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PSEUDO-DYNAMIC TESTING OF LARGE-SCALE R/C BRIDGES IN ELSA

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ABSTRACT

Six large-scale (1:2.5) bridge specimens have been constructed and are being tested pseudodynamically at the ELSA laboratory. The tests are being conducted in the framework of the activities of the integrated European programme of pre-normative research in support of Eurocode 8 involving eighteen research institutions from the European Union. The experimental programme is intended to provide background and improvement to the analysis and design methods. In particular, calibration and improvement of numerical models are sought. The first part of the programme (testing of four bridges) has been devoted to exploring different design solutions aimed at obtaining a more homogeneous ductility demand in irregular bridges. A test with isolation/dissipation devices to check the possibility to obtain a more homogenous ductility demand is being prepared. A test in which asynchronous base-motion will be simulated at the base of the piers is also foreseen. The series of pseudodynamic tests being conducted for the bridges are described. The results obtained for the first part of the testing program (regular and irregular bridges with different reinforcement arrangements) as well as for a cyclic test of a squat pier are presented and discussed.

KEYWORDS

Reinforced concrete bridges, Pseudodynamic testing, Pseudodynamic testing with substructuring, Irregular bridges, Bridge earthquake performance, Large-scale bridge tests.

INTRODUCTION

The Eurocodes -the European provisional standards for constructions- are being prepared by the European Committee for Standardization (CEN, Technical Committee No 250) under mandate of the European Commission. The Eurocodes are being released by the EC as provisional norms (ENV). After a period of three years, during which comments are expected, the Eurocodes will be accepted as European norms (EV). The process of approval of each Eurocode has reached different stages. For the case of Eurocode 8 (EC8), (Eurocode N. 8, 1994a,b), the code relevant to the design of structures in seismic areas, priority needs for supporting research have been identified.

A scientific network has been set up to accomplish a Pre-normative Research Programme in support of EC8 (PREC8), under the Human Capital and Mobility programme of the European Commission. The network groups 18 research organisations in the European Union. The identified priority topics are: *Reinforced concrete frames and walls, Infilled frames, Bridges, Foundations and retaining walls.*

The research programme of the Bridge Group includes the following main tasks (Calvi and Pinto, 1994): *Definition of regularity*, *Definition of design procedures for irregular bridges*, *Seismic isolation devices*, *Asynchronous base motion* and *Soil-structure interaction and second order effects*. The experimental part of the programme includes shaking table tests to be performed in Lisbon (LNEC) and Bergamo (ISMES) and pseudodynamic tests of six bridge models being performed in Ispra (JRC-ELSA) object of this paper. Due to the short length of the paper, only the main results are included. Details of the testing campaign and comprehensive analysis of the test results can be found elsewhere (Pinto *et al.*, 1995b; Negro and Pinto, 1995; Pinto and Negro, 1995).

THE LARGE-SCALE TESTING PROGRAM

The testing program at ELSA includes a preliminary cyclic test of one squat pier, and pseudodynamic tests on six complete bridges. The bridges have three piers with height ranging from 7 to 21 meters and identical cross sections (rectangular hollow section with 400 mm thickness) and a continuous deck (200m long). The scheme of the full-scale bridges is shown in Fig. 1 and Table 1 includes the geometric and reinforcement characteristics of the piers

Table 1. Bridges selected to be tested pseudodynamically

Bridge Identification Label	Pier 1		Pier 2		Pier 3	
	Full-size height [m]	Steel [%]	Full-size height [m]	Steel [%]	Full-size height [m]	Steel [%]
B232	14.0	1.15	21.0	0.62	14.0	1.15
B213A	14.0	0.50	7.0	0.92	21.0	0.50
B213B	14.0	0.50	7.0	1.69	21.0	0.50
B213C	14.0	1.15	7.0	0.50	21.0	1.15
B213A I/D	14.0	0.50	7.0	0.92	21.0	0.50
B213A N/S	14.0	0.50	7.0	0.92	21.0	0.50

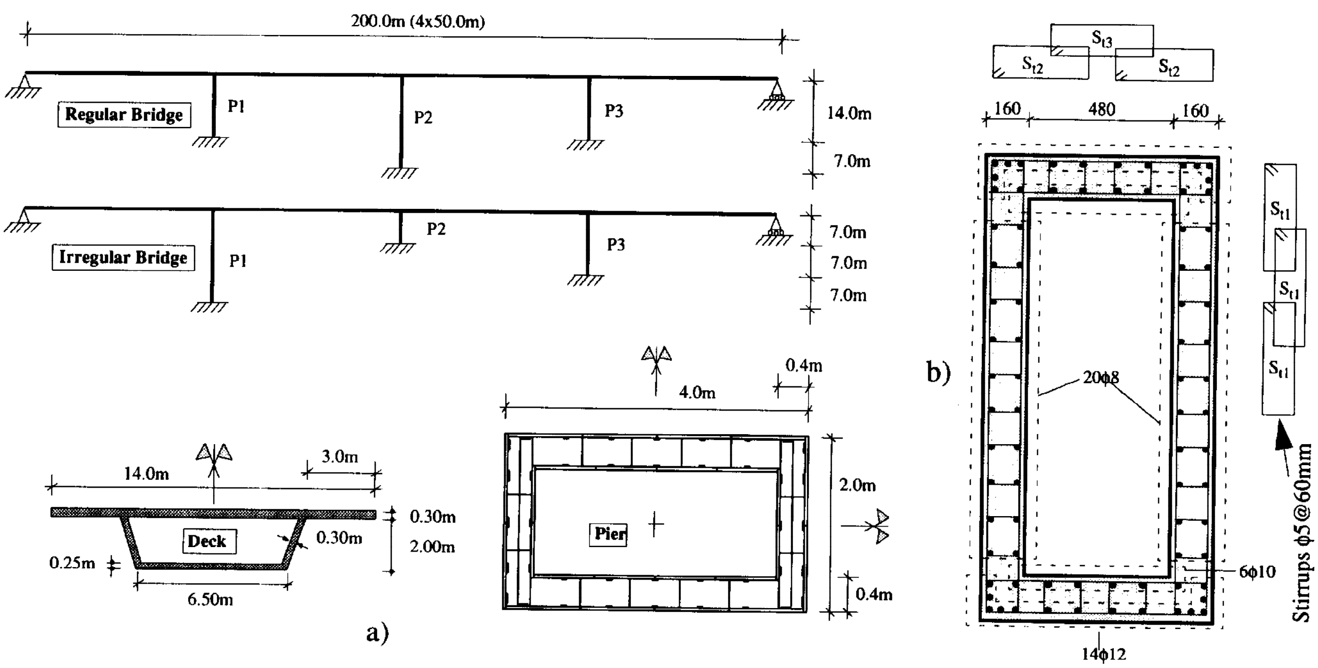

Fig. 1. Bridges being tested pseudodynamically: a) Full-scale scheme b) Typical reinforcement lay-out of a scaled pier (central pier of B213A).

Two different configurations were adopted, having in mind to test the structures in the direction perpendicular to the bridge axis. One represents a typical regular bridge. The regularity is intended in the sense that the fundamental deflected configuration of the structure is similar to the one of the deck alone. The other configuration, the one with the shortest pier in the middle, is regarded as highly irregular, since the stiffness of the deck induces higher forces in the internal stiffer pier.

The letters A, B and C in the label refer to the different design approaches. The label B213A stands for standard design according to EC8, part 2. The bridge B213B corresponds to the most straightforward design solution to the high ductility demands expected for the central pier, i.e. the central pier has been made stronger. In bridge B213C the taller piers have been made stronger instead, in an attempt to relieve the central pier from high forces. The other two bridge specimens (B213A I/D and B213A N/S) are identical to the specimen B213A, but will be tested with seismic isolation devices and with asynchronous base motion respectively.

The bridges are representative of typical multi-span continuous-deck motorway bridges. The deck is a hollow-core prestressed concrete girder, with a full-scale width of 14 m. The deck ends at the abutments with shear-keys to allow end rotations. The bridges have four identical spans of 50 m. The piers have constant rectangular hollow-core cross-section as shown in Fig. 2.

TEST SETUP AND INSTRUMENTATION

The pseudodynamic testing of the bridges have been performed with the substructuring technique which implementation details at ELSA are given in (Buchet and Pegon, 1994). As shown in Fig. 2, the actuator controllers and the computer handling the substructured part were placed in the left side of the figure and the piers are aligned in front of the reaction wall, with the shorter one in the middle. The plinths of the piers are

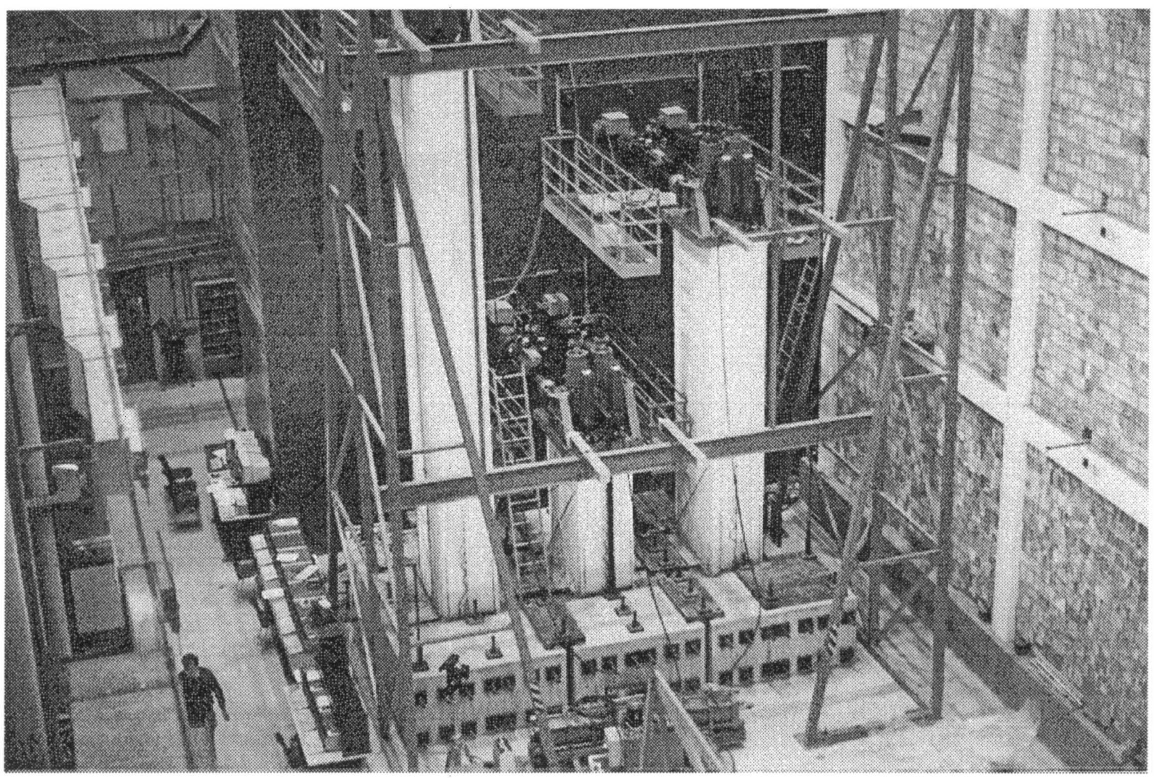

Fig. 2. Test setup - General view.

rigidly attached to the strong floor by steel bars passing through the floor itself. A stiff steel cap is connected with bolts and epoxy resin to the tip of the piers. The cap is used to apply the vertical load corresponding to the weight of the deck as well as to apply the horizontal loads which are required to impose the computed displacements.

The axial force (1700 kN) is applied by means of four actuators, placed inside the hollow core pier and connected to the cap and to gauged steel bars embedded in the pier plinth. The vertical force is imposed at the beginning of the test and remains practically constant, thanks to the hydraulic accumulators placed to compensate the effects of the horizontal displacement.

Two double-acting servo-hydraulic actuators (three for the central pier in the test B213B), capable of a load of 1.0 MN each, are used to apply the horizontal displacements. They are connected by spherical joints to the steel cap at one side and to a steel plate attached to the reaction wall at the opposite side. The imposed displacements are measured with respect to two independent steel frames using digital optical transducers.

The instrumentation of the piers included both active measurements, those which are related to the pseudodynamic control algorithm, and passive instruments. Passive instruments were grouped in the following sets: *Rotations, Diagonal deformations, Horizontal Displacements, Section Warping.*

CYCLIC TEST

In order to prepare the pseudodynamic testing campaign it was decided to carry out a quasi-static cyclic test on one of the short bridge piers (the one corresponding to the B213A bridge). The imposed displacement history included increasing amplitude cycles up to yielding, followed by three cycles for displacement ductility of 1.5, 3.0 and 6.0. A constant axial load of 1700 kN, corresponding to a normalized axial load (ν=0.1), has been imposed on the top of the specimen by means of four pistons with equal pressure. The measured global hysteresis loops are depicted in Fig. 3.

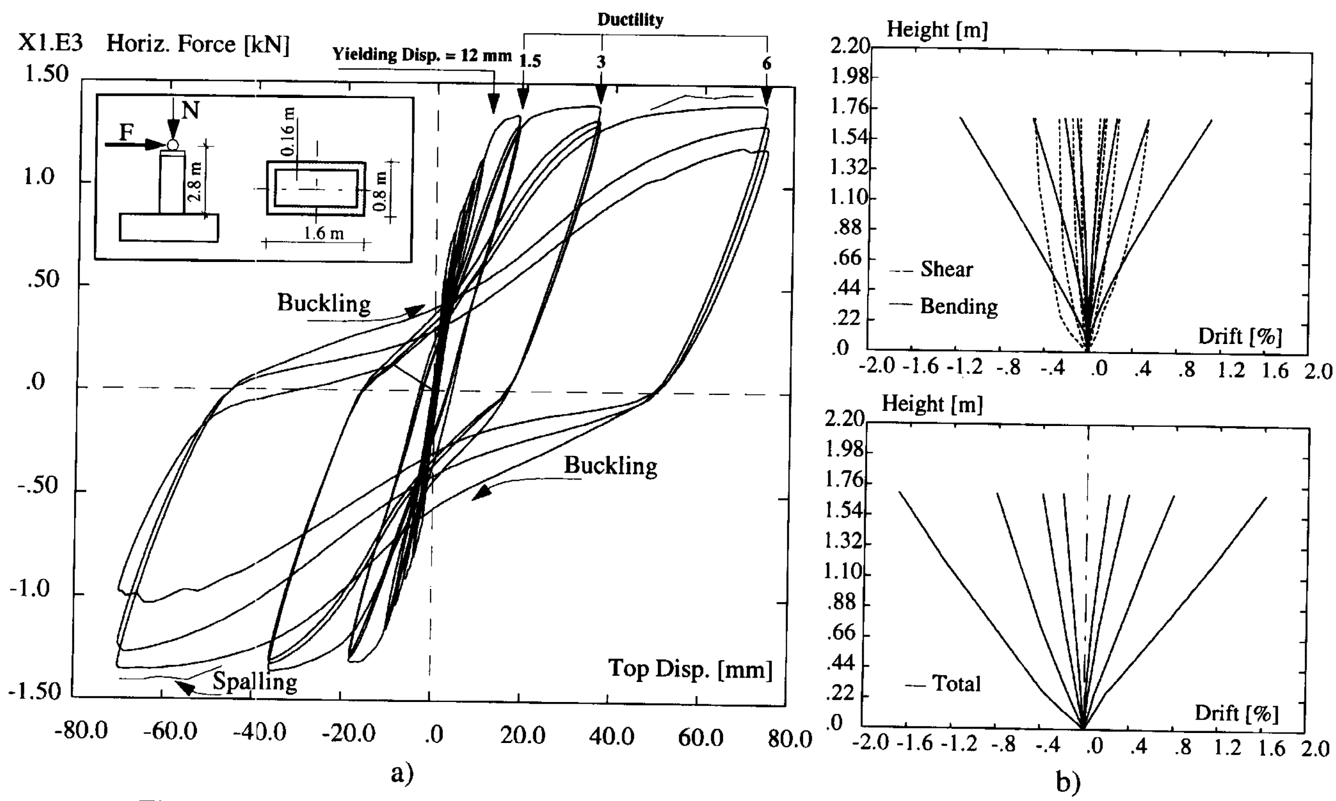

Fig. 3. Results from the cyclic test: a) Force-displacement diagram, b) Bending, shear and total displacement profiles for different ductility levels.

Flexural cracking of the pier started at a displacement of approximately 2 mm, followed by the opening of shear cracks. Yielding took place for a top displacement of about 12 mm. This value was considerably larger than the theoretical value (8 mm), due to the contribution of the deformability of the plinth and the slippage of the bars in the foundation. For the ductility 3.0, increasing diagonal and flexural cracking was observed. From ductility 3.0 to ductility 6.0, spalling of the concrete cover and buckling of rebars at the first reversal took place. The test was stopped after the rupture of the rebars, which experienced large deformations due to buckling in the previous half-cycle. Buckling was also observed at the main diagonal cracks.

The structure exhibited stable and dissipative cycles up to ductility 3.0. Damage was concentrated at the bottom of the pier, without noticeable spread of plasticity. An equivalent plastic hinge length of 260 mm was found which compares well with the values obtained from empirical expressions. The shear deformations were found to be responsible for about 30% of the total top displacement (see Fig. 3). The energy dissipated by shear was, however, small (10~15% of the total). For higher ductility the damage increased, mainly due to buckling of rebars. Failure (reduction of 30% in the load capacity) took place during the third cycle at ductility 6.0. A comprehensive analysis of the results can be found in (Pinto *et al.*, 1995a).

PSEUDODYNAMIC TESTS

Input Motion and Testing Procedure

Pseudodynamic tests have been performed, thus far, on bridges B232, B213A, B213B and B213C. An artificial accelerogram, with a nominal peak acceleration of 0.35 g and which response spectrum fits the EC8 response spectrum for medium soil conditions, was used in the tests (see Fig. 4).

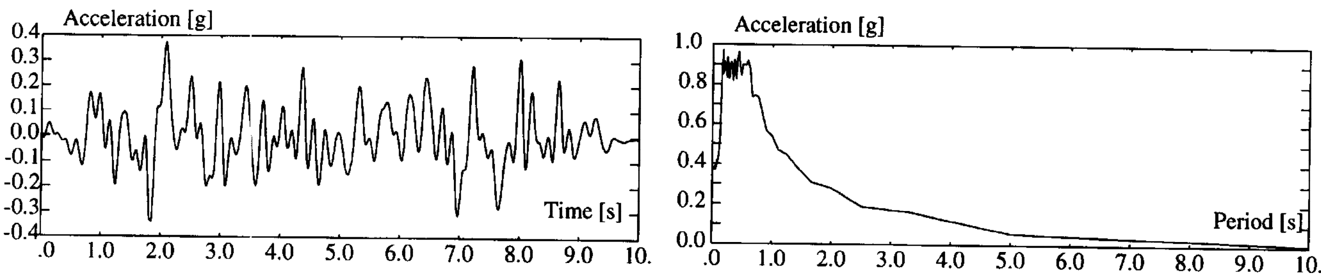

Fig. 4. Artificial accelerogram and corresponding response spectrum (5% damping).

Being the geometrical scale factor, λ=2.5, the mass matrix to be used in the substructured part was scaled by λ^3 and the input motion was accordingly scaled, i.e. the ordinates and the time scale of the accelerogram were respectively multiplied and divided by λ. The original accelerogram of ten seconds became 4 seconds long. Again, a constant value of 1700 kN was applied, to keep the normalized axial load the same as in the real structure (ν=0.1).

No specific testing procedure (type of loading and corresponding intensity) is presently available for dynamic tests of earthquake resistant structures. In principle, this testing procedure is tailored to the general objectives of the tests that must be well clarified. Concerning the PSD testing of the bridges the main objectives are recalled: a) evaluation of the seismic performance of bridges designed in accordance with the present version of EC8, b) comparison of three design solutions for irregular bridges, c) evaluation of safety margins for each design solution.

Thus, it was decided to test the bridges using first the 'design earthquake' as input motion and then with the same accelerogram multiplied by an intensity factor leading to an input motion as close as possible to the one causing failure (high level earthquake). For the set of irregular bridges the intensity of the high level earthquake has been tailored to the weak design solution (bridge B213C). Thus, based on the numerical predictions, the scaling factors adopted for the high level earthquake were 1.2 and 2.0 respectively for the irregular bridges and for the regular one.

Test results and modelling aspects

Typical results from the tests, displacement time histories and global hysteresis loops are given in Fig. 5. Table 2 presents the extreme values of displacement, ductility and damage. The damage index was computed from the Park and Ang model (Park and Ang, 1985).

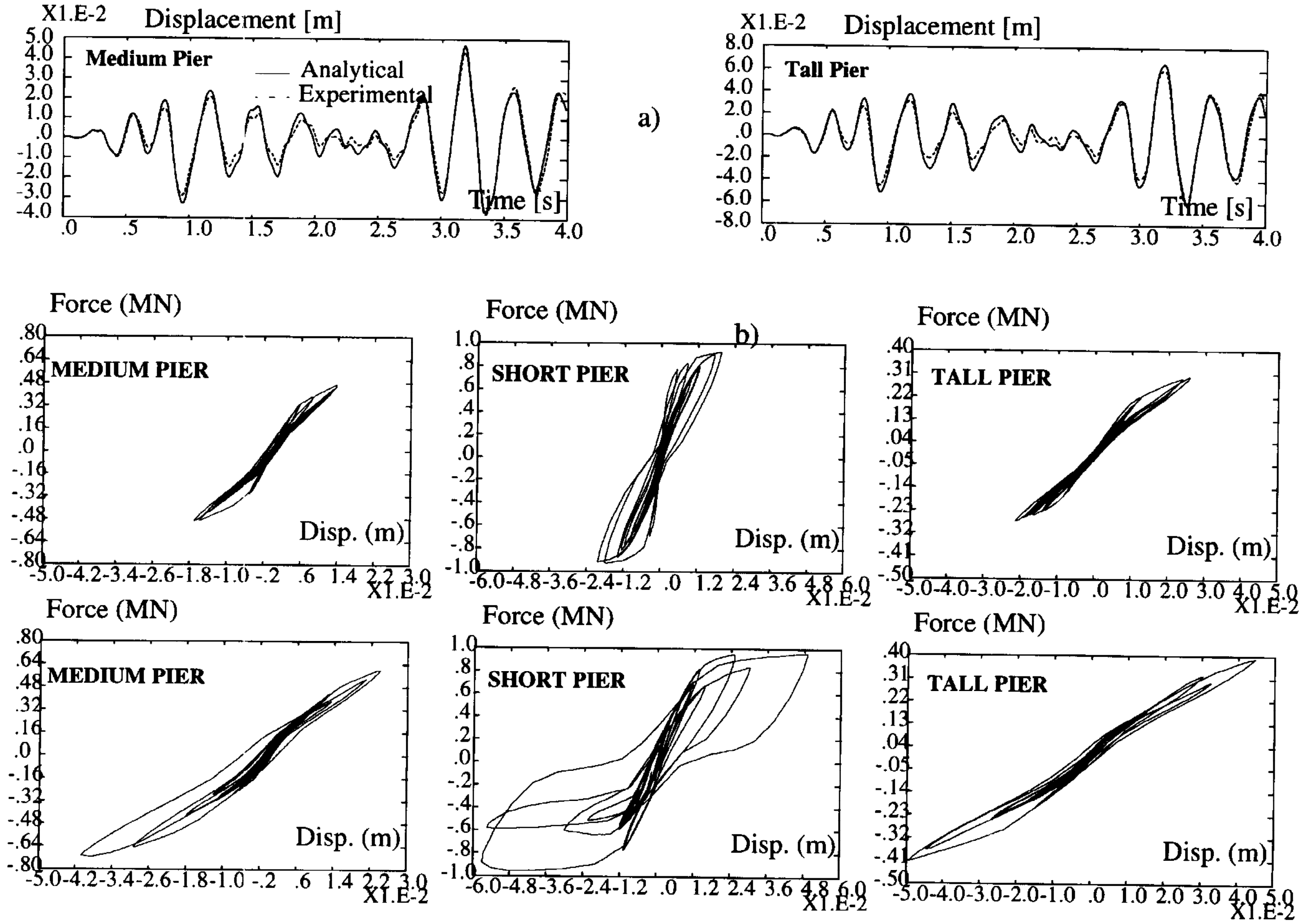

Fig. 5. Typical results from the PSD tests: a) Time histories of the pier top displacement (bridge B232, 1.0xDE) b) Pier force-displacement diagrams of bridge B213C for the two earthquake levels (1.0xDE and 1.2xDE).

Table 2. Seismic response of the bridges for the two earthquake levels

		Design Earthquake			High-Level Earthquake[a]		
BRIDGE	Pier	Top. Disp. [m]	Ductility	Damage Index	Top. Disp. [m]	Ductility	Damage Index
B232	1	0.044	1.4	0.22	0.089	2.8	0.47
	2	0.062	1.9	0.34	0.132	4.1	0.79
B213A	1	0.022	<1	---	(0.029)[b]	---	---
	2	0.024	2.0	0.30	0.052	4.0	0.70
	3	0.028	<1	---	(0.050)	---	---
B213B	1	0.025	<1	---	(0.028)	---	---
	2	0.023	1.5	0.25	0.043	2.5	0.50
	3	0.028	<1	---	(0.048)	---	---
B213C	1	0.017	<1	0.09	0.041	1.3	0.22
	2	0.020	4.2	0.35	0.057	11.9	1.03
	3	0.026	<1	0.09	0.049	<1	0.16

a. 1.2 and 2.0 times the design earthquake respectively for the bridges B213A, B213B and B213C and for bridge B232

b. (.) Values obtained from the numerical model (pier substructured)

The results of the test on bridge B232 comply with the EC8-part 2 assumptions. In fact, dissipation of energy takes place in all piers, and a ductile behaviour could have been assumed. The dissipative mechanism turned out to be stable and efficient, with demands roughly proportional to capacities, and an input signal 2 times larger than the design signal was applied without loss of capacity.

In the case of test B213A the dissipation of energy was limited to the short pier. The ductility demand on the short pier is, however, reasonably small. In the test B213B the short pier, which was stronger than in the previous bridge, attracted much larger forces, but experienced considerably lower ductility demands. Again, energy dissipation took place in the short pier only. The crack pattern was found to be much more spread than in the previous test. In test B213C, the goal to obtain a more regular force distribution was achieved. The distribution of the maximum bending moment among the piers is, in fact, fairly constant. Ductility demands in the short pier are, however, much larger than in test B213A. The advantages of the corresponding design approach are, therefore, debatable.

It is noticed that the maximum values of ductility demand and damage index for the Design Earthquake are quite similar for all bridges (except for the Bridge B213C). However, the irregular bridges, tested with an input motion slightly higher than the design earthquake (1.2 times) and much lower than the intensity used for the regular bridge (2.0 times), have been heavily damaged. Safety against collapse of the irregular bridges is quite low, compared to the safety of the regular bridge. The results confirm that the strategy adopted for the design of the bridge B213B, which corresponds to the use of a lower local behaviour factor, is the 'less worse' and could lead to a suitable design approach for irregular bridges. Alternative solution for these high irregular bridges may lay on the I/D devices which is the object of the next part of the testing campaign in ELSA.

Finally it should be underlined that, after the PSD tests, the bridge piers were tested cyclically until collapse exhibiting a ductile behaviour. This confirms the adequacy of detailing provisions included in EC8 for this kind of structures. Detailed analysis of the pseudodynamic test results can be found in (Pinto *et al.*, 1995b).

Numerical simulations of the pseudodynamic tests have been carried out using different computer programs and models (Calvi and Pinto, 1994). Simplified (Takeda-like) and/or relatively refined (fibre-type) models have been adopted by the partners of the PREC8-Bridge group. The agreement between them and with the test results was in general acceptable thought due to the simplicity of the structures considered . However, some limitations of those modelling have been identified, namely: the steel yielding penetration and base deformability, the contribution of non-linear shear behaviour to the element deformability and the shear flexure interaction for both strength and deformability. The first limitation can have a significant influence on the yield displacement of the taller piers while the second and third play an important role in the case of the central pier of the irregular bridges which aspect ratio is quite low (1.75). The available experimental results are being used for the calibration of improved models taking into account those aspects.

The numerical simulations performed in ELSA used a fibre-type model (Guedes *et al.*, 1994) and the limitations above mentioned were confirmed. In fact, the numerical and the experimental responses of the regular bridge (B232) are in good agreement (see Fig. 5) while the numerical response for the irregular bridge with the central squat pier is unable to follow closely the response obtained from the pseudodynamic tests (Guedes and Pinto, 1995). The base deformability and the yield penetration was taken into account but the non-linear shear behaviour has not been included in the model. An improved strut-and-tie model for non-linear shear behaviour is being developed and the first results are already quite encouraging.

CONCLUSIONS

A testing programme is being carried out as a part of the activities of the Bridge Group of the PREC8 Network. The scope of the tests is to study the implications of irregularity with the q-factor design approach. Up to now, four bridges have been tested pseudodynamically, namely: one regular and three highly irregular bridges designed according to different design philosophies. The test results confirm that safety against collapse of the irregular bridges is quite low compared with the safety of the regular bridge both designed with the same procedure. The bridges testing campaign will be completed with two additional tests -seismic isolation and asynchronous base motion- which are now being prepared. Finally, it is highlighted the potential of the pseudodynamic test method with substructuring used in this test campaign. The on-line analytical simulation of the bridge deck and of some piers have shown the usefulness of the method as implemented in ELSA for the seismic testing of large scale bridges. These PSD tests are the first large scale experiments performed in the world using the substructuring technique.

ACKNOWLEDGEMENTS

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