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RETROFITTING OF REINFORCED CONCRETE BUILDINGS NOT DESIGNED TO WITHSTAND SEISMIC ACTION: A CASE STUDY USING BASE ISOLATION

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SUMMARY

This paper deals with the seismic retrofitting of two reinforced concrete buildings in Eastern Sicily not originally designed to withstand the seismic action. Some special characteristics of the two buildings suggested the choice of a base isolation retrofitting system. Tests performed on the original building materials and also structural analyses performed on the original and on the retrofitted buildings suggested that some stiffening of the superstructure was required. The reasons that led to the retrofitting and to the choice of the retrofitting system will be presented in some detail. The analyses conducted for the evaluation of the seismic resistance and vulnerability of the existing buildings and of the seismic resistance of the retrofitted buildings will also be presented.

INTRODUCTION

Eastern Sicily was only classified as an area of medium seismic hazard in the early '80s. As a result, most of the reinforced concrete buildings erected in the post-war period of economic boom were not designed and constructed to withstand seismic action. Seismological studies have shown that the area has one of highest seismic hazard levels in Italy [1, 2, 3]. The problem of improving the seismic resistance of such buildings has been recognized and several research projects have been supported by Italian Research Agencies to ascertain the seismic risk and to suggest ways to mitigate it. One of the most significant of these was the Catania Project [4] which had the participation of prominent Italian researchers in earthquake engineering. The main findings concerning the vulnerability of the built environment, apart from the main research report [4], have been collected in several volumes published by GNDT-CNR [5, 6, 7] which may be consulted at the web address of the National Institute of Geophysics and Volcanology (INGV): http://gndt.ingv.it/Pubblicazioni/Pubblicazioni_home.htm. Some results of the studies by the present authors have also been reported in the international literature [8].
The vulnerability of the built environment to seismic hazard was highlighted in 1990 by a moderate earthquake that struck the area, resulting in several casualties and significant damage [2]. As a consequence of this earthquake a repair and retrofitting program was carried out in the '90s resulting in

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several reinforced concrete buildings retrofitted by increasing resistance and stiffness mainly by the introduction of shear walls and stiffening cores [9]. Severe damage affected several reinforced concrete buildings in areas characterized by soft soils. The senior author was involved in the retrofitting program of the reinforced concrete buildings belonging to the Autonomous Institute for Council Houses (IACP) in the Syracuse province of Eastern Sicily. Recently this Institute decided to complete two reinforced concrete buildings that had been designed and erected in the middle ‘70s and had never been finished due to bankruptcy of the contractor. Environmental action and acts of vandalism had left the buildings in a very poor condition and their demolition and reconstruction became a reasonable option. However such an option could have posed several legal and psychological problems which it was best to avoid. Fortunately the buildings presented some characteristics that made them particularly suitable for retrofitting by base isolation. The foundation soil consisted of limestone of the Climiti Mountain formation, thus avoiding problems of high spectral responses in the low-frequency range. The foundations were of the strip, or inverted beam, type and there were short columns of about one meter length between the foundation and the first floor slab. This made the cutting of the columns and the support of the building two relatively simple operations. For these reasons base isolation was considered as the most suitable retrofitting system for the buildings under consideration.

The experience of the authors with seismic resistance and vulnerability analyses of reinforced concrete buildings in Eastern Sicily did not suggest that resistance and vulnerability evaluation was worthwhile for the buildings under consideration. Nevertheless, vulnerability analyses were performed in order to assess whether the buildings could withstand the action prescribed by the seismic code in their present state or only with light strengthening.

This work presents the seismic resistance and vulnerability analyses performed on the original buildings, and provides an evaluation of the resistance of the buildings in relation to the current code requirements and their corresponding vulnerability. These evaluations have been performed not only for the seismic hazard specified for the actual site but for all seismic zones considered by the current Italian code [10] and for all local soil conditions considered by Eurocode 8 [11]. For the evaluation of the seismic resistance and vulnerability of the buildings considered the procedure suggested by FEMA [12, 13], as modified by the authors [8], has been used. The same procedure has been used for the evaluation of the seismic resistance and vulnerability of the retrofitted buildings.

DETAILS OF THE SOLARINO BUILDINGS

The buildings were constructed in the second half of the ‘70s in the small town of Solarino in the province of Syracuse in Eastern Sicily. The two buildings are almost identical and were left unfinished, as is shown in the photographs of figure 1, due to bankruptcy of the construction firm. As it may be seen each building has four stories including a soft first story. There are two apartments for each of the stories above the first one to give a total of twelve apartments for the two buildings. The partition and peripheral walls, due to the environmental action and to acts of vandalism, were in a very bad condition and had to be demolished and reconstructed. Therefore only the moment resisting reinforced concrete frame structure was preserved. The foundation structure appears as is shown in figure 2. The small distance between the top of the foundation beams and the bottom of the first floor slab provides the means for a simple support of the building when the short columns have to be cut and the base isolation devices inserted as a replacement.

At the time of construction quality assurance controls for the materials either did not exist or were not effective; therefore the design properties of the materials could not be relied upon and some concrete and steel samples were collected and tested. As a result of these tests the properties of the materials shown in table 1 were found.

Figure 1. The Solarino building.

Figure 2. The foundation structure.

Table 1. Mechanical properties of concrete and steel.

Concrete characteristic cubic strength, f_{cu} (N/mm^2)	**13**
Concrete ultimate design strength, 0.83 f_{cu} (N/mm^2)	10.79
Elastic modulus for concrete, E_c (kN/mm^2)	20.55
Concrete maximum parabolic strain, ε_{co}	0.2%
Concrete ultimate strain, ε_{cu}	0.35%
Steel type	**Fe B 38 k**
Steel characteristic ultimate strength, f_{su} (N/mm^2)	450
Steel characteristic yield strength, f_{sy} (N/mm^2)	375
Elastic modulus for steel, E_s (kN/mm^2)	206
Steel ultimate strain, ε_{su}	1%

SEISMIC RESISTANCE AND VULNERABILITY OF THE EXISTING BUILDINGS

Because the two buildings had identical design and were actually constructed to their design, only one will be considered in the following analyses. In order to prove that seismic retrofitting was required, the seismic resistance and vulnerability to the design earthquake need to be established. For this purpose the procedure outlined in the FEMA documents [11, 12] was applied in the form modified by the authors [8]. This procedure is briefly outlined below.

Procedure for the evaluation of the seismic resistance

The procedure for the evaluation of the seismic resistance of a building outlined by the FEMA documents requires the reduction of the building structural model to a one-degree-of-freedom system. This procedure starts by choosing a control displacement and evaluating the force parameter corresponding in a single-degree-of-freedom model for the chosen displacement. The authors have modified this procedure by choosing the force parameter and evaluating the corresponding displacement. In this way the force parameter, chosen as the base shear of the building, acquires a definite physical meaning. The corresponding displacement is evaluated according to work equivalence.

The single-degree-of-freedom equivalent system

As is well known, the reduction to one-degree-of-freedom of the structural model is carried out on the basis of a pushover curve drawn in terms of base shear and control displacement. The FEMA procedure applies a transformation to the control displacement to obtain the single-degree-of-freedom displacement and another transformation to the story force vector to obtain the corresponding force parameter.

The procedure suggested by the authors could be applied to any multi-degrees-of-freedom model governed by a nonlinear algebraic system of the form

$$\mathbf{K}(\mathbf{u}) \cdot \mathbf{u} = \mathbf{f} \tag{1}$$

where $\mathbf{u}$ is the vector of nodal displacements, $\mathbf{f}$ is the corresponding force vector and $\mathbf{K}(\mathbf{u})$ is the nonlinear stiffness matrix.

The work performed by the external force vector $\mathbf{f}(t)$ is given by

$$W(t) = \int_0^t \mathbf{f}(\tau) \cdot \dot{\mathbf{u}}(\tau) \cdot d\tau = \int_0^t \mathbf{u}(\tau) \cdot \mathbf{K}(\mathbf{u}(\tau)) \cdot \dot{\mathbf{u}}(\tau) \cdot d\tau \tag{2}$$

The base shear components $V_x(t)$ and $V_y(t)$ may be evaluated from the force vector $\mathbf{f}(t)$ by the scalar products

$$\begin{aligned} V_x(t) &= \mathbf{f}(t) \cdot \mathbf{r}_x \\ V_y(t) &= \mathbf{f}(t) \cdot \mathbf{r}_y \end{aligned} \tag{3}$$

where $\mathbf{r}_x$ and $\mathbf{r}_y$ are fixed influence vectors.

The base shear is obtained by synthesis of its components in the form

$$V_b(t) = \sqrt{V_x^{\,2}(t) + V_y^{\,2}(t)} \tag{4}$$

The single-degree-of-freedom equivalent system is defined in terms of the base shear $V_b(t)$ and its corresponding displacement $u(t)$ evaluated on the basis of the following work equivalence

$$W(t) = \int_0^t V_b(\tau) \cdot \dot{u}(\tau) \cdot d\tau \tag{5}$$

The above procedure, when applied to proportional loading, leads to a non-linear relationship described in terms of the two parameters $V_b(t)$ and $u(t)$.
Following FEMA this relationship is transformed into an equivalent bi-linear one from which an initial stiffness and a post-yield stiffness may be derived together with an equivalent ductility ratio. This procedure will be illustrated in detail with its application to the case being considered.

The force vector
The force vector used in the pushover analyses described by equation (1) is evaluated on the basis of the first vibration mode in the direction of the ground motion considered. The identification of such a mode is always possible in a symmetrical system although it may prove a difficult task in the case of an unsymmetrical one. The dependence on the time parameter t is taken of the form

$$\mathbf{f}(t) = t\,\mathbf{f}_0 \tag{6}$$

In the case of the building considered, the first mode in the longitudinal direction and the first mode in the transverse direction were considered. The story force distribution, evaluated according to the first mode of vibration in the two directions considered, is shown in figure 3 together with the standard code distribution.

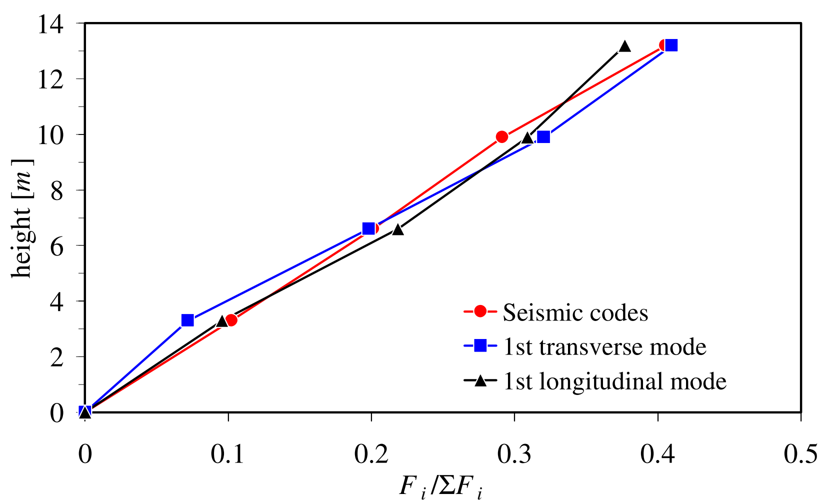


Figure 3. Story force distributions considered in the pushover analyses.

Effective periods and ductility ratios
By applying the procedure outlined in the previous paragraph, the force/displacement relationships shown in figures 4a and 4b are found for the transverse and longitudinal directions respectively. The non-linear relationships are replaced by bi-linear ones on a basis of work equivalence as shown in the same figures. Some relevant parameters are obtained for the bi-linear system among which are the effective yield base shear coefficient, the effective initial stiffness, the hardening stiffness, the effective yield displacement and the collapse displacement. It should be noticed that the collapse state has been associated with the rupture of the first plastic hinge, because it has been assumed that the following stress redistribution would have enacted a sort of chain reaction by which plastic hinges would have ruptured sequentially, rapidly leading to collapse.

The characteristics of the bi-linear systems outlined above enable the evaluation of some significant system parameters, such as the effective period of vibration and the equivalent ductility ratio. For the evaluation of the effective period of vibration, the effective initial stiffness and the equivalent mass of the system are required. In the present work, and in previous ones by the authors, the system mass has been taken as the mass of the building. This is a standard practice when a single-mode model is used to represent the structural behaviour.

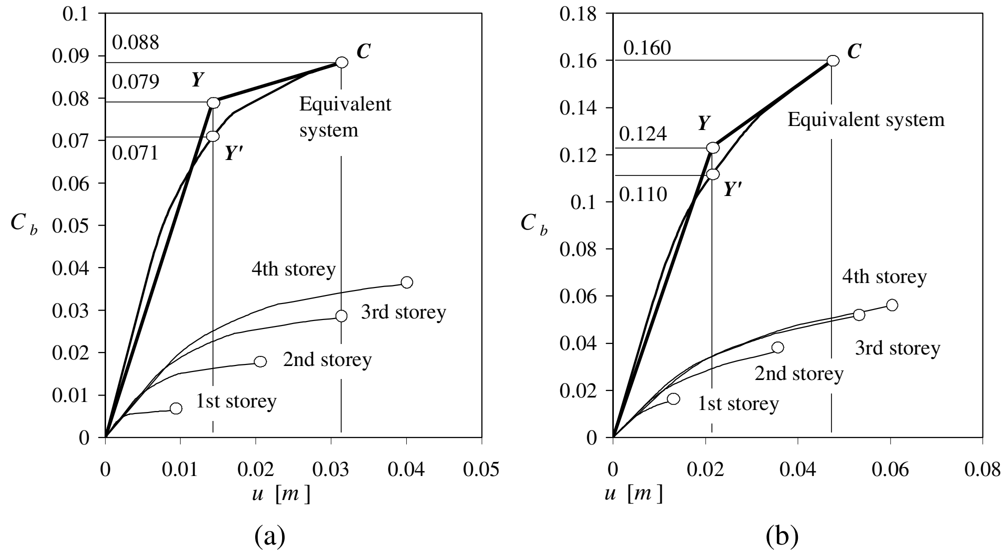


Figure 4. Definition of the single-degree-of-freedom equivalent system: (a) transverse direction, (b) longitudinal direction.

The evaluation of the ductility ratio requires some special considerations due to the presence of the hardening branch of the bi-linear relationship. It is obvious that the ratio between the collapse displacement and the effective yield displacement cannot be used as an effective definition of the ductility ratio; this is because all the relationships which use the ductility ratio were derived for elastic–perfectly plastic systems. An equivalent elastic-perfectly plastic system was used in order to define the effective ductility ratio. This system was again evaluated on the basis of work equivalence by maintaining the collapse force and reducing the collapse displacement as shown in figure 5. As may be seen from this figure the reduction in the ductility ratio may be quite significant because not only the effective collapse displacement is reduced but also the effective yield displacement is increased. The equivalent ductility ratio takes the form

$$\mu = \frac{1}{2}\left(1+\frac{1}{h}\right)\frac{u_c}{u_y^p}+\frac{1}{2}\left(1-\frac{1}{h}\right) \tag{7}$$

where $u_y^p = F_c / K_{eff}$ is the effective yield displacement and h is a hardening measure defined as:

$$h = \frac{F_c}{F_y} = \frac{u_c}{u_y} \tag{8}$$

The main characteristics of the equivalent single-degree-of-freedom systems are shown in table 2.

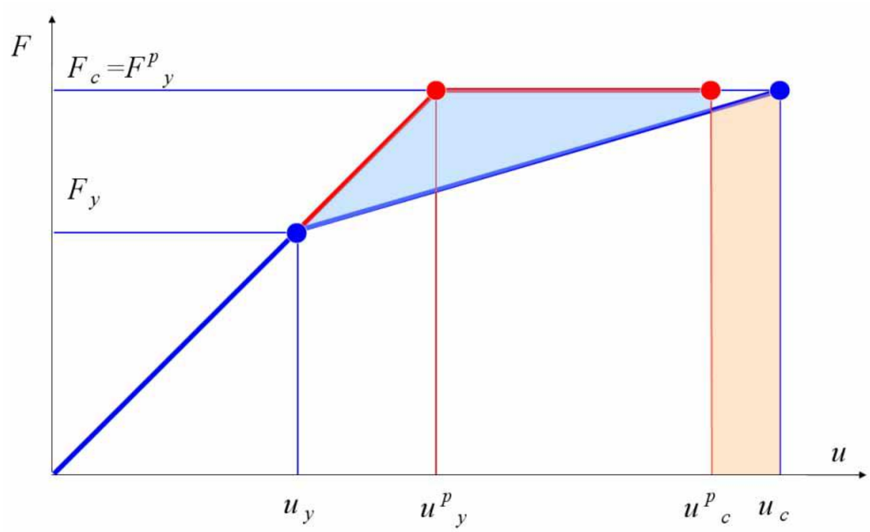


Figure 5. An elastic–perfectly plastic system equivalent to the hardening system.

Table 2. Characteristics of the elastic–perfectly plastic equivalent systems.

Direction	Transverse	Longitudinal
Yielding C_b for the bilinear system, $C_{b,y}$	0.079	0.124
Effective yielding base-shear coefficient, $C_{b,y'}$	0.071	0.110
Collapse base-shear coefficient, $C_{b,c}$	0.088	0.160
Effective yielding displacement, u_y (cm)	1.43	2.15
Collapse displacement, u_c (cm)	3.16	4.78
Effective period, T_{eff} (s)	0.85	0.84
Hardening force ratio, $h = C_{b,c}/C_{b,y}$	1.12	1.30
Global ductility, μ	1.98	1.92

Evaluation of the seismic resistance

The seismic resistance of any structure cannot be independent of the seismic action. It is the seismic action that determines the forces that affect the structure and only the knowledge of these forces enables the structural analyst to ascertain the resistance of the structure to such forces. The seismic action is usually specified in terms of earthquake design spectra. In terms of spectral acceleration these may be described as follows

$$S_a = a_g S \beta_0 \frac{f(T_{eff}, \zeta, S)}{q} \tag{9}$$

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

$$q = \frac{S_e}{S_a} \tag{10}$$

Here S_e is the elastic spectral acceleration corresponding to the effective period T_{eff} considered. The damping ratio ζ in the spectral function $f(T_{eff}, S)$ depends on the structure's damping capacity [13]. It is worth noticing that the behaviour factor q may be expressed as a function of the ductility ratio as is widely reported in the literature [13].
Equation (9) is the basis for the evaluation of the seismic resistance of the building. In fact all the parameters involved may be evaluated from the equivalent one-degree-of-freedom system apart from the peak ground acceleration a_g which is the unknown of the problem. By noticing that S_a can be evaluated from the base shear coefficient at collapse

$$S_a = C_b \cdot g \tag{11}$$

inverting equation (8), it follows that

$$\frac{a_{g,b}}{g} = \frac{qC_{b,c}}{S\beta_0 f(T_{eff}, \zeta, S)} \tag{12}$$

Equation (12) provides the seismic resistance of the building expressed in terms of the effective peak ground acceleration.

Relative seismic resistance, seismic vulnerability and seismic over-resistance
The seismic resistance of the building must be compared to the earthquake demand in terms of effective peak ground acceleration $a_{g,c}$ as specified by the seismic code or otherwise evaluated. A convenient way of making this comparison is through the *relative seismic resistance R* defined as follows

$$R = \frac{a_{g,b}}{a_{g,c}} \tag{13}$$

Accordingly the vulnerability V and the over-resistance OR of the given building are defined by

$$\begin{aligned} V &= 1 - R \quad \text{for} \quad R \le 1 \\ OR &= R - 1 \quad \text{for} \quad R \ge 1 \end{aligned} \tag{14}$$

Application to the Solarino building
The application of the procedure outlined above to the Solarino building provides the results given in table 3 where reference is made to the seismic zones of the current Italian code [14] and to the site conditions specified by Eurocode 8 [10]. Here it suffices to notice that type A soil refers to firm ground while types B and C refer to soft soils of increasing compliance.
It may be noticed that the building considered presents different levels of vulnerability in seismic zones 1 and 2, depending on the soil conditions. In seismic zone 3 the building would be safe only if on firm ground (soil type A), while in seismic zone 4 the building possesses some over-resistance.
A word of caution is needed at this point because the results presented should be used with engineering judgment. They provide a general idea of the seismic resistance of the building considered but could result both in underestimates or overestimates of the actual resistance. For instance the resistance of peripheral and partition walls is not considered in the analyses and the rupture of the first plastic hinge has been assumed as the state of incipient collapse. On the other hand, it is well-known that reinforced concrete

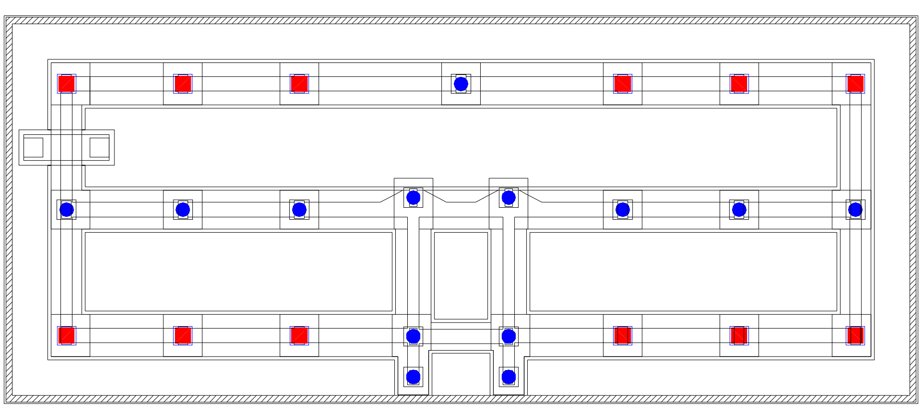
buildings may exhibit node cracking and reinforcement slip and these effects have not been taken into account in the pushover analyses. In the present case an additional cause of overestimation is the presence of the short pillars above the foundation which would probably result in shear failure and lower resistance. These pillars have not been included in the pushover analyses of the superstructure.
The analysis, however, clearly shows that the building considered, located in seismic zone 2 on soil type A, requires seismic retrofitting.

Table 3. Seismic resistance R (%) of the considered building.

Seismic zone	$a_{g,c}/g$	Soil type	Trasverse direction	Longitudinal direction
1	0.35	A	**43**	**66**
		B	**28**	**44**
		C	**24**	**36**
2	0.25	A	**60**	**92**
		B	**40**	**61**
		C	**33**	**51**
3	0.15	A	**100**	**153**
		B	**66**	**102**
		C	**55**	**85**
4	0.05	A	**300**	**460**
		B	**200**	**306**
		C	**166**	**256**

THE RETROFITTING DESIGN

For the reasons given in the Introduction, the choice of the retrofitting system fell on base isolation for which laminated rubber bearings and low-friction bearings were used as shown in figure 6. The properties of the laminated rubber bearings, as provided by the manufacturer, resulted in the elastic force-displacement relationship represented in figure 7 and the damping ratio-displacement relationship shown in figure 8. The friction coefficient for the low-friction bearings derived from test results provided by the manufacturer is somewhat dependent on the applied load but is always less than 1% in dynamic conditions and less than 2% in static conditions.



Laminated rubber bearing   Low-friction bearing

Figure 6. Plan of the foundation structure with the base isolation system.

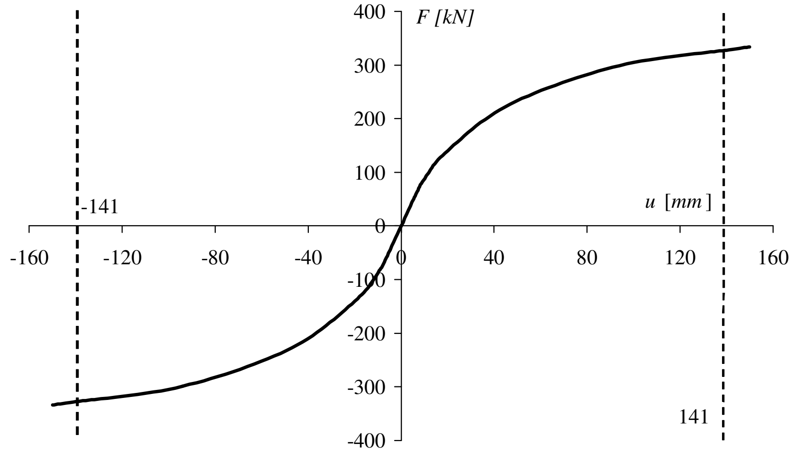


Figure 7. Elastic force-displacement relationship for the 12 laminated rubber bearings.

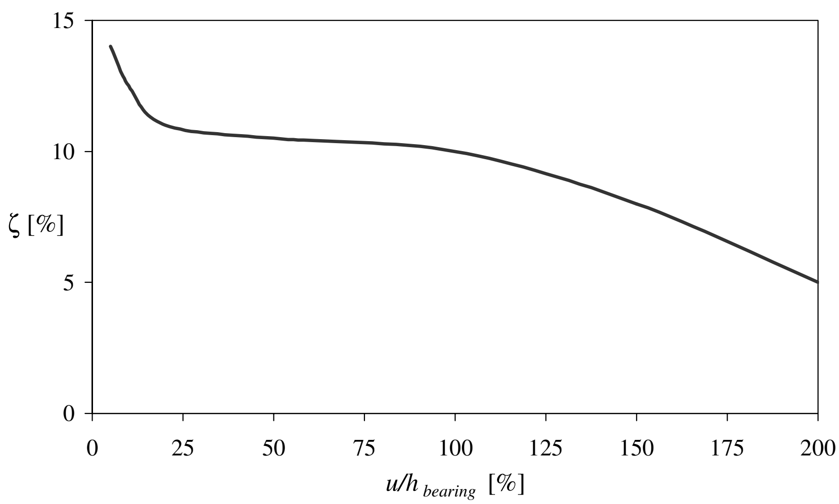


Figure 8. Damping ratio-displacement relationship for the laminated rubber bearing.

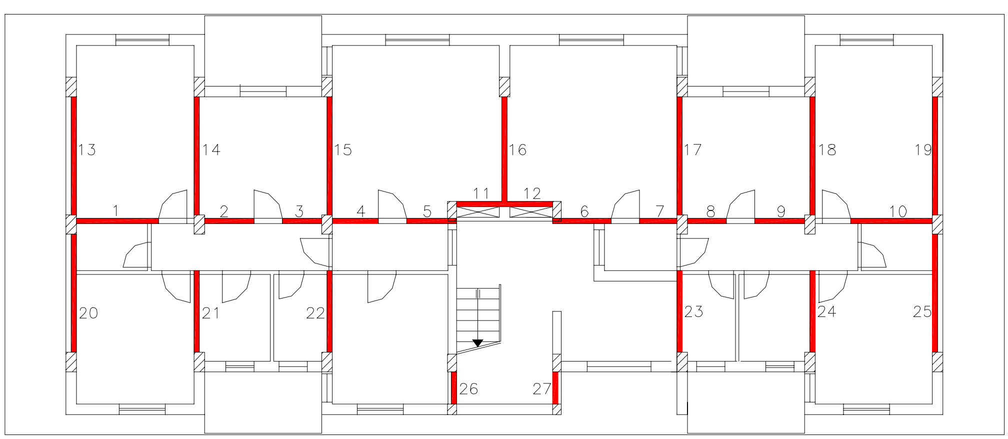


Figure 9. Typical floor plan with location of the thin reinforced concrete walls.

The poor concrete properties evaluated for the existing building required some strengthening of the superstructure which is shown in figure 9. The thin reinforced concrete walls (15 cm thick) provide additional stiffness to the superstructure, limiting inter-story drifts, and provide additional support for the gravity loads.

SEISMIC RESISTANCE OF THE RETROFITTED BUILDING

The procedure used for the evaluation of the seismic resistance of the existing building has been used also for the evaluation of the seismic resistance of the retrofitted building. The results of the pushover analyses are shown in figure 11a for the superstructure and in figure 11b for the basement. These were obtained under the load distributions shown in figure 10. As it may be seen the behavior of the superstructure is elastic up to the ultimate displacement of the base isolation system.

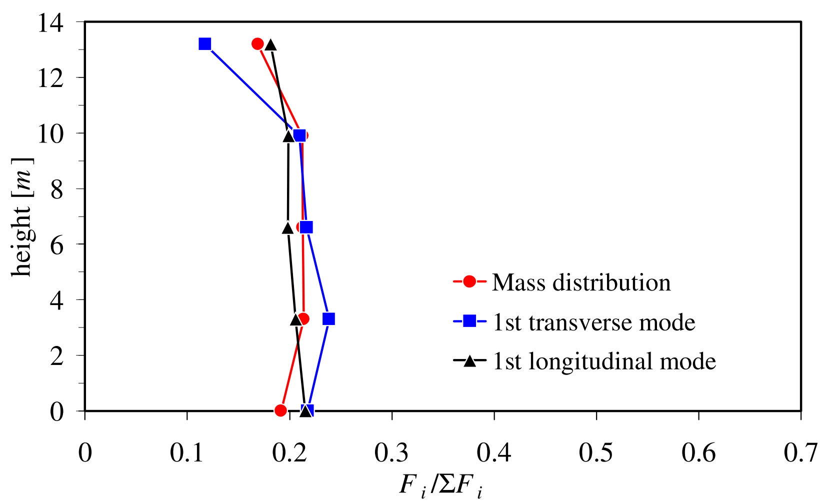


Figure 10. Story force distributions considered in the pushover analyses.

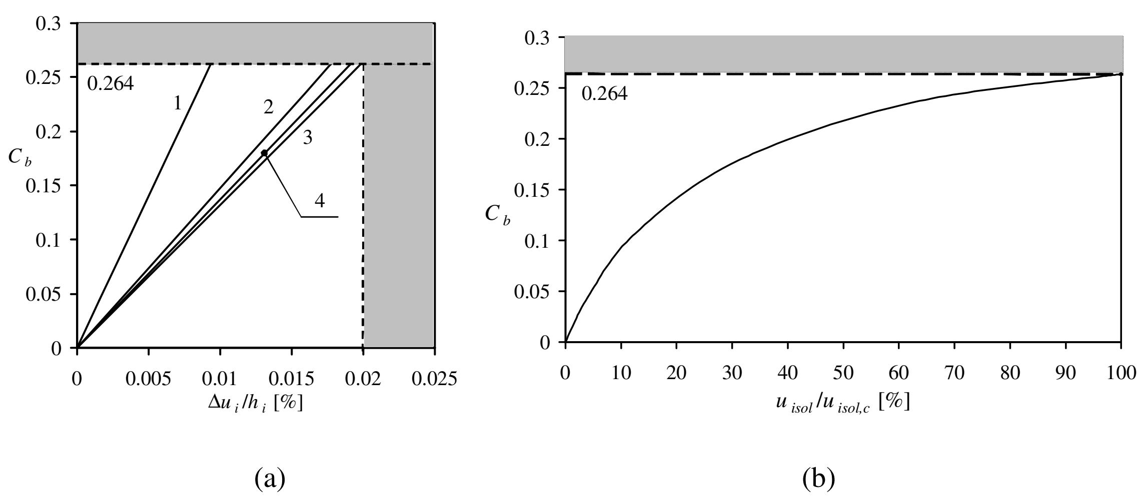


Figure 11. Base shear coefficient against (a) superstructure inter-story drift and (b) lateral displacement of the laminated rubber bearings.

Equivalent two-degrees-of-freedom system

Due to the different behavior of the superstructure and of the isolation system, it is convenient to reduce the actual system to a two-degrees-of-freedom model, one degree-of-freedom representing the superstructure behavior and the other the isolation system behavior (figure 12). The properties of the superstructure model are derived in the same way as shown above with the significant difference that the final result is a linear model rather than a bilinear one. The dynamic properties of the superstructure are shown in table 4.

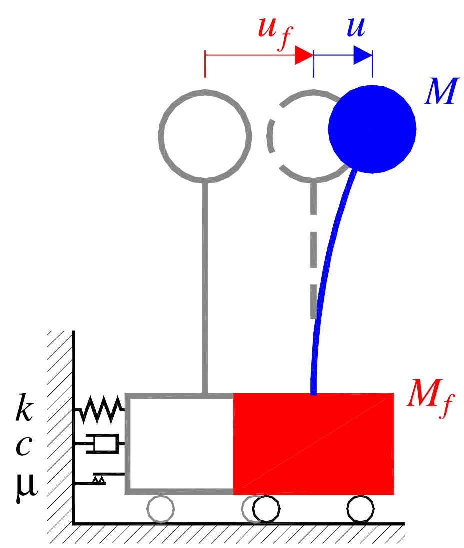


Figure 12. Two-degrees-of-freedom equivalent system.

Table 4. Dynamic characteristics of the superstructure.

Direction	Stiffness (MN/m)	Mass (kNs^2/m)	Period (s)	Damping ratio
Transverse	2769	1214	0.13	2%
Longitudinal	2445	1214	0.14	2%

The properties of the base isolation system are represented by an equivalent nonlinear spring, an equivalent nonlinear viscous damper, and an equivalent friction damper characterized by a static and dynamic friction force. The elastic restraining force and the viscous damping force are obtained respectively from the relationships given in figures 7 and 8 by accounting for the number of laminated rubber bearings. Similarly the friction force is obtained by adding the friction forces of the individual low-friction bearings. Due to the nonlinear characteristics of the base isolation system, the undamped fundamental period of the combined system is evaluated with reference to the ultimate secant stiffness.
The dynamic characteristics of the combined system are shown in table 5.

Table 5. Dynamic characteristics of the two-degrees-of-freedom equivalent system.

Direction	Periods		Friction coefficient	
	T_1 (s)	T_2 (s)	μ_s (static)	μ_d (dynamic)
Transverse	1.46	0.057	2%	1%
Longitudinal	1.46	0.061	2%	1%

Seismic resistance

The seismic resistance of the retrofitted building may be evaluated again by using equation (12). However some care is needed in the evaluation of the parameters that must be used. The structure behavior factor q this time must be set equal to 1 because there is no plastic dissipation anywhere in the structure. Furthermore the spectrum shape function $f(T_{eff},S,\zeta)$ must be evaluated for an appropriate damping ratio. This should account for the damping in the superstructure, the viscous damping in the laminated rubber bearings and the friction damping in the low-friction bearings. Considering that the superstructure behaves almost as a rigid body and that the friction coefficient for large displacements is less than 1%, for design purposes it is convenient to consider only the viscous damping in the laminated rubber bearings which is evaluated at the ultimate displacement.

Results

The results in terms of relative seismic resistance R, over-resistance OR and vulnerability V are given in table 6 for the four zones of seismic hazard of the current Italian regulations and for the three classes of soil conditions specified by Eurocode 8. It should be noticed that no distinction has been made in table 6 for the behaviour in the longitudinal and in the transverse direction because, owing to the presence of the base isolation devices and neglecting the low friction, the building behaviour is essentially isotropic, that is direction-independent. In spite of the fact that the retrofitting was designed for seismic zone 2 and soil condition A, the building shows a considerable over-resistance even for seismic zone 1 and soil condition A. However it shows some vulnerability for seismic zone 1 and soil conditions B and C.

Table 6. Relative seismic resistance R, over-resistance OR and vulnerability V for the retrofitted building in different seismic zones and soil conditions.

Seismic zone	Soil type	R (%)	OR (%)	V (%)
1	A	**135**	**35**	-
	B	**90**	-	**10**
	C	**75**	-	**25**
2	A	**189**	**89**	-
	B	**126**	**26**	-
	C	**105**	**5**	-
3	A	**315**	**215**	-
	B	**210**	**110**	-
	C	**175**	**75**	-
4	A	**944**	**844**	-
	B	**630**	**530**	-
	C	**524**	**424**	-

DYNAMIC BEHAVIOUR

The equations of motion for the two-degrees-of-freedom model of figure 12 may be written in the following form

$$\begin{cases} M\,\ddot{u} + c\,\dot{u} + k(u)\,u = -m\,\ddot{u}_g(t) \\ u_f = 0 \end{cases} \quad \text{for} \quad \left(M + M_f\right)\ddot{u}_g < f_s \tag{15}$$

for the sticky phase, and in the form

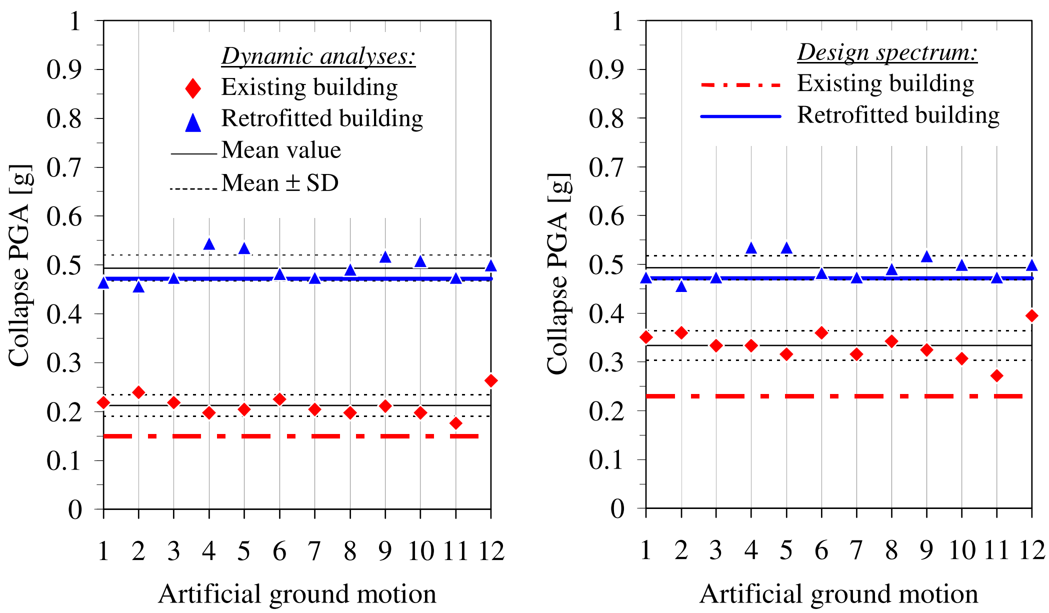
$$\mathbf{M}\cdot\ddot{\mathbf{u}}+\mathbf{C}(u_f)\cdot\dot{\mathbf{u}}+\mathbf{K}(u,u_f)\cdot\mathbf{u}-\mathrm{sgn}(\dot{u}_f)\,f_d\,\mathbf{r}=-\mathbf{M}\cdot\mathbf{r}\cdot\ddot{u}_g(t) \tag{16}$$

for the sliding phase. The pseudo-static displacement vector, **r**, the dynamic displacement vector, **u**, the mass, damping and stiffness matrices, **M**, **C**, **K**, take the form given below.

$$\mathbf{r}=\begin{bmatrix}0\\1\end{bmatrix};\quad \mathbf{u}=\begin{bmatrix}u\\u_f\end{bmatrix};$$

$$\mathbf{M}=\begin{bmatrix}M & M\\ M & M+M_f\end{bmatrix};\quad \mathbf{C}(u_f)=\begin{bmatrix}c & \\ & c_f(u_f)\end{bmatrix};\quad \mathbf{K}(u,u_f)=\begin{bmatrix}k(u) & \\ & k_f(u_f)\end{bmatrix}$$

The symbols in the above matrices and vectors have the following meanings: u, displacement of the superstructure mass relative to the foundation; u_f, displacement of the foundation mass relative to the ground; $\ddot{u}_g$, ground acceleration; M, mass of the superstructure; M_f, mass of the foundation; f_s static friction force; f_d dynamic friction force; c, damping coefficient for the superstructure; c_f, damping function for the rubber bearings; k, effective stiffness for the superstructure; k_f, effective stiffness of the rubber bearings. The equations of motion (15) and (16) have been solved by numerical integration using a fourth order Runge-Kutta method considering 12 spectrum-compatible artificially-generated ground motions. Each ground motion has been scaled up to the point when the collapse of the structure has occurred because the limit displacement for the rubber bearings has been reached. The peak ground accelerations (PGA) at which collapse occurred are shown in figure 13a for the transverse direction and figure 13b for the longitudinal direction where they are compared against their mean value, mean value ± one standard deviation, and the value provided by equation (12). In the same figures the corresponding values for the original building are shown, which exhibit different behaviour in the two directions considered. It may be noticed that the results obtained are generally above those provided by equation (12) and in the few cases when they are below the discrepancy is not significant. The benefit achieved by the retrofitting system may be easily seen.



(a) (b)

Figure 13. Collapse peak ground acceleration: (a) transverse direction; (b) longitudinal direction.

CONCLUSIONS

The procedure used for the evaluation of the seismic resistance and vulnerability of two buildings in Eastern Sicily, not designed to withstand the seismic action, has been described and applied. The retrofitting system applied to the two buildings has been presented and its effectiveness has been demonstrated. During the writing of this paper the rubber bearings and the low friction bearings described in the above text have already been positioned in one building and are being positioned in the other. As a check of the effectiveness of the retrofitting system design and of the proper execution of works a dynamic test has been scheduled in which the design displacement will be imposed and suddenly released. The dynamic characteristics of the retrofitted buildings should be found by such a test. The results will be reported at a later stage.

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