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Seismic response by pseudodynamic tests of RC bridges designed to Eurocode 8 and Italian seismic code

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ABSTRACT: This work is a contribution to the assessment of EC8 capability of providing adequate ductility and low damageability to regular and irregular bridges. Two bridges (one regular and one irregular) have been designed according to EC8 Part 2 assuming a design seismic input motion with a PGA equal 0.35g and a structural behaviour factor equal to 3. Two companion bridges having the same geometry have been also designed according to the Italian Seismic Code DM 24.1.1986 for comparison. Pseudodynamic tests of the four structures have been performed; for each bridge a 1:6 model of the most stressed pier has been tested in laboratory, while the remaining components: deck and piers, have been simulated numerically by substructuring techniques. Reinforcement scaling criteria, based on similitude fulfilment of global quantities: flexural and shear strength, confinement effect, post-elastic buckling and pullout of rebars, are proposed. Two accelerograms have been used: the Tolmezzo E-W component from the 1976 Friuli Earthquake and the Kobe-Kayou E-W component from the 1995 Hansin Earthquake. The structures designed using the EC8 code performed satisfactory for both structures, the ones designed using the Italian code did perform satisfactorily as well although larger damage and degradation was observed. Numerical simulations of the most significant tests did provide a further insight on the mechanical behaviour of the specimens.

1 INTRODUCTION

The new Eurocode 8 Part 2 (EC8.2) for the design of bridges in seismic areas is a European Prestandard (ENV) since 1994. The Code represents an important step forward, with respect to present regulations; it is in fact a rather comprehensive document that considers many of the complex problems involved in seismic response of bridges subjected to intense seismic shaking. In the last years, many numerical and experimental studies have been presented on the effectiveness of the provisions contained in EC8 [15]. The authors of the present paper presented some initial results on the EC8 bridges at 11th WCEE [7]. Two continuous deck bridges designed to EC8, one with a regular configuration and another with an irregular one (see Fig.1), have been subjected to pseudodynamic tests using pier models scaled 1:6 and substructuring technique for the rest of the structures. In the present paper those results are presented and compared to the results relative to laboratory tests carried out on the same two bridges designed to Italian seismic regulations [13]. Numerical analyses using a new fibre beam element with shear modelling [17] are also included providing further information on the mechanical behaviour of R.C. piers.

Testing apparatus was set up in house and a software for instrumentation control and numerical integration was purposely developed [2, 16]. The pseudodynamic test method allows to perform tests on large structural elements where the masses involved are numerically simulated and the static actuators apply the required reactions. Huge dimensions of bridges imply that their scaled model still represent structures of considerable dimensions. Though scale reduction problems are easily handled from a theoretical point of view [1], the crude application of theory can give rise to serious practical problems that complicate the RC specimen fabrication. Moreover, past studies showed that some parameters, such as steel-concrete bond, are hardly controlled and, if not properly scaled, could significantly distort the response. Alternative scaling criteria for large structures such as RC bridge piers are therefore proposed. Similitude criteria between model and prototype are developed with respect to global quantities, such as: flexural and shear strength, confinement effect, post-elastic buckling and pullout of rebars. This approach does not imply the "perfect" geometrical scaling of aggregate granulometry, reinforcing bar diameters and deformed bar shape and spacing, allowing the use of ordinary concrete mixing and commercial reinforcing bars.

Two bridges (one regular and one irregular) have been designed accordingly to EC8: assuming a design peak ground acceleration equal to 0.35g and a structural behaviour factor equal to 3. The irregular bridge was subjected to the E-W component of the 1976 Italian Tolmezzo earthquake (PGA = 0.35g), the regular one to the N-S component of the accelerogram recorded at Kayou Weather Bureau during the 1995 Kobe earthquake (PGA = 0.82g). Preliminary numerical simulations showed that shorter piers of both bridges are expected to experience large inelastic deformation, while the remaining elements should remain in the elastic range. Six reduced models were manufactured, varying the number and the diameter of longitudinal bars and spiral spacing. An improved anchorage detail of the longitudinal bars to improve the bond-slip scaling accuracy has also been investigated.

The performance of the irregular bridge was very satisfactory: significant flexural and shear deformation occurred in the tested pier, which nonetheless showed enough strength and ductility to cope with the imposed seismic action. Comparison between models with and without special anchorage detail showed that the response was highly influenced by pullout scaling. Inelastic deformations and damage to the regular bridge were much smaller than for the irregular one. The overall behaviour of the two structures demonstrates that bridges designed to EC8 have enough safety and low damageability. Perhaps less severe design parameters could pursue adequately safe structures at reduced cost.

Although the last version of the Italian seismic code has been updated in 1996, the 1986 version [13] was used for the design of the other two structures. In this way, models representative of the Italian existing stock built during the ’70 and ’80 and designed according to codes very similar to the 1986 version were obtained. Large differences for both longitudinal and transverse reinforcements were found between the Italian and the EC8 Code for the irregular bridge. For the regular bridge instead, approximately the same longitudinal reinforcement was obtained, although the Italian code still required less transverse reinforcement.

Italian piers have been tested with the same accelerograms and cyclic load history of the Eurocode ones. The regular bridge performed satisfactorily, similarly to the EC8 one, while the irregular one had a different response, showing a higher degradation, given the large difference in longitudinal reinforcement.

2 DESIGN OF THE BRIDGES

2.1 *EC8 Bridges*

Eurocode 8 Part 1 [9] and Eurocode 8 Part 2 [10] for the design of bridges in seismic areas were used with the Eurocode 2 [8] for the general rules on concrete structures. Figure 1 shows both regular and irregular bridges geometry. The bridges have a continuous box girder hinged on the circular section piers and on the abutments. 0.35g peak ground acceleration was assumed for the elastic response spectrum together with a behaviour factor of 3 for horizontal and 1 for vertical component of the input motion. Pier concrete strength was fixed at 25 N/mm^2, steel yielding stress at 500 N/mm^2. Deck self weight and dead load was 200 KN/m. Elastic stiffness of the deck was

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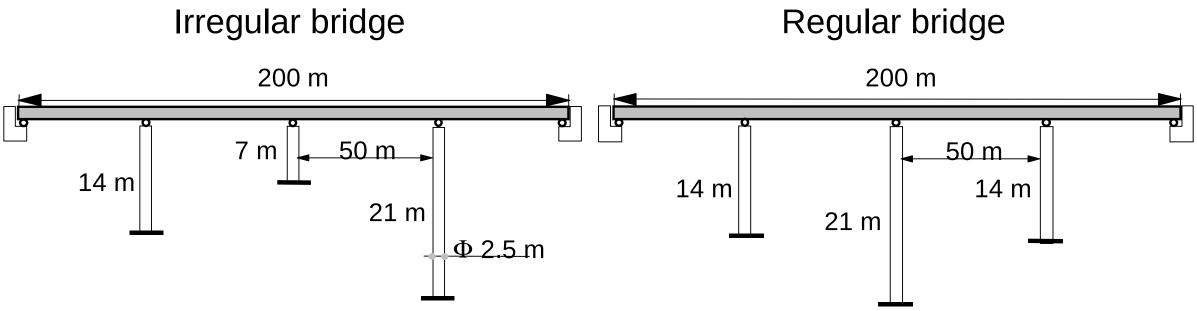

Fig. 1: Layout of designed bridges

calculated on the basis of the gross section; the piers stiffness instead based on the cracked section (50% that of the gross section). Figure 2 shows the central pier of the irregular bridge and the lateral one of the regular bridge, the ones experimentally tested.

2.1.1 *Irregular Bridge*

First mode of the structure had a period of about 0.7 s. Reinforcing details of the central pier are reported in Fig. 2a. While for lateral piers the minimum longitudinal reinforcement (0.8% of the gross section) was sufficient, for the central one, 3.5% of the gross section was required. Capacity design prescribed in this case a dense spiral to avoid shear failure.

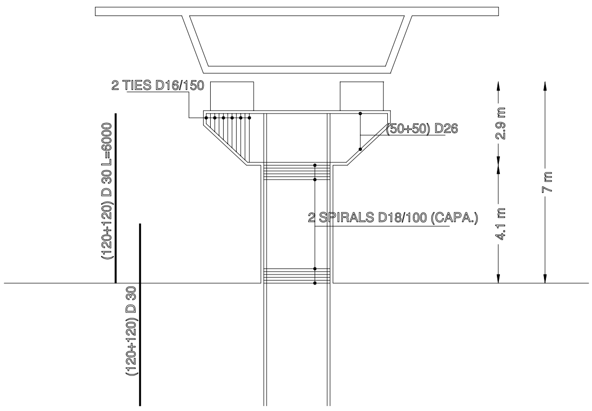

Fig. 2a: EC8 irregular bridge central pier: rebars arrangement (mm)

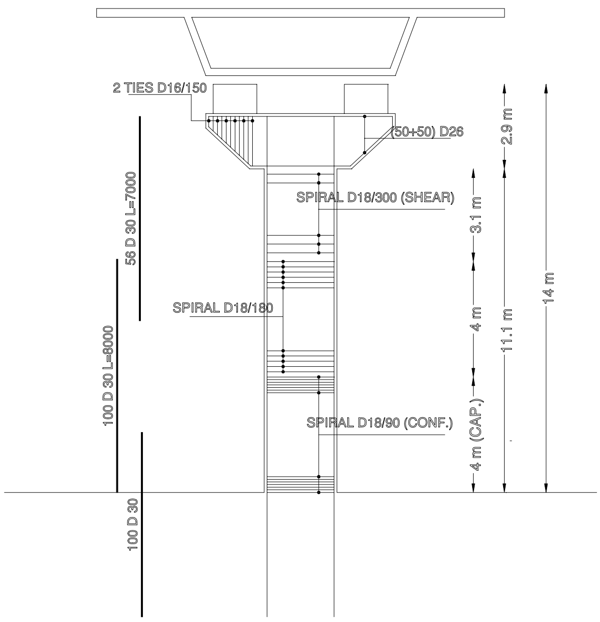

Fig. 2b: EC8 regular bridge lateral pier: rebars arrangement (mm)

2.1.2 *Regular Bridge*

First mode of the structure, using pier cracked stiffness as in the previous case, yielded a period of about 1.3 s. Reinforcing details of the lateral piers are reported in Fig. 2b. Minimum longitudinal reinforcement was sufficient for the central pier; for the lateral one 1.5% of the gross section was required instead. In all piers transverse reinforcement at the base section was determined according to confinement requirements; outside plastic hinge region instead, it was designed by shear verifications.

2.2 *Italian Bridges*

The reference Code was [13] for the design of bridges in seismic areas and [14] for the general rules on concrete structures. Regular and irregular bridge geometry is the one depicted in Fig. 1. The design acceleration response spectrum, used for allowable stress verifications, has a constant intensity of 0.1g from 0.0 to 0.8 s. Allowable stresses in steel is about 1.5 times lower than the ultimate one. Materials characteristics, deck self-weight and dead load were the same as for the EC8 bridges. Elastic stiffness of the deck and of the piers was calculated on the basis of the gross sections, as commonly adopted by structural designers.

2.2.1 *Irregular Bridge*

First mode of the structure, based on gross section characteristics, had a period of about 0.5 s. Minimum longitudinal reinforcement (0.35% of the gross section) was sufficient for the lateral piers, for the central one instead, 0.86% of the gross section was required. Minimum transverse reinforcement was sufficient for all piers. Reinforcing details of the piers, compared with EC8 ones, are reported in the following table. One can note that only 25% of EC8 longitudinal reinforcement is needed, and 8.8% of transverse one.

	Longitudinal reinforcement			Transversal reinforcement		
	Pier H=14	Pier H=7	Pier H=21	Pier H=14	Pier H=7	Pier H=21
Eurocode 8	56 D30	(120+120) D30	56 D30	D18/90-180-300	2 D18/100	D18/90-180-300
Italian code	24 D30	60 D30	24 D30	D12/250	D12/250	D12/250

2.2.2 *Regular Bridge*

First mode of the structure, based on gross section stiffness had a period of about 0.9 s. A longitudinal reinforcement ratio of 0.72% and 1.44% of the gross section was required for the central and lateral piers respectively. Minimum transverse reinforcement was sufficient in all piers. Reinforcing details of the piers, compared with EC8 ones, are reported in the following table. One can see that while the piers have the same longitudinal reinforcement, transverse reinforcement varies from 10% in the lower part of the pier to 50% in the section above.

	Longitudinal reinforcement		Transversal reinforcement	
	Pier H=14	Pier H=21	Pier H=14	Pier H=21
Eurocode 8	100-56 D30	56 D30	D18/90-180-300	D18/90-180-300
Italian code	100-32 D30	50-24 D30	D12/250	D12/250

3 SCALING CRITERIA (FROM[7])

Due to the laboratory hardware limitations, piers were scaled 1:6. Classical scaling [11] consists in a rigorous geometrical reduction of both concrete and rebars dimensions. This approach requires use of microconcrete and steel wires annealed (for yielding) and threaded (for bond-slip).

However, microconcrete and treated steel have been often found to behave differently from the original materials. The large number of longitudinal bars and the small spacing between hoops (Fig. 2), further increasing the difficulties of fabrication using this approach.

The alternative scaling criteria here adopted [3, 4] allowed for the use of ordinary concrete and commercial rebars. According to this criteria, once the geometrical scale factor (l_r) and the longitudinal bar scale factor (ϕ_r) have been fixed, the similitude of prototype (index p) and model (index m) global quantities can be calculated with the remaining scaling factors found accordingly. The longitudinal bar number (N) is given from the similitude of the resisting moments (M_u), corresponding to the stress σ_s in the extreme bar:

$$M_u^{(p)} = \frac{\psi\, D^{(p)}\, N^{(p)}\, \pi\, \phi^{(p)2}\, \sigma_s}{4} = \frac{M_u^{(m)}}{l_r^3} = \frac{\psi\, l_r\, D^{(p)}\, N^{(m)}\, \pi\, \phi_r^2\, \phi^{(p)2}\, \sigma_s}{4 l_r^3} \qquad (1)$$

where $D^{(p)}$ is the pier section diameter, σ_s is the steel stress and ψ is a non-dimensional factor that accounts for the section shape and longitudinal bar layout and $N^{(m)}$ is found as follows:

$$N^{(m)} = \frac{l_r^2\, N^{(p)}}{\phi_r^2} \qquad (2)$$

Spiral spacing (i) is given from similitude of post-elastic buckling of rebars, spiral diameter (ϕ_t) from similitude of shear strength and confinement effect, anchorage length (L) from similitude of concrete-steel bond:

$$i^{(m)} = \phi_r\, i^{(p)} \qquad \phi_t^{(m)} = \phi_t^{(p)} \sqrt{l_r \phi_r} \qquad L^{(m)} = \frac{L^{(p)} \phi_r\, u_{1r}^{\alpha}}{\nu n_b\, q_{1r}^{\alpha}\, l_r^{\alpha}} \qquad (3,4,5)$$

To fulfil the similitude requirement on bond (u_{1r}, q_1 and α are parameters of the Filippou bond-slip model, whose values were obtained on the basis of experimental results published by Eligehausen, see [12]) which requires a reduced anchorage length for the model longitudinal reinforcing bars, two additional bars were added laterally at each rebar through welding in the anchorage zone. In (3,4,5) $\nu = (3\, n_b + 1) / (4\, n_b)$ is a factor that takes into account the bond surface reduction due to bar proximity and welding (n_b=3 is the number of bars welded in parallel instead of the single bar).

3.1 *EC8 Bridges*

For the irregular bridge central pier 4 specimens were manufactured: specimen I71A was arranged with 42 12 mm diameter longitudinal bars and a 6 mm spiral 40 mm spaced with plain anchorage; specimen I72A was arranged with 24 16 mm diameter longitudinal bars and a 8 mm spiral 50 mm spaced with plain anchorage. Specimen I71B and I72B had the same reinforcements of A specimen, but with the improved anchorage detail described before. Two specimens of the regular bridge lateral piers were manufactured: specimen R141A was arranged with 24 10 mm diameter longitudinal bars and a 5 mm spiral 30/60/100 mm spaced with plain anchorage (Fig 3a); specimen R141B had the same reinforcement but with improved anchorage detail (Fig. 3b).

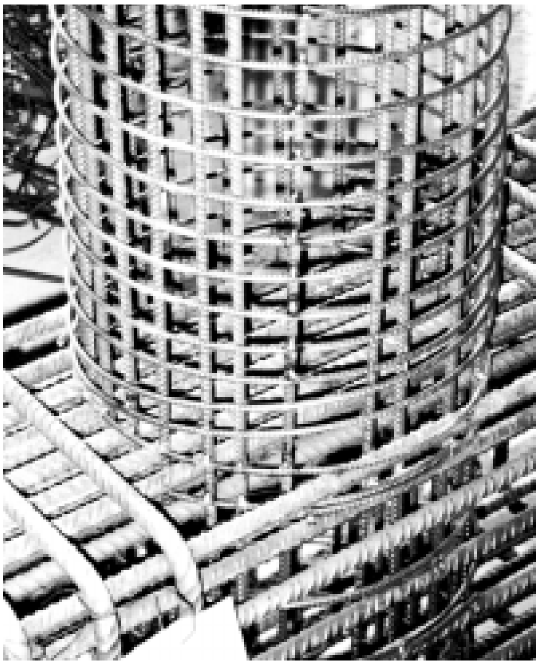

Fig. 3a: EC8 specimen reinforcement detail: plain anchorage

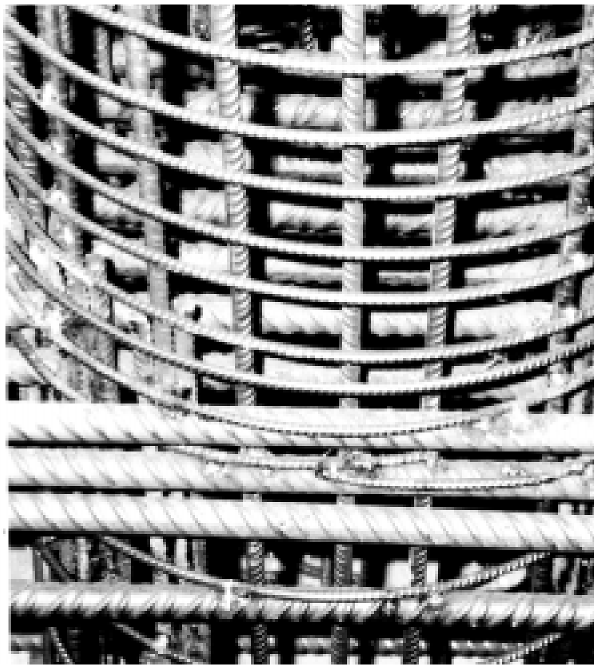

Fig. 3b: EC8 specimen reinforcement detail: improved anchorage

3.2 *Italian Bridges*

Four specimens of the piers were set up, two for each bridge. All specimens were manufactured with the improved anchorage in the foundation. For the irregular bridge two specimen having the same reinforcement were prepared. Longitudinal reinforcement was given by 12 12mm bars, transverse reinforcement by circular stirrups, placed each 12 cm. However due to a construction error, one of the 7m irregular bridge central piers was built without the improved anchorage detail. For the regular bridge also, two specimens were prepared at 1:6 scale. Longitudinal reinforcement was given by 24 12mm diameter bars; in one case (specimen IR14BN2) lap splices were introduced at the column base with a 40 bar diameters overlapping, in the other specimen (IR14AN2) no lap splice was introduced. Transverse reinforcement was given by spiral having 6 mm with a 12 cm pitch.

4 TEST SEISMIC INPUT

4.1 *Irregular Bridge*

Elastic response spectra with 5% damping of about 100 Italian accelerograms were compared in the 0.5-0.8 s period range where the first mode of the structure falls. Tolmezzo (E-W) accelerogram from the 1976 earthquake with 0.35g peak ground acceleration was found to be the most severe for the bridge.

4.2 *Regular Bridge*

Two different registrations: Kobe-Kayou Weather Bureau E-W component (PGA=0.82g) and Amagasaki elevated bridge N-W component (PGA=0.35g) both from the Great Hansin Earthquake of 1-17-1995, were suitable. The first one has been adopted as being more severe, following preliminary numerical simulations (Fig. 5). The differences between design and input response spectra can be appreciated in Fig. 4. Elastic 5% damping response spectra of Tolmezzo, Kobe-Kayou, Amagasaki and EC8 design spectrum reduced by three behaviour factors: 1, 1.7, 3 are shown. The value 1.7 corresponds to the effective reduction resulting from the following effects: 2% damping adopted instead of 5%; design horizontal components, which acted simultaneously in two directions in the design, while only one transversal component was applied in the test; material's mean strengths exceeding design ones.

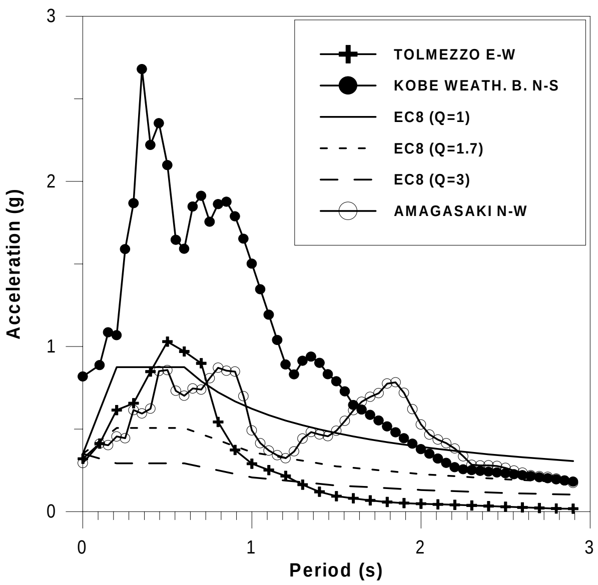

Fig. 4: Natural earthquakes and EC8 spectrum design

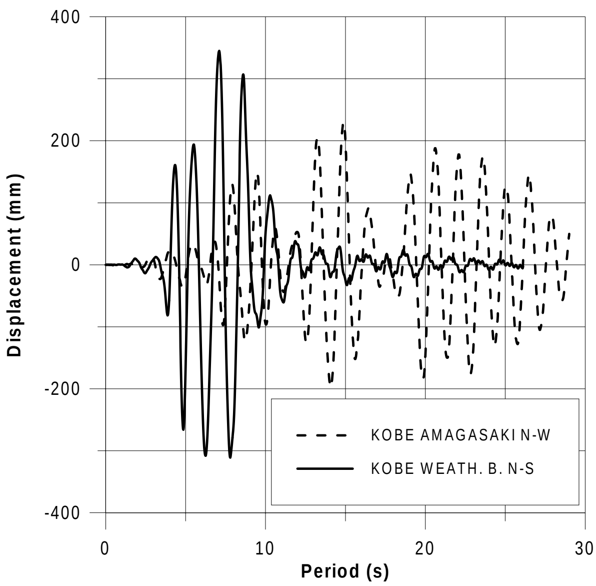

Fig. 5: EC8 regular bridge: numerical simulations response with Kobe earthquake registrations

5 RESULTS

5.1 *EC8 Bridges*

5.1.1 *Irregular Bridge*

Four tests were carried for each of the four specimens: preliminary free vibrations, Tolmezzo pseudodynamic, Tolmezzo scaled to 0.7g pseudodynamic, static cyclic until failure. Results are reported in prototype scale. Tested concrete strength was about the same of the design one, steel yielding varied from 500 to 600 MPa, therefore exceeding the design one [5, 7].

Specimens having same type of anchorage but different bar diameter: I71A and I72A (plain anchorage), I71B and I72B (special anchorage) behaved similarly. These results validated the reinforcement scaling criteria. Instead specimen I71A versus I71B and I72A versus I72B (same reinforcement but different anchorage) behaved very differently. Low intensity free vibrations tests in the elastic range showed the same secant stiffness, equal to 70% of the gross section one, for both plain and special anchorage. Secant stiffness in higher intensity elastic free vibrations was respectively 34% for plain and 50% for special anchorage.

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In the pseudodynamic tests, lateral piers behaved elastically; therefore only the central pier response will be described in the following. As shown in Fig. 6, top displacement for A and B piers during the first 4 seconds was almost the same and remained in the elastic regime. From 4 to 5 s, 4 high amplitude cycles (more than 80 mm) were obtained; the following semicycles until 6.5 s were still large for special anchorage, while decreasing for plain anchorage. In the last 4.5 s, amplitude decreased for B piers and remained the same for A piers. At the end both specimens showed limited damage, whit 45° cracking on the whole height due to intense shear effect. The dissipated energy was higher in special anchorage specimen in all cycles (Fig 8) with specimen B dissipating twice the energy than specimen A (Fig.7).

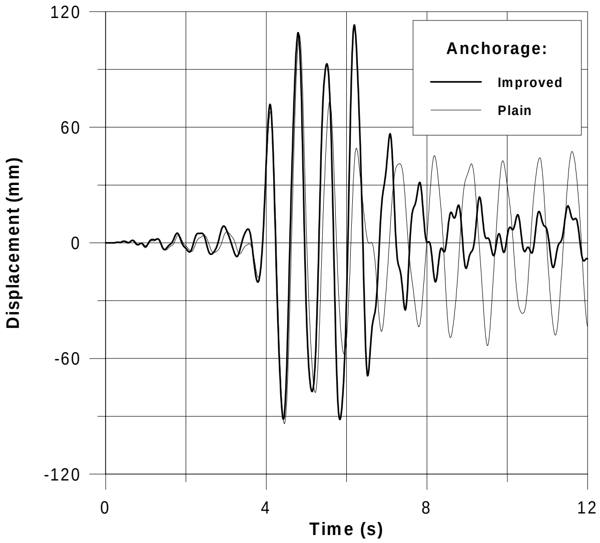

Fig. 6 : EC8 irregular bridge central pier: pseudodynamic test with Tolmezzo - displacement time history

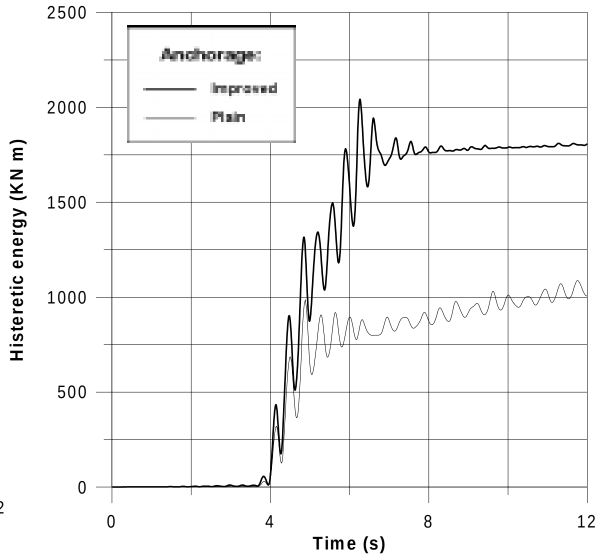

Fig. 7 : EC8 irregular bridge central pier: pseudodynamic test with Tolmezzo - dissipated energy time history

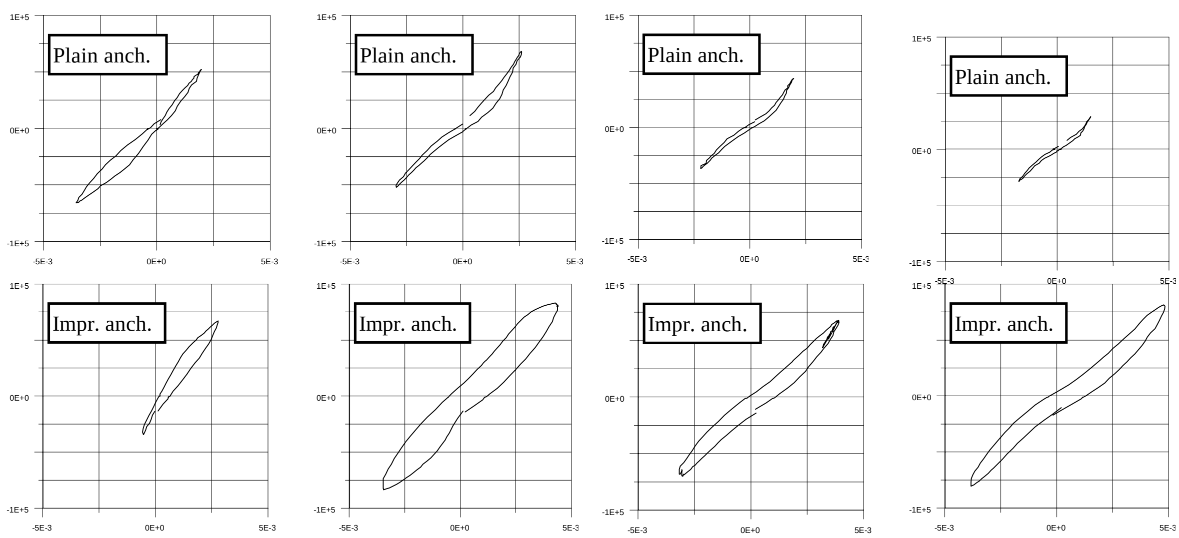

Fig. 8: EC8 irregular bridge central pier – pseudodynamic test with Tolmezzo: Moment (KNm)-Curvature (1/m) cycles from 4 to 6.5 s

Flexural displacements were computed by integrating curvatures along the height, the contribution due to shear was then found as the difference between the pier top total displacement and the computed flexural one. Shear displacement came out to be about 50% of the total one (Fig. 9). This explains why preliminary numerical simulations, based on purely flexural models, leaded to smaller maximum displacements.

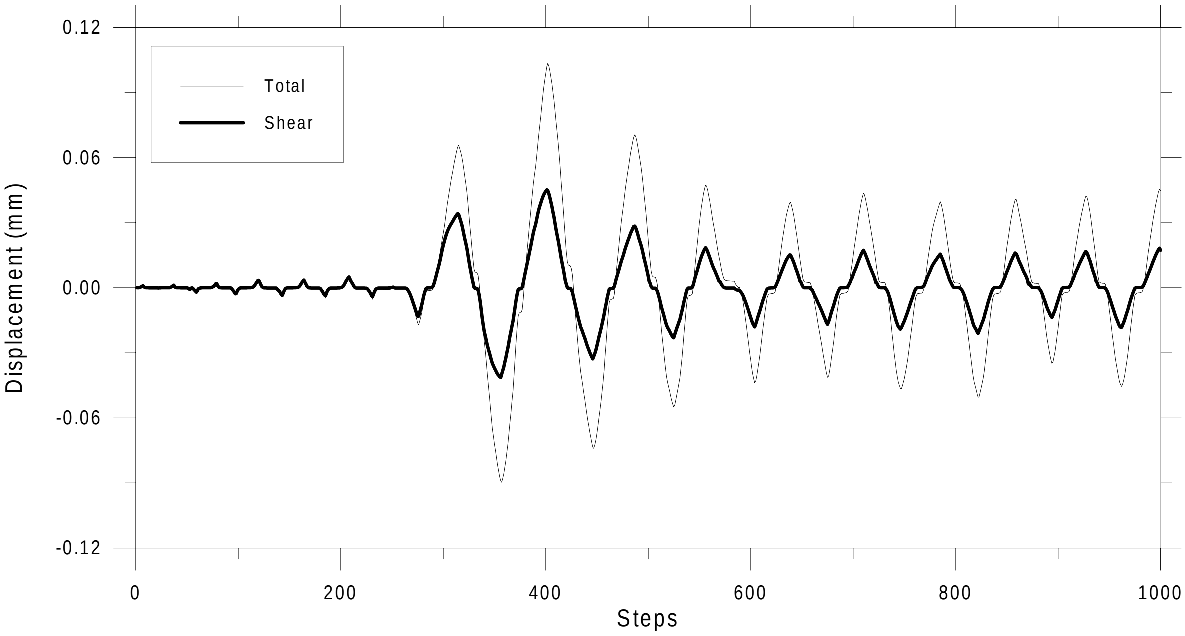

Fig. 9: EC8 irregular bridge central pier with improved anchorage: evaluation of displacements due to shear and flexure.

In the subsequent tests, with Tolmezzo record scaled to 0.7g, smaller differences between A and B specimen have been obtained. Maximum displacement was 0.18 m (at prototype scale) still with limited final damage. In the following static cyclic tests, collapse was reached due to spiral failure immediately followed by longitudinal rebar buckling and breakage.

5.1.2 *Regular Bridge*

Two tests were carried for each of the two specimens: preliminary free vibrations and Kobe Kayou pseudodynamic. Results are reported in prototype scale [6]. R141A and R141B (same reinforcement and different anchorage) behaved similarly. In fact, bar pull-out is less important for slender piers than for squat ones, therefore accurate reproduction is not necessary.

During the pseudodynamic tests the central piers remained in the elastic field. Figure 10 shows the time history displacement of the lateral pier. Significant amplitude cycles (150 to 300 mm) took place between 4 and 16 s. In Fig. 11 the total force-displacement history is reported. Maximum attained ductility was about 4, larger then the design q factor since the spectral ordinate at 1.3 s period of the Kobe record is about 3 times larger than EC8 with q=1.7. At the end both specimens were only slightly damaged, with an incoming crushing of the cover. However, the increase of the bridge period indicated some damage in the rebars. The good performance of the pier is believed to depend mostly on the transverse reinforcement that provided sufficient concrete confinement and avoided rebars buckling.

Preliminary numerical simulations carried out with the purely flexural numerical model, shown in Fig. 5, were in close agreement with the experimental findings. This is due to the fact that no significant shear deformations took place in the tested piers.

5.2 *Italian Bridges*

Only preliminary results for these structures are today available. At present, tests have been performed only on the irregular bridge with plain anchorage and on the regular one without lap splices at the base section.

5.2.1 *Irregular Bridge*

Free vibration response at small amplitude is shown in Fig. 12. The period of vibration of the Italian Bridge is about 0.8 seconds, while the EC8 has a period of about 0.4 seconds. The longer period of the Italian Bridge is strongly influenced by the lack of improved anchorage detail in the foundation causing improper scaling (model bar slippage comparable to real structure one). In Fig. 13 force-displacement diagrams for the Italian Piers are given together with those corresponding to the EC8 piers, in case of plain anchorage. One can see that the stiffness ratio between Italian and EC8 columns is about 0.4. Part of this difference may be due to inaccurate anchorage scaling although it must be recalled that the Italian Bridge has only 25% of the EC8 bridge longitudinal reinforcement. Total inertia of the section can be in fact calculated adding the concrete section inertia to that of the rebars as follows:

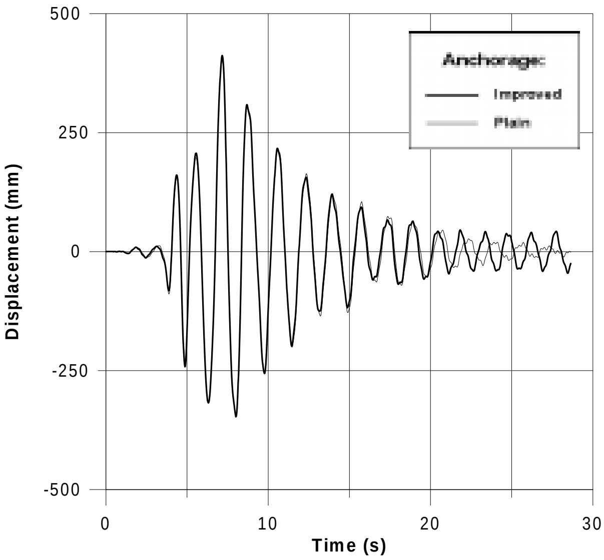

Fig. 10: EC8 regular bridge central pier – Pseudo-dynamic test with Kobe – Displacement time history

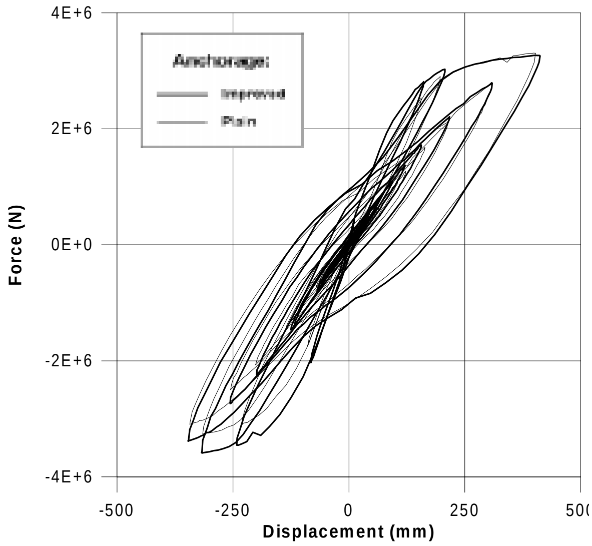

Fig. 11: EC8 regular bridge central pier - Pseudo-dynamic test with Kobe – Force-Displacement cycles

$$J_{tot} = J_c + 2 \cdot \frac{E_r}{E_c} \cdot \rho \cdot J_{gross}$$

with $E_{r,c}$ being the moduli of reinforcement and of concrete, ρ geometrical reinforcement ratio, J_c the inertia of the concrete section and J_{gross} the inertia of the gross section. The second term on the right hand side becomes significant for the EC8 pier where ρ = 0.035.

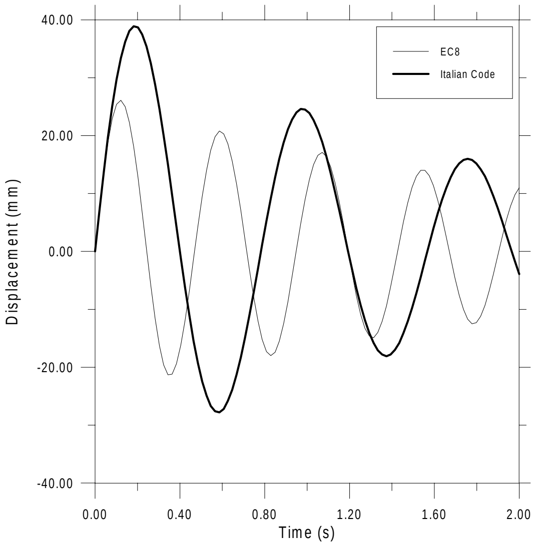

Fig. 12: EC8 and Italian irregular bridge central pier – Free oscillations – Displacement time history

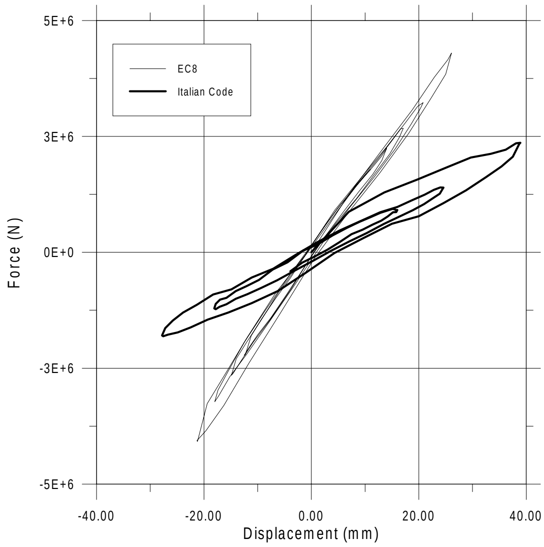

Fig. 13: EC8 and Italian irregular bridge central pier - Free oscillations – Force-Displacement cycles

The response history to Tolmezzo accelerogram is shown in Fig 14. In the same figure, the response of the EC8 Bridge with plain anchorage is plotted for comparison. Fig. 15 shows the pier Force-Displacement response. One can see that the reduction of secant stiffness between elastic free oscillations and large displacement amplitude cycles is greater in the Italian pier than in the EC8 one. Energy dissipation for the Italian structures is obviously smaller given the lower amount of longitudinal reinforcement, as can be seen in Fig 16. With the central pier being the only member to undergo plastic deformations the ratio of dissipated energy to the Bridge total elastic energy is also smaller in the Italian Bridge compared to the EC8 one. The two bridges in fact absorb a similar amount of energy from the ground input motion and, as a result, the Italian Bridge takes longer to damp out the response. As evident from Fig. 14, the Italian Bridge undergoes more cycles in the plastic range compared to the EC8 one since the non-linear behaviour continues after the strong part of seismic shacking is over.

As far as cracking pattern is concerned, there is a substantial difference between Italian and EC8 specimen. In the Italian piers in fact, due to smaller flexural strength, the shear force is proportionally reduced to roughly one fourth of the EC8 case. As a consequence, only a single large diagonal crack at the column base is observed. The crack, initiated in bending develops with a considerable inclination due to shear. Outside the plastic hinge region instead, no other cracks could be observed. Therefore while in the EC8 structure a substantial contribution to total displacement was given by shear cracking along the entire height, in the Italian Bridge plastic distortion were concentrated at the base section. This explain why the Italian pier dissipated more energy per unit amount of longitudinal reinforcement then the EC8 one.

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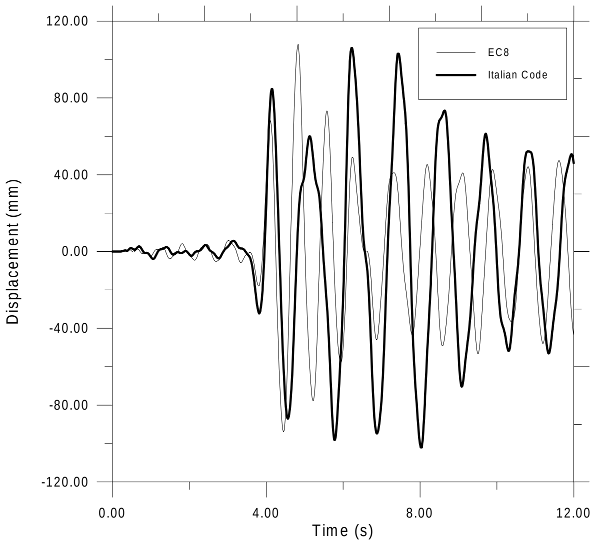


Fig. 14: EC8 and Italian irregular bridge central pier – test with Tolmezzo PGA=0.35g Displacements time history

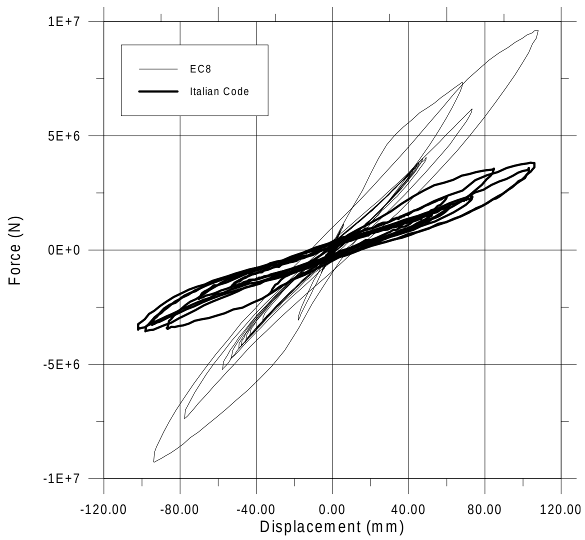


Fig. 15: EC8 and Italian irregular bridge central pier - test with Tolmezzo PGA=0.35g Force-Displacement cycles

The response at 0.7g is shown in Fig 17. For this intensity the structure undergoes larger displacements and damage compared to the EC8 one although the test showed that adequate strength was available to survive to the assigned the seismic input.

Failure of the column has been finally obtained with quasi-static cycles of increasing amplitude. Failure was essentially due to buckling of longitudinal rebars that were not adequately restrained by the hoops contrary to the EC8 plain anchorage specimen where, in fact, buckling was prevented by the close stirrup spacing.

5.2.2 *Regular Bridge*

Figure 18 shows the free oscillation time history of the regular bridge. One can see that curves are almost coincident with those of the EC8 Bridge since the two structures have the same longitudinal reinforcement. The period of vibration is 1.3 seconds.

In Fig. 19 the pseudodynamic response to Kobe accelerogram is shown, compared to the EC8 case. Again the response is very similar although in the Italian pier the final stiffness is reduced with respect to EC8 one. This difference may be due to a larger degradation of the Italian specimen, having less transverse reinforcement, but could also depend on the inevitable scatter that specimen usually exhibit due to differences in manufacturing, material characteristics and so on. It should be remarked in fact that the manufacturing quality of the Italian specimen was not as good as that of the EC8 specimen. Still, at large ductility during the final quasi-static test, rebars buckling and concrete spalling were observed in the Italian pier which did not happened to the EC8 one.

Whether the difference in transverse reinforcement had the same influence at lower ductility (during the pseudodynamic test) cannot be definitely stated.

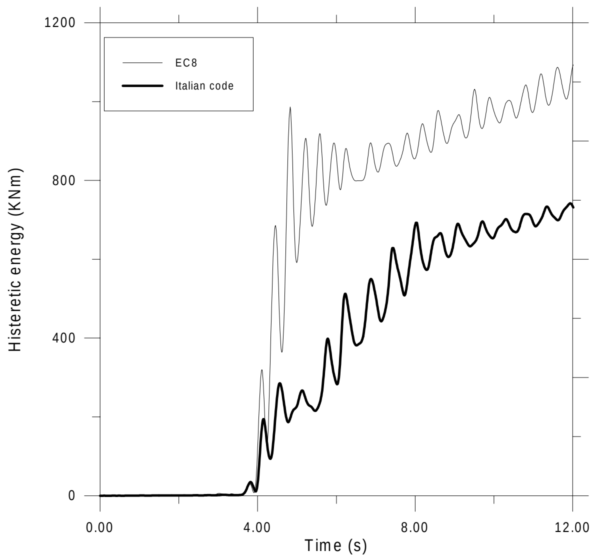


Fig. 16: EC8 and Italian irregular bridge central pier: test with Tolmezzo PGA=0.35g - Dissipated energy time history

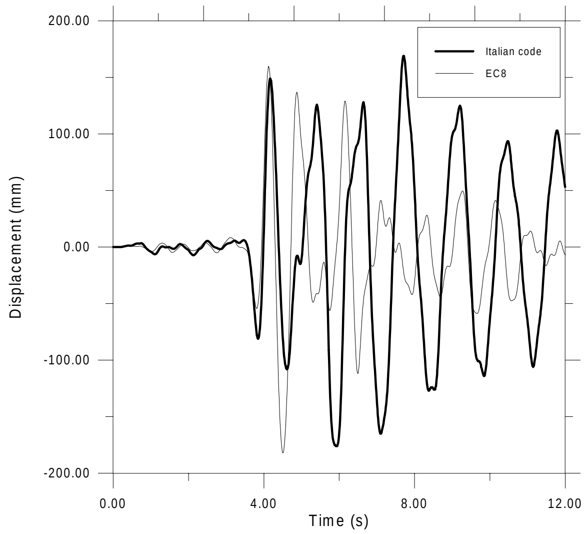


Fig. 17: EC8 and Italian irregular bridge central pier - test with Tolmezzo PGA=0.7g - Displacements time history

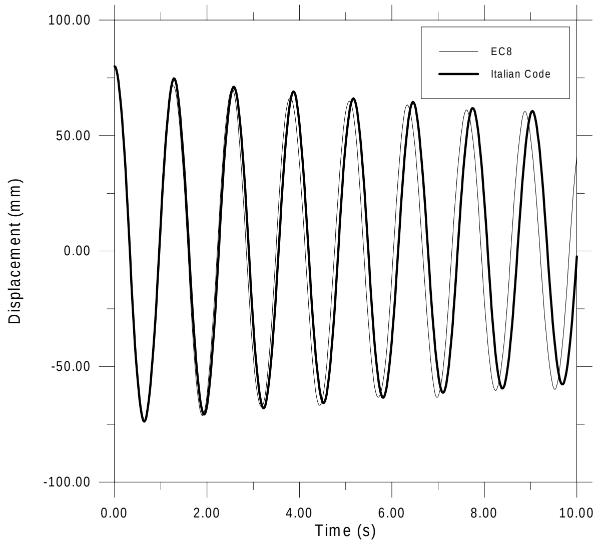


Fig. 18: EC8 and Italian regular bridge lateral pier – free oscillations – Displacements time history

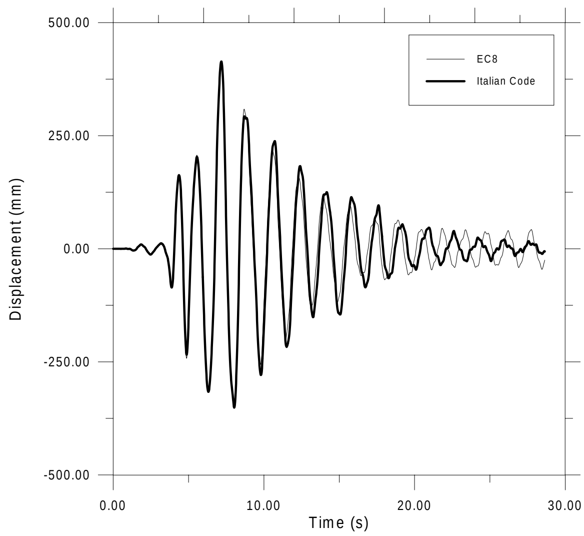


Fig. 19: EC8 and Italian regular bridge lateral pier - pseudodynamic test with Kobe – Displacements time history

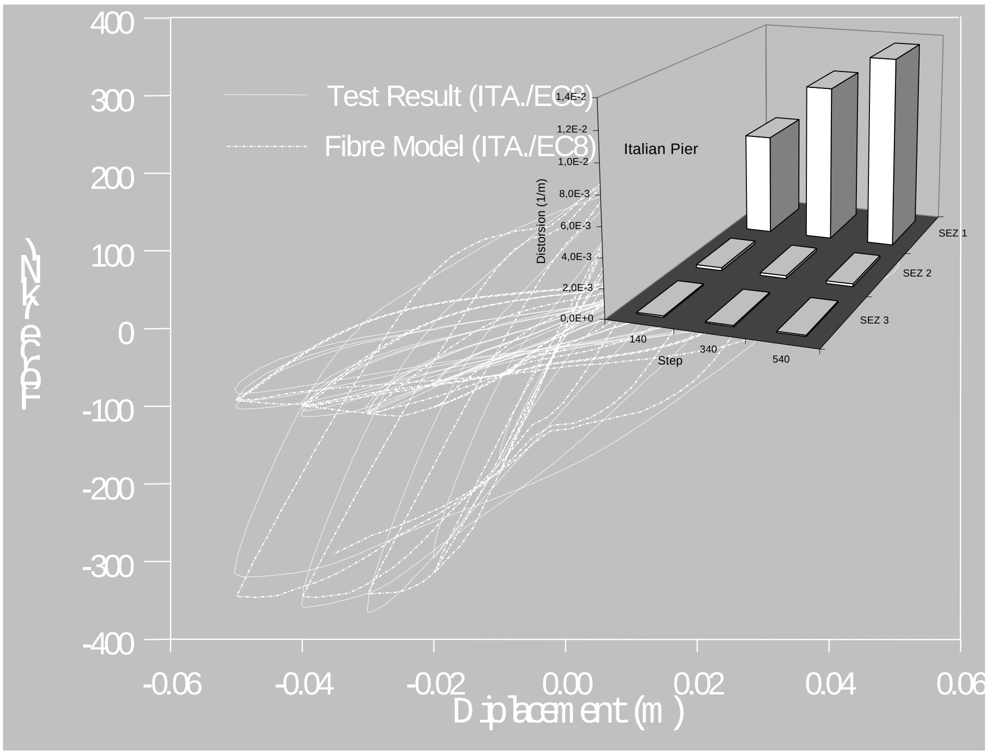
6 THE NUMERICAL ANALYSES

Numerical simulations of the squat pier of the irregular bridges have been performed using a fibre beam element with shear modeling recently developed by the authors [17] and implemented in FIBER finite element code.

The model is a development of the traditional fibre beam element which now accounts, at each fibre, for the shear strain field and for the transverse confinement due to the stirrups. The section strain field is therefore found as the superposition of the plain section hypothesis with a similar kinematic constraint for the shear strain field (user defined shape functions). Transverse strains are found instead by imposing the equilibrium with the stirrups under the assumption of perfect bond.

A Microplane based constitutive model is used to monitor the concrete fibre response. Forces and deformations at the monitoring sections (Gauss's points) are integrated along the element using an equilibrium based approach which allows for damage localization in within the element (plastic hinge regions) [18].

The numerical results found with the model reproduce the experimental findings. The force-displacement response is plotted in Fig. 20, comparing the experimental results with the numerical simulations for the two piers. Since these quasi static tests were performed after the pseudo-dynamic simulations, the piers showed a degraded strength and stiffness which had to be accounted for, in the numerical model, imposing two cycles at the same maximum amplitude (20 mm) reached by the two specimens during the previous tests. Still, yield penetration and bond slip should be included in the model, as they account for a large percentage in the flexural degradation



of the specimens.

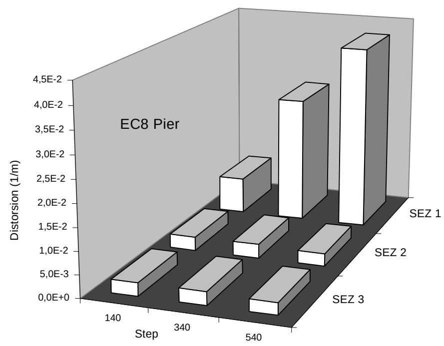
Fig. 20 – Numerical simulation of the quasi-static test for the short pier of the Irregular Bridge

The shear distortion at the integration points (sections) along the element is plotted in Fig. 21 at three different stages of a quasi-static test. Here the different behavior of the two piers as well as the different shear response along their height can be appreciated.

The shear distortion of the pier designed to the Italian code is almost zero outside the plastic hinge region since cracking does not cut trough the section as it happens in the EC8 case. In this case, the shear distortion, following the initial cracking along the whole height, increases at the plastic hinge location only because of bending-shear degrading interaction while remains constant elsewhere as the maximum shear force does not increase during the analysis.

The results shows the difference between members (the Italian pier) where the concrete section shear resistance is in excess of the bending induced shear forces compared with members where it is not. In this case a diffuse shear cracking is observed before the stirrup resisting mechanism can be activated. This phenomenon may have beneficial effect in reducing the structural irregularity but does not contribute to energy dissipation and requires extensive repairs work along the whole height of the piers even after medium intensity earthquakes since, being force driven, it is independent from the maximum ductility reached during the event. In order to avoid this mechanism to develop, a limit on the concrete section shear resistance–bending induced shear force should be introduced. This limit may be more stringent then the actual codes limitations on concrete resisting mechanism in shear which are aimed to prevent the compressive strut failure and in fact may design the concrete section in the plastic hinge region but hardly apply outside it. Shear cracking outside plastic hinge region is instead a tensile type of cracking, especially in r.c. piers subjected to low axial force.

Fig.21 – Shear distortion along the pier height during the quasi static tests



7 CONCLUSIONS

Results have been presented regarding continuous deck r.c. bridges designed to the EC8.2 and to the 1986 Italian Seismic Code. The Bridges designed to EC8.2 have shown a large capability to undergo inelastic deformations with limited damage. This is essentially due to capacity design and small pitch of confining spiral. However possible simplifications with less stringent requirements could probably lead to adequate performances and should be investigated.

The preliminary results shown for the Bridges designed using the 1986 Italian Code showed acceptable performances for the regular bridges configuration, similar to the EC8 one. On the contrary, in case of irregular bridge layout the behaviour of the Italian Bridges has proved to be less satisfactory although still providing sufficient protection. Greater damage has been observed with remarked degradation of the cyclic response. One should be also aware that continuous deck usually has a positive influence on the structural response. Therefore, the results relative to the Italian Bridge may underestimate the damage that can be suffered by the majority of the Italian existing bridges which are often made of simply supported beams.

Shear deformation represents an important contribution to the total displacement and strongly reduces the stiffness of squat piers. This fact can produce positive effects on structural behaviour, as it reduces structural irregularity. The tests and the numerical analyses using the fibre beam element with shear modelling provided a further insight on shear resisting mechanisms inside and outside plastic hinge region.

The tests also confirmed the importance of an accurate scaling of the longitudinal rebars anchorage in the foundation so as to obtain meaningful test results, especially in squat piers. Therefore, further results regarding the Bridges designed to the Italian Code are now under elaboration.

Those presented here refer to structures where longitudinal reinforcement anchorage in the foundation is not properly scaled. Although this fact does not significantly influence slender members as the lateral pier of the regular bridge, it is believed it can have an important influence on squat members as the short pier of the irregular one. For this latter, as for the companion EC8 case, large degradation are expected, increased by the reduction of bond slip in the longitudinal rebars due to the improved anchorage.

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