The European Laboratory for Structural Assessment(ELSA) and its Role for the Validation of European Seismic Codes

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Abstract

The need for European cooperation in earthquake engineering has been recognized since many years, mostly due to the similarities of seismic actions and construction technologies, and the high sophistication and cost of the facilities needed for the experimental study of the problem. Moreover, the need for European collaboration has become urgent with the activation of the process of drafting of the Eurocodes, the harmonized rules for the design of structures.

Even though these reasons stimulated some European collaboration in earthquake engineering research, the construction and operation of the European Laboratory for Structural Assessment (ELSA) has provided much impetus for mobilizing European cooperation in many fields related to seismic research. In particular, the ELSA Laboratory has been used to provide the necessary experimental data for the validation and refinement of Eurocode 8, the European norm relevant to earthquake-resistant design of structures.

This work started immediately after the construction of the Laboratory, with the first seismic test on a complete building designed in accordance with the draft version of Eurocode 8. The pre-normative work has been continued, producing experimental and analytical results for the verification and possible improvement of the Code. Tests were conducted to study problems which were not adequately covered by the code, such as irregular bridges or masonry infilled frames. Other tests were aimed at providing data for new construction materials (high-strength reinforcing steel, composite construction).

Even though Eurocode 8 is presently bound to its conversion from Provisional European Norm (ENV) into a European Norm (EN), the pre-normative work has not been discontinued, since ELSA has become active in the research for the new generation of design codes.

In the paper, the features of the ELSA Laboratory are briefly explained. In particular, a description of the pseudodynamic test method is given. Some of the large-scale seismic tests conducted at ELSA on civil engineering structures are described to highlight the importance of the results in the process of validation and improvement of Eurocode 8.

1. Introduction

There are no doubts that the safety of structures against earthquake is a worldwide problem. The European scientific community has a long-lasting tradition in cooperation in this field, mainly justified by the similarities in seismological features and construction practices, and the high cost of the specialized experimental facilities.

The work needed for the drafting of the Eurocode 8, the European norm for structural design in seismic-prone areas, has provided much impetus to the European cooperation.

It is also true to say that an initiative of the Joint Research Centre (JRC) of the European Commission has contributed to mobilizing multinational cooperation in this field. This initiative has been the launch of the European Laboratory for Structural Assessment (ELSA). ELSA is the name of a large reaction wall facility which represents a unique tool in Europe for large and full-scale testing of structures under static, cyclic and pseudodynamic loading. The decision about the need for this facility included an enquiry through the member states of the European Community and the setting up of an ad hoc working group. As a result of the positive indications obtained as to the usefulness of such large installation, the financial resources were made available, and the facility became a focal point of the European collaboration in this field. Most of the activities taking place at ELSA are Community-wide research projects involving the available expertise, and much of the work is aimed at providing the necessary data for the verification and improvement of Eurocode 8.

2. The Eurocode 8

The Eurocode 8 (EC8) is one of the nine European codes for structural design, and deals with the additional provisions for earthquake safety. After the initial drafting, the European Commission transferred to the European Committee for Standardization (CEN) the work of further development and issue of the Eurocodes.

As the other Eurocodes, EC8 is a performance based code. In this sense, the code states explicitly its objectives as to ensure in probabilistic terms that in the event of an earthquake: a) human life is protected; b) damage is limited; c) vital structures remain operational. These three objectives are converted into two basic requirements, one dealing with the maximum expected earthquake (no-collapse requirement), the other with those seismic actions with return periods comparable with the expected lifetime of the structure (damage limitation requirement). In connection with the two basic requirements, the performance of the structure is defined with reference to the two usual limit states, Ultimate and Serviceability limit states. The Ultimate limit state is associated to the full exploitation of the available ductility, which depends on the choices made by the designer. On the other hand, the Serviceability limit state is related to the damage of non-structural elements and the corresponding economic consequences.

Considering the different importance of the seismic risk in the countries of the Union, EC8 just provides the design rules, leaving with the National Authorities the definition of the coefficients describing the seismic hazard.

The design rules prescribed by the code are based on a number of fundamental concepts, which required, and to some extent still require, an accurate definition and calibration. The fundamental concepts are those requiring experimental confirmation, and are here described in some details.

2.1 Design spectrum

Ductility, i.e the ability of sustaining deformations beyond the elastic limit without loss of stability and bearing capacity, is a fundamental characteristic of earthquake resistant structures. Even though EC8 explicitly refers to the need of relying on the ductile nonlinear behaviour, nonlinear analyses are not required. In its basic formulation, the design process relies on an equivalent elastic analysis, based on the definition of the design spectrum. The design spectrum is obtained from the elastic spectrum by dividing the spectral values by the force reduction factor q. The factor q is defined by the type of structure, ductility class and regularity, and ultimately dictates the magnitude of the seismic forces to be used in the design. Not surprisingly, the calibration of the q values to be used for each structure is the matter most badly needing experimental confirmation.

2.2 Ductility classes

The definition of different ductility classes is possibly the most innovative feature of EC8. The main concepts assumed in the drafting of EC8 is that different combinations of strength and ductility can yield the same required safety level. The code allows the designer to choose among three ductility classes (Low, Medium and High), expected to correspond to structures having the same safety against earthquakes.

The choice of the ductility class corresponds to the definition, by means of the q factor, of the forces to be adopted in the design. The larger the ductility class is, the smaller the seismic forces are, as a consequence of the larger available ductility. On the other hand, the larger the ductility class, the more stringent the detailing rules for the enhancement of ductility.

The matter can be explained with reference to reinforced concrete structures. Ductility class Low structures correspond to the application of Eurocode 2 (the general code for this material), with no or very few provisions for the seismic aspects. Ductility class Medium structures must comply with a set of specific detailing rules to ensure that the behaviour is sufficiently ductile. Ductility class High structures must respect more demanding detailing rules, to ensure that a stable and efficient dissipative mechanism is developed when the structure enters into the nonlinear regime.

The calibration of the clauses and coefficients such that the required safety level is obtained independently from the choice of the ductility class is a unprecedented technical problem. This problem can be solved only in the case that the necessary experimental data are available.

2.3 Capacity design

Capacity design refers to the procedures to be adopted in design for forcing the structure to develop a certain plastic mechanism. These procedures are obviously simplified, and typically defined in terms of a check of the strength of the members connecting onto a joint. To verify whether a stable and efficient mechanism does indeed take place as a consequence of the application of these simplified rules is beyond the possibilities of the designer. Code calibration exercises must therefore include these verifications, and the conclusions can only be confirmed by means of experiments on complete structures.

2.4 Regularity

Whereas regularity is a desirable feature in any structure, for the case of structures to be built in seismic-prone areas irregularity can prevent the structure from behaving in a ductile fashion. For this reason, the level of irregularity in plan and in elevation must be classified, and the corresponding limitations in ductility must be enforced. This is a cumbersome problem, and the need for experimental confirmation is evident.

2.5 Conceptual design

The equivalent elastic procedures in EC8, in particular the definitions of the design spectrum and of the q factor, are based on single-degree-of-freedom concepts. Conceptual design is the only means to ensure that this basic assumption is respected. A building structure can globally behave as a single-degree-of-freedom structure if a simple conceptual design scheme was adopted in design. This may not be appropriate in the case of very irregular structures, or structures with nonductile elements. For the case of bridges, for instance, the application of the equivalent elastic procedure to simple conceptual design schemes may yield contradictory conclusions, as it will be shown in the following sections.

2.6 The EC8 for the next generation

Whereas the process of adoption of EC8 is possibly reaching its end, the code drafting process has not been discontinued. Based on the evident consideration that deformations can lead a structure to failure, not forces, there is much research work aiming at Displacement-Based (as opposed to the presently used Force-Based) simplified design procedures. There are no doubts that this research work, which includes the necessary experimental confirmation, will be finally converted into new code provisions.

3. Experimental techniques

The techniques that are available for large-scale testing of structures are briefly presented. The attention is focused on testing of large structures: design assumptions can only be verified by means of tests on mock-ups representative of the whole structure. The earthquake behaviour of structures is possibly the area in which the understanding of the behaviour and the reliability of the computer models is much less satisfactory.

In earthquake engineering, simplified equivalent-elastic procedures are almost exclusively used. Conceptual design can be applied to the simplified procedure, however, it may lead to misleading conclusions in the case that the basic assumptions which justify the simplified procedures are not satisfied. For instance, the equivalent-elastic design procedures assume that a stable and regular dissipative mechanism is developed when the structure enters into the nonlinear regime: structural testing is the most powerful means to verify this fundamental assumption.

A distinction can be made between tests which are conducted to improve the understanding of the structural behaviour -we may call them basic tests-, and tests intended for the verification of the global response of a complete structure -proof tests. Basic tests are generally conducted on small elements or subassemblages, to capture the progression of damage and to obtain the information needed to model the behaviour of the element in a computer programme. Proof tests are generally needed to verify the adequacy of a complete structure, or of a particular construction method (for instance with new materials, construction processes, connections), or of a design method. Whereas basic tests may be used to address new design problems (to derive information about the mechanical behaviour of new materials, or innovative construction techniques), proof tests may represent the main source of verification for the design codes.

The main experimental techniques which are available for proof-testing of structures are briefly presented. Other experimental methods, such as ambient monitoring or dynamic excitation tests, are not suitable for studying the behaviour of complex structures in the nonlinear regime, therefore will not be considered.

3.1 Shaking table tests

Shaking tables are composed of a rigid platform which is moved along one or more axes by a number of hydraulic actuators. They are classified according to the number of degrees of freedom which are controlled. The most sophisticated shaking tables can reproduce a motion defined by six degrees of freedom. The larger the number of controlled degrees of freedom, the more sophisticated the control system becomes, due to the need to compensate the coupling between the individual degrees of freedom.

Shaking tables are the best means to subject a model structure to any specified base motion. Shaking tables can be commanded to reproduce realistic earthquake motions (either real or artificial) with fairly accurate control. The experiments can be performed in the controlled environment of the laboratory, and can be repeated with increased intensity to achieve the complete failure of the structural system. The specimen can be equipped with any sort of instruments, to record all the mechanical parameters which are needed to reproduce the test results. Shaking table tests are commonly used both for the qualification of systems and equipment and for research and development purposes [1].

The main limitation suffered by shaking tables remains in the size or weight capacity. Since, in general, full size specimens of representative structures cannot be accommodated in the shaking table, specimens must be reduced according to scale laws to meet the capacity requirements. Scaling may result in poor representation of the behaviour of materials and connections. In reinforced concrete structures, scaling may result in changes of the mechanical properties of the rebars, and the bond mechanism would be distorted. In testing masonry structures, it would be impossible to scale the size of the bricks and the thickness of the mortar layers without modifying the global mechanical properties of the composite material. Even in the case of steel structures, scaling may result in the substantially different behaviour of bolted or welded connections. It is clear that the use of the shaking table is an accurate experimental technique when applied to small or light specimens, while it suffers severe limitations whenever the structure has to be scaled to be accommodated by the table.

3.2 Quasi-static tests

Quasi-static tests are performed by imposing quasi-statically predefined displacement or force histories on the specimen [2]. This is generally done for single elements or simple subassemblages, and the test is performed by controlling the displacement, due to the larger uncertainties in predicting the restoring forces in the nonlinear regime.

This testing method has apparently nothing in common with dynamic testing of the structures. However, by imposing cyclic displacements of increasing amplitude and by measuring the corresponding restoring force, one can accurately calibrate a computer model which can be applied in predicting the dynamic behaviour of the structure when subjected to any dynamic input.

Quasi-static tests are simple, relatively inexpensive, and do not require special apparatus. The main limitation arises from the fact that the displacement history has to be defined before the test. The prescribed cyclic displacement history may not cover the range of displacements which the structure would undergo during dynamic action. Moreover, the shape of the loops may affect the behaviour of the structure if the damage depends on the dissipated energy. It would be much more desirable to impose on the specimen the displacement history which the structure would undergo during dynamic action. This is the main idea of the pseudodynamic testing technique. A pseudodynamic test is a quasi-static test in which the displacements to be imposed on the specimen are derived on-line by a computer which solves the dynamic equilibrium equations by making use of the measured restoring forces. The basis of the pseudodynamic testing method will be briefly presented in the following section.

3.3 Pseudodynamic tests

Initially developed and implemented in Japan [3], the pseudodynamic (PsD) test method is a hybrid testing technique [4]. It combines on-line computer simulation of the dynamic aspects of the problem with experimental information about the structure, acquired quasi-statically, to provide realistic dynamic response histories, even for the nonlinear behaviour of severely damaged structures. It uses in principle the same equipment as conventional quasi-static tests, in which prescribed histories of load or displacement are imposed on specimen structures by means of hydraulic actuators. However, the fundamental difference from such tests, from which the method's name is derived, lies in the use of the computer simulation of the dynamic inertia loading. This allows the time-scale of the test to be extended, thus offering a series of advantages which will be illustrated in the following paragraphs.

The PsD method moves away from the analytical techniques used in structural dynamics. In structural dynamics the structural system is considered as an assemblage of elements interconnected at a finite number of nodes. By using a finite element approximation of the continuous problem, a system of second-order ordinary differential equations is obtained:

$$\boldsymbol{M}\boldsymbol{a}(t) + \boldsymbol{C}\boldsymbol{v}(t) + \boldsymbol{r}(t) = \boldsymbol{f}(t), \tag{1}$$

where $\boldsymbol{M}$ and $\boldsymbol{C}$ are the structural mass and damping matrices, $\boldsymbol{a}(t)$ and (t) the acceleration and velocity vectors, $\boldsymbol{r}(t)$ the structural restoring force vector and $\boldsymbol{f}(t)$ the external force vector applied to the system.

The discretization is extended to the time domain, leading to a step-by-step integration technique. In solving the time-discretized equations of equilibrium, one needs to compute the restoring force vector $\boldsymbol{r}(t)$, and this is done by appropriate subroutines which represent the structural behaviour of each element. The lack of adequately refined models for the structural behaviour of the elements is the major source of uncertainty in the numerical analysis of structures, and to some extent, the main reason for the need for testing techniques. The main idea for the PsD method rests on this uncertainty: the PsD method is the numerical solution of the discretized equation of motions, in which the main source of uncertainty - the restoring force vector $\boldsymbol{r}(t)$ - is not evaluated numerically, but directly measured on the structure at certain controlled locations.

Even though Equations (1) can be expressed in terms of any number of degrees of freedom (dofs), the number of the points of the structure to be controlled during the test is necessarily limited (being, in general, equal to the number of actuators attached to the structure). As a consequence, Equations (1) must be reduced - through standard condensation techniques - to the degrees of freedom to be controlled by the test procedure.

It is evident that the PsD testing techniques can be applied most simply to those structures for which the mass can be assumed to be concentrated in certain locations. This is the case for the framed buildings for instance, for which most of the mass is concentrated in the floor slabs, and the slabs are generally stiff enough for their in-plane deformability to be ignored. In this case, the degrees of freedom to be controlled are the horizontal displacements in the floor slabs.

The procedure can be easily illustrated for the case of a building structure. Inertia and viscous damping forces are modelled analytically - a relatively straightforward matter compared to the nonlinear structural restoring forces, which are measured experimentally because of the virtual impossibility of modelling them accurately. The process automatically accounts for the hysteretic damping, due to inelastic deformation and damage to the structural materials, which is usually the major source of energy dissipation. To simulate the earthquake response of a structure, a record of an actual or artificially-generated earthquake ground acceleration history is given as input data to the computer running the PsD algorithm. The horizontal displacements in the building floors (where the mass of the structure can be considered to be concentrated) are calculated for a small time-step using a suitable time integration algorithm. These displacements are then applied to the test structure by servo-controlled hydraulic actuators fixed to the reaction wall. Load cells on the actuators measure the forces necessary to achieve the required storey displacements and these structural restoring forces are returned to the computer for use in the next time-step calculation. Since the inertia forces are modelled numerically, there is no need to perform the test on the real time-scale, and this allows very large models of structures to be tested with only a relatively modest hydraulic power requirement. In this sense, PsD tests are complementary to the more conventional shaking-table tests which are done in real time, but are restricted to components or small-scale models of large structures.

4. Tests on a four-storey R/C building

A full-scale pseudodynamic test was conducted on a four-storey R/C frame [5] designed according to the draft version of EC8. The structure, designed as a ductility class High frame, was subjected to an earthquake action representative of the maximum earthquake for which it had been designed. Afterwards, the building was cyclically driven to failure, to measure its ultimate force and ductility capacities. The testing activity resulted in a number of important conclusions about the global behaviour of the structure.

4.1 The importance of the properties of steel in R/C structures

The first, particularly important conclusion was about the consequences of the adoption of the new type of steel which is becoming dominant in Europe, the steel B500 Tempcore. This steel, thanks to a rather simple and inexpensive thermal process (Tempcore), offers a significantly larger strength (500 MPa characteristic strength), in change for a relatively small reduction in ductility and hardening ratio. This steel did not fulfil the requirements of the 1988 version of Eurocode 8 for ductility class High structures both in terms of strain of failure (lower than the 12% limit prescribed) and failure-to-yield tensile strength ratio (less than the lower limit fixed by the code, even for ductility class Low structures). A pre-mature bond failure of the rebars passing through the internal joints took place, thus greatly reducing the energy dissipation capacity of the structure (Figure 1), and the reason was found in the mechanical properties of the reinforcing steel [5]. The plastic mechanism which was activated did not rely on the formation of plastic hinges at the end of the beams, instead, the mechanism was obtained by the slippage of the rebars at the joint. The development of a stable and efficient (i.e., capable of dissipating a large amount of energy) mechanism is the basis of any seismic design. Based on the findings of the experimental activity, a comprehensive study about the characteristics of steel was initiated [6], and further experimental programmes were launched to assess the adequacy of the Tempcore steel for seismic resistant constructions [7].

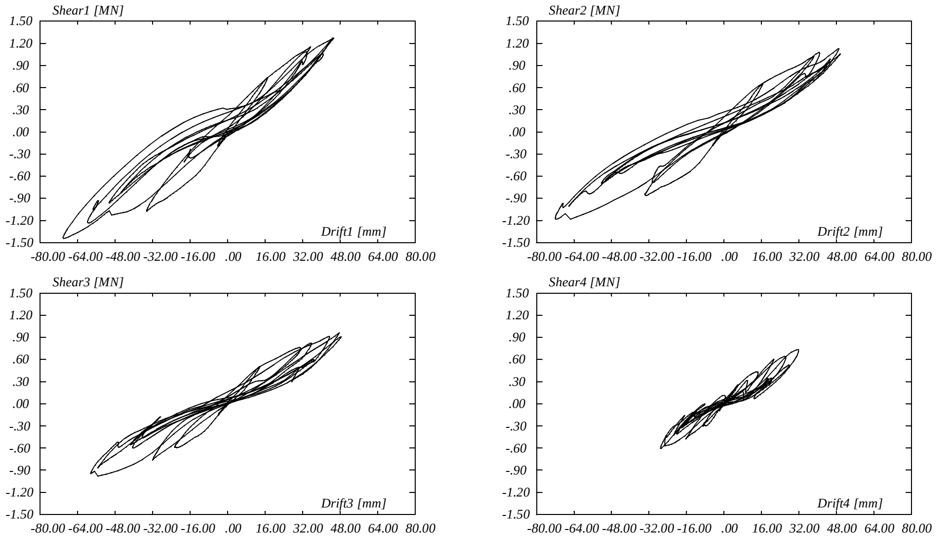

Figure 1: Hysteresis loops measured for the four-storey R/C frame: the area of the loops is particularly reduced as a consequence of the slippage of rebars in the joints.

4.2 Beams and colums. How about joints?

A frame is generally defined as an assembly (planar or three-dimensional) of flexural elements (beams and colums). The joints are the elements connecting beams and columns, however, since they are represented by dimensionless points in the computer model, not much care is typically put in their detailing. On the other hand, design of the joints is required by seismic codes. In such case, the joints of frame structures are designed to remain elastic, so to force the energy dissipation to take place at the end of the beams (i.e., to obtain the so-called strong-column, weak-beam mechanism, which is the sought stable and efficient mechanism).

Another important outcome of the tests performed on the R/C frame, was the failure of the joints connecting beams and columns. The failure of the joints took place at large deformation level, yet before the failure of beams and columns themselves [8]. Even though the structure behaved satisfactorily, one should bear in mind that the failure of the joints does not correspond to a stable and efficient mechanism, and can put into question any simplified design procedures. Simplified design can only be applied when all the structural components are capable of satisfying the global assumptions: in this case, the joints proved to represent the weak links.

4.3 How safe design is?

Possibly, the most important piece of result was the experimental assessment of the maximum strength of the building. One is generally aware of additional sources of strength, such as the contribution of orthogonal elements, or the difference between nominal and characteristic values of material strength or dimensions, and over-strength is generally regarded as an extra, though difficult to quantify, safety coefficient. This is not always the case in earthquake engineering: larger strength corresponds to larger forces and acceleration in the structure, therefore a larger strength does not necessarily lead to a safer structure. A strength 2.5 larger than the one assumed in design was measured in the tests [8]: this may have some important implications in design.

5. Tests on a building equipped with nonstructural masonry infills

Other tests were performed on R/C buildings similar to the previously described one, but equipped with nonstructural masonry panels [9] (Figure 2). Infills are typically neglected in design, therefore it was decided to assess their contribution to the global structural behaviour. The same building was tested bare, with a regular infill distribution (all storeys infilled), and with a vertically irregular distribution (soft-storey pattern). The three buildings (which are normally regarded as the same structure, since infills are not considered in design) behaved very differently. The uniformly infilled frame suffered much smaller deformations (in other words, the damage of the infills protected the building from the seismic forces), whereas the soft-storey configuration resulted in unacceptably large concentrations of deformations in the open storey. It was concluded that the presence of the infills should be included in design, and new rules were indeed developed to achieve this goal [10]. On the other hand, it was proposed to take advantage of the capabilities of regularly distributed infills, to reduce the design forces for the frame. How-

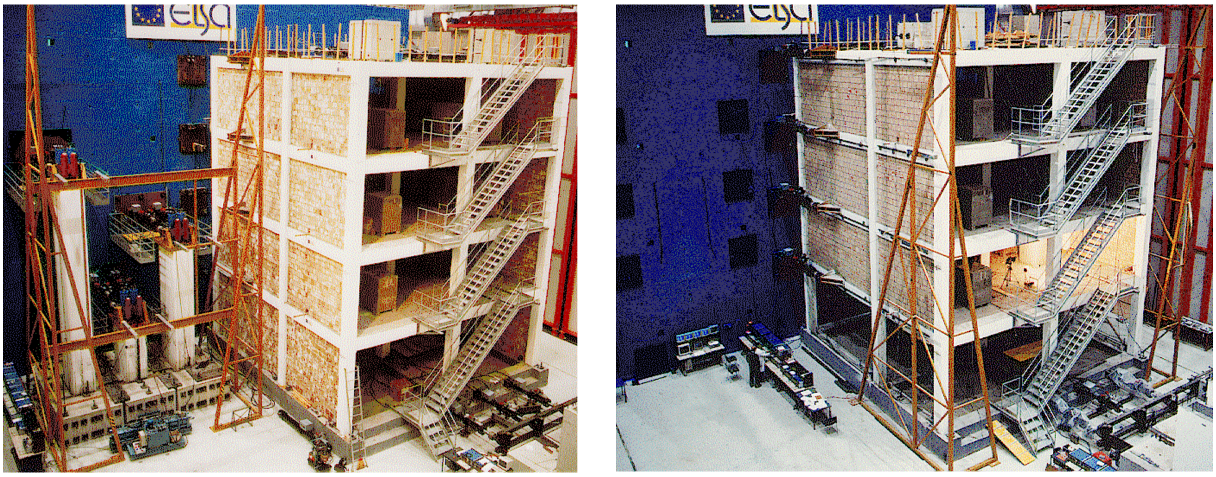

Figure 2: R/C frame with regular and irregular configurations of masonry infills.

ever, subsequent tests demonstrated that the mechanical properties of infills are so variable (as a result of shrinkage or micro-deformations) that it would be unrealistic to design them as real structural elements [11].

6. Tests on continuous-deck bridges

Bridges are structures for which a simple conceptual scheme is generally thought sufficient to guide the whole design process. However, bridges are quite often very irregular structures as far as their seismic behaviour is concerned.

A testing activity was conducted on six continuous-deck bridges with piers of differing heights [12]. Only the elements which were expected to undergo nonlinear deformations (the piers) were physically tested, whereas the deck, which was expected to remain elastic, was simulated by means of a FEM model on-line interacting with the experimental set-up (Figure 3).

These apparently simple structures exhibited dramatically different ductility demands at the bases of the piers. Whereas taller piers remained elastic, the shorter suffered large concentrations of forces and deformations. It turned out that the definition of a unique ductility level (and therefore, a unique force reduction factor to be used in design) is impossible, and the standard equivalent-elastic design methods are a grossly misleading approach. New design methods, based on deformations, were developed on the basis of the conclusions of the experiments [13].

7. How about irregular structures?

Sound structural concepts would lead to simple and regular structures. On the other hand, poor interaction between architects and structural engineers inevitably leads to complex, often quite irregular, structural solutions. Irregular-

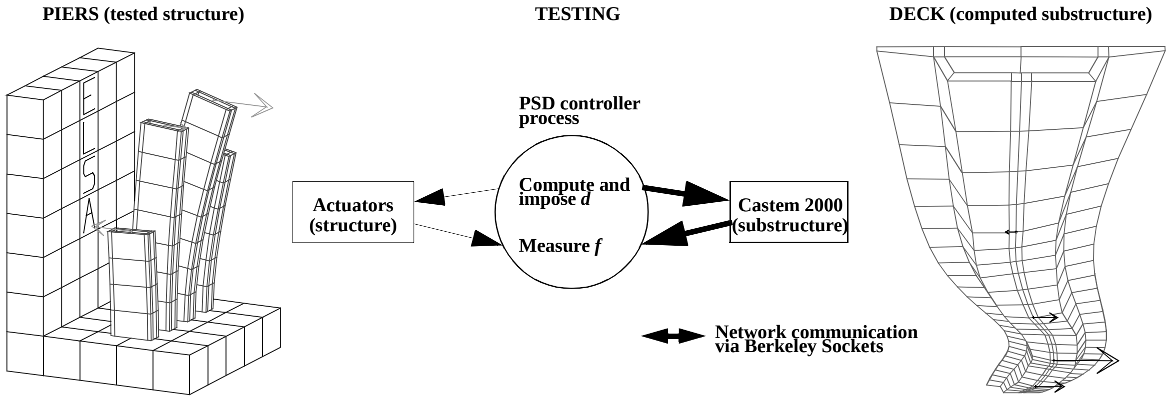

Figure 3: Experimental set-up for the tests on irregular continuous-deck bridges.

ities cannot be taken into account by means of simplified design methods. In earthquake engineering, the consequences of structural irregularities are even more profound, and can be appreciated after any strong earthquake.

Tests were also conducted to study the consequences of structural irregularity. Bi-directional seismic tests were performed on a structure with large torsional eccentricity [14]. The structure was a three-storey full-scale composite building, designed and tested for providing the necessary data for the calibration of the relevant clauses in Eurocode 8. The structure had a large eccentricity in one direction, and appreciable accidental eccentricities were present in the other direction as well. The main conclusion of these tests was that stiffness eccentricities play a very little role in the structural behaviour. What ultimately dictates the structural behaviour is the strength distribution. This general conclusion paves the way to simple strength-based concepts to be used in design [15]. On the other hand, the adoption of design methods solely based on stiffness properties should be discontinued [16].

8. The problem of structural rehabilitation

One of the most difficult problems that engineers have to face today is the structural rehabilitation of existing buildings. This is particularly true for buildings located in seismic prone areas. Many of these buildings have been designed and built according to old design codes and do not possess adequate seismic safety. Old reinforced concrete buildings have typically been built with poor materials (smooth rebars, low-strength concrete), and lack adequate detailing (insufficient confinement, poor detailing of splices and anchors). More importantly, they have large irregularities, and no provisions for ensuring that a stable and efficient dissipative mechanism takes place have been made in the design.

PsD tests were conducted on two frames selected as representative of old European R/C construction practice [17]. The frames (Figure 4) were built with old materials (smooth rebars, low-strength concrete), and exhibited the typical problems of non-seismic design. The strength of the beams was much larger than the strength of the columns (i.e., no capacity design criteria were accounted for). A column line was considerably stiffer than the others, and the strength and stiffness had sharp storey-wise variations. In addition, confinement was insufficient in many potential plastic hinge locations.

Two identical frames were tested, one bare, the other with light nonstructural infills. The bare frame behaved as expected, i.e., showed unacceptable inter-storey deformation at the level corresponding to the variation in the cross-section of the stiffer column. The infilled frame behaved in a much more regular fashion, thanks to regular distribution of infills.

A second testing activity was devoted to assessing the adequacy of possible strengthening techniques. For the infilled frame, based on the findings of the first phase, it was decided to improve the energy dissipation capacity of the infills by means of welded steel meshes and grouted concrete.

For the bare frame, a Selective Retrofitting Technique [18] was adopted. A selective retrofitting technique is a means of increasing either stiffness, strength or ductility capacity, without modifying the other properties (to this extent, the technique may deserve the definition of “conceptual retrofitting”). Modifications were mainly imple-

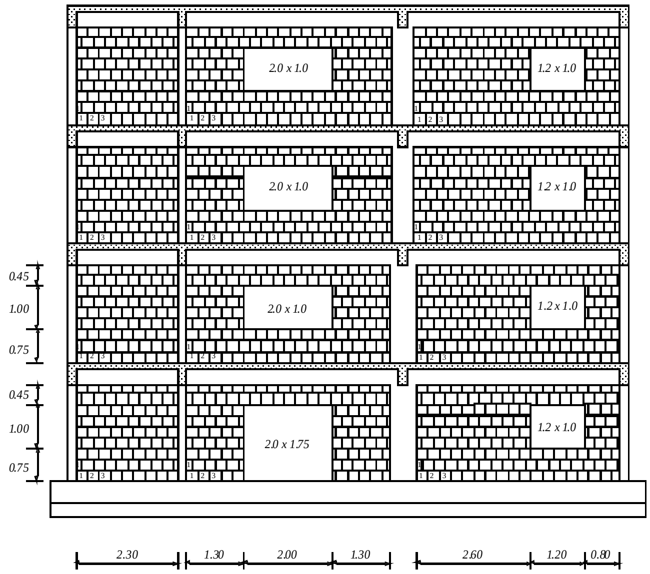

Figure 4: R/C frame for studying assessment, strengthening and repair techniques for existing non-ductile buildings.

mented in the stiffer columns, to provide the necessary confinement (ductility capacity) and to make the storey-wise distributions of strength and stiffness regular. The behaviour in the second test resulted much more regular, thus demonstrating the effectiveness (and sound conceptual significance) of the approach.

9. Conclusions

After a necessarily brief introduction of Eurocode 8, the techniques which are available for testing large structures were briefly presented. Among these, the pseudodynamic test method represents a viable solution for proof-testing. i.e., for verifying whether the basic assumptions made in design were correctly stated. It is believed that this information is what is finally needed for assessing any design approach.

The results of some experimental activities carried out at the European Laboratory for Structural Assessment were summarised, to illustrate the lessons learnt as for the calibration of Eurocode 8

A series of tests conducted on a four-storey building demonstrated that the global behaviour can be completely different from the one assumed in design as a consequence of the adoption of a different reinforcing steel. Also, inadequate detailing of the joints can invalidate the assumption made in the design. Finally, large-scale testing demonstrated that the global strength can be up to 2.5 times larger than the design strength, and the potentially dangerous consequences of this finding were stated.

The importance of including the effects of nonstructural elements such as masonry infills was demonstrated by the tests performed on an infilled-frame building. In particular, the dangerous consequences of any irregular configuration in the infills were presented, and the need to account for them in the design process was stated.

Other tests demonstrated the effects of structural irregularity in the global behaviour. Any design approach is bound to failure if structural irregularity is not properly accounted for in the model. Stiffness irregularity affects the global behaviour to a much lesser extent than strength irregularity. A good design approach should therefore seek a regular distribution of strength.

Finally, the importance of a sound conceptual framework in addressing the rehabilitation of existing buildings was demonstrated. Once again, structural irregularity is often responsible for a unsatisfactory global structural behaviour. Selective limited modifications to the structure can result in a much more regular response.

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