SEISMIC RESPONSE OF REINFORCED MASONRY STRUCTURES

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SUMMARY

Shaking table tests were performed in order to investigate the seismic response of a reinforced masonry reduced scale building (scale 1:3). The testing program included 31 seismic tests (characterised by a design PGA starting from 0.06 g to a final value of 1.42 g, in the scale of the model), alternated with sequences of low intensity dynamic characterisations. The damage evolution was analysed by modelling the structure as a non-linear SDOF oscillator, where the input is the ground motion and the output is the building response, measured at the top. The analysis results highlight a very satisfactory seismic behaviour of the tested r. m. construction technique, in terms of response to low-medium intensity earthquake, and behaviour at the ultimate Limit State.

INTRODUCTION

Unreinforced brick masonry is usually mechanically characterised by a reasonably good ultimate resistance, but by a brittle type of failure that makes it unsuitable for buildings in a seismic area. Including steel bars in the masonry is the obvious way to increase ductility, while preserving the known properties of the construction system (heat insulation and soundproofing, durability and fire resistance, low construction and maintenance costs). Only recently the seismic behaviour of reinforced masonry (r.m.) structures has been systematically studied [Tomazevic and Modena, 1989][Shing et al., 1990][Tomazevic and Weiss, 1992]. Nevertheless, a check at the design stage of the ductility of a r.m. structure still reveals many uncertainties that are often reflected in conservative code specifications. These uncertainties are not mainly linked to the response of the single structural elements, but more to the global response of the whole building, the design of which is still traditionally driven by architectural rather than structural reasons. This is particularly true in the case of low-medium rise buildings.

Recently, the seismic behaviour of buildings in r.m. has been studied within the frame of a research project, co-financed by industry and the European Union [Modena et al., 1997]. Laboratory tests of materials [Bernardini et al.,1998], structural components and overall buildings [Modena et al., 1997] are included as significant steps of the project. Particularly, two identical experimental buildings have been constructed, with a r.m. structure and with an infilled frames structure respectively, with the aim of comparing the mechanical performances and dwelling comfort. They are two-storied buildings 9.38 m long, 6.50 m wide and 6.25 m high that represent the common local architectural typology. In order to study seismic behaviour of the prototype in r.m. a reduced-scale model was built and tested on the shaking table at the ENEA research centre in Rome [Modena et al., 1999]. This paper reports the construction of the model, the seismic testing and its main results.

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EXPERIMENTS

Model construction

The model unit was constructed adopting a scale of 1:3 of the lengths, related to the dimensional capacities and load bearing of the shaking table. Plan dimensions were 3.13 x 2.16 m at ground level, and building height was 2.09 m (Fig. 1). The construction system was based on a new type of lightweight block with a single socket hole in which the vertical reinforcements were fitted and with horizontal reinforcements placed in correspondence with the mortar bed joints. The same building materials as those of the prototype were used, so the scale, relating to the Young's modulus and density of the materials used, assumed a unitary value. The bricks used for the model were obtained from blocks used in the prototype (Fig. 2), cutting them to reach the 1:3 scale. Three types of bricks were made as shown in Fig. 2. As vertical reinforcement smooth steel bars, 8 mm and 6 mm diameter were used at corner locations and intermediate zones, respectively. Horizontal reinforcement consisted of 2-mm diameter steel bars. The model was built on a 120-mm thick concrete slab, constructed to allow it to be transported and coupled to the shaking table by means of steel anchors.

The concrete masonry floor slabs were reproduced to scale and additional masses were placed at the roof and at the floor slab level, to take into account different specific density and simulate variable live loads equal to 600 Nm^{-2} at each storey. After defining the scale factors in relation to the three main dimensions (lengths, stress and mass density), the other factors were calculated (Tab. 1).

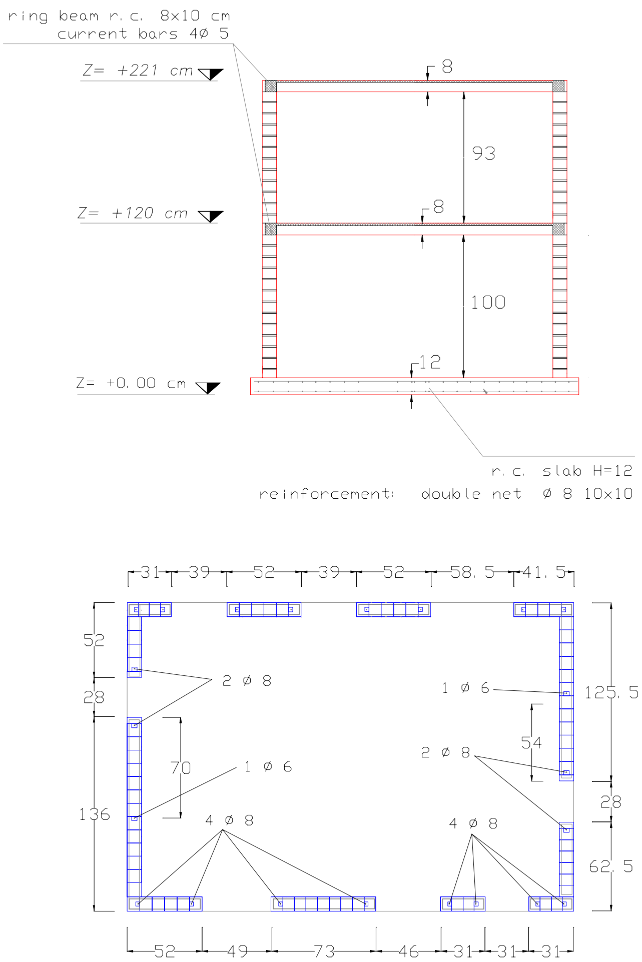

Figure1: Geometrical configuration of the model

Being the acceleration scale factor not equal to 1, the masonry needed to be prestressed, in order to reproduce the same compressive stress at the base of the model as the prototype. Therefore, eight steel tendons, 12 mm in diameter, were anchored to the base slab and to the top of the walls with pre-loaded springs that guarantee the application of an even constant load with moderate displacements of the structure (Fig. 3).

density	$m_r = 1$
modulus E	$E_r = 1$
length	$l_r = 3$
time	$t_r = l_r (E/m)_r^{-1/2} = 3$
frequency	$f_r = l_r - 1(E/m)_r^{1/2} = 1/3$
velocity	$v_r = (E/m)r^{1/2} = 1$
acceleration	$a_r = l_r^{-1} (E/m)_r = 1/3$
stress	$\sigma_r = E_r = 1$
displacement	$u_r = l_r = 3$
force	$F_r = E_r / l_r^2 = 9$

Table 1: Scale factors

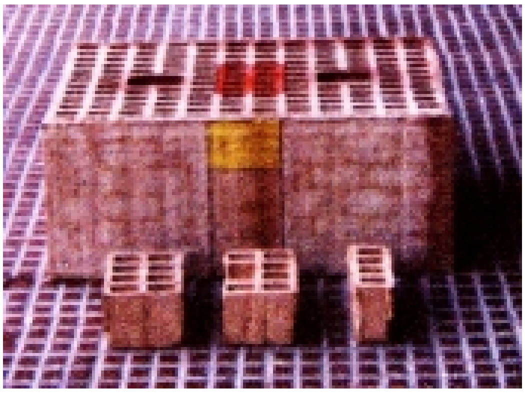

Figure 2: Bricks used in the model compared with that of the prototype .

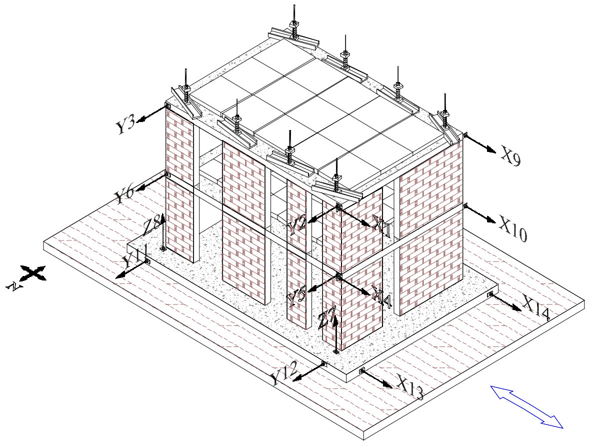

Figure 3: Geometrical configuration of the model and disposition of accelerometers.

Experimental tests

The motion of the most significant points of the structure was measured using 8 piezoelectric accelerometers, positioned as shown in Fig. 3. Four horizontal and two vertical accelerometers (Z8, Z7) were also placed at the base in order to check the motion of the shaking table. The seismic tests were preceded by a series of modal characterisation test. Different experimental techniques were compared (stepped sinusoidal test using an harmonic exciter, shock test using a pulse hammer, random vibration and sine-sweeping test using the shaking table) in order to define a procedure for a rapid and effective characterisation to be repeated after each test. A total of 31 seismic tests were performed at increasing intensity, alternated by sequences of modal characterisations. The same unidirectional ground motion was used for every test, applied to the model in an east-west direction, which corresponds to the direction x in the reference system reported in Fig. 3. The 8 seconds long input accelerogram was generated artificially as response spectrum compatible with the EC8 specifications (Figs. 4, 5). The peak acceleration value was initially fixed at 0.06 g and was increased exponentially in the subsequent tests up to the limits of the capacity of the table. The large number of tests is justified by interest in the evolution of damage following seismic events of medium-low intensity. The first 20 tests were planned with a peak acceleration of less than 0.9 g, corresponding to a real earthquake of 0.3 g. The ultimate limit state was defined as the state of sudden reduction of the base shear in the model response.

The peak accelerations (PGA) in the different tests are reported in Tab. 2, in terms of nominal values of reference signals and effective values achieved with the shaking table. After test 27 (PGA=1.6 g), the control of the table became uncertain and the results unreliable. The damage condition of the building after test 30 suggested the removal of the instruments, making the observations on the seismic response after this testing stage only based on the shock tests information and the cracks pattern inspections.

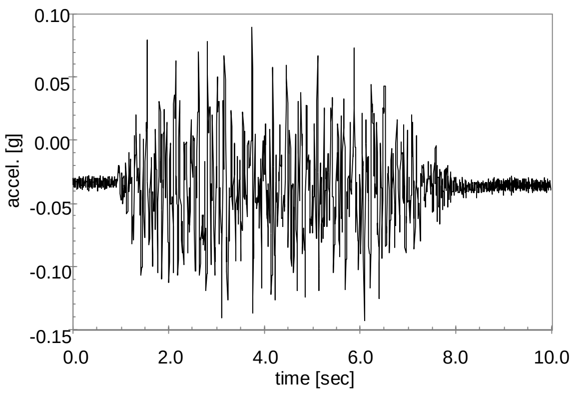

Figure 4: Base input: time history.

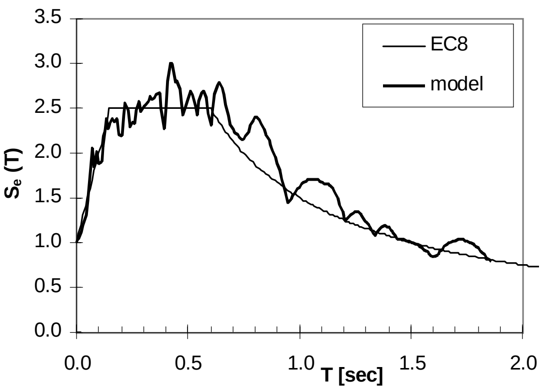

Figure 5: Response spectrum, with ξ equal to 5 %.

QUALITATIVE ANALYSIS OF THE SEISMIC BEHAVIOUR

Predicted failure mechanism

The in-plane failure of a single reinforced masonry pier, under horizontal loads, can be bending or shear dominated. Flexural failure is characterised by the opening of horizontal cracks at the base of the pier, and by the subsequent yielding of the vertical reinforcement and/or by the crushing of the masonry. Shear failure happens by diagonal cracking of the piers. Of course masonry is much more ductile when failure is due to bending than shear [Shing et al., 1990][Tomazevic and Weiss, 1992]. The occurrence of one or the other failure mechanism depends on several parameters, mainly the mechanical properties of the (orthotropic) units and mortar, the distribution of steel reinforcement, the edge supporting conditions and the aspect ratio (i.e. the ratio between the two main dimensions of the piers). In the case study the expected failure mechanism is mainly flexural, as the piers act as vertical cantilevers poorly restrained by the flexible floor slabs.

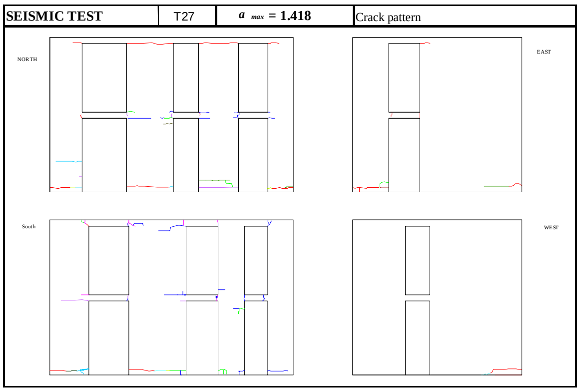

Figure 6: Cracks pattern for Test 27.

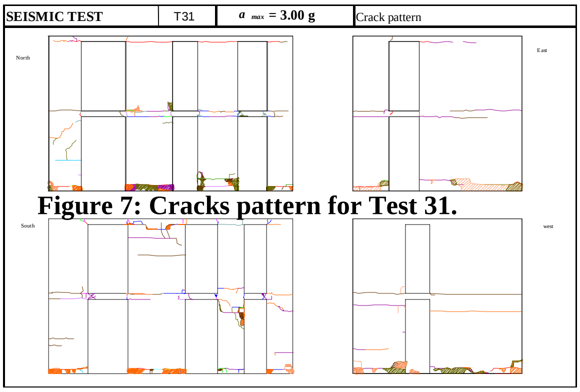

Figure 7: Cracks pattern for Test 31.

Cracks pattern

The evolution of the damage is well described by the trend in the cracks development shown in Figs. 6, 7. The cracks at the base of the piers appeared during the early tests, and by tests 7 and 8 (PGA=0.22 g) small cracks had appeared at the edge beams and at the top the masonry pears.

This pattern did not alter until after the 18th test (PGA=0.62 g), when further movement was noted. From test 22 to test 27 the progressive cracking showed the ductile mechanism of the building: through cracks at the base and at the top of each masonry pier, and plastic hinges at the joint between masonry pier and edge beam at the first floor. This mechanism allows the rigid motion of the masonry piers inducing large structural deformations consistent with the ductility of the vertical reinforcements.

The horizontal reinforcements prevented shear failure, with diagonal cracks noticed only in one pier in the North side and in two piers in the South side, for base accelerations equal to 1.29 g (test 25). With the increasing intensity of the ground motion, residual deformations were observed at the plastic hinges locations (tests 26, PGA=1.411) together with splitting effect at the piers base (Fig. 6, test 27, PGA=1.418 g). No real collapse of the building was experienced even after the most violent shocks (nominal PGA equal to 3.00 g).

Modelling as a single degree of freedom system

When the system is non-linear, the hypothesis of modal superposition is no longer valid [Ewins, 1984] and the “exact” response can be found only by integrating the equations of motion over time. The use of modal decomposition technique provides an optional approach for the analysis of non-linear system as well [Iwan and Yang, 1972]. In the case of multi-storey buildings that presents a regularity in plan (so allowing analysis of the system on a vertical plane) and in elevation, this non-rigorous approach is particularly justified as the system mainly responds according to one of its modal shapes. The system can then be represented as a non-linear SDOF oscillator, where absolute acceleration and relative motion are measured at a reference position (usually the top of the building).

Even if the studied model is not geometrically symmetrical, the distribution of masses and stiffness is so that the first and second modal shapes show displacement components almost exclusively in the major directions x and y, respectively. The structure were therefore modelled as a SDOF oscillator, where the input is the ground motion and the output corresponds to the displacement in direction x of the centre of mass of the second storey, calculated as the average of the response of accelerometers 1 and 9.

Response of a non-linear SDOF system

For non-linear SDOF systems, the equation of the motion, normalised to the mass, is expressed in the general form:

$$\ddot{x} + f(x, \dot{x}) = -\ddot{z}(t) \tag{1}$$

where $\ddot{x}$ is the relative acceleration of the oscillator, $\ddot{z}$ is the input acceleration and $f(x, \dot{x})$ is the generic restoring force. That is:

$$f(x, \dot{x}) = -(\ddot{x} + \ddot{z}(t)) = -a(t) \tag{2}$$

being $a(t)$ the absolute acceleration. The simplest type of non-linear vibrator is the elastic-plastic perfect oscillator (EPP), that is characterised by initial stiffness k_0 and maximum restoring force intensity $F_{\max}$, i.e., in normalised form, by the natural frequency ω_0^2 and by the maximum absolute acceleration. The required ductility is the expression of the maximum displacement in dimensionless terms that occurs during the loading history:

$$\mu = \frac{x_{\max}}{x_y} \tag{3}$$

where x_y represents the yielding displacement. A well known technique for the response assessment of a non-linear oscillator is the Substitute Structure Method [Gulkan and Sozen, 1974][Shibata and Sozen, 1976]. This approximates the response of a generic non-linear system with that of a linear system with equivalent (effective) stiffness and viscous damping. The effective stiffness is the secant stiffness evaluated at the maximum displacement:

$$k_{eff} = \frac{f_{\max}(x,\dot{x})}{x_{\max}} = \omega_{eff}^2 \tag{4}$$

The effective viscous damping is such that the energy dissipated by hysteresis in a cycle in the non-linear system corresponds to the energy dissipated by the viscous restoring force in the equivalent oscillator:

$$\oint f(x,\dot{x})\mathrm{d}x = \oint 2\xi_{eq}\omega_{eff}\, x_{\max}\,\mathrm{d}x \tag{5}$$

The effective load can be calculated in general terms with the equation:

$$m = \int_V \phi \mathrm{d}m \tag{6}$$

where ϕ represents the deflection of the structure, normalised with respect to the position of the reference measurement. In practice, for a multi-storey building this reference measurement is the displacement at the top, and the equation becomes:

$$m = \sum_i \phi_i m_i \tag{7}$$

Where ϕ_i is the normalised displacement and m_i the mass, concentrated at the ith storey. The maximum intensity of the total seismic action measured at the base (base shear) therefore can be expressed as:

$$BS = a_{\max} m = a_{\max} \sum_i \phi_i m_i \tag{8}$$

The normalised displacement varies during the loading history as a consequence of the non-linearity of the structure. The most frequent approach, also adopted by codes, is to assume the deflection shape equal to the first modal shape of the corresponding linear elastic system. In the substitute structure method and with the aim of calculating the maximum base shear, it appears more correct to consider the displacement at the moment of peak acceleration. In any case, the error introduced is limited. The base shear can also be expressed in adimensional form, using the Base Shear Coefficient, defined as:

$$BSC = BS / W \tag{9}$$

W being the weight of the structure.

DISCUSSION OF THE RESULTS

Damage evaluation

The trend of the seismic response of the reduced scale model is represented in Figure 8, by means of synthetic parameters of the response recorded in the location *2cx*, defined not only because of the seismic tests, like the peak of acceleration a_{max} and of the displacement x_{max}, the amplification factor of acceleration *FA*, the base shear coefficient *BSC* and the effective frequency ω_{eff}, but also because of the shock tests, performed after each seismic test, like the natural frequency ω_0 (see Tab. 2).

For a real reinforced masonry structure, the transition from the elastic phase to the plastic phase is not immediate. Structural deterioration is characterised by a loss of stiffness and resistance. This is revealed, in terms of dynamic response, in the lowering of the initial frequency and the reduction of the base shear coefficient. During test 6 the masonry model shows a variation in effective frequency of about 50%, even though yielding of the reinforcing bars has not occurred. It is therefore misleading to calculate the displacement with reference to the initial stiffness: reference is commonly made to reinforcement yielding. An elastic-plastic trend of the seismic response is clearly identifiable, with the beginning of plasticisation being localised in test 9. It therefore seems appropriate to choose the displacement corresponding to test 9 as yielding displacement. The required ductility is then calculated for each of the successive tests. In the successive tests the BSC stabilises around the almost constant value of 0.5 (see Tab. 2 and Fig. 8) and, despite the progressive development of cracks, the resistance characteristics of the system remain essentially unchanged with the increase of the severity of the earthquake.The system reaches a critical phase after the 27^{th} test (PGA=1.4 g), with a significant reduction in the recorded peak of acceleration. The ultimate limit state is therefore identified with the corresponding displacement. About the evolution of the initial frequency versus the PGA (Fig. 8), it should be noted that the frequency remains constant in the initial tests (test 1, 3, 5), and instead it reduces with first cracking (tests 6, 7), and yielding (tests 9). Between tests 10 and 16 there is no significant frequency variation and therefore no deterioration, even though the response is certainly ductile, as shown by the progressive reduction of the effective frequency. In practice, the model experiences a succession of earthquake shocks without visible or measurable damage. This suggests that neither the supplied energy nor the required ductility can be in general indicative of the destructive capacity of an earthquake. The first frequency starts to lower again from test 18, with the appearance of new cracks, reaching the value of 13.35 Hz at test 31. Resistance remains constant until the ultimate Limit State is reached.

Test	*PGA ideal*	*PGA real*	a_{max}	*FA*	x_{max}	*BS*	*BSC*	ω_0	ω_{eff}	μ_r
	[g]	[g]	[g]		[mm]	[kN]		[Hz]	[Hz]	
T01	0.060	0.064	0.185	2.891	0.20	72738	0.148	21.36	21.36	
T02	0.080	-	-	-	-	-	-	-	-	
T03	0.080	0.098	0.285	2.908	0.32	118352	0.240	21.06	21.04	
T04	0.100	0.130	0.375	2.885	0.43	141523	0.287	20.94	20.14	
T05	0.120	-	-	-	-	-	-	-	-	
T06	0.140	0.175	0.485	2.771	1.49	200657	0.407	20.9	19.44	
T07	0.160	0.207	0.505	2.44	1.90	215681	0.438	20.83	18.09	
T08	0.200	-	-	-	-	-	-	-	-	
T09	0.220	0.228	0.493	2.162	2.17	202041	0.410	20.62	17.87	1.0
T10	0.240	0.252	0.490	1.944	2.43	217377	0.441	20.53	17.00	1.1
T11	0.275	0.294	0.500	1.701	3.01	206788	0.420	20.53	16.55	1.4
T12	0.300	0.323	0.505	1.563	3.65	219333	0.445	20.52	16.78	1.7
T13	0.340	0.393	0.510	1.298	5.00	241261	0.490	20.51	15.92	2.3
T14	0.380	0.433	0.345	0.797	5.80	193540	0.393	20.51	16.32	2.7
T15	0.420	0.462	0.525	1.136	6.05	247651	0.503	20.48	15.31	2.8
T16	0.460	0.530	0.350	0.66	7.45	190053	0.386	20.28	13.93	3.4
T17	0.500	0.579	0.534	0.922	7.10	224295	0.455	20.09	15.43	3.3
T18	0.550	0.620	0.534	0.861	7.80	233396	0.474	19.88	14.31	3.6
T19	0.600	0.679	0.540	0.795	7.55	239051	0.485	19.68	12.55	3.5
T20	0.650	0.725	0.535	0.738	7.20	226748	0.460	18.49	13.31	3.3
T21	0.700	0.794	0.545	0.686	8.25	247438	0.502	19.42	12.65	3.8
T22	0.800	0.920	0.555	0.603	7.05	248682	0.505	19.01	10.75	3.2
T23	0.900	1.042	0.525	0.504	11.45	252819	0.513	18.89	10.86	5.3
T24	1.000	1.165	0.530	0.455	10.65	257747	0.523	18.62	9.04	4.9
T25	1.200	1.296	0.540	0.417	10.35	260223	0.528	17.58	8.04	4.8
T26	1.400	1.411	0.530	0.376	13.25	242510	0.492	16.54	5.76	6.1
T27	1.600	1.418	0.534	0.377	16.35	259956	0.528	15.81	5.45	7.5
T28	1.800	-	-	-	-	-	-	15.68	-	-
T29	2.200	-	-	-	-	-	-	15.57	-	-
T30	2.600	-	-	-	-	-	-	14.49	-	-
T31	3.000	-	-	-	-	-	-	13.55	-	-

(-) Results not available

Table 2: Values of the synthetic parameters of the structural response related to base input intensity (PGA) of each seismic test.

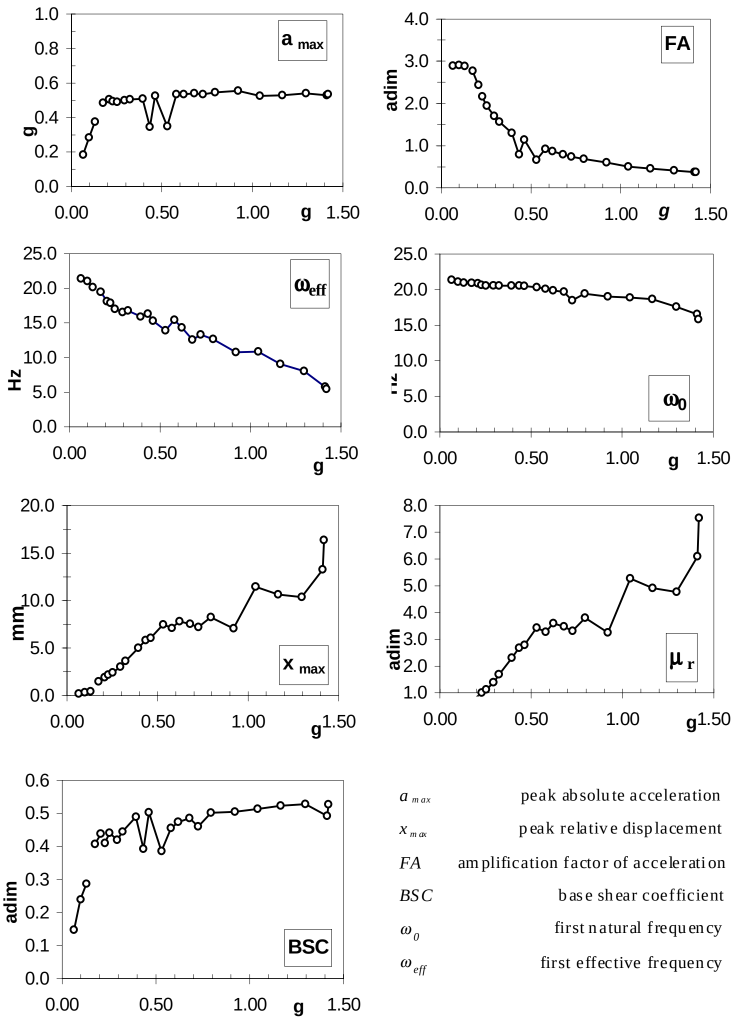

Figure 8: Trend of seismic response related to the base input intensity.

Calculation of the reduction factor

The force-based design procedure is based on a check on a linear elastic model with a conventional resistance. The ductile behaviour of the structure is taken into account by reducing the seismic action by a given q-factor or reduction factor. This is defined as the ratio between the seismic forces that the structure should elastically resist to, and the lowest design seismic forces:

$$q = F_e / F_y$$

(10) (10)

It is a function of the structural parameters like vibration period and damping and so it can be expressed in the generical form:

$$q^*(T,\xi) = \frac{\beta_e(T,\xi)}{\beta_u}$$

(11) (9)

where β_e and β_u represent the amplification factors for the linear elastic response and for the non linear response at the ultimate limit state, respectively.

If we consider the model at yielding point, the natural frequency ω_0 is equal to 20.62 Hz, which implies a period for the real structure equal to 0.145 sec, the equivalent viscous damping ξ_{eq} is equal to 15%, so that the behaviour factor is equal to 5.68. If we instead refer to the initial stiffness, that is a natural frequency ω_0 equal 21.36 Hz (period of the real structure equal to 0.140 sec, damping ξ_{eq} equal to 7.55%), we can obtain a q equal to 7.60.

According to Eurocode 8, for a structural period $T < T_B$ (being T_B equal to 0.15 s), we can derive the following expression of the q factor as:

$$q(T,\xi) = \frac{\beta_0}{\left(1-\frac{T_B}{T}\right)+\frac{1}{q^*(T,\xi)}\left[\eta\cdot\beta_0-\left(1-\frac{T_B}{T}\right)\right]} \qquad (12)$$

where S is a soil parameter, β_0 is the amplification factor equal to 2.5 for 5% viscous damping and $\eta = \sqrt{7/(2+100\cdot\xi)}$ is the damping corrector factor. Assuming the period as the initial one (for the real structure, T = 0.140), q = 5.9 is obtained through (12).

Anyway, the values above obtained for the q factor are greater than the 2.5 minimum value suggested by the Eurocode 8 for reinforced masonry structures. These results mean that the tested building was able to dissipate the input energy performing consistent plastic deformations, that is the construction technique enable the structure to make large dispacement without compromising its mechanical properties.

CONCLUSIONS

A series of 31 shaking table tests, alternated by sequences of modal characterisations, were performed on a 1:3 reduced scale rm. building in order to investigate the seismic behaviour of a new construction system of reinforced masonry. The construction technique was based on a new type of lightweight block with a single socket hole in which the vertical reinforcements were fitted and with horizontal reinforcements placed in correspondence with the mortar bed joints. The overall response of the reduced-scale model was analyzed by means of an effective procedure turned to identify the tested structure as a SDOF inelastic oscillator, where the input is the ground motion and the output is the building response, measured at the top (location *2cx*).

The trend of the seismic response of the reduced scale model was analyzed by means of synthetic parameters of the response recorded in the location *2cx*, defined not only because of the seismic tests, like the peak of acceleration a_{max} and of the displacement x_{max}, the amplification factor of acceleration *FA*, the base shear coefficient *BSC* and the effective frequency ω_{eff}, but also because of the shock tests, performed after each seismic test, like the natural frequency ω_0 and the instantaneous stiffness.

The experimental study presented highlights a very satisfactory seismic behaviour of the tested r. m. construction technique, both in terms of the response to low/medium intensity earthquake, and of behaviour at the ultimate Limit State. Specifically:

1. The building could sustain a sequence of medium earthquake shocks, without showing apparent damage or significant loss of stiffness or resistance; despite the progressive development of cracks, the resistance characteristics of the system remain essentially unchanged with the increase of the severity of the earthquake. This suggests that neither the supplied energy nor the required ductility can be in general indicative of the destructive capacity of an earthquake.
2. A consistent stiffness deterioration was observed at PGA values higher than 1g, in the scale of model (that is 0.33 g referring to the prototype building).
3. The achieved ductility equal to 7.5, evaluated at the ultimate Limit State, was definitively satisfactory.
4. The calculus of the reduction factor *q* gave values higher than 2.5 suggested by Eurocode 8. Therefore, this code seems to be rather conservative for the kind of tested structures.

Finally, the building has shown a behaviour typical of an elastic-plastic system, confirmed both by the inspection of the cracks pattern and by the trend of the response synthetic parameters of the SDOF model. It is understood that the results obtained in this study are strictly dependent on the particular construction technique used, which enabled the structure to make large displacements without compromising its mechanical performances; in order to optimize the design of rm. buildings under strong ground motion, additional investigations are needed.

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