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BASE ISOLATION STRATEGY FOR FRAMED BUILDINGS

P. P. ROSSI*, P. LENZA**, M. PAGANO**

*Istituto di Scienza delle Costruzioni, Facoltà di Ingegneria, Catania 95125, Italy
**Dipartimento di Analisi e Progettazione Strutturale, Facoltà di Ingegneria, Napoli, 80138, Italy

ABSTRACT

We present some design strategies for the base isolation of framed buildings, characterised by a first storey with high deformability, with columns hinged at both ends.
The evaluation of the response of buildings so isolated, in terms of relative displacements and shear, allowed to highlight the behaviour of isolated systems that follow different design strategies, underlining the importance of a mixed strategy that assures a high value of the fundamental period of vibration and, at the same time, guarantees a considerable dissipating capacity by the isolation devices.

KEYWORDS

Base isolation ; Framed buildings ; Dissipating devices

INTRODUCTION

On the occasion of quite strong seismic events, the technique of seismic isolation - by now widely accepted at international level - constantly shows its efficacy in earthquake protection.
The design philosophy of seismic isolation, that was developed some decades ago, even though applied in areas of the world at very high seismic risk, is not yet well regulated and accepted by the scientific community.
As an alternative to a traditional seismic protection strategy based on the ductility concept admitting of a wide damage of the structural elements, the technique of base isolation involves a favourable variation of the dynamic characteristics of the structure by using special devices endowed with low stiffness and high dissipating capacities.
The efficacy of this design philosophy depends on the seismic input in addition to dynamic and strength features and, therefore, represents the goal of a strategy aiming at a careful analysis of the characteristics of the spectra, that is of the earthquakes frequency contents. It also aims at finding a relevant dissipating capacity and an optimal value of the maximum strength that must be transmitted to the structure (Parducci, 1989). All this leads to the production of particularly deformable isolators capable of moving the fundamental vibration period to high values so as to obtain a significant reduction of the spectral acceleration and of the expected input energy (De Luca and Serino, 1988). These isolators are endowed with a strongly non linear behaviour and a remarkable viscous or hysteretic damping permitting the dissipation of most of the input energy and the reduction of the value of the maximum displacement corresponding to the design's high period.

DESIGN PROPOSAL

General Features

The technological solutions proposed for the application of the philosophy of the base isolation of framed buildings with rigid floors (Pagano, 1970) must be considered in the framework of mixed systems that follow both strategies of "oscillation period (T)" and of "yielding threshold (Y)" (Parducci, 1989)
The proposed intervention (Lenza *et al*, 1993) (Lenza *et al*, 1994) (Lenza and Pagano, 1994) (Galasso *et al*, 1995) - that has already been treated in its general aspects by other authors (Carotti and De Miranda, 1986) - includes the construction of normal foundation (direct or indirect) systems and of a first very deformable storey with columns hinged at both ends.
Since it is not possible - as it would be necessary - to completely uncouple the soil motion from the structure motion, that is, to obtain a fully labile storey due to problems of instability, then by means of linear elastic behaviour sub-structures, the first storey is provided with adequate stiffness to ensure an acceptable margin able to balance the destabilising effect of vertical loads. Thus the overall stiffness of such a device is:

$$K' > \alpha P / h$$

being:

- α a safety coefficient
- P the structure weight
- h the height of the hinged storey
- P/h the stiffness required due to the destabilising effect of the vertical load

During the design stage of this study the safety coefficient of instability has ranged between the following values:

$$1.5 \leq \alpha \leq 2$$

The system placed at the level of the first floor is designed so as to be in the elastic field with relative displacement values which are at least equal to the maximum values predicted during the design stage so as to avoid its damage and a decrease of its reliability in case of successive earthquakes.
At the level of the first floor we are introducing an element having a hysteretic non linear behaviour and an initial stiffness K" with a view to provide the structure with a remarkable energy dissipating capacity and thus to keep the values of the maximum relative displacement within the applicability of the isolation system proposed.
In choosing the yielding strength (Fy) as well as that of the initial stiffness of the system (K'+K") it must be taken into account that a too low value would undoubtedly enhance the efficacy of the isolation system, but in the first case, it could involve exceeding the elastic threshold due to forces having an intensity even lower than that of the wind and this could produce a frequent and not always justified damage of the isolation system which hence will have to be replaced, in the second case, it might considerably and dangerously reduce the degree of safety vis-à-vis vertical load instability.
It is thus necessary - at least under serviceability limit state - to provide for a high initial stiffness of the structure reaching the wind force values considered in the codes.
Such a function can be performed - depending on the technological solution chosen - by elasto-brittle elements or by the same elements having an elasto-plastic behaviour with a high initial stiffness.

Design Solutions

Below two technological solutions of the proposed isolation system are described. Both utilise - as an elastic system - bundles of high resistance steel bars that form variable inertia cantilevers placed externally to the building perimeter and built in a small jetty projecting from the supporting wall. They are involved in the structural response due to the first floor that, by contrasting them, cause their strain and reaction.

First solution The system capable of offering adequate resistance and stiffness to wind forces is obtained by exploiting the elasto-brittle behaviour of common bricks arranged either as septa that contrast the external supporting wall or as stiffening elements of the hinges placed on the top of the first order columns.
The dissipating capacity is instead obtained by using mild steel bars put in place in the same way as the elastic ones on the perimeter of the building. They represent a restraining system having an elastic plastic behaviour capable - due to the remarkable ductility of the mild steel - of developing wide hysteretic cycles thus dissipating a non completely negligible amount of energy (Fig. 1-2).
Since extensive damage would involve the need to replace them, it is important to identify the optimal design yielding force which should not be too high so as not to produce a too wide elastic stage and thus insufficient hysteretic cycles; however, it should not be too low so as to prevent the small oscillations of the structure (due to a temporary lack of bricks) from producing the damage of the bars when this is not needed.

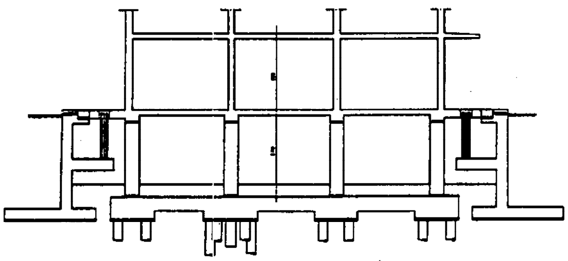

Fig. 1. Excerpt of a section of the isolated building (first solution)

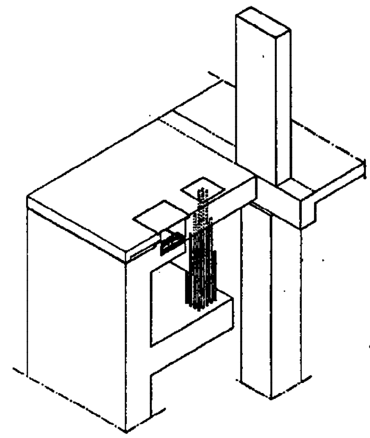

Fig. 2. Assonometric detail of the isolation system (first solution)

Second solution Unlike the first one, the second solution concerns the use of dissipating lead extrusion devices characterised by a markedly rigid-plastic behaviour (Robinson and Greenbank, 1976) (Skinner at al., 1980) (Robinson, 1989) (Skinner *et al*, 1993).
The size of such a device is such that it has yielding threshold almost equal to the pre-set wind forces thus permitting - vis-à-vis higher values - the increase of the natural period of the structure and the dissipation of a remarkable amount of energy through a plastic hysteresis related to the considerable non linearity of the system.

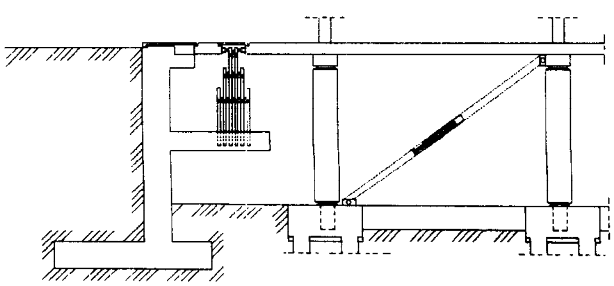

Fig. 3. Excerpt of a section of the isolated building (second solution)

The lead ability to recover its primary mechanic capacities at room temperature immediately after the extrusion process causes the amount of energy it can dissipate not to be limited by the fatigue problem; thus, the device itself becomes capable of undergoing many loading cycles without undergoing too great a damage.
The low dependence of reaction on the application velocity rules out the possibility that under wind loads it can be strained and produce displacements which are not admissible under serviceability limit state.
The device is placed diagonally in some first order bays having lower axial displacements than the displacements of the first floor, but sufficiently high to produce a remarkable action of energy dissipation (Fig. 3)

NUMERICAL TESTS

The Structure: Geometrical Features and Calculation Actions

In order to evaluate the potential of the proposed isolation system, a nine-storey building has been taken into consideration (H = 29.30 mt) having a slightly asymmetric plan.
This building was designed taking only into account vertical, permanent and accidental loads and weak wind forces (non seismic building) (Pagano, 1970).
The main geometric characteristics and the design forces of the chosen building are here reported.

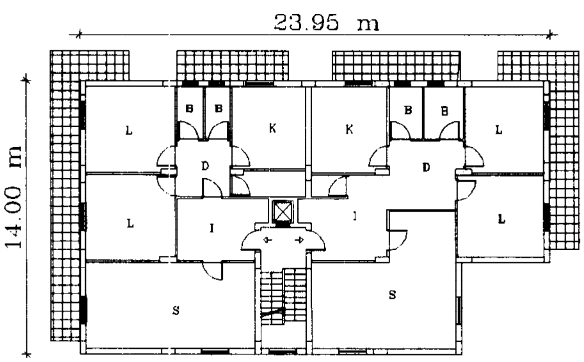

Fig. 4. Plan of the building under examination with its main geometrical dimensions

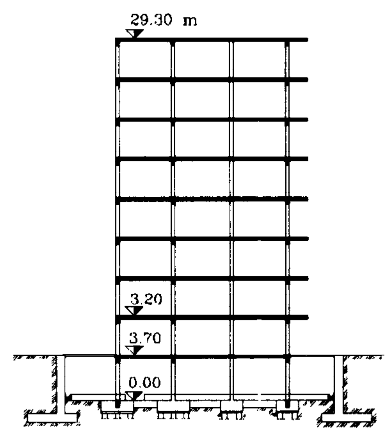

Fig. 5. Section of the building under examination

Table 1 - Design forces

Total weight	(KN)	27720
Wind shear	(KN)	800

Analysis Models

A dynamic step-by-step analysis has been performed solving dynamic equilibrium equations by the Newmark algorithm and establishing a linear acceleration variation within the generic integration step (Lenza, 1988).
At first a high deformability of the first floor in relation to that of the superstructure was expected, thus justifying the study of the structure under examination through a simple model having only one degree of freedom (SDOF).
The simple oscillator response is related to the behaviour of two springs one of which is indefinitely elastic and has K' stiffness, while the other is an elastic perfectly plastic spring with an initial K" stiffness. In analysing the first solution, a fall in stiffness has been taken into account with respect to an assigned shear threshold in correspondence with the breaking of the system having an elasto-brittle behaviour.

Seismic Input

Natural accelerograms have been considered having different dynamic characteristics of duration and spectral contents. They exhibit amplifications in the field of low frequencies as well as of high frequencies.

Table 2 - Accelerometric recordings used for numerical analyses

Earthquake		Station	Component	Duration (sec)	PGA (cm/sec²)
Campano-Lucano	23-11-80	Bagnoli Irpino	WE	79.19	183.57
Imperial Valley	19-5-40	El Centro	S00E	53.80	341.82
Tokaki-Kehoki	16-5-68	Hachinohe	NS	36.00	225.00
Olympia	13-4-49		N80E	30.28	319.31

The corresponding elastic spectra are plotted in terms of accelerations and relative displacements with respect to a 5 per cent damping.

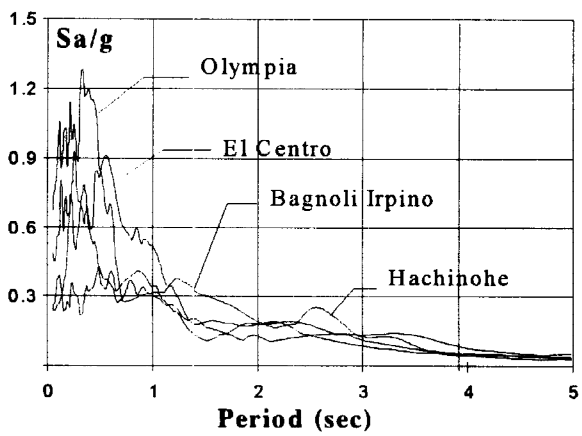

Fig. 6. Elastic response spectrum, in terms of normalized accelerations, for the accelerograms used in numerical analyses ($\nu = 0.05$)

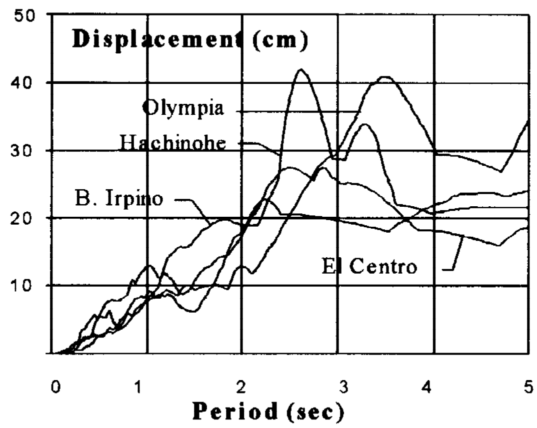

Fig. 7. Elastic response spectrum, in terms of relative displacements, for the accelerograms used in numerical analyses ($\nu = 0.05$)

Results

Unlike what happens in case of conventional designs, the studies, carried out so far concerning seismic isolation, have highlighted - within the structural response in the field of the high periods - the importance of maximum relative displacements of isolation systems because of their technological limitations and also the basic role played in this framework by energy dissipation in view of their more or less marked reduction.
In the non linear analysis of the simple oscillator, R_F denotes the yielding force normalized with respect to the weight of the building while R_K stands for the ratio of the effective elastic stiffness to the total effective one, where by effective stiffness we mean the stiffness of the system amended of the share needed in order to balance the effect P-Δ:

$$R_F = F_y/W \qquad\qquad R_K = K'/(K'+K'')$$

The diagram in Fig.8 shows the maximum cycle of hysteresis of the isolation systems of the structure designed according to two solutions proposed, obtained by means of the SDOF model with El Centro accelerogram and outlines the different behaviour of isolated systems having either a limited or marked non linearity.
The first solution - where R_K=0.5 - represents a typical application of the strategy of the natural period of oscillation with a mild equivalent damping slightly lower than 10 per cent, while the second solution follows a mixed strategy that combines a high period with a considerable non-linearity and thus a high energy dissipation with equivalent damping coefficients which seem to reach values that may also exceed 30 per cent.
It should be stressed that such a type of analysis gets closer to the truth the higher the isolation of the structure and the lesser the non linearity of the system.

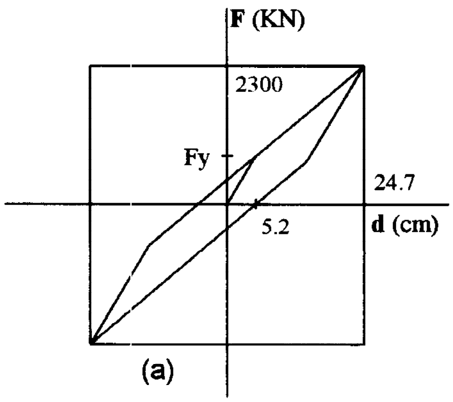

Fig. 8. Maximum hysteretic cycles of the isolation devices of the structure designed where α=2 R_F=0.029 (a) first solution and (R_K=0.5) (b) second solution

For the present yielding threshold of the entire elastic and elastic plastic system - equal to the maximum wind force - Fig 9-12 diagrams show the dynamic response in terms of acceleration and relative displacements when the vibration period changes.

The comparison between the two solutions adopted is made in terms of the secant period that corresponds to the maximum relative displacement vis-à-vis the strongly non linear behaviour of the systems and the suitability of this parameter to represent the average dynamic properties of such structures.

Since it was decided that in the design stage a value ranging between 1.5 and 2 would be set for the safety degree of the instability due to vertical loads - it is possible to identify the segments of the graphs that fulfill this design criterion. It is thus easy to recognise for each design solution and for the range of the α coefficients taken into consideration the values of the equivalent periods that characterise the isolated systems.

Hence the results point to the fact that the first solution involving higher equivalent frequencies is characterised -being the safety coefficients of instability equal - by greater relative displacements than those required by the second design solution; it also implies a higher economic cost for the production of the elastic system. However data need to be validated by a more reliable analysis using a system having more degrees of freedom.

A further numerical analysis made in some previous studies permitted to evaluate the influence of the variations of the vertical force on the structural response. The building schematised by the simple SDOF model simultaneously with the horizontal (EW) and vertical components of the accelerogram of Bagnoli Irpino has revealed the scarce impact of such a factor on the structural response even though further analyses in this direction would be needed concerning earthquakes with stronger vertical components.

The design solutions outlined here describe the applications of strategies both required a considerable variation of the fundamental period of vibration of the structure, but having a low dissipating capacity and a considerable one respectively.

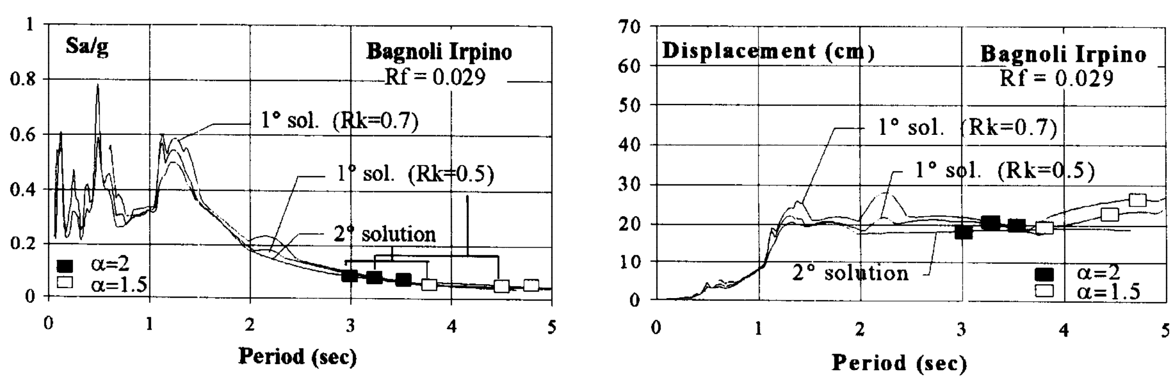

Fig. 9. Response spectrum of non linear SDOF in terms of acceleration and relative displacements for the accelerogram of Bagnoli Irpino (1st and 2nd solutions)

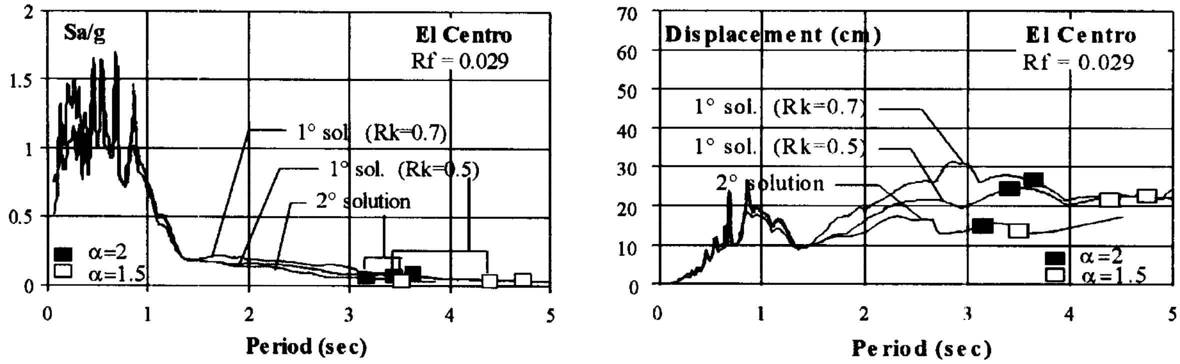

Fig. 10. Response spectrum of non linear SDOF in terms of acceleration and relative displacements for the accelerogram of El Centro (1st and 2nd solutions)

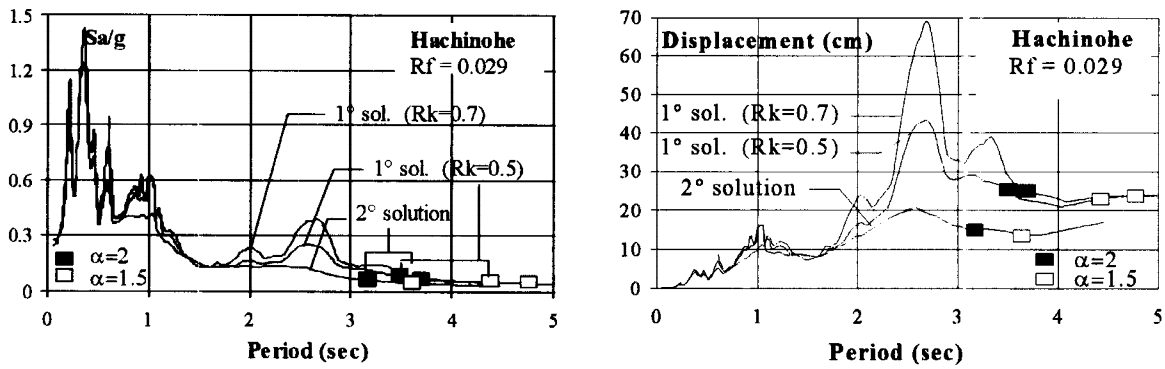

Fig. 11. Response spectrum of non linear SDOF in terms of acceleration and relative displacements for the accelerogram of Hachinohe (1st and 2nd solutions)

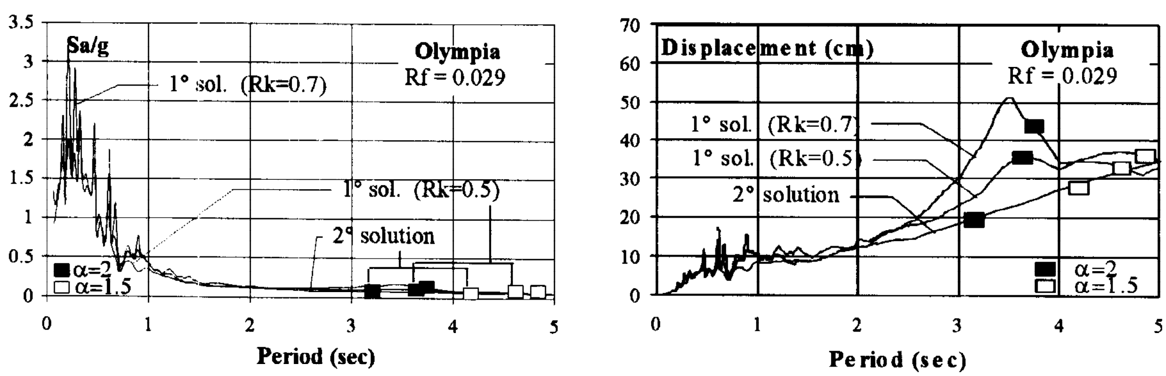

Fig. 12. Response spectrum of non linear SDOF in terms of acceleration and relative displacements for the accelerogram of Olympia (1st and 2nd solutions

It should be noted that the simplified dynamic analysis carried out with the single non linear degree of freedom system - in theory - provides reliable results only in case of low damping systems or of those systems characterised by a slight non linearity. Therefore, its reliability is likely to be ensured in the case of the first design solution having an equivalent damping less than 10 per cent, while it should still be verified in the case of the second solution where a more marked non linearity would seem to question the proposed results.
We are planning an analysis based on a system having more degrees of freedom in order to validate the results so far achieved and to carefully evaluate the influence of superior vibration modes on the structural response and, thus, on the efficacy of the isolation system under consideration.