case B). As expected, cases A and B from the fibre model are associated with envelope curves which bound the more realistic one obtained from the proposed R/C model, with case B providing the best predictions.

Figure 11 reproduces the force-displacement diagram (horizontal direction) predicted under cyclic conditions by the proposed R/C model. From this global response it becomes evident that the Warth bridge pier exhibits poor ability to dissipate energy under cyclic conditions.

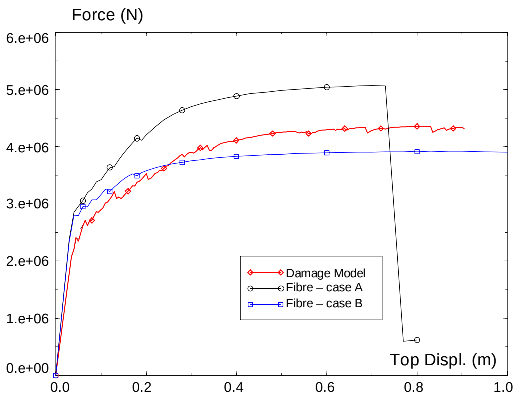


Figure 10. Monotonic loading.

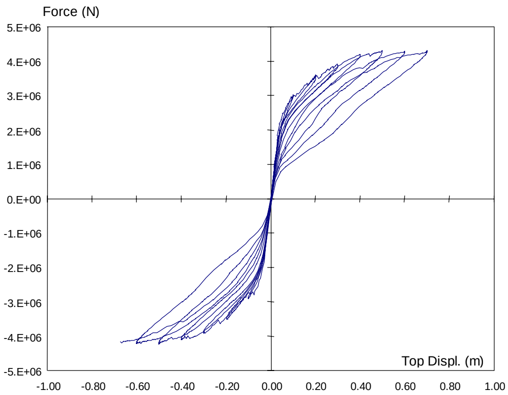


Figure 11. Cyclic behaviour.

CONCLUSIONS

In this paper the most relevant aspects of a constitutive model suitable for the modelling of the non-linear behaviour of reinforced concrete bridge piers (rectangular hollow sections included) were presented. For the concrete itself, a constitutive law founded on the Continuum Damage Mechanics was used, combined with a 2D finite element discretization. For the cyclic behaviour of the steel reinforcement the formulation proposed by Giuffré-Menegotto-Pinto was adopted, and prescribed on 2-noded truss elements which reproduce the steel rebars quite accurately. The validation of the model was performed on the basis of a reduced scale hollowed section bridge pier experimentally analysed during a pseudodynamic test. The comparison of the model predictions with the available experimental results showed a very good agreement, and evidenced the ability of the proposed R/C model to simulate the seismic behaviour of this kind of bridge piers. Preliminary analysis of a representative pier from the Austrian Talübergang Warth bridge was performed, aiming to check its behaviour under cyclic loads. A comparison of the numerical results obtained with the proposed R/C model with the ones from a fibre model was performed, which put into evidence the ability from the former to reproduce the strut-and-tie mechanism which is observed in concrete members close to failure conditions, due to the influence of shear. Besides, the interference on the strut-and-tie mechanism due to the interruptions of the longitudinal reinforcement was also fully captured by the proposed R/C model.

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