Paper No. 1310. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

SEISMIC RESPONSE BY PSEUDODYNAMIC TESTS OF RC BRIDGES DESIGNED TO EC8

A. DE SORTIS[1] and C. NUTI[2]

[1]Servizio Sismico Nazionale, Via Curtatone 3, 00185 Roma, Italy
[2]Dip. di Scienze, Storia dell'Architettura e Restauro, Università "G. D'Annunzio", 65127 Pescara, Italy

ABSTRACT

This work is a contribution to the assessment of EC8 capability of providing adequate ductility and low damageability to regular and irregular bridges by pseudodynamic tests on whole bridges. Two bridges (one regular and one irregular) have been designed according to EC8 Part 2: design PGA was 0.35 g, the structural behavior factor was 3. 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. Scale problems to prepare RC pier models have been deeply studied. Reinforcement scaling criteria, based on similitude fulfillment of global quantities: flexural and shear strength, confinement effect, post-elastic buckling and pull-out of rebars, are proposed. Two accelerograms have been used in the tests: Tolmezzo E-W, from 1976 Friuli Earthquake to test the irregular bridge, and Kobe-Kayou E-W from 1995 Hansin Earthquake, for the regular. Results show good performance of the bridge under severe seismic action.

KEYWORDS

Pseudodynamic tests; Eurocode 8; small scale; RC model; RC bridge; RC pier; experimental test.

INTRODUCTION

The new Eurocode 8 part 2 (EC8.2) for the design of bridges in seismic areas is an European Prestandard (ENV) since 1994 and will become an operative code in a few time. In the last years many numerical and experimental studies have been presented on the effectiveness of the provisions contained in EC8 (Nuti *et al.*, 1994). The broadest and more comprehensive is certainly the one supported by the European Commission under the Prenormative Research Program (Calvi *et al.*, 1994) whose first results are now becoming available. In the present study bridges, having the same geometry of those subjected to pseudodynamic testing in the Prenormative Research, have been analyzed numerically and experimentally in order to investigate some of the aspects not included in that study. The first considered item was the optimization of the pseudodynamic with extensive use of substructuring technique, the second was the refinement of the scaling criteria, reducing costs and obtaining reliable results, finally different shape of the section of the piers were considered with respect to Prenormative research.

Testing apparatus was set up in house (Shing *et al.*, 1991; Colangelo *et al.*, 1994) and a software for instrumentation control and numerical integration was purposely developed. The pseudodynamic test method allows to perform tests on large structural elements, because the masses involved are numerically simulated

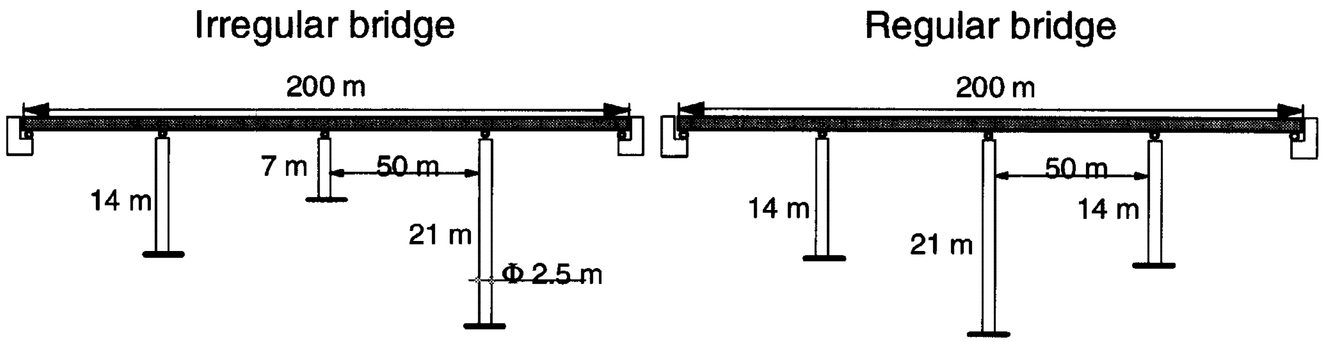


Fig. 1. Layout of designed bridges

and the strong reactions are easily carried by static actuators. 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 (Buckingham, 1914), the crude application of theory can give rise to serious practical problems which 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. Therefore alternative scaling criteria for large structural elements to be experimentally tested, such as RC bridge piers are proposed. Similitude criteria between model and prototype were developed with respect to global quantities, such as: flexural and shear strength, confinement effect, post-elastic buckling and pull-out of rebars. This approach does not imply the "perfect" geometrical scaling of aggregate granulometry, reinforcing 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: design peak ground acceleration was 0.35 g, the structural behavior was 3. The irregular bridge was subjected to 1976 Italian Tolmezzo E-W earthquake (PGA = 0.35 g), the regular one to 1995 Japanese Kobe-Kaiyou Weather Bureau N-S (PGA = 0.82 g). 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 the spiral spacing, or adding a anchorage detail to improve bond-slip scaling accuracy.

Bridge performance of the irregular bridge was very satisfactory: significant deformation occurred in the tested pier, which anyway showed enough strength to cope with the imposed seismic action. High shear deformations, that is hardly predictable by analytical models, occurred. Comparison between models with and without special anchorage detail showed that the response was highly influenced by pull-out scaling. Also the regular bridge showed very small damages at the end of the test. In this case numerical models accurately predicted the response, that was mainly influenced by flexural behavior. Test results suggested that bridges designed to EC8 had enough safety and low damageability. Perhaps less severe design parameters could pursue adequately safe structures at reduced cost.

DESIGN OF THE BRIDGES

The reference Codes were Eurocode 8 Part 1 (EC8.1, 1993) and Eurocode 8 Part 2 (EC8.2, 1993) for the design of bridges in seismic areas, Eurocode 2 (EC2, 1991) for the general rules on concrete structures.

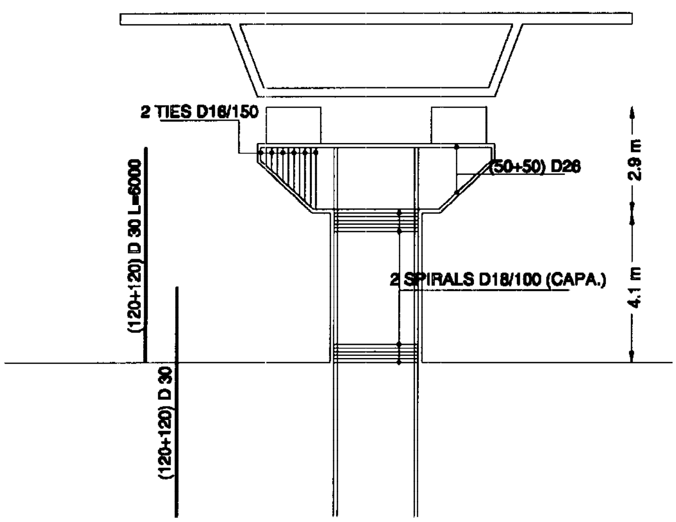


Fig. 2a. Irregular bridge central pier:
rebars arrangement (mm)

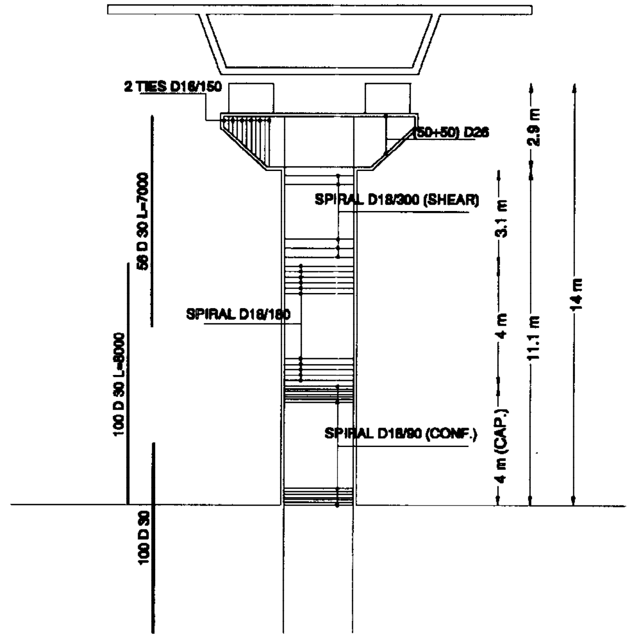


Fig. 2b. Regular bridge lateral pier:
rebars arrangement (mm)

Figure 1 depicts both regular and irregular bridge geometry, with continuous box deck hinged on the cantilever circular section piers and on the abutments. The elastic response spectrum was assumed to have 0.35 g peak ground acceleration. The behavior factor was 3 for horizontal and 1 for vertical component of input motion. Materials characteristics were: pier concrete strength 25 N/mm^2, steel yielding stress 500 N/mm^2. Deck self weight and dead load was 200 KN/m. Elastic stiffness of the deck was calculated on the basis of the gross section; for piers the stiffness of the cracked section (one half that of the gross) was assumed. Figure 2 shows the central pier of irregular bridge and the lateral piers of regular one, experimentally tested.

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 3.5% of the gross section was required. For the latter, capacity design prescribed a dense spiral to avoid shear failure. For construction simplicity such transversal reinforcement was continued in a short part outside the plastic hinge.

Regular Bridge

First mode of the structure had 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 was required a 1.5% of the gross section. In all piers transversal reinforcements at the base were due to confinement effect; outside plastic hinge they were determined by shear verifications.

SCALING CRITERIA

Because of the laboratory hardware limitations, piers were scaled 1:6. Classical scaling (Krawinkler *et al.*, 1982) consists in a rigorous geometrical reduction of both concrete and rebars dimensions. That implies the need of microconcrete and steel wires annealed (for yielding) and threaded (for bond-slip).

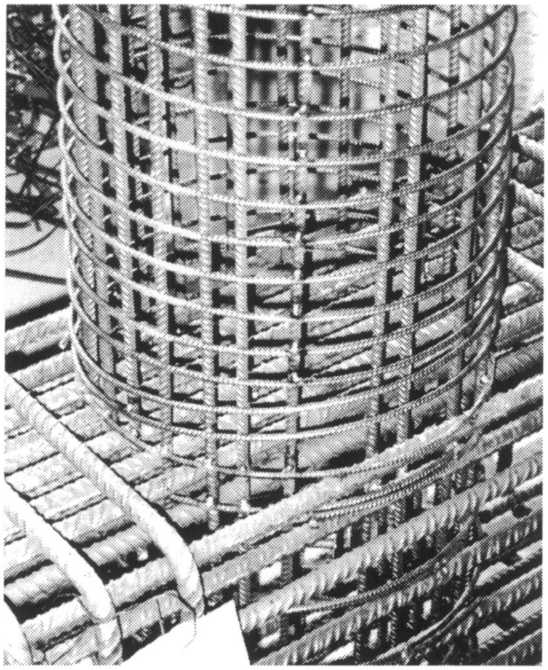


Fig. 3a. Specimen reinforcement detail: plain anchorage

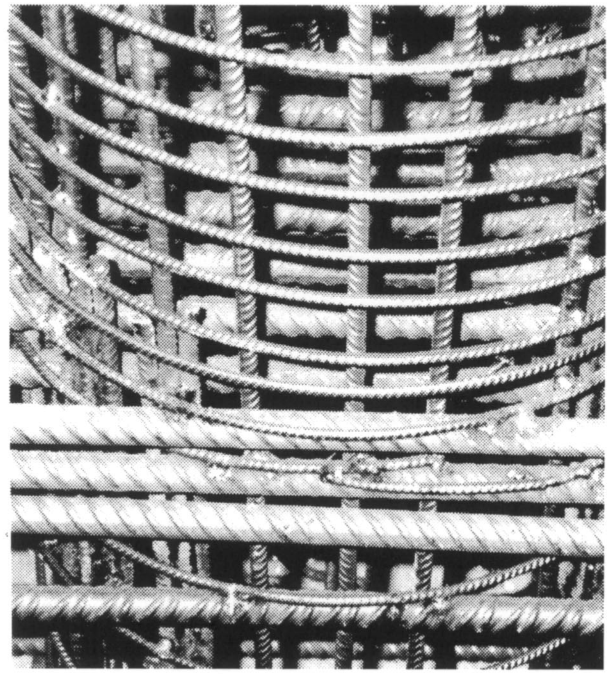


Fig. 3b. Specimen reinforcement detail: improved anchorage

However, the behavior of microconcrete and treated steel was often found to be different from that of original materials. The great number of longitudinal bars and the small spacing between transversal ones (Fig. 2), highlight the difficulties of fabrication arising from that approach.

The alternative scaling criteria here adopted (De Sortis *et al.*, 1994; De Sortis, 1995) permitted the use of ordinary concrete and commercial rebars. Given the geometrical scale factor (l_r) and the longitudinal bar scale factor (ϕ_r), the similitude of prototype (index p) and model (index m) global quantities leaded to remaining scaling factors. 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 y is a non dimensional factor that accounts for section form and longitudinal bar disposition),

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

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 fulfill the latter requirement (u_{1r}, q_1 and α are parameters of the Filippu bond-slip model, whose values was obtained on the basis of experimental results published by Eligehausen, see Monti, 1994) with limited anchorage length for the longitudinal bar diameters to be used in the models, two bars were added laterally at each rebar through welding in the anchorage zone. In the formula $\nu = (3\ n_b + 1) / (4\ n_b)$ is a factor that takes in 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 rebar).

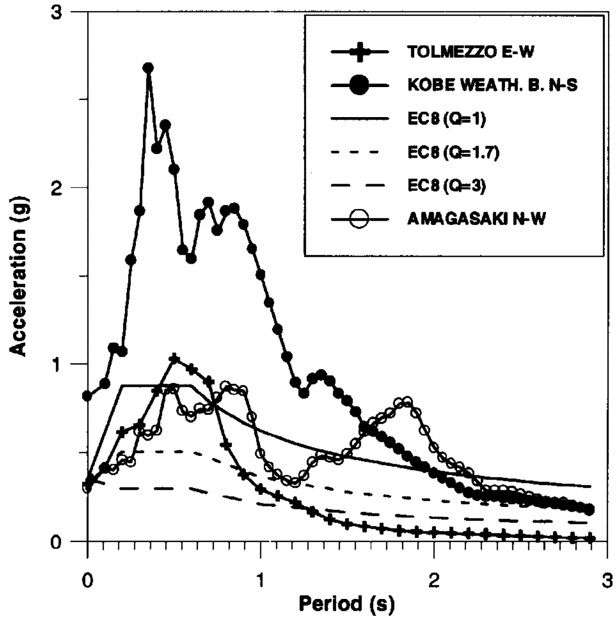


Fig. 4. Natural earthquakes and EC8 design response spectrum

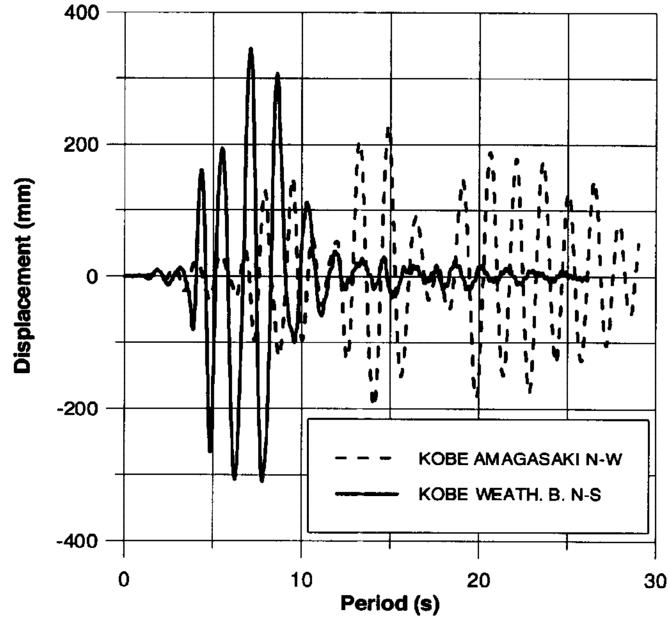


Fig. 5. Regular bridge: numerical simulations with Kobe earthquake registrations

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. 2 specimens were manufactured of the regular bridge lateral piers: 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 (Fig. 3b).

TEST SEISMIC INPUT

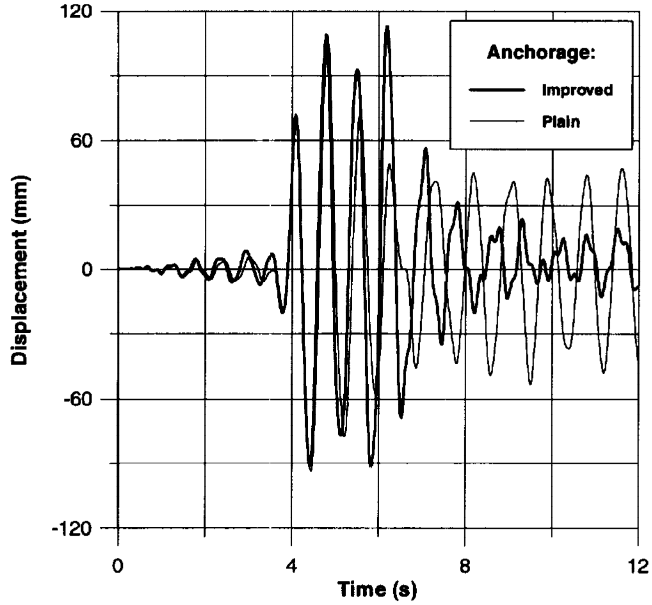
Irregular Bridge

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

Regular Bridge

Two different registrations: Kobe-Kayou Weather Bureau E-W (PGA=0.82 g) and Amagasaki elevated bridge N-W (PGA=0.35 g), from Great Hansin Earthquake of 1-17-1995, were suitable. By numerical preliminary simulations (Fig. 5) the latter resulted as the most severe and adopted.

The differences between design and test action can be appreciated in Fig.4. Elastic 5% damping response spectra of Tolmezzo, Kobe-Kayou, Amagasaki and EC8 design spectrum reduced by three behavior 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, that exceeded design ones.



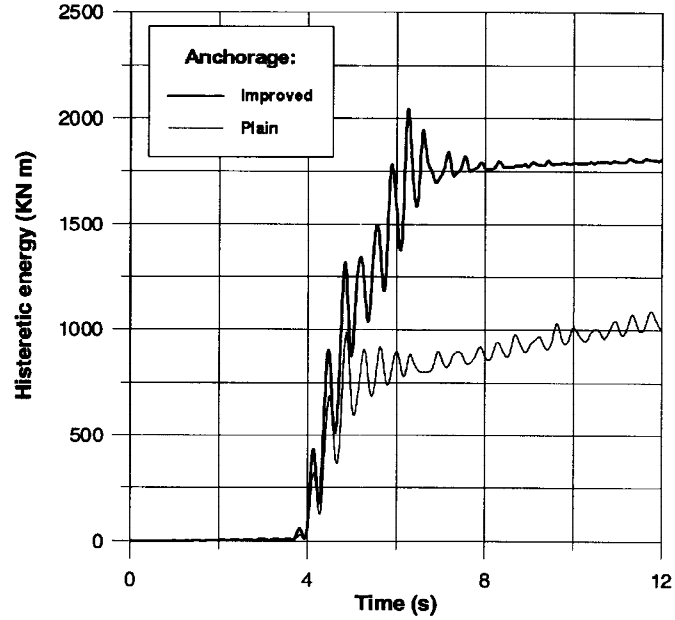


Fig. 6. Irregular bridge central pier: pseudodynamic test with Tolmezzo - displ. time history

Fig. 7. Irregular bridge central pier: pseudodynamic test with Tolmezzo - energy time history

RESULTS

Irregular Bridge

Four tests were carried for each of the four specimens: preliminary free vibrations, Tolmezzo pseudodynamic, Tolmezzo scaled to 0.7 g pseudodynamic, static cyclic until failure. Results are reported in prototype scale. Tested concrete strength was about the same of the design, steel yielding varied from 500 to 600 MPa, therefore greater than the design one (De Sortis, Monti and Nuti, 1995).

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

In pseudodynamic tests, lateral piers behaved elastically, therefore in the following only central pier response will be described. As shown in Fig. 6, top displacement in A and B piers from 0 to 4 s, was nearly the same and elastic; from 4 to 5 s, 4 high amplitude (more than 80 mm) cycles were obtained; the following semicycles until 6.5 s were still large for special anchorage, while decreasing for plain. In the last 4.5 s, amplitude decreased for special and remained the same for plain. At the end both specimens showed low damages, whit 45° cracking on the whole height, due to intense shear effect. The dissipated energy was higher in special anchorage specimen: at any cycle (Fig 8) and the total (Fig.7), B dissipated twice than A Displacements due to flexure only were computed by integrating curvatures on the height, the contribution due to shear was then derived by difference from the total. Displacement due to shear was about 50% of the total. This explains why preliminary numerical simulations, based on flexural models, leaded to smaller maximum displacements.

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

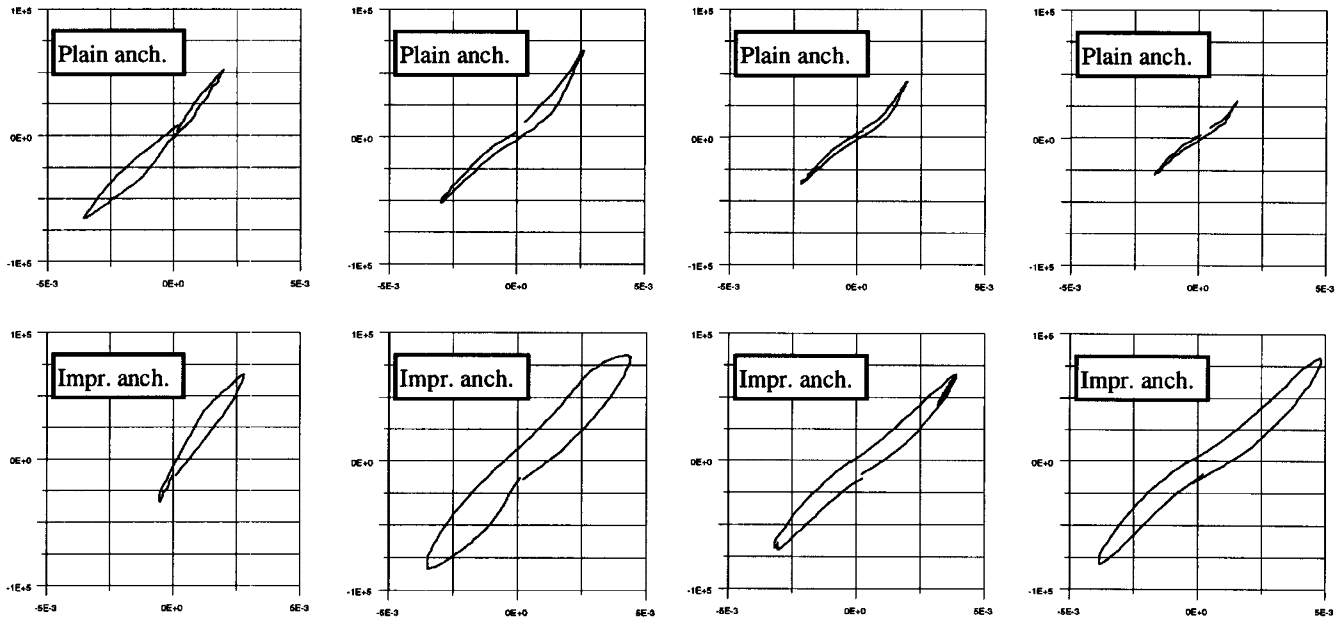


Fig. 8. Irregular bridge central pier - pseudodynamic test with Tolmezzo
Moment (KNm)-Curvature (1/m) cycles from 4 to 6.5 s

Regular Bridge

Two tests were carried for each of the two specimens: preliminary free vibrations and Kobe Kaiyou pseudodynamic. Results are reported in prototype scale (De Sortis and Nuti, 1995). R141A and R141B (same reinforcement and different anchorage) behaved not very differently. In fact pull-out is less important for slender piers than for squat ones, therefore accurate reproduction is not necessary.

In pseudodynamic tests, central piers remained in the elastic field. Figure 9 shows time history on the top of the lateral pier. Numerical preliminary simulation, shown in Fig. 5, was very similar to experimental results. In fact no shear deformations appeared, and the numerical model which only accounts for bending is therefore accurate. Significant amplitude cycles (150 to 300 mm) were from 4 to 16 s. In Fig. 10 the total force-displacement history is reported. Maximum attained ductility was about 4, grater than the ratio of Kobe and EC8 (q=1.7) spectral ordinate at 1.3 s period which is about 3. 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 of rebars. The good performance of the pier essentially depended on transversal reinforcement that gave sufficient concrete confinement and avoided buckling of rebars.

CONCLUSIONS

EC8.2 provides 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.

Accurate scaling of anchorage in the foundation is of large importance to obtain meaningful results in tests, especially in squat piers.

Shear deformation represents an important contribution to the total and strongly reduces the stiffness of squat piers.

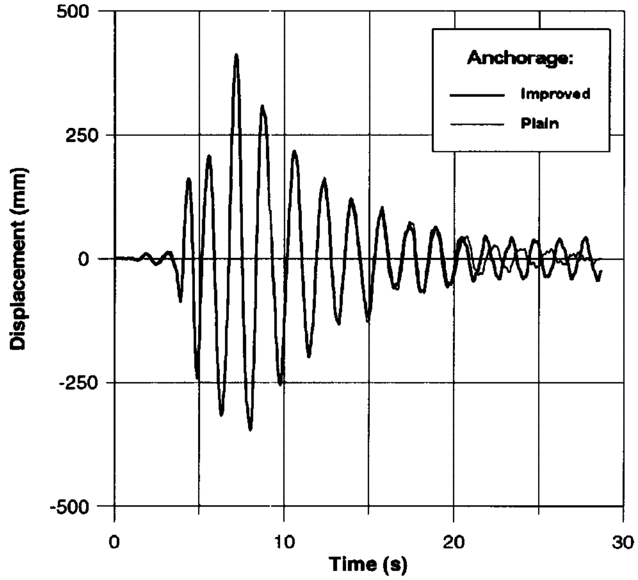


Fig. 9. Regular bridge central pier - pseudodynamic test with Kobe - displ. time history

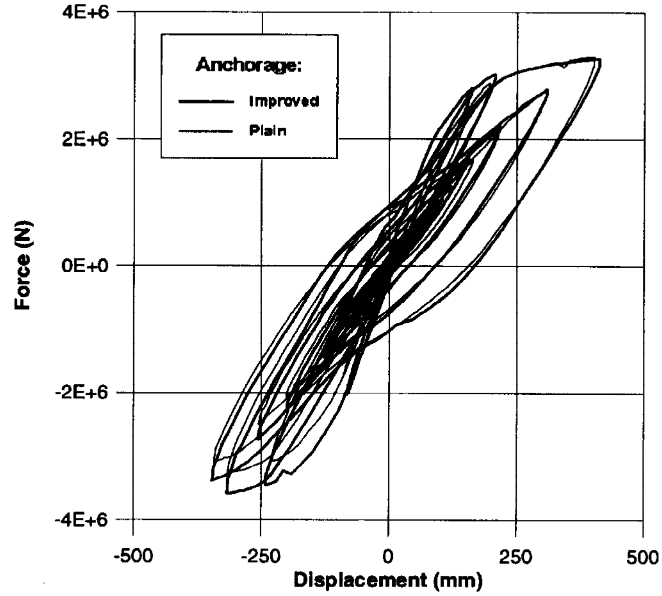


Fig. 10. Regular bridge central pier- pseudodynamic test with Kobe - Force-Displacement cycles

REFERENCES

Buckingham E. (1914). On physically similar systems; illustrations of the use of dimensional equations, *Phys. Review*, **4**, 345-376.

Calvi, G.M., E.C. Carvalho and P.E. Pinto (1994). Irregular Bridges Designed According to Eurocode 8. Numerical and Experimental Verifications. In: *Proc. 2nd Inter. Workshop on Seismic Design and Retrofit of r.c. bridges,* Queenstown, New Zealand.

Colangelo, F. and A. De Sortis (1994). *Esperimenti Pseudodinamici: Studio Preliminare e Realizzazione su Sistemi ad un grado di libertà.* Rapporto n. 1/94, Dip. Ing. Str. Acque e Terr., Univ. dell'Aquila, Italia.

De Sortis, A., G. Monti and C. Nuti (1994). Problemi di scala nella sperimentazione pseudodinamica di pile da ponte in c.a. In: *Atti del Worksop su Danneggiamento, Prove Cicliche e Pseudodinamica*, Dip. Analisi e Prog. Str., Università "Federico II", Napoli, Italia.

De Sortis, A. (1995). *Sperimentazione Pseudodinamica e Modellazione in Scala Ridotta del Comportamento dei Ponti in Calcestruzzo Armato Soggetti ad Azioni Sismiche.* Tesi di dottorato di ricerca, Dip. Ing. Str. e Geot., Università "La Sapienza", Roma, Italia.

De Sortis, A., G. Monti and C. Nuti (1995). Comportamento sismico di ponti progettati secondo Eurocodice e sottoposti a prove pseudodinamiche in scala ridotta. In: *Atti del 7° Conv. Naz. Ing. Sismica*, Siena, Italia

De Sortis, A. and C. Nuti (1995). Prova pseudodinamica di un ponte progettato con EC8.2 e sottoposto al terremoto di Kobe del 1995. In: *Atti del 7° Conv. Naz. Ing. Sismica*, Siena, Italia.

Eurocode 2, Design of Concrete Structures, Part 1: General Rules and Rules for Buildings (1991).

Eurocode 8, Structures in Seismic Zones, Design, Part 1: General Part and Buildings (1993).

Eurocode 8, Structures in Seismic Zones, Design, Part 2: Bridges (1993).

Krawinkler, H. and P.D. Moncarz, Similitude requirements for dynamic models (1982). In: *Dynamical Modeling of Concrete Structures*, Ch. SP73-1, A.C.I., U.S.A.

Monti, G. (1994). *Modellazione agli Elementi Finiti di Nodi Trave-Pilastro in Cemento Armato con Scorrimento delle Barre di Armatura.* Tesi di dottorato di ricerca, Dip. Ing. Str. e Geot., Università "La Sapienza", Roma, Italia.

Nuti, C. and G. Monti (1994). Seismic Analysis of Bridges, a state of the art. In: *Proc. of 10th Eur. Conf. on Earth. Eng.*, Vienna, Austria.

Shing, P.B., M.T. Vannan and E. Cater (1991). Implicit time integration for pseudodynamic tests. *Earth. Eng. Str. Dyn.*, **20**, 551-576.