PRACTICAL PERFORMANCE-BASED ASSESSMENT OF AN EXISTING PLAN-WISE IRREGULAR BUILDING

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ABSTRACT :

In the framework of the research activity of the ELSA Laboratory of the Joint Research Centre, pseudo-dynamic testing of a real-size plan-wise irregular 3-storey frame structure, both in the as-built and in two retrofitted configurations, was carried out as the core of the research project SPEAR (Seismic PErformance Assessment and Rehabilitation of existing buildings). The experimental activity carried out on the SPEAR structure allowed a one-of-a-kind wealth of data to be collected.
Once in the post-test phase, it became clear that a good understanding of the complex features of the response of the specimen was difficult to be obtained, due to the effects of double eccentricities, adding up to poor structural detailing and lack of ductility.
Still, it was felt that a kind of easily applicable and extendable lesson should be derived from that complexity, and that a fully performance-oriented interpretation and evaluation of the results could be attempted, in order to provide an effective tool to compare the effectiveness of different possible retrofitting strategies in complex scenarios.
A performance-based assessment exercise was thus carried out. This included the estimations of the costs of the different damage states, provided by the engineering practice. The probability of attaining each damage state was obtained by combining the experimental skeleton curves with the probabilistic definition of the expected intensities based on the very recent seismicity maps of Italy, as defined in the latest Italian Code for Constructions, on a local, continuously varied basis, as opposed to the older 4-zones classification of the Italian territory.
Conclusions were finally drawn on the effectiveness of the retrofitting interventions in terms of reduction of the expected loss during the lifetime of the building.

KEYWORDS: Performance-based design, assessment, rehabilitation, irregularity, torsion, FRP

1. THE SPEAR EXPERIMENTAL ACTIVITY

The SPEAR structure is a simplification of an actual three-storey building representative of construction practice of the ‘60s and ‘70s in Southern European Countries, without specific provisions for earthquake resistance. A thorough description of the aims and the experimental and numerical activity of the SPEAR project is given elsewhere [Negro et al., 2004], [Molina et al., 2004].
The structure is regular in elevation: it is a three-storey building with a storey height of 3 meters. The plan configuration is non symmetric in two directions (Fig.1), with 2-bay frames spanning from 3 to 6 metres; the presence of a balcony on one side and of two offsets increases the plan irregularity, shifting the centre of stiffness away from the centre of mass.
Eight out of the nine columns have a square 250 by 250 mm cross-section; the ninth one, column C6 in Fig. 1, has a cross-section of 250 by 750 mm, which makes it much stiffer and stronger than the others along the Y direction, as defined in Fig. 1, which is the strong direction for the whole structure.
The centre of stiffness (CR) (based on column secant-to-yield stiffness) is eccentric with respect to the mass

centre (CM) by 1.3 m in the X direction (~13% of plan dimension) and by 1.0 m in the Y direction (~9.5%).

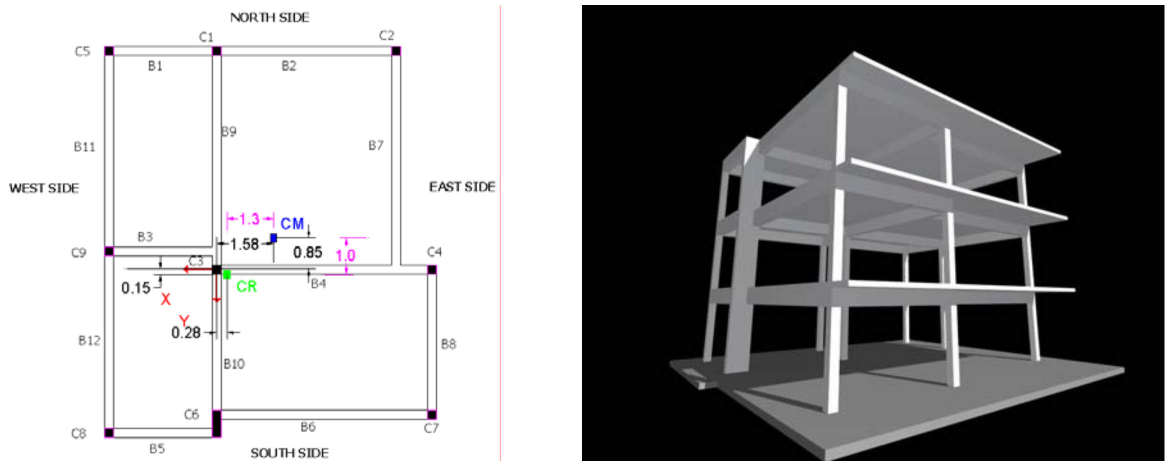

Figure 1: The SPEAR structure

Given the bi-eccentricity of the specimen, a bi-directional PsD test had to be carried out, with the application of both components (longitudinal and transverse) of the chosen input accelerogram; three degrees of freedom (DoFs) per storey were thus taken into account: two translations and one rotation along the vertical axis.

Two different retrofitting techniques were applied to the specimen: the first one consisted in increasing the ductility supply of the vertical elements and of selected joints by means of Fiber Reinforced Polymer (FRP) wrapping. Glass fiber wraps were used, with unidirectional fiber orientation for the columns and quadriaxial fiber orientation for the joints and the strong column C6, expected to exhibit significant shear damage.

After removing the FRP wrapping, the second retrofitting intervention was carried out, consisting of RC-jacketing of selected vertical elements (columns C1 and C4 in Fig. 1, along their whole height), with the aim of relocating the centre of strength of the structure to reduce torsional effects in the seismic response.

The choice to try and relocate the centre of strength rather than the centre of stiffness complies with the recent trends being discussed within the scientific community [Rutenberg and Tso, 2004]. According to the recent proposals, the practice, adopted by all codes, to modify the distribution of seismic forces to take into account the stiffness eccentricity should be abandoned, on the ground that the response of the structure is dominated by its nonlinear behaviour rather than by its initial stiffness properties. The consequences of this approach would be profound: if the strength distribution is made regular, the nonlinear response would not be affected by the irregular distribution of initial stiffness. In order to tackle only the strength distribution with the least possible modifications of the stiffness eccentricity, a very thin layer of concrete was added, covering a large amount of longitudinal corrugated steel rebars.

Three rounds of tests were thus carried out: the first one on the structure in the as-built configuration, the second one in the FRP-retrofitted configuration and the third one in the RC-jacketed configuration. The accelerograms used as input were the Montenegro '79 Herceg-Novi records for the longitudinal and transverse component, artificially fitted to the EC8 spectrum, scaled to different levels of PGA: 0.15g and 0.20g for the original structure, 0.20g and 0.30g for the specimen in the two retrofitted configurations, and applied to the structure according to the combination of direction and orientation that would maximize torsional effects, based on a number of pre-test numerical simulations. Moreover, a preliminary test at low PGA level (0.02g), carried out in each of the three configurations, allowed the first mode shapes of the structure, along with its initial frequencies and modal damping values to be estimated [Molina et al., 1999].

Global quantities, such as displacement and rotation at the CM at each floor, base shear, storey shears, storey drifts, absorbed energy at each floor and for each degree of freedom, were measured. Local instrumentation was also set at the top and bottom of a number of columns and beams, in the most meaningful locations.

In the following, only the results of the 0.20g PGA intensity test will be briefly discussed. For a more detailed presentation of the experimental results, see [Fardis and Negro, 2005], [Mola and Negro, 2006], [Mola, 2007].

2. THE SPEAR EXPERIMENTAL ACTIVITY: HIGHLIGHTS OF THE RESULTS

2.1. Original Structure

In the PsD tests in the as-built configuration, as expected, torsional effects strongly affected the response, in an often unpredictable way: a comparison between the drift measured at the CM and the drifts of the edge columns, the most deformed ones, clearly showed that the effects of torsion on the drifts of the edge columns were remarkable in both directions. In the X direction, where the structure was more flexible and the drift at the CM was already quite large, the maximum drift reached at the CM was 55 mm, whereas the maximum drift reached at the edge columns was about 70 mm, a difference that is not negligible. In the Y direction the maximum drift reached at the CM was 45 mm, whereas the maximum drift of the edge columns was above 70 mm, (+ 50%).
These increases are larger than could be expected when considering the values of plan eccentricity of the structure in themselves: an eccentricity of around 10%, often dismissed as ‘minor’.
Moreover, the failure mechanism developed by the specimen was of the soft-storey kind but at the second floor and not at the first one, as was reasonably expected due to the higher axial load of first storey columns that would negatively affect the ductility capacity of their cross sections: the effects of higher modes and the dynamic amplification of eccentricity were clearly pointed out by this behavior.
Finally, it became quite obvious that the almost total lack of ductility of the vertical elements (due to the poor structural detailing design) was a key factor in leading to failure.
The retrofitting strategies that were thus conceived aimed at tackling on one side the shortcomings of the vertical elements in term of ductility supply (FRP wrapping), on the other side at reducing the ductility demand through the reduction of the rotational component of the response (RC jacketing).

2.2 FRP-wrapped specimen

The same displacements as those of the unretrofitted specimen were reached in both the X and Y directions without loss of strength or stiffness; also, the initial slope of the loop did not increase, meaning that, as assumed in design, structural stiffness was not significantly affected by the intervention. Finally, the relative importance of rotational DoFs with respect to translational ones was also unchanged, meaning that no reduction of the torsional effects was effected by the intervention, as, in fact, this was not the aim to be pursued by this approach. The maximum absolute values of rotation reached in the FRP-wrapped configuration were even higher than those in the as-built one, for the same values of torque.
The damage developed in the structure under this level of excitation was negligible: only light cracking of vertical elements outside of the wrapped area was visible.

2.3 RC-Jacketed structure

In this case, the relative importance of the rotational DoF with respect to the translational ones in the energy dissipation process decreased; the attained maximum rotations were also reduced.
The initial slope of the loops increased, as the retrofitting intervention modified the global structural stiffness, affecting in particular the X direction, which was originally significantly weaker than the Y direction, thus being more sensitive to the increase in the cross-section of the two retrofitted vertical elements.
The damage developed by the specimen at this level of excitation was more intense in comparison with that developed by the structure in the FRP-wrapped configuration. In this case, damage of the vertical elements was fairly visible, mainly at the top of second and first storey columns. The damage pattern was thus very similar to that developed by the as-built structure, mainly concentrated on the elements with the highest axial load ratio, with heavy spalling of concrete cover and initiation of buckling of vertical rebars.

2.4 Discussion of the experimental results

The original structure was tested at differing levels of peak ground acceleration, attaining a severe level of damage for 0.2 g. The test sequence was not continued any further, to allow for repair, rehabilitation and further testing; however, due to the intrinsic lack of ductility of the structure, one might realistically assume that 0.2 g is close to the ultimate peak ground acceleration capacity.
The rehabilitation approach aimed at increasing global ductility by means of the application of FRP wrapping in all the potential plastic hinge zones proved to be particularly effective. The FRP-wrapped structure, in fact, survived the 0.2 g earthquake without visible damage, and the same held true for the test with 0.3 g. The test sequence was discontinued, again to allow for further repair, rehabilitation and testing, therefore no experimental measure of the ultimate peak ground acceleration capacity was made. An estimate was made by calibrating a push-over model against the experimental PGA-drift curves, and assuming that failure would occur at the attainment of the curvature corresponding to the maximum concrete strain assumed in the design of the FRP wrapping, which was the case for a PGA of about 0.32 g.
The rehabilitation approach aimed at reducing demands by reducing the strength eccentricity also proved to be effective. The maximum drifts turned out to be significantly reduced as compared to the original structure, thus confirming the advantages of tailoring the strengths of the vertical members so to minimize the global strength eccentricity rather than accounting for existing stiffness eccentricity. The major drawback of this approach remains with the lack of a unique definition of eccentricity and the rather high level of expertise which is needed to design the intervention.
In spite of the reduction in the maximum interstorey drift, the structure was severely damaged at the end of the 0.2 g test, which can realistically be taken as its ultimate PGA capacity.
The conclusion which can be drawn from the experimental results is that the rehabilitation approach aimed at increasing the global ductility by confining all potential plastic hinge zones can significantly extend the maximum PGA which can be sustained by the building, whereas the strength relocation is more effective in reducing the damage corresponding to lower PGA values, but does not significantly extend the strength, being the latter ultimately limited by the insufficient global ductility capacity.

3. PERFORMANCE-BASED ASSESSMENT EXERCISE

To provide clearer insights about the relative advantages and disadvantages of the two approaches, a practical cost-benefit analysis was carried out.
To be able to refer to a practical case, the structure was ideally converted into a real building of the same age (Figure 2). The building was intended as an office building, had a continuous glass façade and light internal partitions (these choices were dictated by the fact that no infills were present in the frames during the tests).
The performance-based assessment consisted into the evaluation of the costs associated to each rehabilitation measure and the expected losses during the remaining life-span of the building for all the defined limit states.

Figure 2: Rendering of the 'realistic' SPEAR building

In the evaluation of the costs of the rehabilitation measures and of the repair/replacement actions needed after each damage state, both the cost of the technical intervention and the costs associated to the limitation in use of the building were considered.
An assumption, affecting the cost estimates, was made: the floor system was of the floating type (i.e., easily dismountable and re-mountable), and partitions and ceilings were amenable to easy access to the structure.
The expected life-span for the building was assumed as 20 years.
The seismicity was defined with reference to a particular location in northern Italy at the border from Veneto and Friuli regions (ID N. 10083 in the recent map of seismicity of Italy, [DM 14-01-2008], corresponding to a peak ground acceleration of 0.25 g for a 475 years return period). The map provides a set of values of the peak ground acceleration for a discrete set of return periods, along with an interpolation formula, so that the peak ground acceleration can be obtained for any return period.
The skeleton curves providing the peak ground acceleration vs. maximum interstorey drift in each configuration were obtained from the experimental results. From the interstorey drift thresholds defined for each limit state, the peak ground acceleration was obtained. By means of the seismic map, the peak ground acceleration values were converted into return periods and then into probability of exceedance.
Summing up the contribution of each defined limit state the total expected loss was obtained, so that a comparison of the two rehabilitation techniques could be done in terms of reduction of the total expected loss with respect to the original structure.

3.1 Evaluation of costs

The costs associated to each rehabilitation measures have been evaluated with reference to the real building and occupancy.
FRP-wrapping will require 70,000 EUR for the placement of the laminates, 6,000 EUR for dismounting, remounting and painting, plus 32,000 EUR for day-off costs (these include: moving the contents of the offices forth and back, providing new services and publicity, loss of productivity, renting of a similar building for the time required, estimated in three weeks for the very intervention, but affecting the office activity for four months). The total estimated cost of the intervention is therefore about 108,000 EUR.
RC-jacketing will take place with no disruption, or with limited disruption of the office activity. The very cost of the intervention is 10,000 EUR. The day-off cost was estimated by assuming that the activity in the building would not be discontinued; however, a small new office would still have to be rented. The intervention will last 9 weeks and will affect the activity for three months. The total day-off cost is then 30,000, summing up to a total cost of 40,000 EUR.
A number of limit states were defined, and the associated costs were computed.
The first limit state was defined as low-damage, i.e., damage to the glass façade and partitions. This was estimated as a percentage of the cost of the façade (180,000 EUR), in 36,000 EUR.
The second limit state was defined as heavy damage/loss of the façade, whose associated cost was 180,000 EUR.
The third limit state was defined as severe structural damage and, due to the intrinsic fragility of the structure in any configuration, was taken as coincident to the fourth limit state to be considered, i.e. loss-of-the building/collapse. The corresponding cost is the demolition-reconstruction cost, 570,000 EUR. The estimate of the day-off costs was computed on the basis of a 16 months period as 90,000 EUR, summing up to 660,000 EUR.
The engineering measure defining damage was selected as the maximum interstorey drift at the most affected columns (this to account for the torsional response, also considering that the most affected storey turned out to be the second rather than the first). The drift thresholds defining low-damage and heavy damage/loss of the façade were provided by the producer of the façade system as 1% and 2% respectively.

3.2 Evaluation of the probabilities of exceedance

The PGA values corresponding to the drifts, including the drift at failure, were obtained from the skeleton curves

obtained from the tests (Figure 3), and were treated deterministically. It has to be recalled that the ultimate values for the FRP-wrapped structure was obtained by extrapolating the monotonic curve as explained before. Also, it has to be noted that the initial branch of the skeleton curve for the FRP-wrapped structure is softer than the one of the original structure, which can only be explained by the damage suffered throughout the test sequence. For consistency, it was decided to bring the skeleton curve for the FRP-wrapped structure up to a PGA of 0.2 g coincident with the one of the original structure.
The corresponding values are given in Table 1.

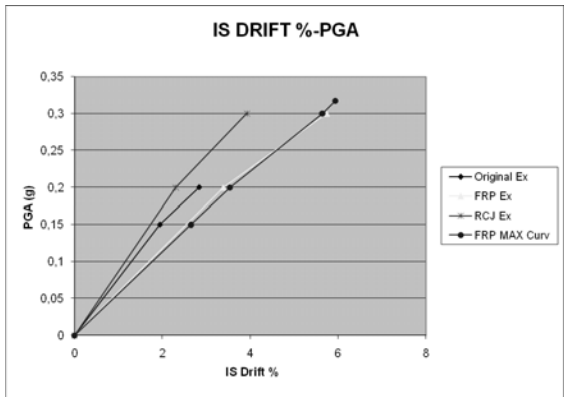

Figure 3: Experimentally derived skeleton curves for the three configurations

Table 1. Summary of PGA and drift values for 4 limit states in the three configurations

Limit State	Original Structure		FRP-Wrapped		RC Jacketed	
	Drift(%)	PGA(g)	Drift(%)	PGA(g)	Drift(%)	PGA(g)
1	1	0.0773	1	0.0773	1	0.0870
2	2	0.1533	2	0.1533	2	0.1739
3/4	3.54	0.2	5.94	0.3173	2.29	0.2

The following step was to compute the probability of exceedance for the so obtained PGA values. The new Italian seismic map provides the PGA values for a discrete set of return periods, starting from a return period of 30 years. It also provides a formula for interpolating those values; moreover, the user is advised that the formula is valid only for return periods larger than 30 years, which is the case at hand. The interpolation formula provided is thus applied:

$$\log(a_g) = \log(a_{g1}) + \log\left(\frac{a_{g2}}{a_{g1}}\right) \times \log\left(\frac{T_R}{T_{R1}}\right) \times \left[\log\left(\frac{T_{R2}}{T_{R1}}\right)\right] \qquad (3.1)$$

where a_g is the generic PGA value for which the corresponding return period T_R must be computed and a_{gi} and T_{Ri} are the two closest tabulated values of the parameters. By solving the formula for T_R, with the previously determined a_g values, the corresponding return periods were determined. The probability of exceedance in N years R_N was then obtained from the formula:

$$R_N = 1 - \left(1 - \frac{1}{T_R}\right)^N \qquad (3.2)$$

in which N was set to the expected lifespan of the structure, i.e., 20 years.
The probabilities of exceedance in 20 years could therefore be computed and are shown in Table 2.

Table 2. Return periods and probabilities of exceedance for the four limit states in the three configurations

Limit State	Original Structure		FRP-Wrapped		RC-Jacketed	
	T_R (years)	R_{20}(%)	T_R (years)	R_{20}(%)	T_R (years)	R_{20}(%)
1	40,72	39.18	40,72	39.18	49,26	33.65
2	151,84	12.38	151,84	12.38	199,23	9.573
3/4	276,20	6.997	851,52	2.406	276,20	6.997

3.3 Evaluation of total expected loss

Having computed the costs C_i associated to the attainment of each limit state, and the associated probability of exceedance R_{20i}, one can compute the total expected loss L in each configuration by direct application of the total probability theorem as:

$$L = \sum_i C_i \left(R_{20i} - R_{20i+1} \right) \tag{3.3}$$

The total loss expected in 20 years is given in Table 3.

Table 3. Total expected losses and investments in 20 years for the three configurations

	Original Structure	FRP-Wrapped	RC-Jacketed
Total Loss (EUR)	40,263	9,990	34,211
Investment (EUR)	0	107,500	39,500

In terms of reduction of total expected loss, the FRP-wrapped solution turns out to be by far the most effective, since the total expected loss is reduced to one fourth of the one of the original structure.
On the other hand, the advantage in terms of reduction of the deformability due to torsional response which was offered by the RC-jacketed structure is not reflected in the corresponding total expected loss, which is reduced by a mere 15% with respect to the original structure.
In terms of return of investment, it has to be noticed that none of the rehabilitation strategies has economic justification. However, it has to be recalled that neither possible casualties were accounted for in the analysis, nor the risk of loss of the contents was considered. These issues, if properly accounted for, might change the last conclusion.
It must also be recalled that the same comparative assessment of the two rehabilitation strategies, if conducted for larger eccentricities, might yield different conclusions.

4. CONCLUSIONS

A practical, cost-based, assessment method was used to compare the performance of the SPEAR building in the two conceptually different rehabilitated configurations. The method was based on the direct application of the total probability theorem. The costs associated to each rehabilitation measures were assessed according to engineering practice, including the consequences of the occupancy limitations. With the same approach, the losses corresponding to the attainment of a number of damage limit states were computed. The maximum interstorey drift was assumed as the measure of damage, and was related to the peak ground acceleration in a deterministic way by using the experimentally obtained skeleton curves. The values of the peak ground acceleration were converted into return periods by using the most recent Italian seismic map, and those were converted into probabilities of exceedance. Finally, by summing the contributions of all the limit states, to total expected loss over the assumed remaining life-span.
The reduction of the total expected loss with respect to the original structure was taken as the most important performance indicator, within the limitations of the exercise.
It can be concluded that the rehabilitation measure aimed at increasing the global available ductility performed

better in reducing the expected losses. The rehabilitation measure aimed at reducing the torsional response was successful in cutting the losses associated to the lower limit states but was not as effective as the one aimed at increasing the global available ductility. This conclusion should be restricted to the case at hand (small torsional eccentricity), and might be different for more irregular structures.
The measure to increase the ductility proved to be efficient with reference to the ultimate limit state, whereas the reduction of the eccentricity was effective for the lower limit states. It might therefore be concluded, even if no direct confirmation is available, that combining the two approaches would optimize the performance.
In terms of cost-benefit analysis, none of the rehabilitation measures proved to be economically justifiable. However, different conclusions could be reached if casualties would be taken into account, or even if the probability of the loss of the contents would be considered.

5. ACKNOWLEDGEMENTS

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