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DEVELOPMENT OF REGULATIONS FOR SEISMIC ISOLATION AND PASSIVE ENERGY DISSIPATION OF BUILDINGS AND BRIDGES IN ITALY AND EUROPE

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SUMMARY

The approach to design of structures equipped with advanced passive seismic protection systems, like base isolation or energy dissipation systems, is discussed. Reference is made to the recent developments in the seismic European codes. A comparison is made among the European codes and the US Code IBC 2000 (ICC [1]), in order to evaluate the consistency of the approach to the problem. Moreover, the convenience in terms of design forces is verified, by comparing isolated and fixed structures forces, based on the Italian code. Finally, the applicability of the indications of the Italian seismic code to design structures equipped with EDBs is investigated.

INTRODUCTION

A great activity in the field of seismic isolation and passive energy dissipation during the eighties led Italy to be the country where the biggest number of applications to bridges (about 150 long and important bridges and viaducts) were made. Consequently, the Italian industry developed and experimented a great variety of devices based on different concepts. More recently, during the last decade, such a strong impulse was considerably slowed down, due to the absence of a specific seismic norm and the need to get the approval of the National Authority, which usually needed more than one year. For this reason, very few applications were realized in the nineties in Italy, although passive control technologies were getting a growing interest all over the world and in Europe and some strong earthquakes proved the effectiveness of passive control in limiting or even avoiding damage too structural and non structural parts.
Very recently, the European seismic code (EC8, CEN[2]) was converted from ENV to EN and in March 2003, a new seismic code has been introduced in Italy (OPCM[3]), which is mostly based on EC8, like in other European seismic countries. Similarly to EC8, the Italian code contains a chapter exclusively devoted to seismic isolation, both in the part relevant to buildings and in the part relevant to bridges. It must be underlined that also in many other countries in the world the seismic codes have been updated and deal with the design of seismic isolation systems, as reported by Dolce [4,5]. On the other hand, the design and analysis methods for both new structures and for the retrofit of existing structures makes it possible to use energy dissipation devices while fulfilling the requirements of the code. This makes the

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application of seismic isolation and passive energy dissipation in Italy no more requiring the approval of the Authority and will surely encourage designer to protect buildings and bridges with passive control systems.
As far as seismic isolation is concerned, the aspects relevant to simplified analyses and to the definition of the role and functioning of different devices in an isolation system have been considerably improved with respect to the previous Italian Guidelines. The code is consistent with EC8, but several additional aspects have been dealt with, to permit an easy design of strongly non-linear isolation systems for buildings and bridges.
In this paper, after a comparative review of the main aspects of the new Italian code IC2003, of EC8, and of IBC 2000, comparative analyses will be presented to evaluate the implication of the new rules in terms of design and construction of structures protected with passive control systems.
The content of a seismic code is generally based on four fundamental points: performance levels, seismic actions, analysis methods, and specific rules on devices. In this paper, several aspects of the norm relevant to the first three points are analyzed.

PERFORMANCE LEVELS

In the Italian seismic code, like in EC8 (CEN [2]), there are two main limit states to comply with, when designing a seismic isolated structure. The "no-collapse" requirement, i.e. the Ultimate Limit State (ULS), is referred to a design seismic action with return period TNCR, whose recommended value is 475 years. The Service or Damage Limit State (DLS) is checked with respect to the limits of interstorey drift in both substructure and superstructure. These limits are 0.005 h (where h is the storey height), if the building has brittle non-structural elements attached to the structure, and 0.0075 h if non-structural elements are not in contact with the structure or are able to accommodate deformations without failure. The return period TDLR for this limit state is approximately 95 years.
In the IBC-2000 the performance levels are referred to a Design Basis Earthquake (DBE) and a Maximum Credible Earthquake (MCE), having a return period of 475 and 2500 years respectively. The MCE is used only for the safety checks of the isolation units, while the DBE provide the design displacements of the structure. Also EC8 and IC2003 take additional cautions for the checks of isolation devices and this is made by multiplying the actions by a factor of 1.2, thus indirectly increasing the return period to a value of about 750 years.

SEISMIC ACTIONS

The approach of EC8 and IC2003 to the calculation of the seismic actions for both isolated and fixed structures is practically the same, and is referred to a 5% damped elastic response spectrum. The spectrum amplitude is described by the peak ground acceleration a_g, consistent with the return period in the considered area, multiplied by the importance factor of the structure γ_I (whose value is in the interval 1-1.4). The spectrum shape, which is described by the transition periods T_B, T_C, T_D, is determined on the basis of the soil conditions. Amplification site effects are accounted for by means of the soil factor $S \geq 1$. It is assumed equal to unity if the soil is classified as engineering bedrock (ground class "A"). There are, then, five classes of ground (A to E), classified on the basis of the velocity of shear waves in the top 30 m of depth. A damping different from 5% is taken into account by means of a reduction coefficient $\eta = \sqrt{10/(5+\xi)}$, where ξ is the effective viscous damping of the isolation system in percent. The minimum allowed value for η is 0.55, which implies that the maximum value of damping to take into account in the spectrum calculation is ξ=28%. It is, also, worth to mention that the Italian code increases the safety of an isolated structure by increasing the last transition period T_D from 2.0 s to 2.5, thus shifting forward the cusp point of the spectra and increasing spectral ordinate.

The only additional provision of EC8 is the need to generate site specific spectra if the building has an importance class I and is close to a potentially active fault with a magnitude $M \geq 6.5$. These ground motions can generate unexpected great displacements at the isolation level, Jangid [6]. A similar approach is used in IBC-2000, where site specific spectra are needed if the building is in an area with spectral acceleration S_1 greater than 0.6g.
Using EC8, the design of structural elements in building structures can be made by applying a behavior factor q=1.5 (taking into account structure ductility an damping), that divide the above said actions. In IC2003 and IBC-2000 the value of the structure factor (behaviour factor) depends on the type of seismic force resisting system, although, for most of the building structures the behaviour factor results to be of the same order as in EC8.

MODELING AND STRUCTURAL ANALYSIS

Because a reliable modeling of an isolated structure goes through a reliable modeling of the isolation system, there are provisions about the mechanical and physical values of the isolation devices to be considered in the global structural analysis. The most unfavorable values of mechanical and physical values attained during the lifetime of the structure shall be used, considering the dependency of them on:

1. rate of loading
2. magnitude of the simultaneous vertical load
3. magnitude of the simultaneous horizontal load in transverse direction
4. temperature
5. change of properties over the projected service life (ageing effects).

The evaluation of inertial effects shall be based on the maximum stiffness and minimum damping and friction coefficients of the isolation system, while displacements shall be determined accounting for the minimum values of the above quantities. These general rules are the base of all the three codes discussed in this paper.

Equivalent linear analysis

Generally speaking, the behaviour of an isolation systems under cyclic actions are more or less non-linear. Under certain conditions, an equivalent linear visco-elastic force-deformation relationship can be assumed for the isolation system and a linear static or a modal dynamic analysis can be carried out. In all the three codes this possibility is given, taking effective values of stiffness (K_{eff}) and damping (ξ_{eff}) to be evaluated at the total design displacement d_{dc}. EC8 requires the following conditions to be met:

1. the effective stiffness (secant) of the isolation system is at least 50% of the stiffness at 0.2 d_{dc} (where d_{dc} is evaluated at the stiffness centre of the isolation system).

2. the effective damping of the isolation system is lower than or equal to 30%. In fact, high values of damping can introduce modal coupling and then increasing floor accelerations and base shear, neglected in standard dynamic modal and static analysis, as illustrated in Kelly [7].

3. the mechanical characteristics do not vary by more than 10%, due to the rate of loading and vertical load variations in the range of the design values.

4. the increase of force in the isolation system for displacement between $0.5d_{dc}$ and d_{dc} is at least 2.5% of the total superstructure weight, to provide a minimum restoring effect.

These rules are kept in the Italian code (IC2003), where only condition n. 4 is different, requiring a lower increase of the restoring force (1.25% W). Moreover condition n.3 is more precise, stating the amplitude of the range of the loading rate ($\pm$30%) in which the mechanical characteristics must not vary by more than 10%.

Greater differences can be found in IBC-2000, where condition n.1 is less severe, asking that K_{eff} at 0.2 d_{dc} is at least 33% that at $d_{dc.}$ Moreover, IBC requires that the ratio of the MCE and DBE isolation

displacement is at least equal to 3/2. This clause is aimed to limit the hardening of the isolation system behaviour. Condition n.2 is the same in EC8 and in IC2003, but it is formulated in a different way (as explained later) in IBC-2000. Condition n.3 is only qualitative in IBC-2000, while condition n.4 is the same.
A general agreement is then found on this point, related to the possibility to model the isolation system as equivalent linear visco-elastic, being IBC-2000 the less stringent.

Simplified linear analysis (Static)
This kind of analysis is made through two horizontal translations, with additional torsion effects about the vertical axis taken into account separately. It can be called *static* because it considers only the first vibration mode, assuming that the superstructure is a rigid mass above the isolation system. The overall system vibrates with a period $T_{eff} = 2\pi\sqrt{\frac{M}{K_{eff}}}$, where M is the total superstructure mass. The possibility to model the isolation system as linear visco-elastic is an important condition to perform the simplified analysis, but other conditions on the superstructure characteristics, soil profile, seismic area, are required, depending on the seismic code. These conditions are summarized and compared in table 1.

Table 1. Conditions to meet to apply static linear analysis.

		IC2003	EC8	IBC-2000
1	Maximum mass - stiffness centres eccentricity	8.0%	7.5%	Not specified
2	Limitation on site seismicity	Not required	Distance from M≥6.5 faults >15 Km	S_1<0.6g
3	Plan regularity of building and symmetry	Required	Required	Required
4	Maximum plan dimension	50 m	50 m	Not required
5	Maximum superstructure height	20 m	Not specified	19.8 m
6	Maximum number of stories	5	Not specified	4
7	Period range of T_{eff}	$4T_f$ – 3.0s	3Tf – 3.0s	3Tf – 3.0s
8	Ratio between vertical and horizontal stiffness K_v/K_{eff} of isolation system	≥800	≥150	Not required
9	Maximum vertical vibration period T_v	0.1 s	0.1 s	Not required
10	Limitation of the soil class	None	None	A, B, C or D
11	Tension in isolation devices	Not allowed	Allowed	Allowed
12	All devices must be located above elements of substructure that support vertical load	Not required	Required	Not required

Condition n.1: EC8 and IC2003 have about the same value of eccentricity between the centers of mass, (plan projection) and of isolation system stiffness, including the accidental eccentricity. IBC-2000 states the general rule that the mass must be accounted for in the most disadvantageous location in terms of eccentricity.

Condition n.2: Both EC8 and IBC-2000 provide rules on site seismicity, although in different forms (distance from active fault and spectral acceleration).

Conditions n.3-4-5-6: EC8 does not limit the superstructure height nor the floor number, neglecting possible participation of higher modes, which can increase inertial effects and modify force distribution

along the building height. In this case IBC-2000 and IC2003 are almost equivalent, except for the plan dimensions. In general, IC2003 is the most conservative.

Condition n.7: IC2003 imposes a more conservative minimum isolation degree (4.0) with respect to EC8 and IBC-2000 (3.0). This leads to a greater consistency of the hypothesis that the superstructure behaves like a rigid mass, Skinner [8].

Conditions n.8-9: the vertical flexibility of the isolation system can increase vertical vibrations and, mainly, can make the overturning moment to induce a rocking rotation of the structure, that frustrates the beneficial effects of seismic isolation. The EC8 limit on the vertical vs. horizontal stiffness ratio K_v/K_{eff} is much less than the Italian code, although the resultant vibrating period is the same (0.1 s). With a simple calculation it can be shown that the Italian limit ($K_v/K_{eff} \geq 800$) is aimed to guarantee that up to $T_{eff} = 3.0$ s, the limit on condition n. 8 practically leads to the fulfillment of condition n.9. In fact:

$$T_v = \frac{T_{eff}}{\sqrt{800}} = \frac{3.0}{28.3} = 0.106$$

IBC-2000 does not provide any limitation, although the rocking rotations at the isolation interface can make the simplified analysis method inadequate.

Condition n.10: is specified only in IBC-2000. It seems inessential, as the amplification effects of the ground can be accounted for by means of site specific spectra.

Condition n.11: is missing in EC8. Tension or uplift in the isolators, implying strongly nonlinear behaviour, is difficult to be taken into account with this simplified method.

Condition n.12: This condition is contained in EC8 only. It is, probably, to be intended for devices carrying vertical loads (e.g. rubber or sliding isolators) but not for auxiliary dissipating or re-centering devices (see IC2003).

It should be remarked that for this kind of analysis, IBC-2000 does not specify any limit for the damping of the isolation system, instead provided for the response spectrum analysis and equal to 30% of critical. Moreover this simplified analysis, when applicable, is mandatory in IBC-2000 and constitutes the minimum actions to apply in the design.

The static linear analysis is applied by following the procedure reported below.
According to EC8 and IC2003, the design displacement of the stiffness center is calculated as:

$$d_{dc} = \frac{M \cdot S_e(T_{eff} \cdot \xi_{eff})}{K_{eff,\min}} \quad \text{(eq. 1)}$$

with reference to the elastic response spectrum. The superstructure force distribution is proportional to the floor masses:

$$f_j = m_j \cdot S_e(T_{eff} \cdot \xi_{eff}) \quad \text{(eq. 2)}$$ being m_j the j-th floor mass.

For the calculation of the total design displacement of each isolation unit in a given direction, amplification factors shall be applied that depend on the position of the unit, on the total mass-stiffness eccentricity in the direction normal to the seismic action and on the torsional stiffness radius of the isolation system. The design of the structural elements shall be made by dividing the calculated stresses by the above discussed behaviour factor q.
The approach of IBC-2000 is similar, passing through the calculation of the design displacement d_{dc}, starting from the spectral acceleration, damping and stiffness of the isolation system. Then, the total base shear is:

$$V_s = \frac{K_{D\max} \cdot d_{dc}}{R_I} \quad \text{(eq. 3)}$$

where K_{Dmax} is the stiffness of the isolation system at the design displacement and R_I the behaviour factor, which is taken equal to 3/8 of the behaviour factor of the same fixed base structure. However the behaviour factor R_I must be always in the range $1.0 \leq R_I \leq 2.0$ The base shear force has to be distributed along the building height, assuming an inverted triangular distribution of accelerations, like for fixed structures. This assumption disregards that an isolated structure behaves like a rigid mass on the isolation system, then experiencing an almost constant acceleration distribution, as assumed by EC8 and IC2003.
In all the three codes, a behaviour factor equal to 1 shall be taken for the design of the substructure elements, with the aim to get an overstrength that minimize possible differential displacements and avoid large deformations of the isolation storey.

Modal dynamic linear analysis
If the isolation system may be modeled as linear, but some of the conditions of table 1 are not met, the structural system shall be analyzed at least with a modal dynamic analysis, where both the superstructure and the isolation system are modeled as linear elastic.
In IC2003 the modal dynamic analysis is applicable even if all the conditions in table 1 are not met, with the only care of considering the simultaneous vertical component of the seismic action, if condition n.8 is not satisfied. In EC8 there is no specific care to be taken even if all the conditions in table 1 are not met.
In IBC-2000, the modal analysis may be applied only if condition n.10 on soil class is met, otherwise a time history analysis is necessary. A static linear analysis is necessary in any case, since the results of the modal analysis shall be compared with those of the simplified analysis. In particular the design displacement shall be at least 90% and the base shear at least 80% of the corresponding quantities resulting from the simplified analysis.
Although the isolation system can have any value of the effective damping, in this kind of analysis it is possible to account for a value not greater than about 30% in all the three codes. In IBC-2000 spectral acceleration can be divided by a factor B, taking into account the damping of the isolation system. This factor depends on the effective damping and it is limited within the range $0.8 \leq B \leq 2.0$. An approximate formula for this factor is $1/B = 0.25 \cdot (1 - \ln \beta)$ in which β is the damping ratio, Naeim [9].
No special differences are detected in the three codes for the execution of this kind of analysis.

Time history analysis
If it is not possible to model the mechanical behavior of the isolation system as equivalent linear, a time history analysis is needed, with only the isolation system modeled as non linear, while a linear model is kept for the structure. The non linear model shall represent the actual constitutive law of the isolation system in the actual range of deformations and velocities related to the seismic design situation. The approach is the same in all the three codes analyzed in this paper. In IC2003, however, an extension to nonlinear system of the simplified method described in 4.2 is allowed. If only the isolation system does not meet the conditions discussed in 4.2, as it presents a highly non linear mechanical behaviour, then it is possible to perform the time history analysis on a non linear single degree of freedom system, by assuming the structure to behave as a rigid mass mounted on the isolation system. The displacement obtained from this analysis will be the design displacement, while the maximum acceleration on the rigid mass will replace the term $S_e(T_{eff} \cdot \xi_{eff})$ in eq. 2 for the calculation of floor forces.

DESIGN SEISMIC ACTIONS ON FIXED BASE AND ISOLATED BUILDINGS

The question of the cost of seismic isolation is often raised when this strategy is proposed for the seismic protection of buildings and bridges. Although the problem should be correctly addressed, looking at the

overall expected costs, thus including the cost of repair and of destroyed contents, as well as casualties and social costs, nevertheless it is also important to get an estimation of the initial costs, which is an important component of the overall expected cost. The additional cost of seismic isolation is due to the cost of the devices and of the structural arrangement at the isolation interface, while possible savings can be obtained from the reduction of the seismic forces acting on the superstructure. Since the additional costs are made of two parts, one fixed, the other dependent on the seismic forces, while the possible savings are strictly related to the seismic forces, it appears interesting to compare the seismic forces on a fixed base and on a similar isolated structure. Such a comparison is strongly related to the seismic regulations and allows the designer to make the basic choice whether to utilize or not seismic isolation and, eventually, to optimize its application.

In this paragraph the actions on a fixed base framed R/C structure and on a similar isolated structure are compared in terms of base shear, referring to IC2003. This comparison is reported in terms of ratio of design spectral accelerations multiplied by the effective mass ratio of a fixed base structure Sd(Tf) and of a similar seismic isolated structure Se(Tis), where Tf is period of the fixed base structure and Tis the period of the isolated structure. The effective mass ratio is taken into account with a value of 0.85, as prescribed for the equivalent linear static analysis, if the structure has at least 3 stories and its vibrating period is Tf < 2Tc. Obviously it is taken equal to 1 for the isolated structure.

The comparison is referred to both the ultimate limit state (ULS) and the damage limit state (DLS). For the ULS of the fixed base structure, the design spectral ordinate depends on the behavior factor *q,* which is related to the ductility class, high (CD "A") or low (CD "B"), and to the regularity of the superstructure along the height. By combining these two conditions, there are four possible values of the behavior factor. The comparison is made for the most and the less favorable cases. Thus, a case with $q=4.5\cdot\alpha_u/\alpha_1$, corresponding to the condition of high ductility and regularity, and a case with $q=0.7\cdot4.5\cdot\alpha_u/\alpha_1$, related to low ductility and structural elevation irregularity, are examined. For the isolated structure, the behaviour factor is $q=1.15\cdot\alpha_u/\alpha_1$. The multiplier α_u/α_1 is practically ignored, as it is considered and equally evaluated also for the isolated structure. Reference is made to a typical rubber isolation system, having an equivalent viscous damping ratio equal to 10%, resulting in a 0.816 reduction factor of the spectral ordinate.

In this paper the comparison is limited to an intermediate soil condition, relevant to the soil profiles B, C, E.

In fig. 1 there are shown the ULS mass acceleration ratio (fixed / isolated) for four different fixed base periods (0.5, 0.7, 1.0, 1.5 secs.), in a diagram with the isolated structure period at the abscissa. This ratio is as much favourable to seismic isolation as the higher it is. Only the values relevant to reasonable the isolation ratios (Tis/Tf ≥ 2) and isolation period (Tis ≥ 1.5 s) are reported. As could be easily predicted, this ratio is an increasing function of the isolation period and a decreasing function of the fixed base period, i.e. an increasing function of the isolation ratio Tis/Tfb. Focusing the attention on the usual range of application of rubber isolation, 1.5 s ≤ Tis ≤ 3.0 s, it can be seen that the acceleration ratio varies between 0.57 (Tf=0.7s, Tis=1.5s) and 1.92 (Tbf=0.5s, Tis=3.0s), when a regular high ductility structure is considered, and between 1.02 and 3.42, when an irregular low ductility structure is considered.

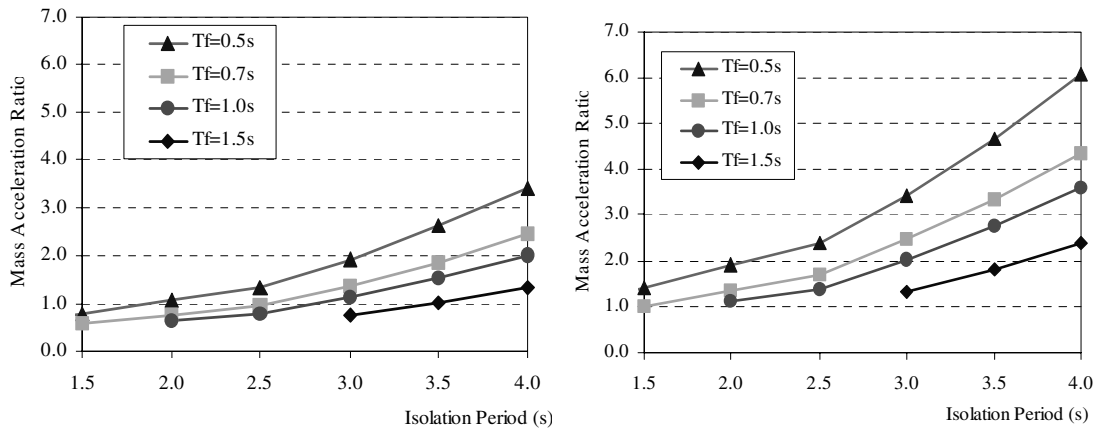

Figure 1 - ULS mass acceleration ratio for (a) regular, high ductility structures, (b) irregular, low ductility structures.

As far as high ductility structures are concerned, it should be observed that considerable additional costs are implied by the capacity design and special detailing rules, resulting in a considerable increase of steel flexural reinforcement in columns (of the order of 20-40%) and shear and confinement reinforcement in beams and columns.

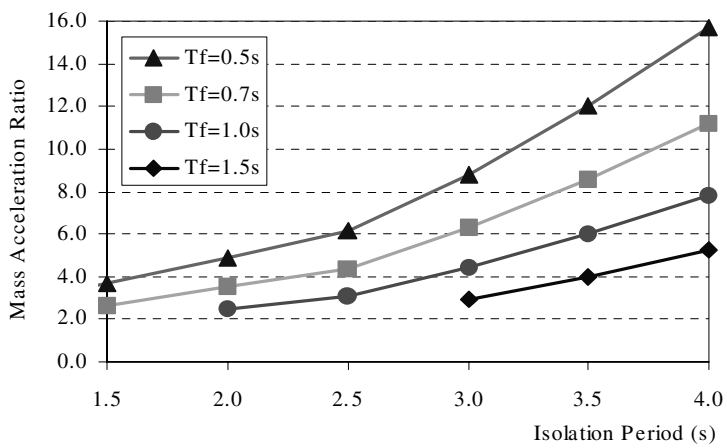

Figure 2 - DLS mass acceleration ratio for both regular - high ductility and irregular - low ductility structures.

The effectiveness of seismic isolation in reducing seismic effects is even much greater when looking at the DLS diagram, reported in fig. 2. This occurs because the actions on fixed base and isolated structures are referred to the same design elastic spectrum, whose ordinates are exactly 2.5 smaller than the ULS elastic spectrum, irrespective of the type of structure. Moreover the isolated structure can also benefit of the increased damping, in this case leading to a further about 20% force reduction. Focusing, again, the attention on the usual range of application of rubber isolation, it can be seen that the acceleration ratio varies between 2.45 (Tf=1.0s, Tis=1.5s) and 8.82 (Tbf=0.5s, Tis=3.0s). Since DLS is essentially related to deformations produced by seismic actions, the considerably lower inertial forces produce considerable advantages in terms of flexibility requirements of isolated structure, resulting in possible savings due to the reduced sectional areas of the structural members.

DESIGN OF STRUCTURES EQUIPPED WITH ENERGY DISSIPATING BRACES (EDBs)

The protection of buildings with seismic isolation is not always the best solution and sometimes it is not applicable for the lack of the fundamental prerequisites that make it possible to exploit the advantages of this technique. In such cases, the protection of buildings can be made by introducing dissipating braces in the structural frames, which help the structure to dissipate the seismic energy, thus reducing displacement and ductility demands. Some types of braces, Dolce[10,11], are able also to provide a beneficial recentering capacity, in order to avoid residual deformations at the end of the quake. As said above, the new Italian and European seismic codes do not contains specific chapters or indications devoted to design of structures embedding dissipating braces. In spite of this, it is possible to design structures with dissipating braces, by using the static non-linear analysis methods, such as the one reported in EC8 and in the Italian code.

The method consists of a static pushover analysis plus a capacity spectrum analysis, to be carried out according to the following steps:

1. construction of the force-deflection behaviour of the structure in terms of base shear F_b vs. displacement of a checkpoint d_c (usually the roof center of mass).
2. transformation of this curve in a bilinear path, describing the behaviour of an equivalent Sdof system
3. calculation of the maximum displacement response based on the elastic code spectrum
4. conversion of this displacement in the deformed shape of the structure and checking of the compatibility of the displacements (ductile elements), strength (fragile elements) and the deformations of the protection devices.

This method is applicable to regular buildings in plan and in elevation and to non-regular buildings if stiffness evolution methods are used to perform pushover analysis in step no.1.

Once the force-deflection curve F_b- d_c is obtained, as in step 1, force and displacement of the equivalent bilinear Sdof system can be calculated by eq. 4.

$$F^* = F_b / \Gamma$$
$$d^* = d_c / \Gamma \qquad \text{(eq. 4)}$$

where Γ is the participation factor defined as $\Gamma = \dfrac{\Sigma m_i \Phi_i}{\Sigma m_i \Phi_i^2}$.

Moreover, for the bilinearization of the curve F^*- d^* the coordinates of the yielding point $F_y{}^*$ and $d_y{}^*$ shall be determined with the following equations 5:

$$F_y{}^* = F_{bu} / \Gamma$$
$$d_y{}^* = F_y^* / k^* \qquad \text{(eq. 5)}$$

in which F_{bu} is the ultimate strength of the structure and k^* is the stiffness obtained by the equivalence of the areas as in figure 3.

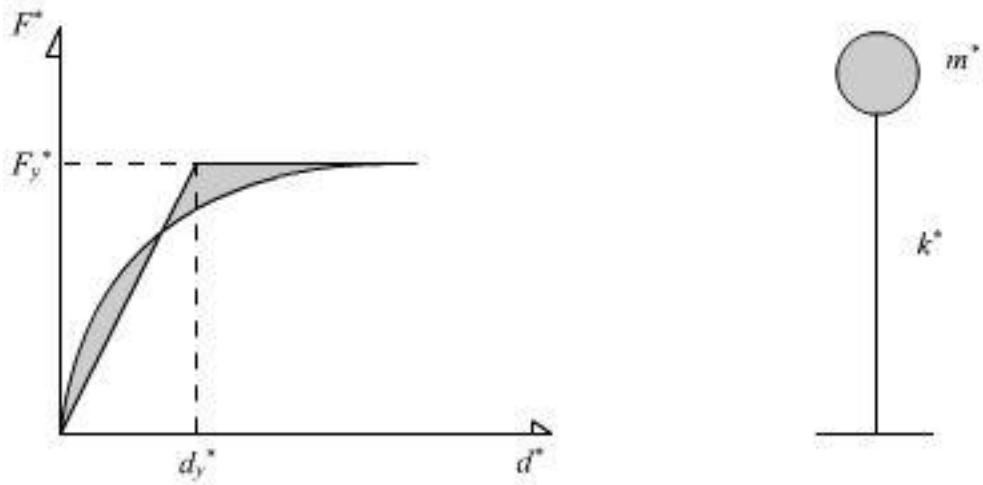

Figure 3 - Force-deflection characteristics of the equivalent bilinear Sdof system.

The elastic period of the equivalent Sdof system will be

$$T^* = 2\pi\sqrt{\frac{m^*}{k^*}} \qquad \text{(eq. 6)}$$

where $m^* = \Sigma m_i \Phi_i$

The displacement of the elastic system with stiffness $k_e = k^*$ must be converted into the displacement of the bilinear system, by using the elastic response spectrum. Two cases shall be considered:

1. If the equivalent period $T^* \geq Tc$, then the equivalence of displacements shall be applied, i.e. $d^*_{max} = d_{e,max}$ (Tc is the transition period at which the design elastic spectrum reduces after the horizontal segment)
2. If the equivalent period $T^* < Tc$, the displacement of the bilinear system will be greater than the elastic one, and can be calculated with the following equation

$$d^*_{\max} = \frac{d^*_{e,\max}}{q^*}\left[1 + (q^* - 1)\frac{T_c}{T^*}\right] \qquad \text{(eq. 7)}$$

where $q^* = \dfrac{S_e(T^*) \cdot m^*}{F^*_y}$ is, in practice, the ductility demand.

Finally the displacement of the real structure will be $\Gamma \cdot d^*_{\max}$.

CONCLUSIONS

The general rules on the seismic isolated structures contained in EC8, are inherited in the new Italian seismic code, with some modifications, in order to increase safety and simplify applications. A comparison with the U.S. code (IBC-2000) is made, focusing on the principal rules for the application of the different analysis methods. A specific requirement of the U.S. code, which is not contained in the other two codes, is that a simplified analysis shall be carried out even when the needed conditions for this kind of analysis

are not met, in order to make a preliminary design of the isolation system and, mainly, to check the displacement and base shear obtained with more sophisticated analyses. On the other hand, the Italian Code pays much attention on the non linear isolation systems, which have a great potential in the applications to buildings and bridges.

In general, however, it can be said that the international current state of practice indicates a considerable consistency of the different codes, as far as at least design aspects are concerned. Some differences can be detected when the experimental tests and the safety requirements of the isolation devices are concerned, but this can be justified by the differences in the experimental facilities and in the most frequently used technologies in the different countries. It should be mentioned, here, that the European Standardization Committee (CEN) is setting up a special norm for antiseismic devices, separated from the seismic code, which should standardize experimental tests and safety requirements in Europe.

A comparison of the design seismic forces acting on fixed base and base isolated structures has been made, referred to the Italian Code rules and typical design situations using quasi-elastic isolation systems. It leads to interesting results regarding the range of convenience of seismic isolation, as far as the initial cost of the structure is concerned. For most of the typical Italian buildings, having vibration period of about 0.5 s, it is sufficient to get an isolation period of at least 2.0 s to reduce the design actions with respect to a fixed base regular structures with high ductility. In the same conditions the design forces of an irregular low ductility fixed base structure are about twice the forces on the isolated structure. Moreover, it should be taken into account that: i) the actions related to the service limit state, mainly governed by structural flexibility, are greatly reduced (about five times for the above conditions), ii) no high ductility rules, related to the capacity design criteria, must be satisfied for base isolated structures. These considerations lead to further savings, with respect to those indicated by the comparison of the seismic forces only, and can make seismic isolation convenient even in terms of initial costs.

An important question is whether the design and construction of isolated structures should be subjected to special checks and, in this case, which kind of checks. In Italy the submission of the design documents to a Ministry committee has led, in the past, to an almost complete stop of seismic isolation applications, due to the long time required to get the approval. With the new code, specific experience is required to the designer and to the inspection engineer, but no submission of the project is required. With IBC a peer review on the entire project has to be performed by professional engineers and others experienced in seismic isolation.

Regarding the application of other passive seismic protection techniques, like energy dissipating braces, it must be remarked that the Italian codes implicitly allows it and provides general analysis methods which are appropriate for the design of energy dissipating systems. On the other hand no rules related to qualification and acceptance of specific devices for energy dissipating braces are provided, although the provisions given for auxiliary devices of seismic isolation systems can be suitably extended to this purpose.

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