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SEISMIC RETROFITTING OF REINFORCED CONCRETE BUILDINGS USING TRADITIONAL AND INNOVATIVE TECHNIQUES

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ABSTRACT

The seismic retrofitting of reinforced concrete buildings not designed to withstand seismic action is considered. After briefly introducing how seismic action is described for design purposes, methods for assessing the seismic vulnerability of existing buildings are presented. The traditional methods of seismic retrofitting are reviewed and their weak points are identified. Modern methods and philosophies of seismic retrofitting, including base isolation and energy dissipation devices, are reviewed. The presentation is illustrated by case studies of actual buildings where traditional and innovative retrofitting methods have been applied.

KEYWORDS: Pushover Analyses, Seismic Vulnerability, Seismic Retrofitting, Base Isolation

INTRODUCTION

Seismic retrofitting of constructions vulnerable to earthquakes is a current problem of great political and social relevance. Most of the Italian building stock is vulnerable to seismic action even if located in areas that have long been considered of high seismic hazard. During the past thirty years moderate to severe earthquakes have occurred in Italy at intervals of 5 to 10 years. Such events have clearly shown the vulnerability of the building stock in particular and of the built environment in general. The seismic hazard in the areas, where those earthquakes have occurred, has been known for a long time because of similar events that occurred in the past.

It is therefore legitimate to ask why constructions vulnerable to earthquakes exist if people and institutions knew of the seismic hazard. Several causes may have contributed to the creation of such a situation. These are associated to historical events, fading memory, greed, avarice, poverty and ignorance.

Among historical events particularly relevant are wars, epidemics, and natural disasters which may limit, in a significant way, the available resources of a country. In such circumstances there is a tendency to build with poor materials and without too much attention to good construction techniques and safety margins. A situation of this kind occurred in Italy and in Japan after the Second World War and similar situations have occurred in Italy many times in the past. In such a situation it is possible that the phenomenon of fading memory occurs and past memories are easily erased.

In Italy commercial profits often result from the employment of poor material and workmanship rather than of the optimal utilization of the production factors. The depressing situation of poor quality control and material acceptance also falls into this framework, which, in most cases, results only in paperwork devoid of substantive value. Marginal propensity to expenditure sometimes ensures that even the owner prefers a low quality product to save resources for more immediate needs.

Among causes arising from ignorance there may be both an inadequate knowledge of the seismic hazard and design errors due to insufficient knowledge of the earthquake problem; also the inability to correctly model the structural response to the seismic action.

While considerable progress has been made in recent years by the research community in dealing with the above problems, it has become more difficult to transfer the results to the seismic engineering profession and the situation can only deteriorate in the near future.

Recent changes in the curricula of engineering schools are leading to a general impoverishment of the basic knowledge and operational capabilities of our engineering graduates.

A final cause of vulnerability is connected with the maintenance of constructions; it is obvious that if a construction is not regularly maintained, much as happens for a motorcar, the mechanical properties of the materials may undergo local and global degradation with a significant loss of resistance of the structural members and of the entire construction. Also, changes in service conditions, often made arbitrarily, may lead to substantial changes in the structural behaviour resulting in a degradation of the structural response to the expected loading conditions.

On the basis of what has been presented so far, it is not surprising that in areas long known to be subject to the seismic hazard it is not infrequent to find constructions vulnerable to earthquakes. These constructions need to be retrofitted to allow them to withstand the effects of the earthquake ground motion expected at the site considered. In the following sections some procedures used for the evaluation of the seismic resistance and vulnerability of reinforced concrete buildings will be described together with traditional and innovative techniques of seismic retrofitting of the same structures. The paper ends with a description of the seismic retrofitting of two reinforced concrete residential buildings in the village of Solarino, near Syracuse, in Sicily. The buildings belong to the Institute Autonomo Case Popolari (IACP) of Syracuse.

As will be clear from following arguments the aim of the paper is not to discuss in depth the state-of the-art of seismic retrofitting, but rather to give a general overview. The aim is also to focus on a few specific procedures which may improve the state-of-the-art practice for the evaluation of seismic vulnerability of existing reinforced concrete buildings and for their seismic retrofitting by means of innovative techniques such as base isolation and energy dissipation.

SEISMIC ACTION

Seismic vulnerability is not an absolute concept but is strongly related to the event being considered. The same construction may not be vulnerable to one class of earthquakes and yet be vulnerable to another. Therefore, before attempting a seismic vulnerability evaluation of a given construction, the seismic action that will affect that construction must be fully specified.

All seismic codes specify the seismic action by means of one or more design spectra. These are a synthetic and quantitative representation of the seismic action which, besides depending on the characteristics of the ground motion, depends on some intrinsic characteristics of the structure such as the fundamental mode of vibration and its energy dissipation capacity.

The elastic design spectrum depends on the vibration periods of the structure and on the available damping. In Figure 1 the elastic spectrum of Eurocode 8 (CEN, 1998) is drawn for three different values of damping. A new draft of Eurocode 8 (CEN, 2003) became available in 2003, but is not being used here because some of the Eurocode 8 material relevant to the present work is still questionable and not generally accepted.

The value of the spectral pseudo-acceleration, corresponding to a vanishing small period, corresponds to the peak ground acceleration (PGA). In fact, for $T = 0$ the structure is rigid and, therefore, subject to the same acceleration as the ground. This acceleration, called the maximum effective ground acceleration or PGA, depends directly on the seismic hazard at the construction site and acts as the anchoring acceleration of the spectrum. This value is generally prescribed by seismic codes as a function of the seismic hazard at the construction site.

Furthermore, four regions may be identified for the elastic spectrum, each defined by a lower and upper period. In the first region, $(0 \le T \le T_B)$, the spectral ordinates increase linearly with the period; in the second $(T_B \le T \le T_C)$, these are independent of the period; in the third $(T_C \le T \le T_D)$, the spectral ordinates decrease rapidly with the period, that is with the reciprocal of the period T according to Eurocode 8; and finally in the fourth region $(T \ge T_D)$, they decrease even more rapidly, with the reciprocal of the period squared according to Eurocode 8. More details on the elastic design spectrum may be found in the seismic codes (CEN, 1998), in specialized publications and in the treatises on dynamics of structures and seismic engineering (Chopra, 2001; Clough and Penzien, 1993). The separation periods T_B, T_C, T_D depend on seismological factors and on local site conditions. For instance Eurocode 8 specifies them as a function of three subsoil classes: A (firm soil), B (medium soil), C (soft soil).

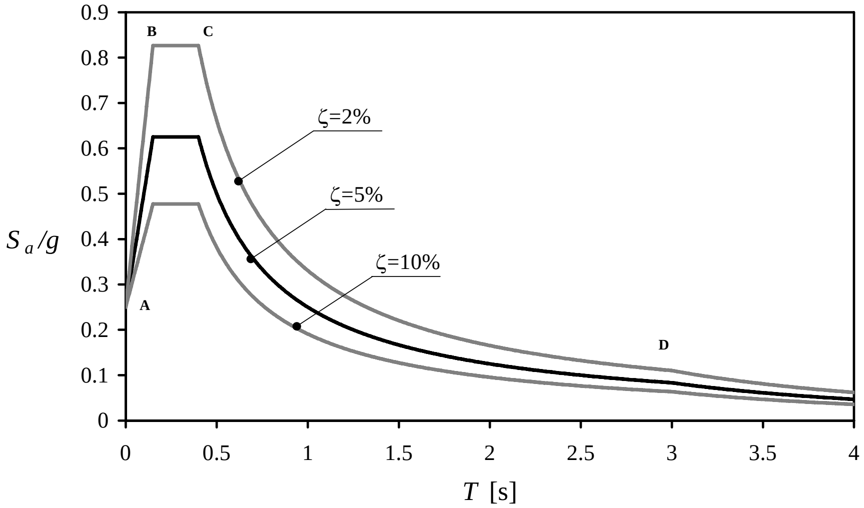


Fig. 1 Elastic design spectrum of Eurocode 8, soil type A

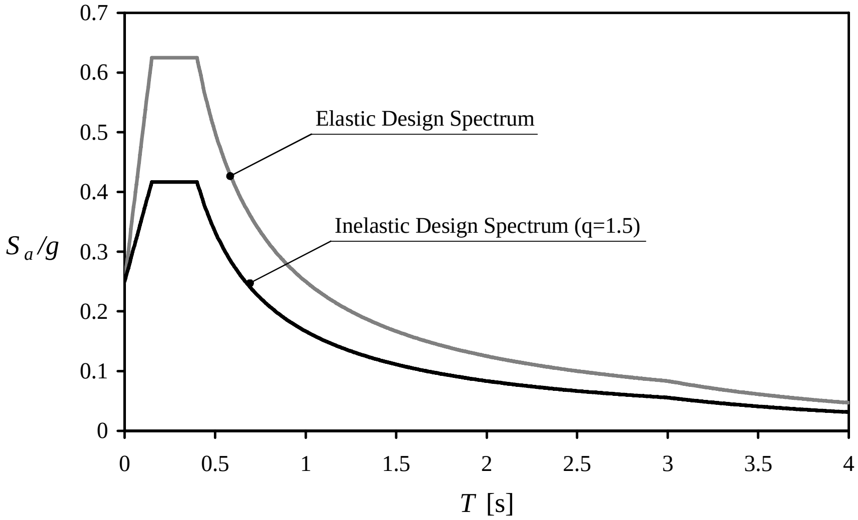


Fig. 2 Elastic and inelastic design spectra of Eurocode 8, soil type A

In traditional seismic design the energy dissipation capacity of the structure deriving from plastic deformations is generally considered. Including the inelastic resources of a structure allows for a considerable reduction of the spectral ordinates in the design spectrum. This reduction generally depends on the available ductility and on the vibration period. Eurocode 8 considers that this reduction is mainly dependent on a factor related to ductility and it is described as structure behaviour factor or simply structure factor. Typical values of the structure factor q may fall in the range 1 to 5 for reinforced concrete structures (CEN, 1998). As may be seen from Figure 2, the use of the inelastic resources of a structure allows for a considerable reduction in the spectral ordinates and therefore in the design strength.

SEISMIC RESISTANCE AND VULNERABILITY

Because it is necessary to retrofit only constructions vulnerable to the design earthquake, a vulnerability evaluation is obviously needed before attempting any seismic retrofitting. In the following, a definition of seismic resistance is provided and the corresponding vulnerability of a construction to the design earthquake is also defined. As has been seen, the design earthquake is specified by means of a design spectrum which depends on the energy dissipation capacity through the structure behaviour factor. Assuming that the structure behaviour factor for the structure being considered can be evaluated, the design spectrum can be drawn. An example of such a spectrum is shown in Figure 3.

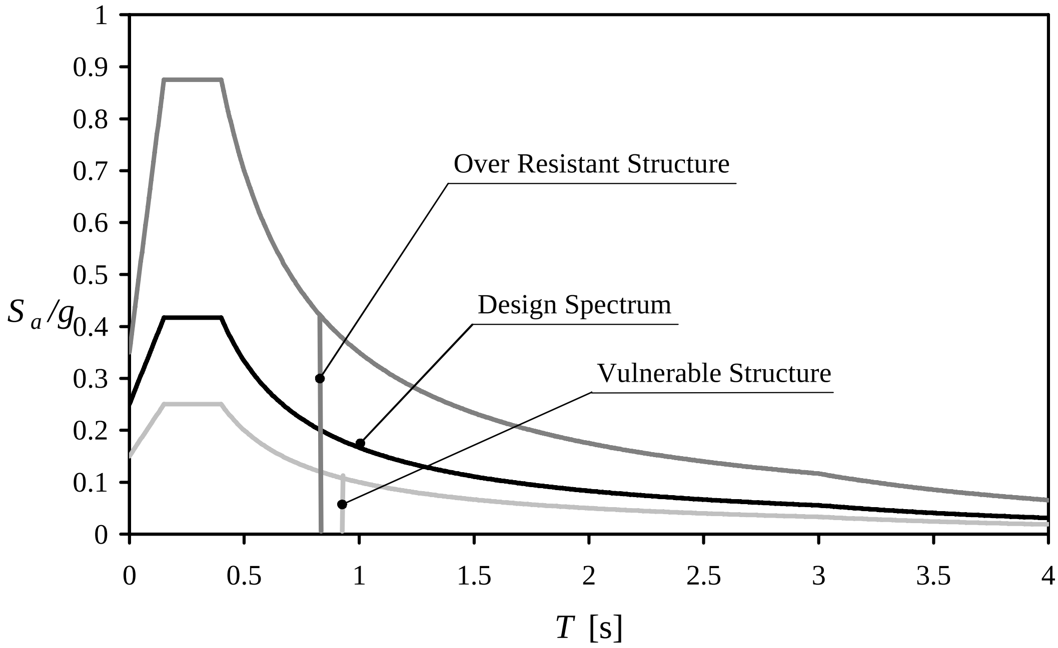


Fig. 3 Comparison between seismic resistance and seismic demand

If a structure exhibits seismic resistance larger than that required by the design earthquake, it obviously possesses an over-resistance and therefore is not vulnerable. This is the case shown by the longer ordinate in Figure 3. A structure with the resistance specified by such an ordinate is capable of withstanding an earthquake with an anchoring acceleration larger than that associated with the design earthquake. Instead if the seismic resistance of the structure corresponds to the shorter ordinate in Figure 3, it is obvious that the resistance capacity is smaller than the demand that the earthquake places on it and the structure is vulnerable to the design earthquake. In this second case the structure can only withstand an earthquake with an anchoring acceleration smaller than the design one. It is, therefore, necessary to retrofit the structure to allow for the satisfaction of the design inequality:

$$\text{Capacity} \geq \text{Demand} \tag{1}$$

The design inequality above must be satisfied not only in terms of strength or resistance, but also in terms of stiffness. The stiffness capacity of the building must not be less than the stiffness demanded of it by the earthquake. If it were not so, displacements would be too large, especially inter-story drifts, and damage could result to non-structural components. The stiffness control is usually performed indirectly by checking the inter-story drifts. The methods for the evaluation of the seismic resistance will be discussed later. Now the traditional methods of seismic retrofitting will be discussed briefly.

TRADITIONAL METHODS OF SEISMIC RETROFITTING

Traditional methods of seismic retrofitting fall essentially into two categories, one based on the classical principles of structural design which requires an increase of strength and stiffness, and the other based on mass reduction. Thus the first one tends to satisfy the design inequality by an increase of the capacity while the second one achieves the same result by a reduction of the demand. Since seismic design is different from ordinary design, both techniques may turn out to be quite ineffective as is shown in the following.

With reference to the first method, that is increase of strength and stiffness, the concept involved in its application can be understood using Figure 4. Suppose that the fundamental period of the structure is T_{nr}, to which corresponds a demand S_{anr} in pseudo-acceleration terms, which the structure cannot satisfy. On applying a strength and stiffness increment, the fundamental period will shorten from T_{nr} to T_r, to which corresponds a demand S_{ar} much larger than the original one. It is, therefore, possible that the structure will be less safe in the new condition than in the original one.

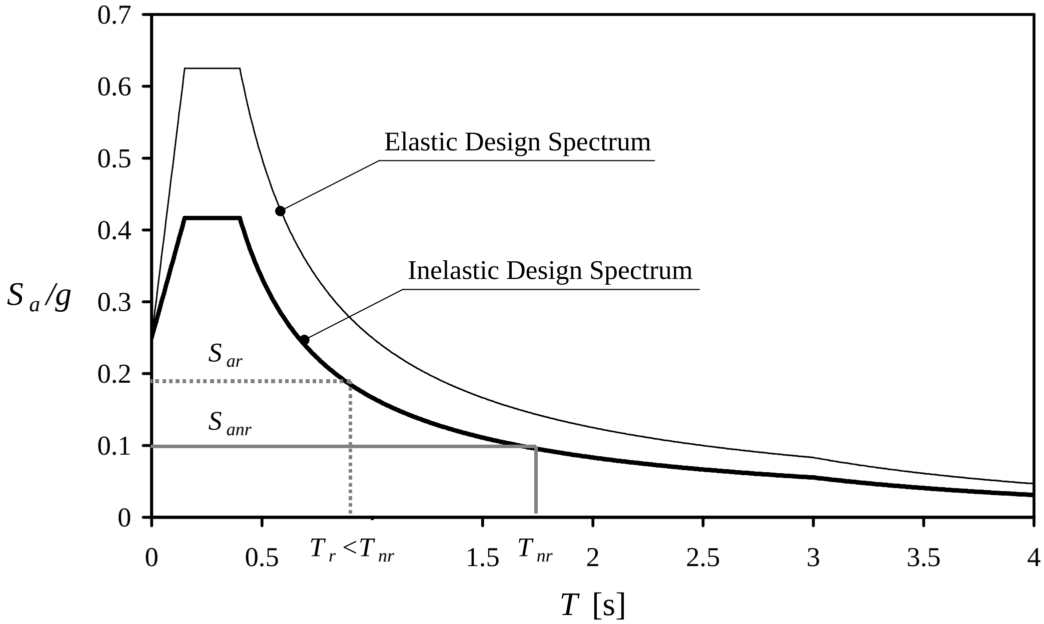


Fig. 4 Increase of the seismic demand following an increase of seismic resistance

Only after stiffness and strength have been increased up to a level where the fundamental period corresponds to the constant branch of the design spectrum, is it possible to achieve a condition where the design inequality is satisfied. It is, therefore, evident that an attempt to increase the seismic resistance capacity in this way only results in an increase of the seismic demand. When, in the end, the procedure converges, it is at the expense of a considerable expenditure of resources.

A similar situation occurs with reference to mass reduction. This may be achieved, for instance, by removal of one or more storeys as shown in Figure 5. In this case it is evident that the removal of the mass will lead to a decrease in the period, i.e. $T_r < T_{nr}$, which will lead to an increase in the required strength, i.e. $S_{ar} > S_{anr}$. Therefore the advantage acquired by the mass reduction is partially cancelled by the period shortening through the increase in the demand as shown in Figure 4.

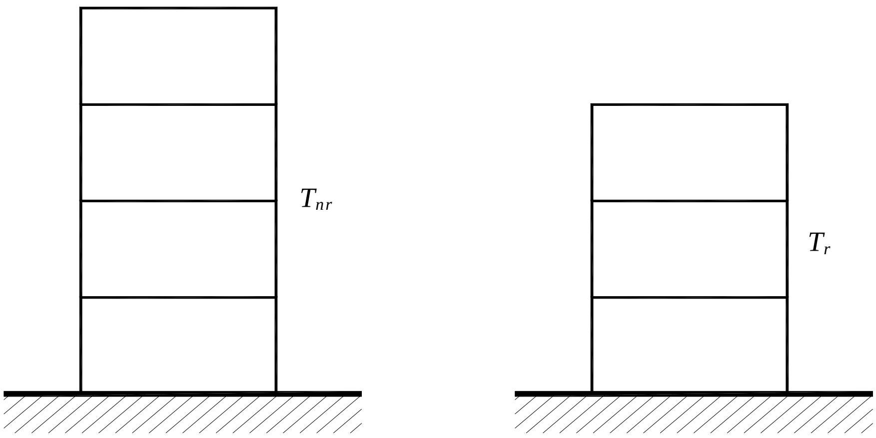


Fig. 5 Seismic retrofitting by mass reduction (removal of a storey)

In conclusion, both of the traditional methods of seismic retrofitting, although effective, are rather expensive. It must however be pointed out that, as in the case of low buildings, the fundamental period may already fall within the constant branch of the design spectrum and a period shortening may not result in an increase of the seismic action. Another situation when traditional methods of seismic retrofitting may be rather effective is in the case of soft soil conditions, where higher spectral ordinates occur at relatively longer periods, see for instance Clough and Penzien (1993) (Chapter 25, Sections 2 and 3) and the design spectra of Eurocode 8 (CEN, 1998).

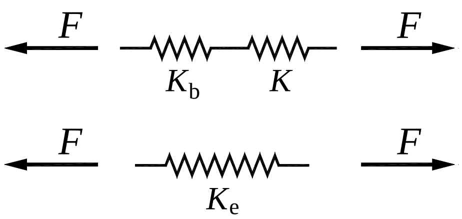
INNOVATIVE APPROACHES TO SEISMIC RETROFITTING

The main innovative methods of seismic retrofitting may be grouped into the following classes:

- Stiffness reduction
- Ductility increase
- Damage controlled structures
- Composite materials
- Any suitable combination of the above methods
- Active control.

For equal mass the 'stiffness reduction' produces a period elongation and a consequent reduction of the seismic action and therefore of the seismic strength demand. The stiffness reduction may be achieved by the principle of springs in series whereby the equivalent stiffness of two springs in series is smaller than either of the single springs as shown in Figure 6. In general it may be assumed that base isolation is a special case of the stiffness reduction approach. Although very effective, this method must be used with a pinch of salt. Too low a stiffness may result in large displacements, especially inter-story drifts, which may conflict with the functioning of the building and cause damage to non-structural components. Therefore deformability checks are always a must. Instances in which this method may not be effective are the cases of long period structures or of stiff structures on soft soils. In the first case the advantages gained by a reasonable increase in period may be negligible; in the second case the stiffness reduction may be counterproductive by leading to an increase of spectral ordinates. An application of the 'stiffness reduction method' will be shown in some detail in a further section.

A 'ductility increase' may be achieved locally by confinement of reinforced concrete flexural as well as compressed structural members. Although this method has a long history, it may now be applied easily using new materials such as fibre reinforced polymers (FRP). These materials are distinguishable by the type of fibre and the most common are denoted by CRP, GRP, ARP, indicating respectively reinforcement with carbon (C), glass (G) and aramidic (A) fibres.



$$\frac{1}{K_e} = \frac{1}{K_b} + \frac{1}{K}$$

$$K_e < K_b \qquad K_e << K$$

Fig. 6 Stiffness reduction by the principle of springs in series

One of the most important developments to surface in earthquake engineering in the last 10 years is the introduction of the concept of designing 'damage controlled structures' (Huang et al., 2001). According to this concept the structural system consists of two parallel structures as shown in Figure 7.

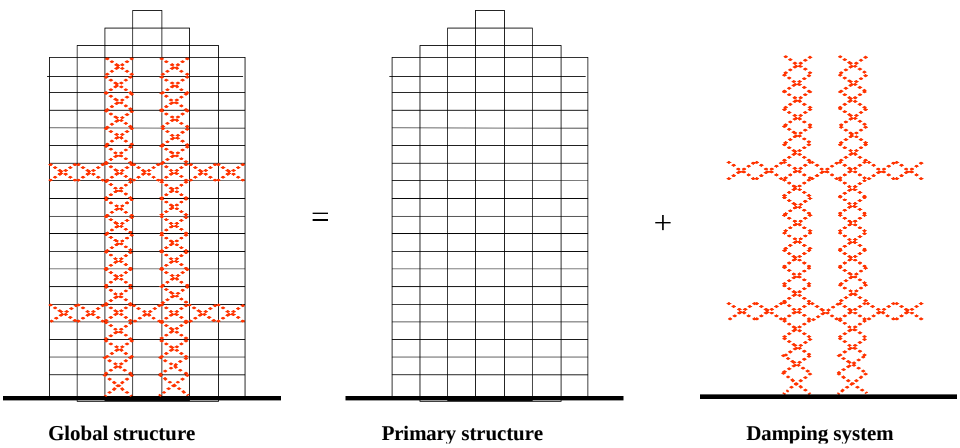


Fig. 7 Damage controlled structure

The primary structure will behave elastically under the most severe design earthquake while the auxiliary structure, shown by the damping system in Figure 7, will respond to the seismic action. The concept is applicable to new as well as to old buildings. The auxiliary structure introduces a stiffness increment and a large energy dissipation capacity. Damage occurs only in the auxiliary structure in which damaged elements may be replaced after the earthquake. It is important to realize that, with this seismic design criterion, the structure remains operative even under the most severe design earthquake. A comparison of the behaviour of a traditional system and of a damage controlled system is shown in Figure 8.

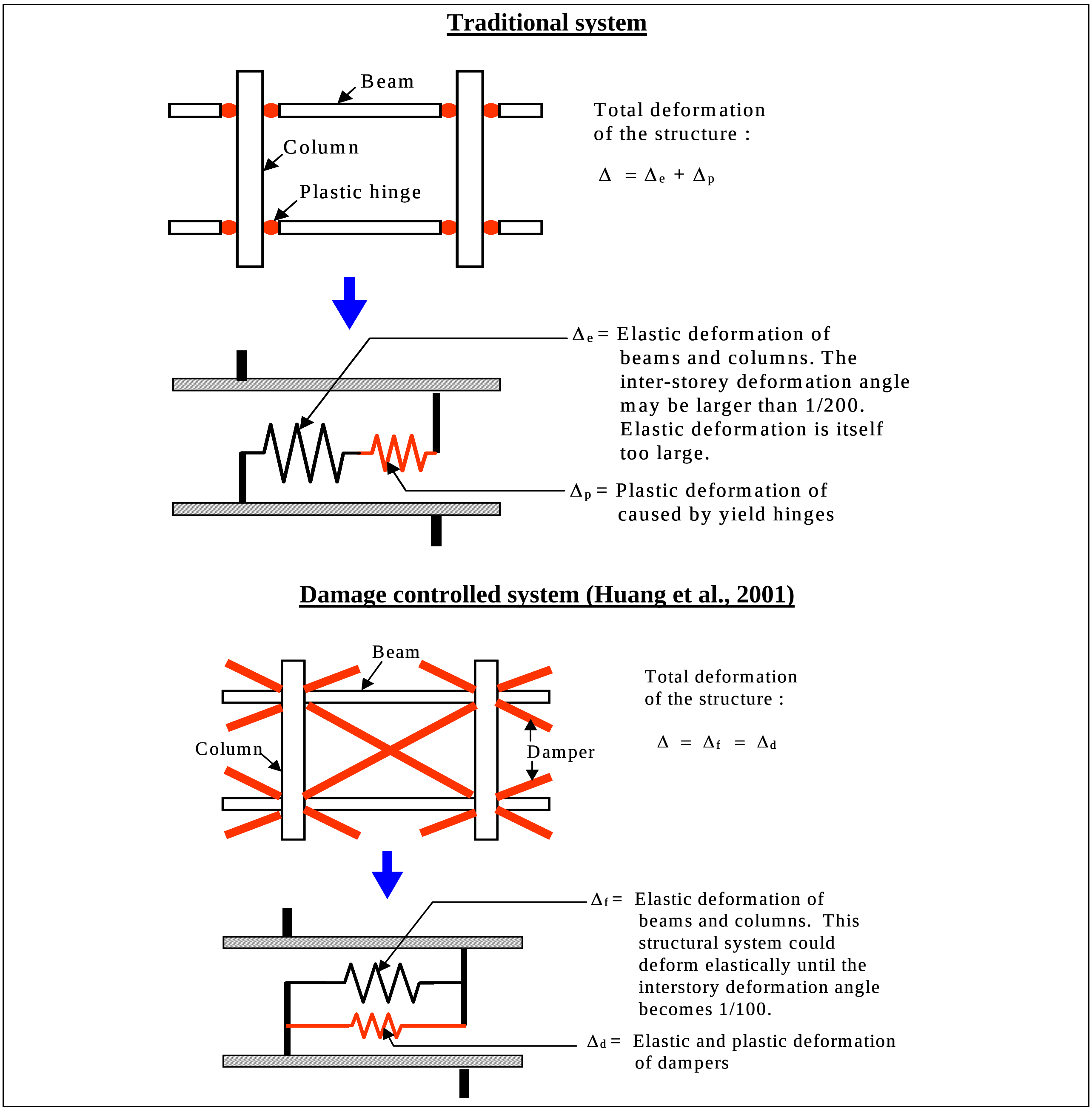


Fig. 8 Comparison of a traditional system and a damage controlled system

In the traditional system, elastic deformations of beams and columns and plastic deformations occur in series, so that the total deformation is the sum of the two. The total deformation, measured as an angular inter-story drift, may be larger than 1/200 and therefore too large to be tolerated without serious damage (see explanatory graph in Figure 8).

In the damage controlled system, the primary structure and the damping system are in parallel, so that the total deformation is the same for both. In this system the primary structure can deform elastically until the angular inter-story drift is as large as 1/100. Plastic deformations occur in devices specially designed for such a purpose without affecting the primary structure.

A comparison from the viewpoint of the constitutive behaviour is shown in Figure 9. Under a small or moderate earthquake the traditional system behaves elastically, while under a severe earthquake it undergoes large elastic and plastic deformations and the structure as a whole may be so damaged as to be no longer operative. With the damage controlled system, even under small or moderate earthquakes, while the primary structure remains elastic, the auxiliary structure reacts to the seismic action by dissipating an amount of energy proportional to the deformation amplitude. Under a severe earthquake the primary structure continues to behave elastically while the auxiliary structure dissipates a much larger amount of energy than in the previous cases. After the earthquake the primary structure will continue to be operational and at most it might be necessary to replace damaged elements in the auxiliary structure.

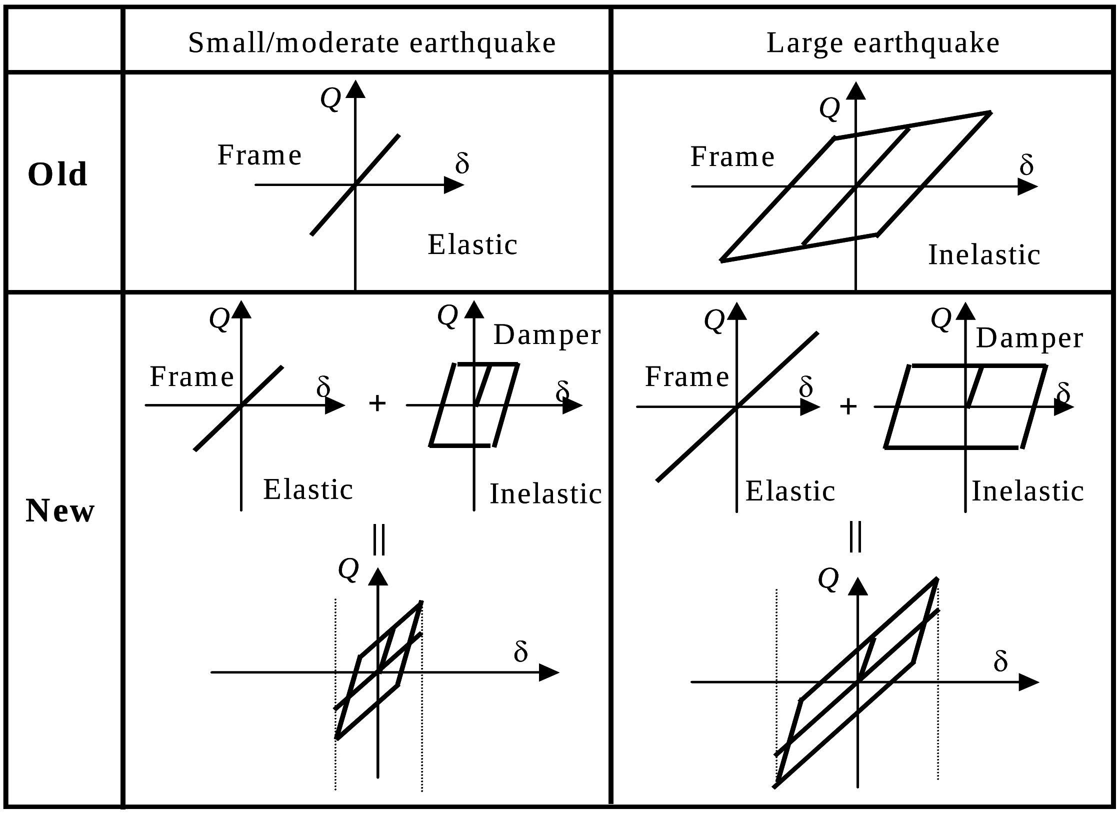


Fig. 9 Comparison of old system with new system (Huang et al., 2001) on a constitutive level

While there is no doubt that this concept can be applied advantageously for new buildings, its application to existing buildings may give rise to a few problems. The compatibility of the auxiliary structure with the primary structure must be carefully ascertained. The auxiliary structure will change the stress distribution in the primary structure and this must be capable of withstanding the new stress distribution. Also for the auxiliary structure to become effective, some flexibility may be required in the primary structure. It would be of no use if the auxiliary structure started to work when the primary structure was already seriously damaged.

Composite materials promise interesting applications for the seismic retrofitting of old constructions, especially masonry. Crack opening with associated degradation of strength and stiffness constitutes a major limitation for the use of masonry in seismic areas. The application of composite laminae to masonry panels confers a strong traction resistance to masonry, limiting crack extension and width and favouring the closure of open cracks. The phenomenon of strength and stiffness degradation is, therefore, strongly reduced, if not removed.

Active control is performed by means of servo-actuated devices capable of applying opposite forces to the seismic action. Sensors are required to read the motion of the structure; and hardware and software are required for the evaluation of forces to be applied for the minimization of the structural response. An energy source must always be available for the functioning of all the equipment including the generation of the controlling forces. These systems are still in the research stage and no significant application yet exists in the field of earthquake engineering. At the moment their application in practice appears somewhat dubious.

EVALUATION OF SEISMIC RESISTANCE AND VULNERABILITY

1. Definition of SDOF Equivalent Systems

The seismic resistance and, consequently, vulnerability of reinforced concrete constructions may be evaluated by means of a procedure proposed within some documents of the Federal Emergency Management Agency (BSSC, 1997a, 1997b). These documents have been subsequently upgraded to pre-standard level, FEMA 356 (BSSC, 2000); however, while document FEMA 356 (BSSC, 2000) is intended to supersede document FEMA 273 (BSSC, 1997a), document FEMA 274 (BSSC, 1997b) remains the basic commentary also to the pre-standard. The FEMA procedure has been modified by some research work carried out at the University of Catania (Oliveto et al., 2001). The results that will be obtained within the present paper use the modified procedure. An elastic-plastic incremental analysis of the structure under the seismic action is a necessary prerequisite. The seismic action is defined in terms of the forces corresponding to the first few modes of vibration of the structure or in terms of the pseudo-static forces prescribed by seismic regulations. The results of the incremental analysis come in the form of storey force-displacement curves commonly known as push-over curves. On the basis of these curves a single-degree-of-freedom (SDOF) equivalent system is defined.

Before describing the procedure in some detail it is appropriate to notice that the procedure may be used for the evaluation of the seismic resistance of existing buildings as well as that of new ones (in the design stage). As such the procedure may also be used for the evaluation of the effectiveness of seismic retrofitting projects. Figure 10 shows a reinforced concrete building before and after retrofitting according to the stiffness and resistance increment concept. Besides demonstrating the type of retrofitting system which has been used in this case, the pictures illustrate the complexity of the structure on which the incremental analysis must be performed.

Fig. 10 Building owned by IACP of Syracuse in the Saline district of Augusta (seismic retrofitting by increment of strength and stiffness provided by newly inserted reinforced concrete cores)

Further details on the procedure used for the design of the retrofitting systems for a class of buildings of the type shown in Figure 10 may be found in Oliveto and Decanini (1998). For the sake of clarity it should be noted that the building in Figure 10 was retrofitted in the early nineties, before the FEMA procedures became available and before the subsequent studies by the senior author and his co-workers. The building is shown here to provide an example of seismic retrofitting by increase of resistance and stiffness and to illustrate the complexity of systems on which push-over analyses must be performed. This is the reason why the push-over analysis described below was not performed on this building but on a four storey building described in detail in Oliveto et al. (2001).

The storey force-displacement (push-over) curves have been constructed using commercial and research computer programs. The use of commercial programs has been undertaken in order to ensure a quick transfer of the research results to the seismic engineering profession. More details and the relevant literature may be found in Oliveto et al. (2001). The analyses have been performed along two orthogonal directions roughly corresponding to the axes of symmetry of the plan of the building; in fact the chosen directions were those of the corresponding first modes of vibration of the building. The analyses have been performed, using approximations described in detail in Oliveto et al. (2001), on 3D models of the buildings considered.

The results of the push-over analyses are shown in Figure 11. Here the storey force-displacement curves are shown for each of the storeys of the building considered, together with the work performed by the storey forces as functions of the base shear of the building. Because the floors are considered as rigid for in-plane strains and the building is nearly symmetrical, any floor point may be considered in the construction of the storey force-displacement curves in Figure 11. For each step of the incremental (push-over) elastic-plastic analysis the storey forces are known and the corresponding floor displacements are calculated. The analysis is stopped when the first plastic hinge breaks, on the assumption that this leads to a stress redistribution and subsequent plastic hinge failures as in a chain reaction. The displacement of the SDOF equivalent system is evaluated on the basis of the work equivalence. The equivalence is established in incremental as well as in global terms and the result is shown in Figure 12. The shaded area in Figure 12 is the sum of the shaded areas in Figure 11. The work equivalence defined above is not limited to symmetrical buildings with in-plane rigid floor slabs, but can be established for any structural system. A mathematical equivalence for general multi-degree-of-freedom (MDOF) systems may be found in Oliveto et al. (2004a).

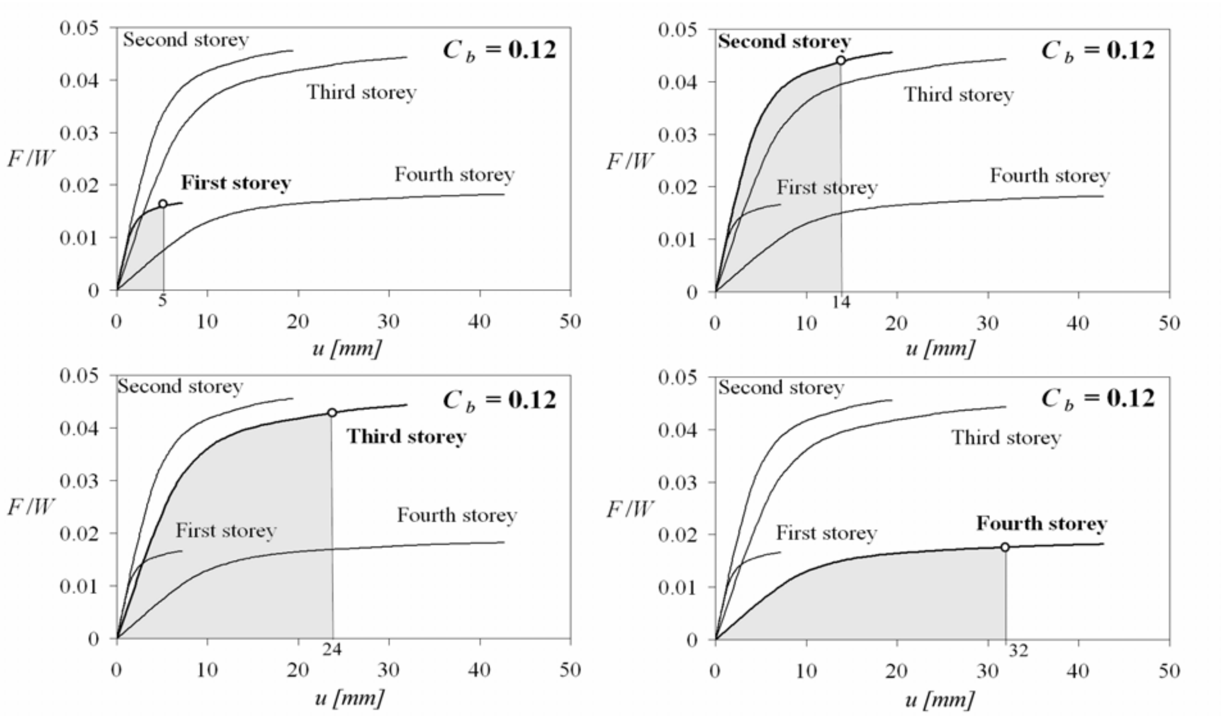


Fig. 11 Storey force-displacement (push-over) curves for the construction of the equivalent SDOF system

The graph in Figure 12 defines the equivalent SDOF system of the building in terms of base shear and the corresponding displacement as established by using work equivalence. Perhaps it may be worth noticing at this point that the base shear coefficient $C_b = 0.12$ in Figure 11 and in Figure 12 is an arbitrarily chosen value of the ratio between the base shear force and the weight of the building in the interval $0 \le C_b \le C_{b,c}$ with $C_{b,c} = 0.125$ being the collapse base shear coefficient. Given the weight *W* of the building, to each C_b there corresponds a specific base shear force and specific storey forces as shown in Figures 11 and 12 for $C_b = 0.12$.

Obviously the equivalent SDOF system should be defined for the two principal directions of the building. Therefore at least two equivalent SDOF systems of the form shown in Figure 12 must be evaluated for each building according to the previously outlined procedure.

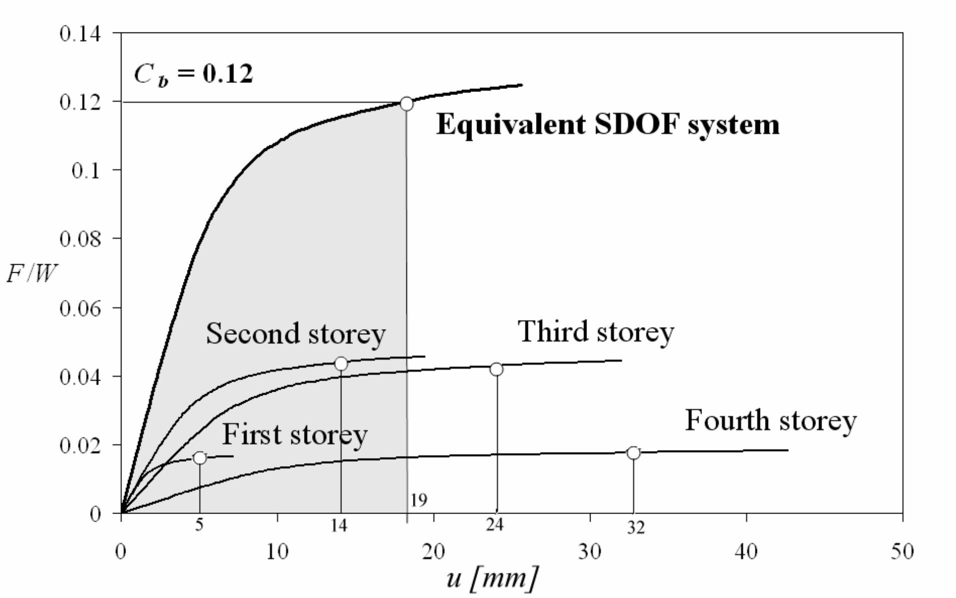


Fig. 12 Evaluation of the equivalent SDOF system on the basis of the storey force-displacement (push-over) curves

2. Seismic Resistance in Terms of Effective Peak Ground Acceleration (PGA)

The characteristics of the force-displacement curve of the equivalent SDOF system are used to establish the seismic resistance of the building in terms of the effective PGA. For this some preliminary considerations relating to the design spectrum and to the interaction between the spectral ordinates and the PGA are required. This interaction is clearly shown in Figure 13.

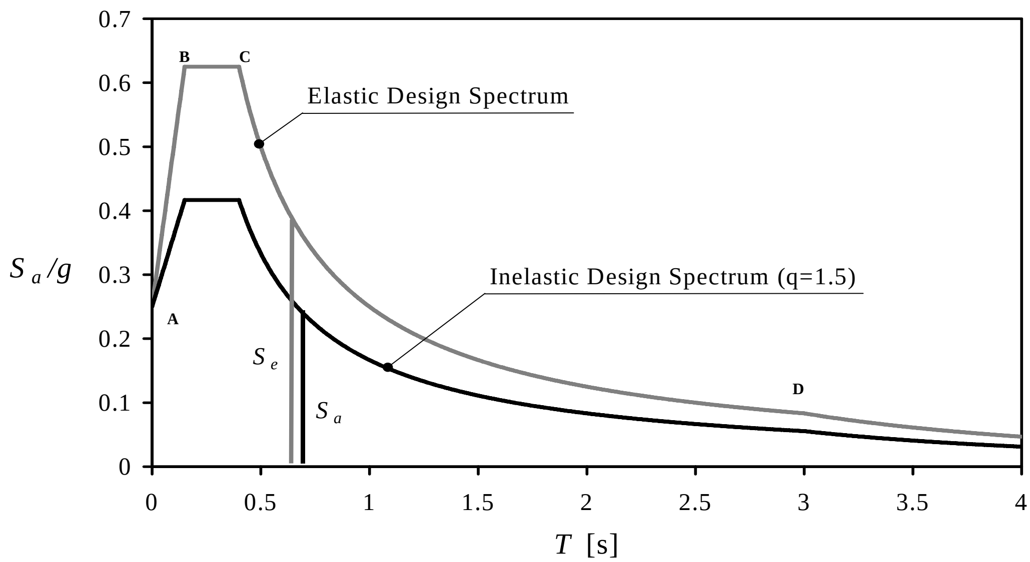


Fig. 13 Elastic and inelastic design spectra

The ordinate of the inelastic spectrum S_a is related to the effective PGA a_g through the equation:

$$S_a = a_g S \beta_0 \frac{f(T_{eff}, S, \zeta)}{q} \tag{2}$$

where S is a local site condition factor, β_0 is a dynamic magnification factor usually set equal to 2.5, $f(T_{eff}, S, \zeta)$ is a function defining the spectral shape, ζ is the damping ratio and q is the structure behaviour factor defined as:

$$q = \frac{S_e}{S_a} \tag{3}$$

The relationship between the spectral ordinates and the characteristics of the equivalent SDOF system is easily established if one recalls that the base shear coefficient occurring on the ordinates of curves of Figure 12 and Figure 13 is defined as:

$$C_b = \frac{S_a}{g} = \frac{a_g}{g} S \beta_0 \frac{f(T_{eff}, S, \zeta)}{q} \tag{4}$$

It is evident from Equation (4) that if $C_b, S, \beta_0, f(T_{eff}, S, \zeta)$ and q were known, the only unknown would be a_g. As a matter of fact S and β_0 are known, while the spectral shape $f(T_{eff}, S, \zeta)$ is a known function of T_{eff}, S and ζ. The problem may be considered to be well defined if it is possible to determine the values of C_b, T_{eff} and q. The SDOF equivalent system defined by the force–displacement curve of Figure 12 is used for this. The evaluation may be performed by using a procedure which may be described with the help of Figure 14.

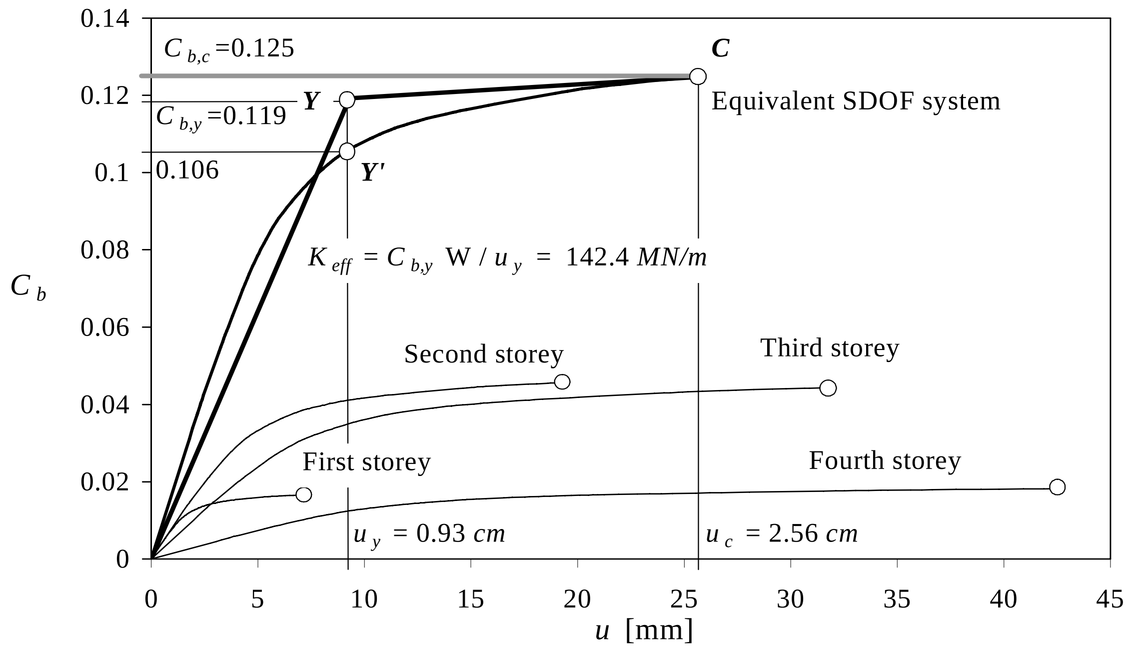


Fig. 14 Substitution of the equivalent SDOF system by a bi-linear system

The first operation that must be performed on the equivalent SDOF system is the substitution of the continuous non-linear curve with a bi-linear one. Of the two linear segments, the first one is considered elastic while the second is elastic-plastic with hardening. The substitution is achieved by using the work equivalence and the condition that the second linear segment should be tangent to the actual curve at point *C*. In this way three characteristic points are identified, *Y’*, *Y* and *C*, two of which belong to the original system, that is *Y’* and *C*, and two belong to the new one, that is *Y* and *C*. Point *C* corresponds, at the same time, to the maximum base shear and to the corresponding equivalent deformation. The value $C_{b,c}$ of the base shear coefficient corresponding to point *C* defines one of the unknown parameters in Equation (4). Point *Y*, corresponding to the vertex of the bi-linear system, is related to the definition of the effective elastic stiffness of the equivalent SDOF system. As shown in Figure 14, this may be evaluated as:

$$K_{eff} = C_{b,y} \frac{W}{u_y} \tag{5}$$

where $C_{b,y}$ is the base shear coefficient corresponding to first yielding and u_y is the corresponding displacement. Both of these may be obtained as coordinates of point *Y*, while *W* is the seismic weight of the building. Having evaluated the effective elastic stiffness of the equivalent SDOF system, it is straightforward to evaluate its effective period, which is given by the formula:

$$T_{eff} = 2\pi \sqrt{\frac{W}{K_{eff} \cdot g}} \tag{6}$$

The structure behaviour factor q, which is related to the ductility factor, still remains to be evaluated. The ductility factor of an elastic-perfectly plastic SDOF system is defined as the ratio between the collapse displacement u_c and the first yielding displacement u_y:

$$\mu = \frac{u_c}{u_y} \tag{7}$$

In the case of a hardening elastic-plastic system the definition is not strictly applicable since the dissipated energy is less than that of an elastic perfectly-plastic system with a yield strength equal to the maximum strength of the first system. A work-equivalent elastic-perfectly plastic system could be defined and the corresponding equivalent ductility factor determined (Marletta, 2002; Oliveto et al., 2004a). However in the systems considered, plastic hardening is very small and the error in using Equation (7) is well within engineering uncertainties. The structure behaviour factor q, or equivalently the response reduction factor R_y, is a function of μ and T_{eff} (Chopra, 2001: Chapter 7, Section 11). Here it is sufficient to note that the following holds:

$$q = \mu; \quad \text{for } T_{eff} \geq T_C \tag{8}$$

where T_C is the vibration period that separates the acceleration-sensitive region from the velocity-sensitive region of the elastic design spectrum, corresponding to point *C* in Figure 14. Eurocode 8 provides the value of q in a different but, nevertheless, almost equivalent manner.

Once all the required parameters have been evaluated, the seismic resistance of the building in terms of effective PGA is given by the expression:

$$\frac{a_{g,b}}{g} = \frac{qC_{b,c}}{S\beta_0 f(T_{eff}, S, \zeta)} \tag{9}$$

It should be noticed that Equation (9) accounts for the local site conditions through the coefficient S that modifies the spectral shape $f(T_{eff}, S, \zeta)$ and the spectral amplitude $S\beta_0$.

2.1 A Note on the Spectral Shape Function $f(T_{eff}, S, \zeta)$

The spectral shape function $f(T_{eff}, S, \zeta)$ appearing in Equation (9) has different expressions for the elastic and the inelastic design spectra in the range of periods $T \geq T_C$ according to Eurocode 8 (CEN, 1998). In the same range of periods, relevant literature (Chopra, 2001) suggests using the same spectral shape for elastic and inelastic behaviour. For the sake of simplicity the elastic spectral shape of Eurocode 8 has been used in the following numerical applications.

3. Seismic Resistance and Vulnerability

The seismic resistance defined in terms of effective PGA by means of Equation (9) represents a measure of the maximum ground motion that a building can withstand at the threshold of collapse. It is interesting to compare this value with the corresponding value that the seismic regulation prescribes for the construction site. By denoting the latter with $a_{g,c}$ the relative seismic resistance may be defined as:

$$R = \frac{a_{g,b}}{a_{g,c}} \tag{10}$$

It is evident that the building can withstand the seismic action if and only if $R \geq 1$. If instead, $R < 1$, the building is vulnerable to the design earthquake. Therefore the seismic vulnerability and over-resistance may be defined respectively as:

$$V = 1 - R; \quad \text{for } R \leq 1 \tag{11}$$

$$OR = R - 1; \quad \text{for } R > 1 \tag{12}$$

4. Application to Buildings in the Village of Solarino

The procedure described in the previous sub-sections has been applied to two almost identical buildings in the village of Solarino of the province of Syracuse. The relevant results of the push-over analyses and other properties of the buildings are shown in Table 1 (Marletta, 2002). The meaning of the symbols is the same as introduced in Sub-section 2 above and shown in Figure 14. The ductility ratio shown in Table 1 was evaluated with the refined method proposed by Marletta (2002) and Oliveto et al. (2004a) and is somewhat smaller than the value predicted by Equation (7).

Table 1: Results of the Push-over Analyses Performed on the Original Solarino Buildings

Direction	Transverse	Longitudinal
First Effective Yielding Seismic Coefficient, $C_{b,y}$	0.079	0.124
Collapse Seismic Coefficient, $C_{b,c}$	0.088	0.160
First Effective Yielding Displacement, u_y **(cm)**	1.43	2.15
Collapse Displacement, u_c **(cm)**	3.16	4.78
Mass, M **(kN·s^2/m)**	578	578
Effective Stiffness, K_{eff} **(MN/m)**	32.0	33.1
Effective Period, T_{eff} **(s)**	0.85	0.84
Ductility Ratio, μ	1.98	1.92

Table 2: Relative Seismic Resistance *R* (%) for the Building of Figure 15

Seismic Zone	Soil Type	Transverse Direction	Longitudinal Direction
High Seismic Hazard	A	43	66
	B	28	44
	C	24	36
Medium Seismic Hazard	A	60	92
	B	40	61
	C	33	51
Low Seismic Hazard	A	100	153
	B	66	102
	C	55	85

According to present seismic regulations the building site is in an area of medium seismic hazard and local site conditions may be classified as Type A according to Eurocode 8. For research purposes the analysis has also been repeated for the zones of low and high seismicity and for the soil conditions of Types B and C, thus covering the complete spectrum of seismicity and site conditions covered by Eurocode 8. The results in terms of relative seismic resistance are shown in Table 2. The regional seismic hazard is specified in Italy in terms of effective PGA as follows: PGA = 0.35*g* for the areas of high seismic hazard, PGA = 0.25*g* for the areas of medium seismic hazard, and PGA = 0.15*g* for the areas of low seismic hazard. Just recently a fourth area of minimal seismic hazard has been proposed with PGA = 0.05*g*.

From an examination of Table 2 it appears that the building would be vulnerable to the design earthquake independently of local soil conditions if located in a zone of high or medium seismic hazard. If the building were situated in a zone of low seismic hazard it would be able to withstand the design seismic action only on firm soil, that is Type A soil condition. From the same table it may be seen that the transverse direction is the one with less seismic resistance. The same results in terms of seismic vulnerability are given in Table 3.

Table 3 provides a vulnerability index for the building with reference to the design earthquake. The '0' value shows that the building is not vulnerable while '1' (100%) indicates that the building has no seismic resistance. Intermediate situations have an obvious meaning. Data on seismic over-resistance has not been shown because it was available in just one case.

Table 3: Seismic Vulnerability *V* (%) for the Building of Figure 15

Seismic Zone	Soil Type	Transverse Direction	Longitudinal Direction
High Seismicity	A	57	34
	B	72	56
	C	76	64
Medium Seismicity	A	40	8
	B	60	39
	C	67	49
Low Seismicity	A	0	0
	B	34	0
	C	45	15

It is, at this point, appropriate to point out that the seismic resistance evaluated using the procedure outlined above may represent an upper limit of the actual resistance exhibited by the building because the computer programs for non-linear analyses nowadays do not account for some of the damage mechanisms occurring in the actual behaviour. For instance, most commercial programs do not account for beam-column joint failure, often observed in post-earthquake inspection, or for reinforcement slippage. Furthermore the conditions that lead to plastic hinge formation in general account for the interaction of force resultants only in an approximate way. Nevertheless the procedure presented here, coupled with engineering judgment, provides a useful tool in the design office. From the examples presented it is easy to appreciate that the procedure can also be used to check the resistance of a new building during the course of the design process.

SEISMIC RETROFITTING BY STIFFNESS REDUCTION

The buildings owned by IACP of Syracuse in the village of Solarino, the seismic vulnerability of which has been evaluated in the previous section, have been considered for seismic retrofitting by means of stiffness reduction, and one of the original buildings is shown in Figure 15. The IACP buildings in Solarino seemed to invite the designer to retrofit by stiffness reduction. In fact, by looking at the original foundations shown in Figure 16, it was clear how easy it would be to support the building, to cut the short columns between the foundation and the first floor slab, and to insert the devices that would ensure the stiffness reduction. Also, a detailed geological study confirmed the rocky nature of the foundation soil, thus excluding high long period components in the expected ground motion and confirming Class A soil condition according to Eurocode 8 (CEN, 1998). The devices for stiffness reduction as used in the present case are shown in Figure 17.

As may be seen from Figure 18 the building is supported by 12 elastomeric bearings and by (9+4 = 13) low-friction bearings. The elastomeric bearings, commonly known as seismic isolators, besides contributing to the stiffness reduction, introduce also a significant energy dissipation capacity. The low-friction bearings, which could rightly be called seismic isolators, have the function of transmitting vertical loads to the foundation, while limiting any possible horizontal action to the bare minimum.

Preliminary investigations on materials and structural members have shown an excessive deformability at local and global levels, so much so that the structure would not have been safe under gravity and seismic loads even after the stiffness reduction. For this reason a further retrofitting action has been undertaken to reduce this high deformability. The proposed design is shown in Figure 19.

Fig. 15 Building owned by IACP of Syracuse in the village of Solarino

Fig. 16 Foundations of an IACP building in Solarino

Elastomeric Bearing Devices Low-Friction Bearing Devices

Fig. 17 Stiffness reduction devices used for the seismic retrofitting of two IACP buildings in Solarino

The building stiffening by thin reinforced concrete walls, of thickness 15 cm, allows not only for an improvement of the vertical load carrying capacity and for the deformability limitation, but also for a much better behaviour of the stiffness reduction mechanism and of the entire building. It should be noticed here that the inserted reinforced concrete walls stiffen and strengthen only the superstructure while the overall stiffness is essentially determined by the base isolation system. Therefore the overall system, while attracting lower seismic forces, is better suited to withstand the seismic forces affecting the superstructure with well-controlled inter-storey drifts.

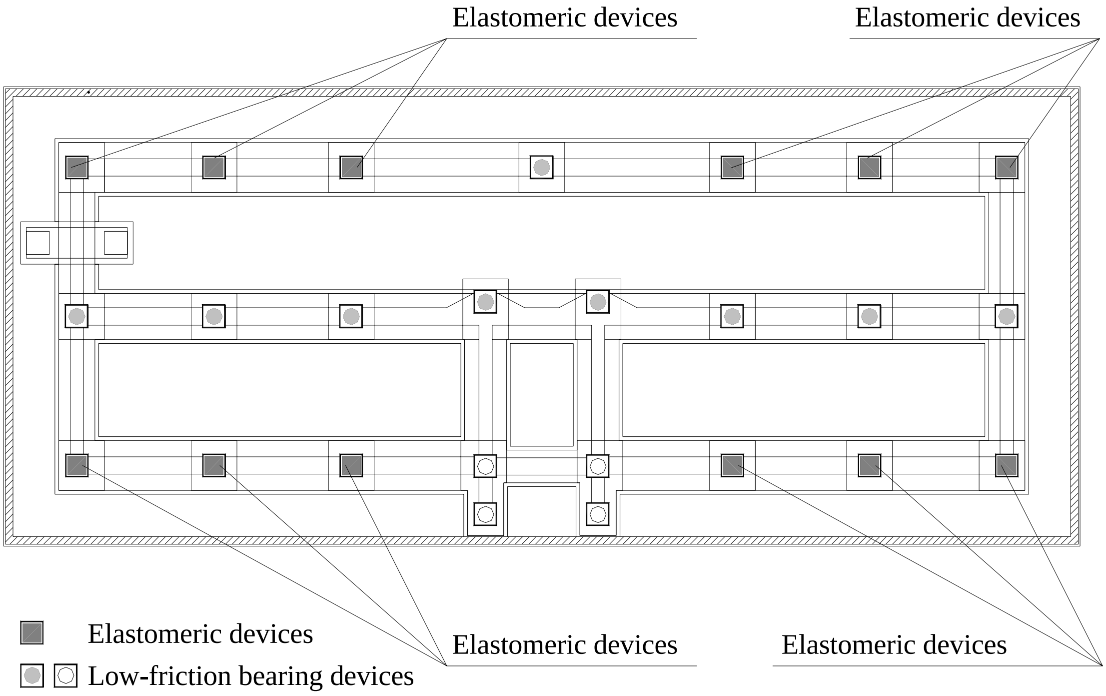


Fig. 18 IACP building in Solarino with devices for stiffness reduction

1. Modelling of the Retrofitted Building

The same procedure used for the evaluation of the seismic resistance and vulnerability of the existing building was usefully employed for the safety check of the retrofitted building. In the case being considered, as well as in modelling the one-dimensional structural members, the problem of modelling the reinforced concrete walls also arises. A detailed description of the modelling of the retrofitted building as well as that of the existing one may be found in Marletta (2002); here only the fundamental aspects will be considered briefly. The reinforced concrete framework has been modelled using the commercial program SAP-2000 while for the walls the parallel multi-component discrete model by Vulcano and Bertero (1986, 1987) has been used. The constitutive model of the horizontal shear component of this model is qualitatively shown in Figure 20.

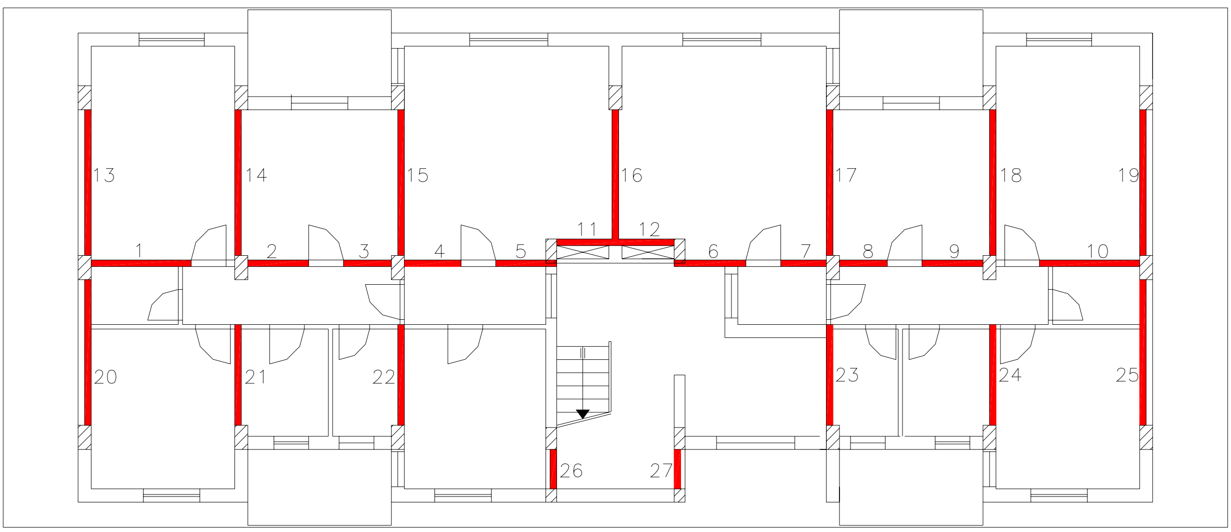


Fig. 19 Deformability reduction by reinforced concrete thin walls; first storey

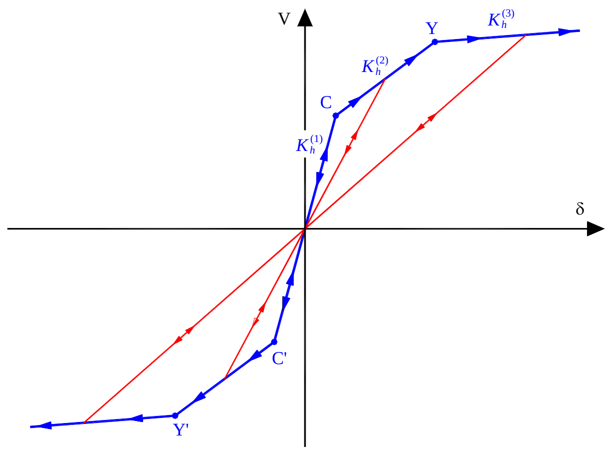


Fig. 20 Constitutive model of the horizontal shear component of the MCP model

The global model, including the original structural framework, the thin stiffening walls and the devices for stiffness reduction has been constructed by using the commercial structural program SAP-2000. This model has been used to produce the push-over curves by non-linear incremental analyses. Because of the presence of the stiffness reduction devices, it is not advisable to try to replace the overall model by an equivalent SDOF system. It is preferable to reduce the structural model to a two-degrees-of-freedom (2-DOF) system that accounts for the deformation of the superstructure and of the stiffness reduction devices (seismic isolators). The proposed model is, therefore, the one shown in Figure 21. In this model the non-linear relationship existing between displacement-resistant forces and displacements for the base isolation system is clearly visible. Also the equivalent damping ratio is specified by the bearing supplier as a non-linear function of the vibration amplitude. Both stiffness k_f of the equivalent spring and damping coefficient c_f of the equivalent damper depend on the amplitudes of displacement. The characteristics of the superstructure have been evaluated by means of the push-over analysis shown in Figures 22 and 23 with reference to the transverse direction. The push-over analysis has been terminated when the limit displacement has been reached in the base isolation bearings. The push-over analysis in the longitudinal direction provides similar results (Marletta, 2002).

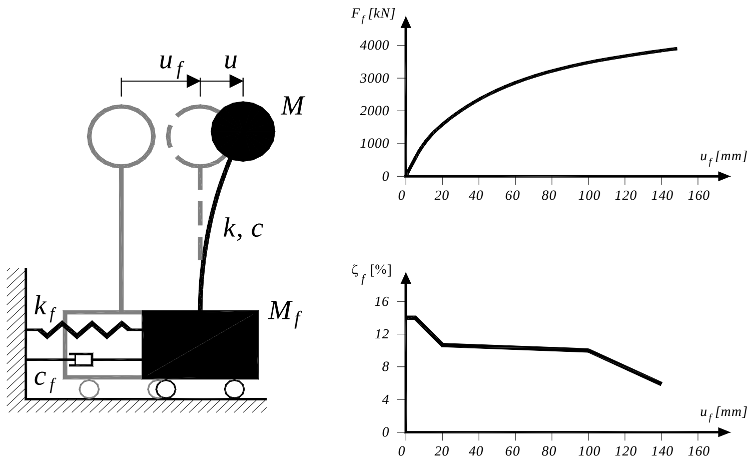


Fig. 21 2-DOF equivalent model for the retrofitted system

The characteristics of the superstructure may be summarized as follows:

- fundamental period in the longitudinal direction $T_l = 0.140$ s
- fundamental period in the transverse direction $T_t = 0.132$ s
- global ductility ratio in both directions $\mu = 1$

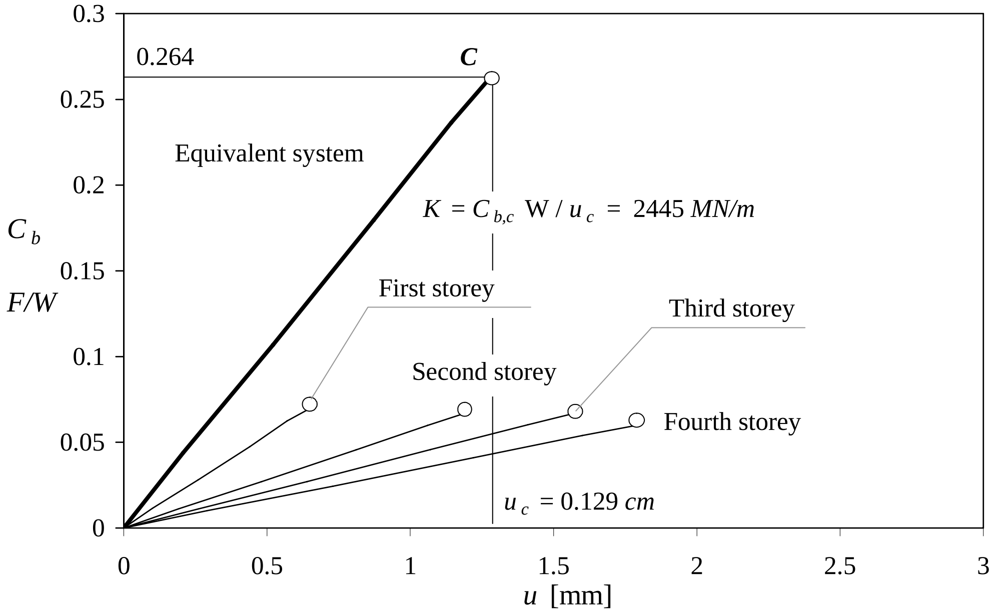


Fig. 22 Push-over analysis on the retrofitted Solarino building: superstructure

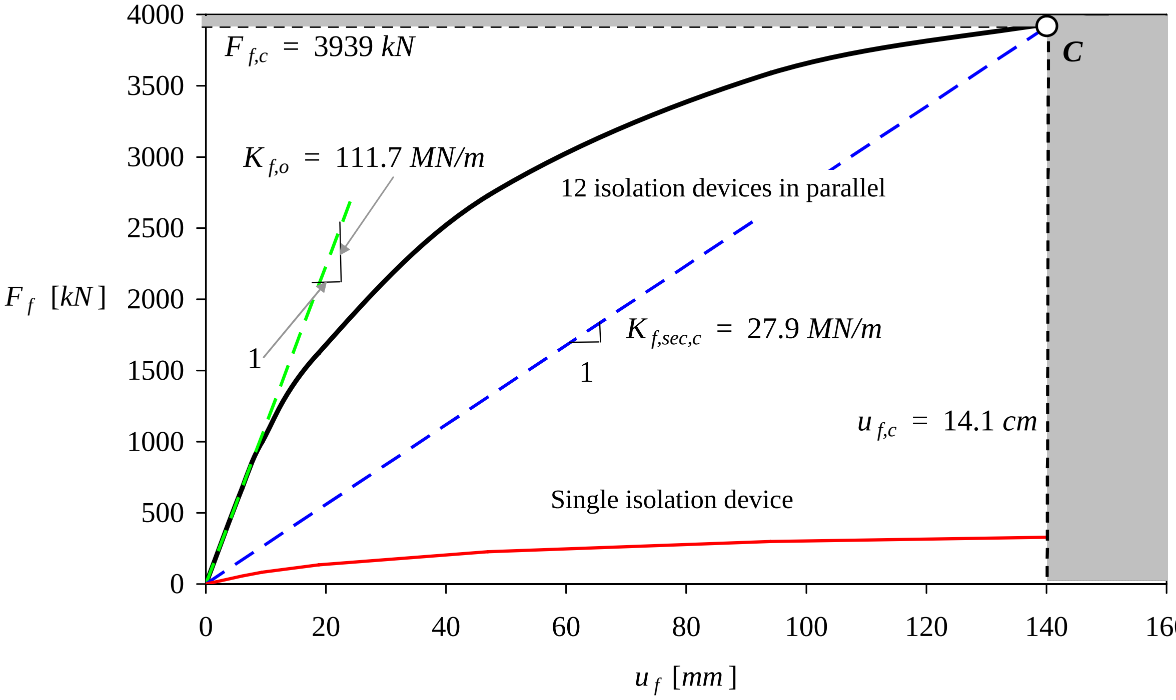


Fig. 23 Push-over analysis on the retrofitted Solarino building: base isolation system

The effective periods above may seem at first sight too short. However the building still appears to behave linearly when the limit displacement for the base isolation system is reached (Marletta, 2002). This explains also the unitary value of the ductility ratio μ (see Figure 22).

The global characteristics of the retrofitted system may be summarized as follows:

- initial fundamental period T_T = 0.743 s
- secant period at collapse T_S = 1.46 s
- displacement at incipient collapse u_{fc} = 14.1 cm

The periods shown above were determined (Marletta, 2002) using the force-displacement relationship provided by the supplier for the base isolation bearings chosen. The displacement at incipient collapse is the limit displacement provided by the bearing supplier. It should be observed at this point that the friction force introduced by the low-friction bearings has been considered as negligible in the above considered push-over analyses.

2. Resistance and Vulnerability

The resistance analysis conducted with the previously outlined procedure has produced the results as shown in Table 4. The force distribution used in the push-over analyses was again that corresponding to the first vibration mode in the direction considered, which in displacement terms is practically constant with all displacement occurring at the level of the base isolation bearings and the building essentially behaving as a rigid body.

For the three classes of seismic hazard considered in Italy, the results based on soil conditions of Type A, that is the soil condition existing at the construction site and best suited for retrofitting by the base isolation, are shown. Besides the results referring to the building retrofitted with walls and isolators (denoted by SR+W, the symbols indicating stiffness reduction plus walls), those referring to the original building are given for comparison along with those of the hypothetical building strengthened by the presence of the walls. It is evident that the retrofitted building has an over-resistance for all the classes of seismic hazard, which is obviously decreasing as the level of seismic hazard increases. Overall the building strengthened only with thin walls would be safe only in the areas of low seismic hazard. The situation shown in Table 4 in terms of seismic resistance is reconsidered in Table 5 in terms of vulnerability and in Table 6 in terms of over-resistance.

Table 4: Relative Seismic Resistance *R* (%) of the Retrofitted Building

Seismic Zone		Transverse Direction	Longitudinal Direction
High Seismicity	**Original Building**	43	66
	Building + Walls	75	60
	SR+W	135	135
Medium Seismicity	**Original Building**	60	92
	Building + Walls	104	84
	SR+W	189	189
Low Seismicity	**Original Building**	100	153
	Building + Walls	174	140
	SR+W	315	315

Table 5: Seismic Vulnerability *V* (%) of the Retrofitted Building

Seismic Zone		Transverse Direction	Longitudinal Direction
High Seismicity	**Original Building**	57	34
	Building + Walls	25	40
	SR+W	0	0
Medium Seismicity	**Original Building**	40	8
	Building + Walls	0	16
	SR+W	0	0
Low Seismicity	**Original Building**	0	0
	Building + Walls	0	0
	SR+W	0	0

It may be remembered here that the results for the original building are the same as shown in Sub-section 4 of the previous section, those for the building retrofitted only by walls (Building + Walls) were obtained in Marletta (2002), and those for the building retrofitted by base isolation and walls (SR+W) have been obtained considering the superstructure as elastic $(\mu = 1)$ and the secant period at the limit base displacement. It should also be mentioned that in the cases of the original building and of the building plus walls, a damping ratio $\zeta = 5\%$ was used in the definition of the spectral shape. In the case of the (SR+W) system an equivalent damping ratio $\zeta = 10\%$ was used to account for the damping of the base isolation system that consisted of high damping rubber bearings coupled with low-friction sliding bearings. For the sake of completeness, the characteristics of the (Building + Walls) system are shown in

Table 7. Again the refined method of Marletta (2002) and Oliveto et al. (2004a) was used for the evaluation of the ductility ratio.

Table 6: Seismic Over-resistance *OR* (%) of the Retrofitted Building

Seismic Zone		Transverse Direction	Longitudinal Direction
High Seismicity	**Original Building**	0	0
	Building + Walls	0	0
	SR+W	35	35
Medium Seismicity	**Original Building**	0	0
	Building + Walls	4	0
	SR+W	89	89
Low Seismicity	**Original Building**	0	0
	Building + Walls	74	40
	SR+W	215	215

Table 7: Results of the Push-over Analyses Performed on the Solarino Building + Walls

Direction	Transverse	Longitudinal
First Effective Yielding Seismic Coefficient, $C_{b,y}$	0.549	0.591
Collapse Seismic Coefficient, $C_{b,c}$	0.623	0.660
First Effective Yielding Displacement, u_y **(cm)**	0.57	0.48
Collapse Displacement, u_c **(cm)**	0.68	0.79
Mass, M **(kN·s²/m)**	1214	1214
Effective Stiffness, K_{eff} **(MN/m)**	1144	1458
Effective Period, T_{eff} **(s)**	0.20	0.18
Ductility Ratio, μ	1.05	1.48

3. Displacement Control

The analysis of the previous sub-section satisfies the resistance requirements of the building; however it might happen that the base displacement exceeds the limit displacement for the isolation bearings, or that the inter-storey drifts exceed tolerable values. The base displacement can be checked fairly easily by assuming that the displacement response is dominated by the first mode. The maximum displacement is provided by the relationship:

$$S_d = S_a(T)/\omega^2 = S_a(T)\frac{T^2}{(2\pi)^2} \tag{13}$$

where the period T takes a value within the interval:

$$T_T \le T \le T_S \tag{14}$$

Because for $T \le 3$ s the pseudo spectral acceleration varies with the reciprocal of T according to Eurocode 8 (CEN, 1998), it is obvious from Equation (13) that the maximum displacement occurs for $T = T_s$ and the limit displacement is reached for the base isolation system. From Table 6 it may be seen that the building has an over resistance whatever class of seismic hazard is considered in the Italian territory; therefore in each case the base displacement will be smaller than the limit displacement of the base isolation system, thus satisfying the fundamental design equation (see Equation (1)). The actual displacement, due to the non-linearity of the system force-displacement relationship, may only be calculated through an iterative procedure. Dynamic analyses described in the next sub-section confirm the above argument.

Inter-storey drifts can be controlled much in the same way by using the results of the push-over analyses. Maximum drifts occur when the limit base displacement has been reached. Because this is never attained for the present building, even when the high seismic hazard zone is considered, the inter-storey drifts will be always smaller than those pertaining to the limit base displacement. Therefore upper bounds for the inter-storey drifts are given by the values shown in Table 8. It may be easily recognized how those values are much smaller than the limits given by seismic codes. Also, it may be worth observing how the work equivalence establishes a one-to-one correspondence between the storey displacements and the base shear, therefore allowing for the control of key displacement variables such as inter-storey drifts.

It is important to keep in mind that for elastic-plastic systems ($\mu > 1$), the displacement calculated using Equation (13) must be amplified by a factor equal to the ductility ratio μ (Chopra, 2001).

Table 8: Angular Inter-storey Drifts (%) at Limit Base Displacement

Direction	First Storey	Second Storey	Third Storey	Fourth Storey
Transverse	0.09	0.18	0.20	0.19
Longitudinal	0.20	0.16	0.12	0.07

4. Simulations under Spectrum-Compatible Earthquakes

The 2-DOF equivalent model has been used for simulations under spectrum-compatible earthquakes. Twelve acceleration samples have been generated whose compatibility with the design spectrum is shown in Figure 24. In the same figure is also shown a sample of the artificially generated acceleration histories. The response of the 2-DOF equivalent model under one of the 12 artificially generated acceleration samples is shown in Figure 25. Results refer to the acceleration applied in the longitudinal direction, and to a PGA = 0.25*g* corresponding to the value prescribed by the Italian seismic code for the Solarino area. The parameters examined are the displacement of the superstructure relative to the foundation and the total displacement of the foundation. The relative displacement of the superstructure is the absolute displacement of the building strengthened only with the thin walls, but with a fixed base. It may be observed that the two displacements differ by an order of magnitude. This implies that the displacement of the building with base isolation is negligible if compared with that of the building strengthened only with walls. Furthermore, the behaviour of the superstructure with base isolation is elastic while the cycles of plastic deformation in the absence of base isolation are shown in the left lower part of Figure 25. In the central part of Figure 25 the displacement of the foundation is shown; it is evident that the small deformation of the superstructure is possible because of the high deformability of the foundation. On one hand this gives the advantage of keeping the superstructure in the elastic range, while on the other hand a significant displacement of the foundation must be accommodated.

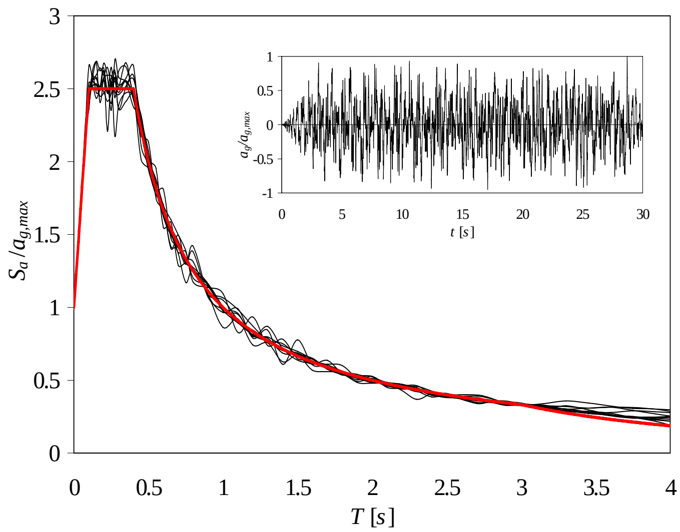


Fig. 24 Response spectra of 12 spectrum-compatible acceleration samples

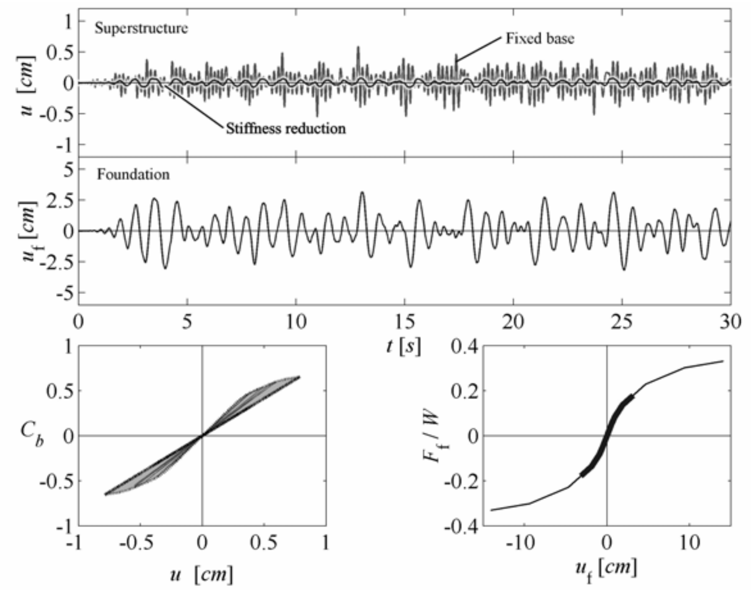


Fig. 25 Response of the retrofitted building to a spectrum-compatible acceleration sample

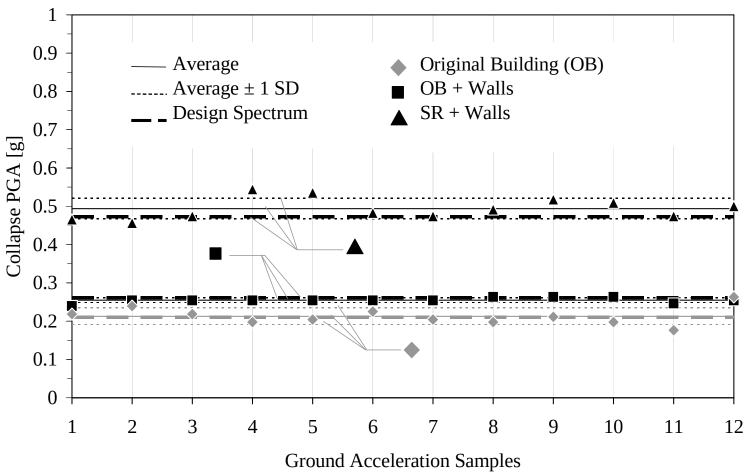


Fig. 26 Seismic resistance in terms of pseudo-acceleration for an IACP building in Solarino (excitation in the transverse direction)

In the lower part (on the right) of Figure 25, the history of the foundation displacement is shown as a function of the base shear. It is important to notice that, in spite of the fact that the acceleration sample is spectrum-compatible, the maximum foundation displacement is only a small fraction of the maximum design displacement. A similar behaviour is also observed in the transverse direction and for all other acceleration samples considered. If the PGA for areas of high seismic hazard had been used instead, the enclosed area in the left lower part of Figure 25, depicting structural damage in the non-isolated building, would be comparatively much larger. The base displacement in the isolated building would, thus, also be much larger, but still within the limit displacement of the bearings as indicated by the over-strength shown in Tables 4 and 6.

Figure 26 provides a concise description of the building behaviour in the transverse direction. For ease of comparison, the seismic resistance of:

- the original building is shown by rhomboidal symbols;
- the building strengthened only with walls is shown by square symbols;
- the retrofitted building with walls and base isolation is shown by triangular symbols.

The thick broken lines indicate the seismic resistance evaluated by the modified FEMA procedure. The symbols indicate the maximum acceleration of the scaled spectrum-compatible acceleration sample, while the solid thin lines represent the mean value of the seismic resistance evaluated from the twelve samples considered. The thin broken lines indicate the mean ± standard deviation value.

From the observations of Figure 26, it may be concluded that strengthening by using only walls does not significantly improve the seismic resistance of the building in the transverse direction, while the addition of base isolation produces a considerable over-resistance. It should also be noted that the simulations with spectrum-compatible acceleration histories generally provide results reasonably close to the prediction of the FEMA procedure.

The seismic resistance in the longitudinal direction is shown in Figure 27. The representation is similar to that described for the transverse direction. However, some significant differences in the behaviour may be noted. The building retrofitted with walls and base isolation exhibits the same seismic resistance in the longitudinal direction as in the transverse direction. The hypothetical building strengthened only with walls exhibits instead a slightly lower resistance than the original building, and a definitely lower resistance than the seismic resistance in the transverse direction. This behaviour may be easily explained. The walls inserted in the longitudinal direction increase the building stiffness, and produce shortening of the period. This shortened period corresponds to the constant part of the spectrum, thus causing larger seismic forces. As a consequence, the seismic resistance of the strengthened building diminishes compared to that of the original building. Such behaviour is more evident if the comparison is limited to the numerical simulations. At any rate, it is important to observe that the building retrofitted with walls and base isolation exhibits a dynamic behaviour independent of the excitation direction.

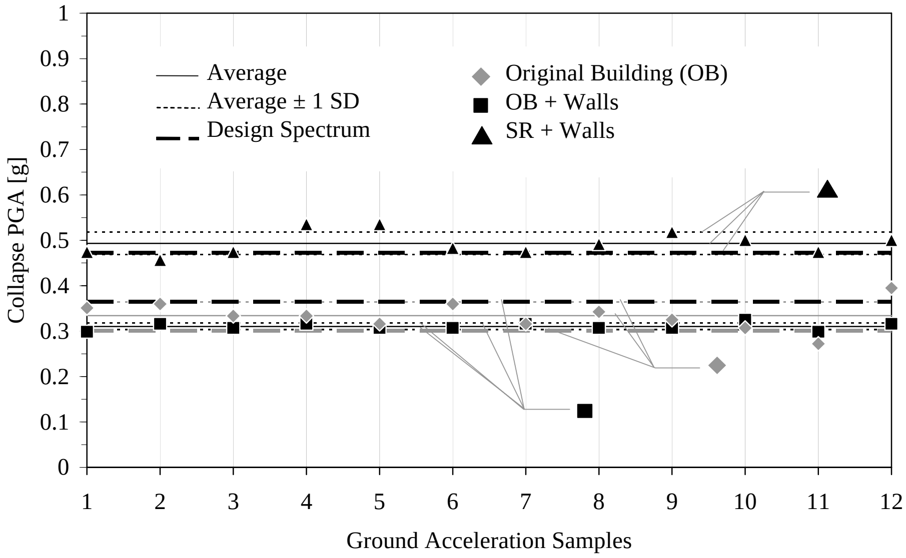


Fig. 27 Seismic resistance in terms of pseudo-acceleration of the IACP building in Solarino (excitation in the longitudinal direction)

5. Experimental Tests

Several free vibration tests were performed on one of the Solarino buildings in July 2004 by statically applying base displacements of various amplitudes up to the design displacement and then suddenly releasing the building. The tests were repeated publicly on July 16 in the presence of world renowned experts in earthquake engineering. The preliminary results have been reported in the literature (Oliveto et al., 2004b) and studies are still underway for the complete identification of the mechanical properties of the building and of the base isolation system.

It was interesting to see that the resulting vibration period was amplitude-dependent as would have been expected because of the dependence of the system stiffness on displacement amplitude that is longer periods at higher amplitudes and shorter periods at lower amplitudes. The periods were also much longer than those obtained in the present paper. The reason is quite simple and easily predictable: the overall stiffness of the rubber bearings used for base isolation was much lower than the nominal value provided by the manufacturer and used for the analyses presented in this paper, but in line with the design prescriptions. The main conclusion is that the seismic forces affecting the retrofitted building will be lower than those assumed in this paper.

Equivalent viscous damping calculated by logarithmic decrement was also dependent on displacement amplitude with values ranging from about 15% of critical damping in the proximity of the design displacement (nearly 14 cm) to more than 20% at low amplitudes (around 4 cm). It is the opinion of the authors that this behaviour may be more due to friction than to an actual decrease of damping in the rubber bearings with increasing amplitude. It is easy to show how the equivalent coefficient of viscous damping for friction damping is proportional to the reciprocal of the displacement amplitude.

Another interesting feature shown by the tests has been an acceleration jump at any sign change in the velocity. This should allow for the evaluation of the dynamic friction force in low-friction bearings used in combination with the rubber bearings in the base isolation system of the building. More information may be found in the paper already quoted (Oliveto et al., 2004b) and even more should appear in the literature in near future.

SUMMARY AND CONCLUSIONS

After an introduction which explains why there are so many vulnerable structures in areas of high or moderate seismic hazard around the world, the authors consider the specific case of Eastern Sicily. The paper proceeds with an illustrative description of the seismic action and then addresses the problem of evaluating the seismic resistance and vulnerability of engineering structures. The application of the methodology presented to reinforced concrete buildings in Eastern Sicily clarifies the concepts discussed. In particular, the concepts of seismic resistance, seismic vulnerability and seismic over-resistance become easily understood and appreciated.

The paper then considers the retrofitting of buildings vulnerable to earthquakes and briefly describes the main traditional and innovative methods of seismic retrofitting. Examples drawn from the professional, editorial and research activity of the senior author are used to illustrate the problems in a simple way. Among all the methods of seismic retrofitting, particular attention is devoted to the method which is based on stiffness reduction. This method is carried out in practice by application of the concept of springs in series, leading in fact to base isolation. One of the two springs in series represents the structure and the other represents the base isolation system.

The application of the concept to two buildings in Eastern Sicily concludes the presentation. The enhanced resistance of the buildings to the design earthquake clearly shows the effectiveness of the method, while a generally improved seismic performance also emerges from the application.

In conclusion it is hoped that the material presented in this paper will be useful in increasing the understanding of the earthquake engineering problem and of seismic retrofitting.

ACKNOWLEDGEMENTS

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REFERENCES

1. BSSC (1997a). “FEMA 273: NEHRP Guidelines for the Seismic Rehabilitation of Buildings”, Building Seismic Safety Council, Washington, D.C., U.S.A.
2. BSSC (1997b). “FEMA 274: NEHRP Commentary on the Guidelines for the Seismic Rehabilitation of Buildings”, Building Seismic Safety Council, Washington, D.C., U.S.A.
3. BSSC (2000). “FEMA 356: Pre-Standard and Commentary for the Seismic Rehabilitation of Buildings”, Building Seismic Safety Council, Washington, D.C., U.S.A.
4. CEN (1998). “Eurocode 8: Design Provisions for Earthquake Resistance of Structures”, Comité Européen de Normalisation, Brussels, Belgium.
5. CEN (2003). “Eurocode 8: Design of Structures for Earthquake Resistance”, prEN 1998-1 (Draft No. 6), Comité Européen de Normalisation, Brussels, Belgium.
6. Chopra, A.K. (2001). “Dynamics of Structures – Theory and Applications to Earthquake Engineering”, Prentice Hall, Upper Saddle River, U.S.A.

7. Clough, R.W. and Penzien, J. (1993). "Dynamics of Structures", McGraw-Hill, New York, U.S.A.
8. Huang, Y., Wada, A., Iwata, M., Mahin, S.A. and Connor, J.J. (2001). "Design of Damage-Controlled Structures" in "Innovative Approaches to Earthquake Engineering (edited by G. Oliveto)", WIT Press, Ashurst, U.K., pp. 85-118.
9. Marletta, M. (2002). "Vulnerabilità Sismica e Adeguamento di Edifici in Cemento Armato", Ph.D. Dissertation, University of Catania, Catania, Italy.
10. Oliveto, G. and Decanini, L.D. (1998). "Repair and Retrofit of a Six Storey Reinforced Concrete Building Damaged by the Earthquake in South-East Sicily on 13th December 1990", Soil Dynamics and Earthquake Engineering, Vol. 17, pp. 57-71.
11. Oliveto, G., Caliò, I. and Marletta, M. (2001). "Seismic Resistance and Vulnerability of Reinforced Concrete Buildings Not Designed for Earthquake Action" in "Innovative Approaches to Earthquake Engineering (edited by G. Oliveto)", WIT Press, Ashurst, U.K., pp. 119-201.
12. Oliveto, G., Caliò, I. and Marletta, M. (2004a). "Retrofitting of Reinforced Concrete Buildings Not Designed to Withstand Seismic Action: A Case Study Using Base Isolation", Proceedings of the 13th World Conference on Earthquake Engineering, Vancouver, Canada, Paper No. 954 (on CD).
13. Oliveto, G., Granata, M., Buda, G. and Sciacca, P. (2004b). "Preliminary Results from Full-Scale Free Vibration Tests on a Four Storey Reinforced Concrete Building after Seismic Rehabilitation by Base Isolation", Proceedings of the JSSI 10th Anniversary Symposium on Performance of Response Controlled Buildings, Yokohama, Japan, Paper No. 7-2 (on CD).
14. Vulcano, A. and Bertero, V.V. (1986). "Nonlinear Analysis of R/C Structural Walls", Proceedings of the 8th European Conference on Earthquake Engineering, Lisbon, Portugal, Vol. 3, pp. 6.5/1-8.
15. Vulcano, A. and Bertero, V.V. (1987). "Analytical Models for Predicting the Lateral Response of RC Shear Walls: Evaluation of Their Reliability", Report UCB/EERC-87/19, Earthquake Engineering Research Center, University of California, Berkeley, U.S.A.