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ASSESSMENT OF EC8 PROVISIONS FOR REINFORCED CONCRETE BRIDGES

G. M. CALVI[(1)] and P. E. PINTO[(2)]

[(1)]Dipartimento di Meccanica Strutturale, Università degli Studi di Pavia, Pavia, Italia
[(2)]Dipartimento di Ingegneria Strutturale e Geotecnica, Università di Roma "La Sapienza", Roma, Italia

ABSTRACT

The main results of extensive experimental and numerical research performed in the last years on the seismic response of RC bridges are presented. The topic addressed are the deficiencies in existing bridges and code provisions, the effects of method of analysis, modelling assumptions, soil-structure interaction, non-synchronous input motion on ductility and displacement demand, the development of alternative design methods for standard and isolated bridges.

KEYWORDS: Eurocode 8, bridges, ductility demand, regularity, soil interaction, non-synchronism, isolation, displacement-based design

INTRODUCTION

Bridge structures have always occupied a special place in the affection of structural engineers, probably because in bridges the structural conception is more strictly related to aesthetics and functionality than in any other construction type. For the same reason bridges give the impression of being rather simple structural systems, whose seismic response could be easily predicted. On the contrary, in recent earthquakes bridges have not performed well, indicating thus the need for additional research and understanding of different potential problems and collapse mechanisms.

In Europe the development of the appropriate code of practice (EC8/2, 1994) has been accompanied by an intensive research work (see for example Calvi et al., 1989) which rather than ending with the approval of EC8/2 has found more sources of energy in the need of improving the code in the three years required to transform the pre-standard into a European standard and in the problems arising from the Northridge and Kobe earthquakes.

In this paper a brief overview of the most recent progress in bridge design, assessment and retrofitting is offered, with emphasis on the implementation into a code of standard, nameley EC8/2. A preliminary discussion of the most relevant problems will introduce the discussion of present EC8 provisions, and of some proposals for improvement emerged from recent research.

DEFICIENCIES IN EXISTING BRIDGES AND IN CODE PROVISIONS

An overview of the reasons of damage and collapse in recent earthquake clearly offers a reliable guideline on the deficiencies in bridge design, to be considered in improved code provisions and in support research. An overview of the reasons of seismic damage to bridges can be found in (Priestley et al., 1996), while in (Priestley et al., 1994 and Seible et al., 1995) specific examples of problems encountered in the Northridge (1994) and Kobe (1995) are discussed.

In general it has been noticed that most of the problems arise from design problems, rather than from construction or material inadequacies, namely from the explicit application of elastic design concepts or from the heritage of the elastic design philosophy used in the recent past.

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The following typical failure modes have been observed in recent earthquakes:
- span failure due to unseating (insufficient seat length and/or restraining force capacity);
- flexural failures, due to insufficient strength and/or ductility capacity, and often triggered by inappropriate termination of longitudinal reinforcement (not only in piers, but also in the cap beam of frame bent);
- shear failure, often due to the inappropriate consideration of the concrete contribution to shear strength in presence of large ductility demand;
- joint failure: a properly designed joint reinforcement was almost never provided;
- footing failure, due to omission of top reinforcement, inadequate shear strength, insufficient anchorage of pier reinforcement and inadequate connection between pier and footing.

It has also been observed that unusual types of seismic motions and soil effects have produced damage and collapses. These comprise soil filtering and amplification at long period of vibration, liquefaction, abutment slumping, extremely high PGA for near field events, and non-synchronous input motion.

In Europe it has been assessed that in most cases foundations and piers of existing bridges have enough strength resources to stand realistic acceleration levels even without significant ductility capacity. The main problems are therefore related to displacement capacities at joints and at fixed and movable bearings.

PROPOSED INPROVEMENT OF EC8 DESIGN PROVISIONS

Methods of analysis

The discussion of what methods of analysis should be used for the final verification of the response of a bridge structure is an old topic, with no practical merit until only linear multimodal analysis will be reliably used in design offices. There are two important exception to this situation, which deserve some comments.

The first exception can be found in the US, where in the last two years there has been growing interest in an event scaling procedure, used to determine the sequence of inelastic actions, the formation of local mechanism and the formation of a global collapse mode. The main merit of this approach, well known in principle for years, is to draw the attention to the actual nonlinear response, accepting a coarse geometric modelling of the structure and renouncing to simulate its dynamic response. The method, often referred to as "push-over" analysis, consists in a sequence of linear elastic analyses with a step-wise changing structural system. Special purpose programs (SC-push 3D, 1995) have been developed to iterate for axial load effects, update the structural stiffness as a function of events such as cracking, formation of flexural hinges, yielding of soil springs and perform the sequence of linear analyses.

The second exception is in EC8/2, where a non linear dynamic analysis is essentially compulsory in case of bridges with isolation/dissipation devices. This topic will be further discussed later, but it is now worthwhile to observe that a non linear dynamic analysis can be so simplified in this case that its complexity is not necessarily greater than a standard linear multimodal analysis. Also in this case special purpose programs have been developed and successfully used (Giannini et al., 1992).

Extensive linear and nonlinear analysis simulations (Calvi et al. 1994, Alarcon, 1995, Calvi and Pinto, 1994), large scale speudodynamic tests (Pinto et al. 1996) and shaking table tests (Casirati et al. 1996) have shown that in most cases a single reduction factor (q, in Europe) approach combined with linear dynamic analysis gives results sufficiently accurate for design purpose. The exception is given by special cases of highly irregular bridges, as discussed further in the paper.

Detailing and capacity design provisions

Little attention has been devoted to this topic in recent research in Europe, likely because it is felt (and it has been confirmed by the tests mentioned in the previous section) that the present code provisions are effective in avoiding any kind of brittle failure mode. Only in recent years it has been understood that design rules developed for new structures cannot be blindly applied to assessment and strengthening of existing structures. The assessment of strength and displacement capacity of ill designed and detailed elements is therefore an art not yet diffused in Europe and will be the object of future research, together with the development and assessment of strengthening techniques.

Influence of bridge regularity on the expected ductility demand

The approach presently adopted in EC8/2 for bridge design is based on a single behaviour factor (q) that reduces the seismic forces to take into account the non linear response and in general the energy dissipation capacity of the bridge structure. This approach assumes that the structure will respond regularly, with predictable ductility capacity and a rather uniform ductility demand throughout the different piers. It is well known that a ductile behaviour can be obtained with the application of capacity design principles, capable of creating a hierarchy in the potential failure modes, with the ductile modes always preceeding the brittle modes. Unfortunately it has been shown that unexpected concentrations of ductility demand may nevertheless result when the bridge configuration is "irregular". It has also been discussed that this is particularly the case when the dominant mode of vibration of the deck alone is significantly different from the dominant modes of the whole bridge. In this case the deck mode can prevail after significant softening of the piers, inducing larger than expected displacements in specific piers. A first attempt to give a measure of the bridge regularity resulted in the following index, to be computed preliminarly on the base of two eigenvector analysis (Calvi et al. 1994):

$$I_R = (1/n)\ \Sigma_i\ \Sigma_j\ (\delta_{ij}\ \phi_i^D\ M\ \phi_j^B - (1-\delta_{ij})\ |\phi_i^D\ M\ \phi_j^B|) \qquad (1)$$

where ϕ_j^B is the eigenvector of the entire bridge, ϕ_i^D is the eigenvector of the deck alone, M is the mass matrix, and δ_{ij} is the Kroenecker delta.
Extensive parametric analyses, performed using non linear time integration techniques, showed that a threshold limit of about $I_R = 0.5$, below which larger than expected ductility demand may result in some pier. The result of several non linear dynamic analyses are shown in figure 1. All the bridges considered were designed for a target ductility demand equal to 3.

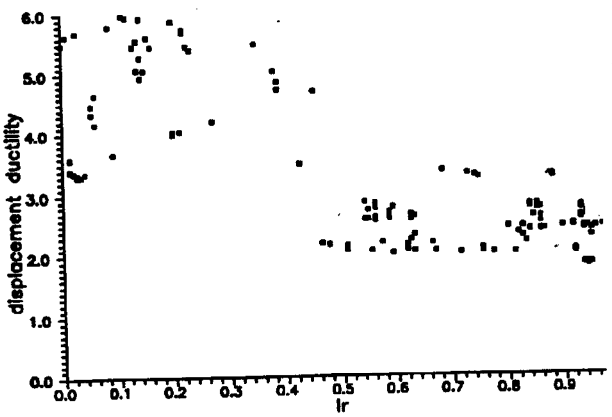

Fig. 1. Displacement ductility versus regularity factor (see eq. 1, (Calvi et al. 1994))

The implications of the above discussion is that the correct application of capacity design principles can avoid brittle failure modes, but only appropriate design and analysis methods can avoid excess of damage concentration in ductile collapse mode. For irregular bridges a single force reduction factor uniformly applied to an elastically designed structure is clearly unable to assure an acceptable response. In this last case the only present alternatives consist in the use of very conservative q-values or in nonlinear analysis verifications (with an appropriate number of input accelerograms).

Influence of different modelling assumptions on the expected response

Several numerical simulations (Elnashai and Mc Lure, 1995) showed that different conceptual modelling of the bridge structure may result in significantly different ductility demand distribution. For example consider how modelling of the connection between pier and deck with a hinged, or fixed, or fixed with a limiting moment connection, may lead to significantly different results. This was also confirmed in some kind of international benchmark numerical tests [Park (Editor), 1994]. The already mentioned shaking table tests (Casirati et al., 1996) confirmed the difficulties in a correct numerical simulation of the pier-deck connection. The connection was in principle a fixed connection with a limiting moment and shear keys; the relative horizontal displacement were nevertheless significant, with transfer function generally assuming values larger than 2.

Influence of soil-structure interaction (SSI) on the expected response

EC8/2 states that SSI effects "should be considered when the displacement due to soil flexibility is greater than 30% of the total displacement at the centre of mass of the deck". Clearly the emphasis is on the potential for increased displacements rather than on the relevance of SSI on inelastic response in general and ductility demand in particular. An extensive parametric analysis (Ciampoli and Pinto, 1995) of a single bridge pier allowed to produce figures 2 and 3, where the ratio between top displacement (η) and ductility demand (μ) obtained considering and negelcting SSI are shown. The parameter represented in abscissa in both plots expresses the relative stiffness of foundation and soil: $\sigma = V_s T / H$, where V_s is the shear wave velocity of the soil, T is the rigid-base period, and H is the height of the pier. As expected, the added foundation flexibility leads to increased top displacements, but limited to less than 25% in more than 90% of the cases. The sensitivity to variations of η is also very low. SSI also has very limited effect on the ductility demand, as shown in figure 3. These results do not exlcude the possible existence of peculiar cases where SSI effects are important, in general however, bridges are rather flexible structures, with moderate masses and limited dimensions of the foundations. It is then confirmed that consideration of SSI effects may only be necessary in case of low soil stiffness, with the main purpose of a better estimate of the displacement demand.

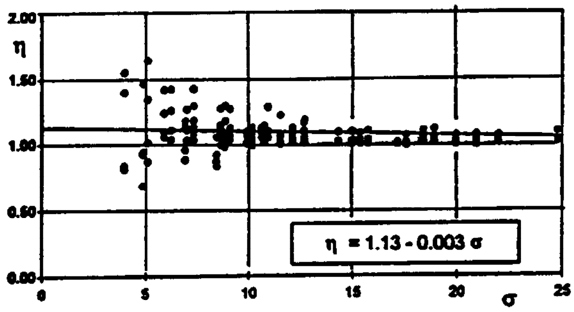

Fig. 2 Ratio between maximum displacement with and without SSI (Ciampoli and Pinto, 1995)

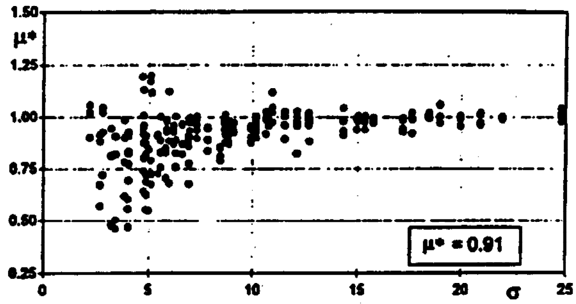

Fig. 3 Ratio between displacement ductility demand with and without SSI (ibidem)

On another aspect of the soil effects on bridge response, it is worth noticing that the Northridge and Kobe earthquakes have clearly shown that there is a need of improving the design spectra suggested in the codes, particularly for what concerns long periods of vibration. Again this is a topic for future research. A warning on the potentially abnormal amplification resulting from special soil stratigraphy is already present in EC8.

Influence of non-synchronous input motion on the expected response

EC8/2 is the only code known to the authors where consideration is given to the spatial variability of the ground motion and some guidance is given on how to account for it. The present version of EC8/2 requires to consider the effects of non synchronism for all bridges longer than 600 m, and when the soil presents marked geological or topographical variations. Three methods are suggested, in decreasing level of complexity:
- to describe the soil motion at different points as components of a random field, homogeneous in space and stationary in time, and defined by its covariance matrix;
- to consider the components of a simplified random field, defined by independent motions allocated at the nodes of a square mesh;
- to use a purely kinematic model, consisting of a set of static relative dispalcements expressed as:

$$d(x) = x\, V_g / C_p < 2^{1/2} d_g \qquad (2)$$

where x is the distance between two points, V_g and d_g are the peak ground velocity and displacement, and C_p is the velocity of the compressive wave.

Very few studies are internationally available on this topic (see, for example, Der Kiureghian and Neuenhofer, 1992), and generally tend to concentrate on the solution algorythm and to be limited to a linear response. A systematic study of the response of bridges designed with a standard q-factor approach to non-synchronous input has been completed recently (Monti et al. 1995). A series of six-span continuous bridges, with differern pier heights, different soil flexibility and different q-factors was designed. The input was defined using a coherency function of the following form:

$$\gamma(x,\omega) = \exp(-(\alpha\omega x / V_s)^2) \exp(i\omega x_L / V_{app}) \tag{3}$$

where x is the distance between the two of interest, ω is the angular frequency, α is a parameter, V_s is the shear wave velocity, x_L is the projected horizontal distance between along the direction of wave propagation, and V_{app} is the apparent velocity of propagation of the wave train. The first term has a stochastic nature, and alone would produce wave trains differing by random phase angles; the second term has a deterministic nature and would produce identical processes shifted in time. The two main parameters adopted have been the shear wave velocity and the apparent velocity. Typical results are shown in figures 4 and 5. It seems possible to conclude that conventional design provides a global upper bound of the response and is therefore globally conservative. However, further studies on different geometrical configuration are needed before drawing general conclusions.

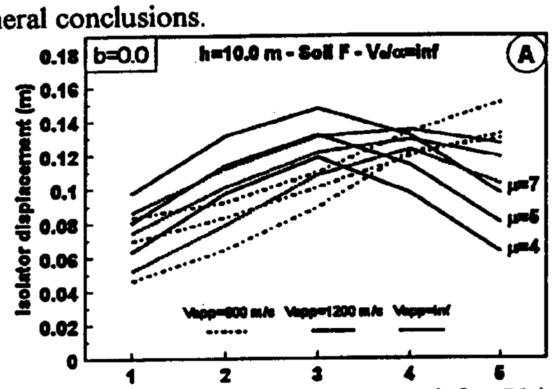

Fig. 4 Displacement ductility demand for V_s/α = 300 m/s (Monti et al. 1995)

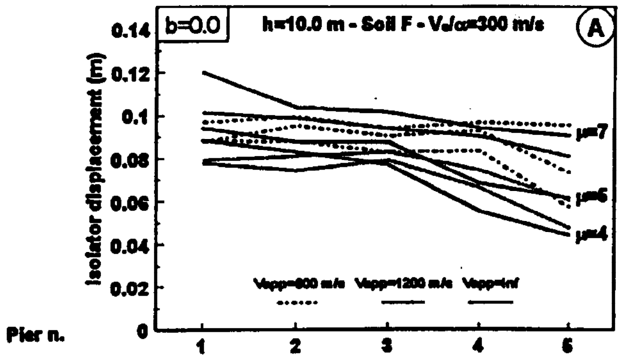

Fig. 5 Displacement ductility demand for V_s/α = infinite (ibidem)

Seismic isolation

It is believed that approximately 300 isolated bridges exist in the world and more than half of them are in Italy. No code of practice on isolated bridges exists in Italy (with the exception of the document "Guidelines [...], 1991), and only recently a section on this topic has been introduced in EC8/2. Several kinds of devices have been designed and successfully tested, but effective design methods have still to be developed and a sound base to relate safety level and devices features needs further studies. It is therefore true that in this field practice and technology went farther than research and codes. The limited experience on response during severe earthquakes is certainly contributing to this dichotomy, even if there is a good capacity of simulating the nonlinear response of isolated structures and a high and reliable technological production level. On the other hand broad probabilistic studies to estimate the safety level associated with different protection factors and response parameters have not yet been completed and are an important future research field.

For the time being possibly larger safety factors should be used when designing an isolated bridge, nonlinear time history analysis should be performed in most cases, the I/D devices should be always tested, a maintenance program with periodic inspection and possible re-testing of devices should be scrupulously followed.

Provided that this general philosophy is accepted and the derived specific requirements are met, dynamic isolation constitutes a powerful, flexible and economical tool for designing new bridges, and, to a even larger extent, for retrofitting existing bridges. The extreme variability of the potential design variables allows clever solutions to difficult problems and specific protection of elements without ductile details. The same wide possible selection of design parameters makes the preparation of simple design rules to be blindly applied difficult and confirms from another point of view the need for detailed analysis.

Two specific topics deserve special attention: the known sensitivity of isolated structures to the characteristics of soil motion, and their potentially high sensitivity to non-synchronous input. The first problem addresses to the recourse to site-specific studies rather than using standard spectral shapes. On the second topic a pioneering work (Monti et al., 1995) seems to indicate that the variability of the motion does not alter the order of magnitude of the displacements, which however increase systematically particularly at the piers closer to the bridge ends. The combination of spatial variability and input frequency content different from that assumed in the design phase may well result in displacement increase on the order of 50 %.

ALTERNATIVE DESIGN APPROACHES FOR IRREGULAR AND ISOLATED BRIDGES

A displacement-based design approach

There has been growing interest recently in the development of displacement based seismic design (DBD) procedures for bridges (Kowalski and Piestley, 1996, Calvi and Kinglsey, 1995). This has stemmed partly from the recognition that, under seismic actions, displacements provide a more fundamental expression of structural response than forces, and that the structural design process should be oriented accordingly. Since structural damage can be considered to be related directly to displacement demands, it can be controlled more efficiently through the imposition of displacement (or drift) limits rather than strength limits. Furthermore, displacement-based design offers the ability to control explicitly the displacement demand in each member rather than assigning a single, force-based behaviour factor to the entire structure. It is therefore felt that limits of the q-factor approach, particularly in case of irregular bridges, may be overcome.
The fundamental conceptual shift required in switching from a force-based to a displacement-based design approach has been presented by Kowalski and Priestley (1996) for the case of a single degree-of-freedom (SDOF) bridge pier structure. In summary, in a traditional force-based design the period of vibration of a structure is estimated (or calculated based on previous trial design), and design pseudo-acceleration response spectra are entered to determine, for a given level of damping, the elastic design force level on the structure. Alternatively, the required design displacement demand for the structure can be specified, and the displacement response spectra consulted to determine the required period of vibration (stiffness) necessary to achieve it, provided that the structure has been modelled assuming a linear behaviour and a viscous damping equivalent to the actual non-linear response. In this manner, strength and stiffness become end-products of the design rather than primary design goals. For simple SDOF structures, the process outlined has been applied successfully to bridge piers over a range of possible design parameters. The application of the method to more complex MDOF structural systems is not necessarily straightforward, and requires some additional steps (Calvi and Kingsley, 1995).
The development of an equivalent SDOF representation of a MDOF structure has been suggested, to allow SDOF design spectra to be used. This is achieved by imposing a pre-defined displaced shape on the structure, and establishing the other properties of the equivalent SDOF structure accordingly. Fundamental problems are the definition of appropriate target displacements, or drifts, and of reliable displacement response spectra. The first choice depends on the limit state, on the accepted level of damage and on the current design philosophy: for example from a comparison of the target drifts used by American and European researchers, the European tendency to stiffer piers results clearly. The second problem is related to the already mentioned inadequacy of current code spectra at long periods of vibration.
The method has already proven to be more efficient than standard force based design methods (Calvi and Kingsley, 1995). It is in general not possible to pre-assign any desired deformed shape, for example imposing a simultaneous optimal drift to all piers. It seems that the imposed shape should be similar to the fundamental (and possibly dominant) mode of the bridge. Improved techniques based on strain rather than on displacement are under development, to allow a direct control of the reinforcement elongation.

A design approach for isolated bridges

Designing an isolated bridges is in principle simpler than designing a non-isolated bridge. Essentially one single capacity design principle will be applied, i.e. the isolating system will be the only element where a non linear response will be expected. This means to set the yielding strength of each isolator at a strength not greater than 85% of the lowest strength (flexure, shear, footing, etc.) of the corresponding pier. In most cases it will be required to set all the isolator at the same strength, and to compensate the stiffness of each pier with the stiffness of the corresponding isolator, to obtain a perfectly regular system, where the deck should remain undeformed during the earthquake. This philosophy will be applicable to new and existing bridges, but in the last case it may imply a significant loss in strength, since the weakest pier will govern the strength of all piers. A better exploitation of the piers strength will require more refined non linear analyses and possible iterations of the final analysis process. In general it will be possible to obtain a regular structure with a single dominating mode, where the energy dissipation capacity of the isolation system will dominate the global equivalent damping of the structure. On this base it has been possible (Calvi, 1996) to develop a special version of the displacement based design method described by Calvi and Kingsley (1995), which seems to

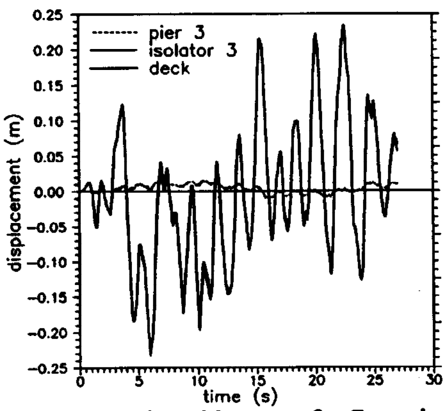

Fig. 7 Displacement time history of a 7 m pier and its isolator (Calvi, 1996)

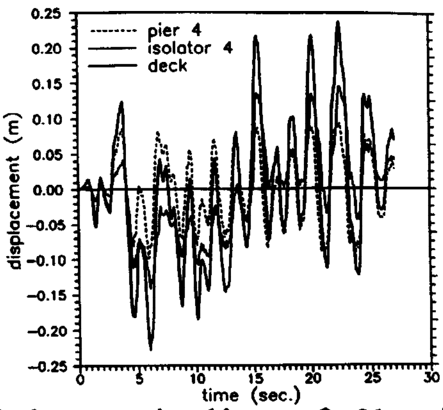

Fig. 8 Displacement time history of a 21 m pier and its isolator (ibidem)

produce reliable results. An example of the nonlinear dynamic response obtained from two piers with very different heights (7 and 21 m), part of the same bridge, is shown in figures 7 and 8. It is evident how the isolators regularize the response, producing a similar global displacement history.

OPEN PROBLEMS AND RESEARCH NEEDS: ASSESSMENT AND STRENGTHENING

The problem of assessing the safety of existing structures has been traditionally absent from codes of practice, assuming that the methods accepted for designing new structures could be applied straightforwardly, and that lower protection factors, i.e., a lower safety against failure, should be accepted. Both assumptions are actually wrong, since assessment of existing bridges deserves a revision of limit states, methods of analysis, capacity design principles and methods to calculate strength and deformation capacity of members, and there is no reason why existing structures should be more dangerous than new structures. Lower (or higher) protection factors might be appropriate, though, because of different level of knowledge of the structural systems. For example structure geometry and material strength could be known without uncertainties, if detailed measures have been taken and tests results are available. It is therefore possible that lower protection factors do correspond to the same level of protection against failure. In case of limit states against other kinds of undesired events, which do not imply collapse and potential loss of human life, lower protection factors could be accepted because of the economical implication of demolition and reconstruction.

Capacity design principles also deserve some revision and generalization to the case of existing bridges, as attempted in the case of buildings (Priestley, 1995). Of particular interest is the evaluation of the real response of underdesigned elements, since a simple separation of acceptable and unacceptable details will be useless in the case of existing bridges. The interaction of shear and flexure is an example of problem which has gained attention in the last years. Prioritization schemes to identify and rank all high-risk bridges in a region, to allow an optimum allocation of resources for retrofit, should also be developed and improved. Very little research on this topic has been performed in Europe, while several documents are available in Japan, in New Zealand and in the US. A specific topic where a European contribution will be particularly valuable is the evaluation of global earthquake damage scenarios, where the vulnerability of a single bridge to such limit states as ininterrupted transitability or repairability is evaluated in relation to the global post-earthquake situation.

There is very little experience on bridge strengthening in Europe, on one side because there have been no strong earthquakes requiring post-earthquake repair and strengthening to bridges for several years, on the other side because of the blind attidute, common to most countries in the world, to wait for an earthquake rather than assess and strengthen to get prepared to it.

Column retrofit techniques, such as jacketing using steel, concrete or composite materials, have been studied in detail in the US. A special problem, where more studies are required, is the movement joint retrofit. Actually several kinds of restrainers and seat extenders have been proposed and used, but the considerable length of modern continuous deck bridges, particularly popular in Europe, requires detailed theoretical and experimental studies of the effects of possible inpacts at the movement joint. In Japan it was assumed that "damage to expansion joints does not cause serious damage to the bridge" (Manual for Menshin design, 1994) and, on the contrary, impact could provide an additional source of energy dissipation. This opinion may have changed after the Kobe earthquake. EC8/2 requires expansion joints capable to accomodate the seismic displacement, and this may result in very large (and expensive) joints.

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