STANOVENÍ ZRANITELNOSTI A ZESILOVÁNÍ ŽELEZOBETONOVÝCH BUDOV, KTERÉ NEBYLY NAVRŽENY S OHLEDEM NA SEISMICITU

VULNERABILITY ASSESSMENT AND STRENGTHENING OF R.C. BUILDINGS NON-SEISMICALLY DESIGNED

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Keywords: RC buildings, non-linear seismic analysis, vulnerability assessment, seismic retrofitting.

Anotace: Seismická zranitelnost existujících budov postavených do roku 1970 v městských oblastech většiny jihoevropských zemí, s mírnými až vysokými seismickými riziky, je krajně důležitá. V té době se železobetonové budovy navrhovaly a stavěly bez ohledu na zemětřesení a představují tak významné riziko. Nedávná velká zemětřesení po celém světě dokázala, že existující budovy tohoto typu, které nemají vhodnou odolnost proti zemětřesení, jsou velmi zranitelné. Jejich modernizace má snížit jejich zranitelnost a z ní plynoucí riziko na dnes přijatelnou úroveň. Přes zřejmé výhody propracovaného nelineárního dynamického modelování konstrukčního vlákna je nutno připustit, že tato metoda může být někdy pracná a nákladná. Zjednodušené nelineární statické modely, které berou v úvahu právě jednu DOF (jako je metoda kapacitního spektra) nejsou často schopny přesně stanovit nepravidelné konstrukční systémy. Tyto skutečnosti podporují vývoj méně složitých strukturálních modelů, aniž by snižovaly základní rysy dynamické odezvy. Demonstruje to nedávno navržená zjednodušená metoda nelineární dynamické analýzy budov založená na multimodální spektrální seismické odezvě, a optimalizační nástroj pro navrhování zesílení existujících budov.

Abstract: The seismic vulnerability associated with existing buildings, constructed until the late 1970's, in urban areas in most southern European countries, with moderate to high seismic risk, is of extreme importance. In that period, reinforced concrete buildings were designed and constructed without considering earthquake provisions, constituting a significant risk. Recent major earthquakes around the world have evidenced that this type of existing buildings lacking appropriate seismic resisting characteristics are very vulnerable. Their retrofit should be made in order to reduce vulnerability, and consequent risk, to currently accepted levels. Despite the evident advantages of a refined non-linear dynamic structural fibre modelling, it must be admitted that this approach can frequently become elaborated and costly. Simplified non-linear static models considering just one DOF (such the Capacity Spectrum Method) are frequently not able to assess accurately irregular structural systems. These facts sustain the development of less complicated structural models without debasing the essential features of dynamic response. It is demonstrated a recently proposed simplified methodology for non-linear dynamic analysis of buildings based on the multi-modal spectral seismic response, and an optimisation tool for the strengthening design of existing buildings.

1. INTRODUCTION

In Europe, many structures are potentially seismically vulnerable due to the late introduction of seismic demands into building codes. Therefore, there is an evident need to investigate the seismic behaviour of existing reinforced concrete (RC) buildings, in order to assess their seismic vulnerability and ultimately to design optimum retrofitting solutions.

The most common causes of inadequate response of building to seismic loading are associated with: i) stirrups/hoops, confinement and ductility; ii) bond, anchorage, lap-splices and bond splitting; iii) inadequate shear capacity and failure; iv) inadequate flexural capacity and failure; v) inadequate shear strength of the joints; vi) influence of infill masonry; vii) vertical and horizontal irregularities; viii) higher modes effect; ix) strong-beam weak-column mechanism, and, x) structural deficiencies due to architectural requirements. However, it should be noted that normally structural damages and failures are associated to the combination of several of these factors. In this paper, is proposed a model for the simulation of the non-linear shear behaviour.

2. BRIEF DESCRIPTION OF THE TESTING CAMPAIGN

In the framework of the ICONS Topic 2 - Assessment, Strengthening and Repair - research programme [Pinto et al., 2002], two full-scale four-storey reinforced concrete frames were tested pseudo-dynamically at the ELSA laboratory. The frames, representative of the common practice of design and construction until the late 1970’s in most European Mediterranean countries, have been constructed and tested in order to assess the vulnerability of bare and infilled structures and to investigate various retrofitting solutions. This experimental study aimed at assessing the original capacity of existing structures, with and without infill masonry, and to compare the performance of different retrofitting solutions.

The general layout of the building frame model is shown in Figure 1. It is a reinforced concrete 4-storey full-scale frame with three bays, two of 5 m span and one of 2.5 m span. The inter-storey height is 2.7 m and a 0.15 m thick slab of 2 m on each side is cast together with the beams. Equal beams (geometry and reinforcement) were considered at all floors. The columns, all but the wider interior one, have equal geometric characteristics along the height of the structure. A comprehensive description of the frames, tests on material samples used in the construction (steel reinforcement and concrete) and PsD test results can be found in [Pinto et al., 2002; Varum, 2003].

The materials considered at the design phase [Carvalho et al., 1999] were a low strength concrete, class C16/20 (Eurocode 2) and smooth reinforcing steel (round smooth bars) of class FeB22k (Italian standards). The reinforcement detailing (lap-splice, stirrup, etc.) adopted is representative of the non-ductile reinforced concrete structures of ~50 years ago. Vertical distributed loads on beams and concentrated loads on the column nodes were considered in order to simulate the dead load other than the self-weight of the frame. These correspond to the following vertical loads: weight of slab 25 × 0.15 = 3.75 kN/m^2, weight of finishings 0.75 kN/m^2, weight of transverse beams 2.5 kN/m, weight of masonry infills 1.1 kN/m^2 of wall area, and live load 1.0 kN/m^2 (quasi-permanent value).

The input seismic motions were defined in order to be representative of a moderate-high European seismic hazard scenario [Campos-Costa and Pinto, 1999]. Hazard consistent acceleration time series (15 seconds duration) were artificially generated yielding a set of uniform hazard response spectra for increasing return periods. Acceleration time histories for 475, 975 and 2000 years return periods (yrp) were used in the tests (PGA of 218, 288 and 373 cm/s^2, respectively).

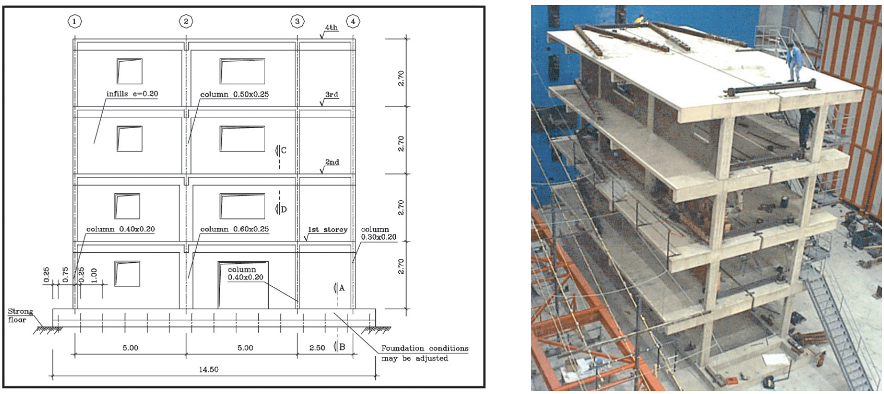

Figure 1. Tested frames: a) elevation views of the frames; b) models in the ELSA laboratory

3. RESULTS FROM THE TESTS ON THE BARE STRUCTURE

The bare frame (BF), was subjected to one PsD earthquake test corresponding to 475-yrp and subsequently to a second PsD test carried out with a 975-yrp input motion using pseudo-dynamics testing techniques.

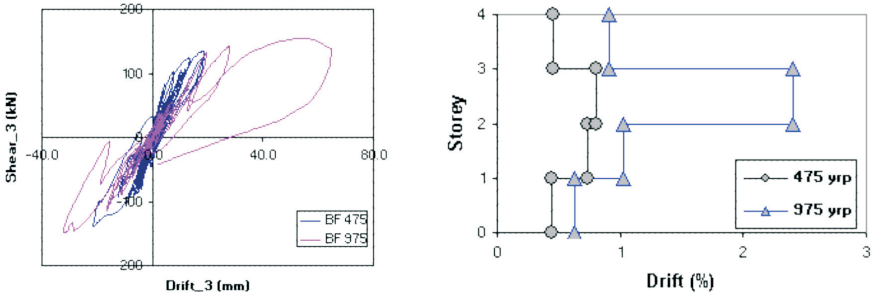

Figure 2. BF test results: a) shear-drift diagram at the 3rd storey;
b) maximum inter-storey drift profiles

The 975-yrp test was stopped at 7.5 seconds, because imminent collapse was attained at the 3rd storey. The significant reduction in terms of stiffness and strength from the 2nd to the 3rd storey (vertical irregularity), coupled with the inadequate lap-splicing and shear reinforcement, induced the concentration of larger inter-storey drift demand, and consequently damage, in the 3rd storey, developing a soft-storey mechanism at the 3rd storey. Results from these tests are given in Figure 2, in terms of storey shear versus storey drift at the 3rd storey and maximum inter-storey drift profiles.

4. RESULTS FROM THE TESTS ON THE STRENGTHENING STRUCTURE

Following the two earthquake tests on the bare frame, the damaged parts of the structure were repaired. A selective retrofitting scheme (SR), proposed by a research group from the Imperial College of London [Elnashai and Pinho, 1999] was applied. The retrofitted solution was based on a rational intervention, which balances strength, stiffness and ductility according to the requirements for increased seismic performance. The selective retrofitting solution involved two types of interventions in the wide internal column. A strength-only intervention was implemented in the wide column at the 3rd and 4th storeys to reduce the large flexural capacity difference. A ductility-only intervention was accomplished at the first three storeys in the wide column, where large inelastic deformation demand is expected. This intervention was achieved by the addition of external confining steel plates at the critical zones (at the base and at the top of the column). Furthermore, to minimize the risk of shear failure, additional plates were also added at mid-height of the columns. The selective retrofitting scheme adopted effectively solved the vertical irregularity identified in the structure, as can be observed in the maximum storey drift profiles plotted in Figure 3, resulting in a much more uniform maximum storey drift profile. Furthermore, the retrofitted frame was able to withstand input motion intensity corresponding to a return period of 2000 years (1.8 times the nominal one, 475-yrp, in terms of PGA), without collapse and with limited and reparable structural damages, maintaining its load carrying capacity, while the bare frame collapsed for an input motion 1.3 times the nominal intensity. The deformation capacity of the retrofitted structure is, at least, double that of the original one.

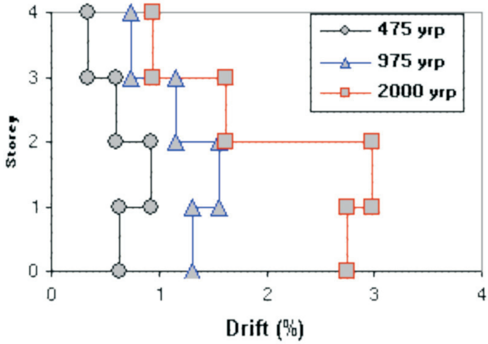

Figure 3. SR test results: maximum inter-storey drift profile

5. RESULTS FROM THE TESTS ON THE INFILLED STRUCTURE

Figure 1 shows the general layout of the structure including infill panels and the type and location of the openings. The 150 mm thick infill-walls (non-load bearing) were constructed after the reinforced concrete frame. Representative materials and construction techniques were used, namely: Italian hollow clay (ceramic) blocks horizontally perforated, with dimensions: 0.12 m thick, 0.245 m base-length and 0.245 m height. The mortar joints are approximately 1.5 cm thick and a 1.5 cm thick plaster was applied on both sides of the walls. The same mortar proportioning was used for bed joints and plaster (1:4.5 - hydraulic binder:sand). The infilled frame specimen was subjected to three consecutive PsD earthquake tests corresponding to 475, 975 and 2000-yrp. During the 2000-yrp PsD test, the masonry infills at the 1st storey collapsed and the test was stopped at ~5 seconds. Results from these tests are given in Figure 4 in terms of storey shear-drift and maximum inter-storey drift profiles. For the 475-yrp test, overall, the infilled frame structure behaved very well. The 975-yrp earthquake caused a significant damage to the infill alls in the bottom storey, with some minor damage to the concrete beam-column joints and columns at this level. Smaller amount of damage in similar locations were noted in the 2nd storey. No significant damage was observed in the upper two stories. It was recognised that the infill frame had become, at the end of the 975-yrp test, a soft-storey infill frame structure. Nevertheless, it was subjected to the 2000-yrp earthquake signal in order to study how gradually the lateral strength dropped off with increasing drift. The storey shear versus drift hysteresis loops clearly illustrate that the load deflection characteristics approach those of the bare frame as the drifts increase to values in excess of 1% (see Figure 4). The infilled frame demonstrated completely different behaviour compared to the bare frame. Infills protect the RC structure but also prompt storey mechanisms and cause shear-out of the external columns in the joint region.

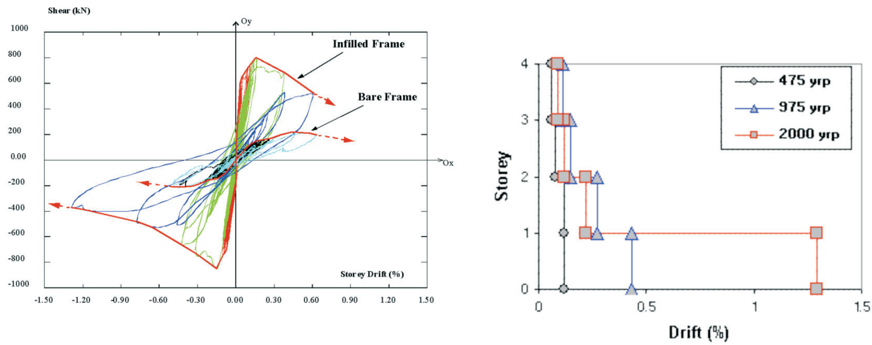

Figure 4. IN test results:
a) 1st storey shear-drift diagrams and envelope curves (comparison with the BF);
b) maximum inter-storey drift profiles

6. A MDOF NON-LINEAR DYNAMIC MODEL FOR STRUCTURAL ASSESSMENT

In this section it is summary described a simplified MDOF dynamic procedure [Varum, 2003] for non-linear dynamic analysis of buildings based on the multi-modal spectral seismic response. The model accounts for two levels of non-linearities, namely: a) storey behaviour in terms of shear-drift; and, b) damping as a function of deformation. The procedure assumes that a non-linear MDOF system can be represented by an equivalent linearized system with element stiffness given the secant stiffness. Consequently, linear spectral analysis can be used and multi-modal response methods with quadratic combination can be applied. The procedure is based on a generalization of the substitute-structure method, proposed by [Shibata and Sozen, 1976], which states that the response of a non-linear SDOF system can be accurately approximated by the response of an equivalent linear system with an equivalent period corresponding to the secant stiffness.

The non-linear damping can be modelled in two different ways: a) variable (damping defined for each storey); and, b) global structural level. It was included the possibility of participation of several natural modes for the structural response, with their quadratic combination. The building structure is idealised as a bi-dimensional (2D) cantilever model (shear building), with a number of horizontal translational DOF's equal to the number of storeys. The structural model is fixed at the base, as represented in Figure 5, and each node is fixed against rotation. The shear force-displacement relationship of each beam-element represents the curve storey shear versus inter-storey drift.

In this model, represented schematically in Figure 5, the mass distribution of the building is defined for each floor level accounting for the mid-height storey masses and lumped at floor level (equivalent total storey masses). The storey damping is labelled ξ_i. The force vector {F} is expressed in terms of the storey shear forces. The relative inter-node displacement vector {D} is expressed in terms of lateral deformation of the element (inter-storey drift). The storey shear force (Fi) acting on a beam element and the inter-storey drift (Di) are related by the non-linear Fi-Di curve. In the iterative step-by-step procedure, for each step, the calculations are made with constant secant stiffness and damping at the storey levels. The proposed simplified MDOF non-linear dynamic method for assessment of multi-storey building structures calls for a relatively small number of DOF's (one per floor), compared to a detailed FE model. Evident advantages come out, for example, fast parametric studies with a good level of confidence can be carried out with the model.

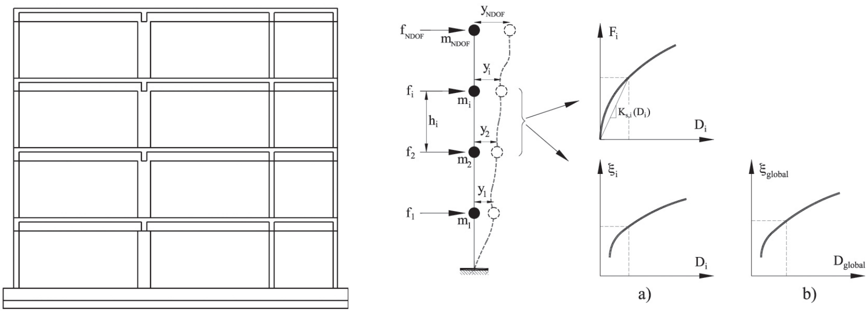

Figure 5. MDOF simplified model with concentrated masses at storey levels being connected by shear beam elements: a) damping defined for each storey, b) global first mode damping

The proposed MDOF non-linear dynamic model was used to estimate global parameters (such as top-displacement, maximum inter-storey drift, maximum storey shear, and equivalent damping) measured in the full-scale PsD tests performed on the irregular and regular structures, bare and strengthened structures, respectively. The structures were analysed for input motions corresponding to the maximum accelerations of the earthquakes considered in the tests, namely 218 and 288 cm/s2 for the BF (corresponding to 475 and 975-yrp). For the structure under analysis, four DOF are considered, being the storey masses considered for the first three storeys (m1, m2 and m3) 44.6 ton, and for the fourth storey (m4) 40.0 ton. The envelope storey shear-drift behaviour curves, obtained from the PsD earthquake tests, were adopted as capacity curves.

To perform a structural assessment, it is essential to define accurately the damping as a function of the deformation demand. In the literature, there are some proposals of damping functions for new buildings, but not for existing ones. In this study, it was estimated the damping from the test results on the frame representative of existing RC structures [Varum, 2003]. The best-fit logarithmic curve obtained, in terms of storey equivalent damping, as a function of the maximum inter-storey drift, is plotted in Figure 6-a. Even for considerable deformation levels, for existing structures, a low value of damping was estimated (maximum value less than 11%), which confirms that existing structures, with reinforcing plain bars, have a small energy dissipation capacity.

The inter-storey drift profile obtained from the numerical analyses performed with the proposed simplified MDOF non-linear dynamic method is plotted in Figure 6-b, for the BF structure, and for the 975-yrp earthquake input motion. In this figure, it is also plotted, for comparison, the maximum inter-storey drift profile observed in the corresponding PsD test. The structural response was estimated considering the participation of one and four natural modes of the equivalent linear system, in order to analyse the influence of the number of natural modes in the global response. A good estimative of the maximum response was achieved, with the simplified non-linear dynamic model, considering a small number of DOF (4 versus 372 DOF's for the refined 2D FE model). Therefore, this displacement-based methodology can be an effective numerical tool to perform fast non-linear analyses, which could allow for parametric studies and rapid screening (seismic vulnerability assessment) of existing building classes.

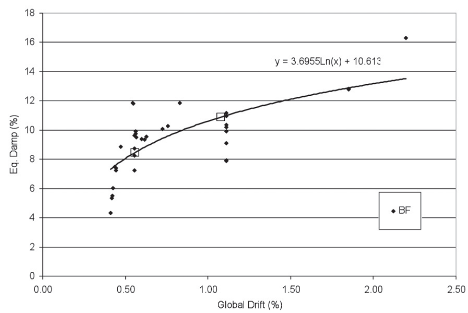

Figure 6. MDOF simplified model: a) equivalent global damping versus global drift for the BF structure; b) maximum inter-storey drift profile computed and test results for BF structure (975-yrp earthquake)

7. OPTIMIZATION TOOL FOR THE STRENGTHENING DESIGN

Since structural optimization problems are characterized by computationally expensive function evaluations, it is common to generate a sequence of convex, separable sub-problems, which are then solved iteratively [Chickermane and Gea, 1996].

It was judged appropriate to develop a methodology that can address the strengthening design of MDOF structural systems, generating optimal distribution (location) of the strengthening in the structure components (at storey level). Three methodologies for optimum redesign of existing structures were proposed and programmed. The optimization algorithms are based on the convex approximation methods, such as the Convex Linearization Method developed by Fleury [1989] and Braibant [1985], and the Method of Moving Asymptotes. These optimization algorithms can deal with non-linear objective functions (minimum cost of intervention) and allows to impose constrains on the design variables (strength, stiffness or damping) and on any other response variable depending on the design variables, such as inter-storey drift, top-displacement, etc.

The optimization procedure requires several structural response evaluations, namely of the objective function, of constraints, and of their derivatives. The calculation of the structural response is required many times during the optimization process, which would be unfeasible with a refined FE model. The simplified model allows for spectral analysis, which constitutes a great advantage over the multi-series analyses. The model is able to estimate the response of irregular structures, those we address with the optimization of the retrofit. Therefore, the simplified MDOF dynamic method, presented in the previous section, was incorporated in the redesign optimization algorithms here presented.

For the optimization problems here proposed, it is assumed that the behaviour of a multi-storey RC existing building (non-seismically designed) subjected to a certain level of earthquake action can be represented by the multi-modal model proposed in the previous section. Buildings are modelled with one DOF per storey, linked by beam elements that represent the storey behaviour. The beam elements have an equivalent secant stiffness corresponding to the maximum deformation point in the non-linear storey constitutive curve. Furthermore, response spectra modal analysis with concentrated and/or distributed damping is used to compute the seismic response for each step of the optimization procedure.

A seismic performance objective is formed by combining a desired building performance level (a damage limit-state) with a given earthquake ground motion (level of hazard). The objective of this analysis is to find the optimum retrofitting solution in order to comply with a certain seismic demand-level defined for each limit-state. The optimization problem, in generic terms, is to minimise the total strengthening requirements in the structure, whilst satisfying the upper limits for the inter-storey drifts and strengthening at each storey. The objective function for each problem is the sum of the control variables (additional strengthening costs) at each storey level. The inequality constraints are upper inter-storey drift limits (to restrain the damage at storey level) and upper storey strengthening limits (to restrict the strengthening within acceptable values).

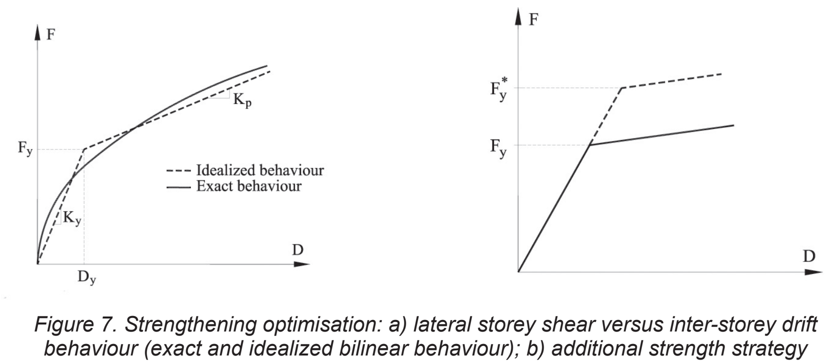

Figure 7. Strengthening optimisation: a) lateral storey shear versus inter-storey drift behaviour (exact and idealized bilinear behaviour); b) additional strength strategy

The optimization procedure requires previous identification of simplified (bilinear) storey shear-drift constitutive relations, as represented in Figure 7-a. Three design optimization structural strengthening problems were established in this work. They were conceptually based on the strengthening strategies commonly used in practice, which call for the control variables (at storey level): i) the additional strength (controlled by the yielding shear force, ΔFy), see Figure 7-b; ii) the additional pre-yielding stiffness (ΔKy); and, iii) the yielding strength of the energy dissipation devices (Fydev).

A numerical example is herein presented in order to illustrate the proposed optimal retrofit design methodology. From the experimental tests performed on the original four-storey RC building bare frame, it were calculated the envelope curves of storey shear versus inter-storey drift and approximate for the best-fit idealized bi-linear curves. The original storey shear-drift curves were approximate for the idealized bi-linear curves, maintaining the dissipated energy and the maximum shear load. The adopted storey shear-drift curves are plotted in Figure 8-a.

The example of optimization problem presented in this section assumes as control variables the additional storey strength. The objective function to be minimised is the total structural additional strength, i.e. the sum of the storeys additional strength. It is intended to find the optimal distribution of strengthening in the building, whilst satisfying the restrictions in terms of maximum storey strengthening and maximum allowable inter-storey drift. The constraint conditions for this structural optimization problem are (ATC-40, 1996): a) maximum admissible drift of 3.0 cm (1.1%), for every storey; and, b) upper limit of 500 kN for each storey additional strength, that do not restraint the solution, and minimum zero (not additional strength). The pre-yielding and pos-yielding stiffness are assumed to be constant, as represented in Figure 7-b. The optimization problem converges after 12 iterations. In Figure 8-b are represented the storey strength profiles of the original structure and of the optimum strengthening, to accomplish with a performance objective corresponding to the earthquake of 975-yrp and the drift limit of 3.0 cm.

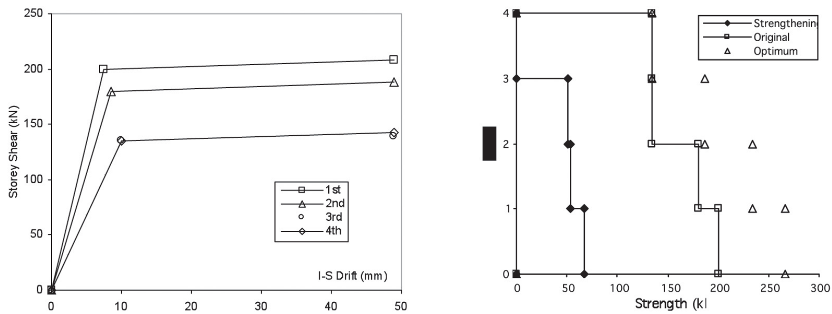

Figure 8. Example: a) storey shear-drift curves adopted from the experimental tests; b) storey yielding strength of the existing structure and optimum strengthening distribution

8. REMARKS

The pseudo-dynamic tests performed on the 4-storey full-scale RC models have shown that the vulnerability of existing reinforced concrete structures constitute a high risk source for human life. Furthermore, it was demonstrated that retrofitting solutions adequately selected, designed an implemented can reduce substantially that risk to levels currently considered in modern design.

The numerical results obtained with the proposed non-linear displacement-based model are in good agreement with the experimental ones, even for the irregular structure. The proposed optimization methodology deal with non-linear objective functions and allow to impose constrains on the design variables (strength, stiffness or damping) and on any other response variable, depending on the design variables, such as inter-storey drift, top-displacement, etc, generating the optimum strengthening storey distribution, for one or multiple performance objectives. These simplified models can be useful design tools, as a preliminary step, in the seismic vulnerability assessment and global structural strengthening decision, which could allow for parametric studies and rapid screening of existing building classes.

9. ACKNOWLEDGMENTS

This research has been partially developed under the ICONS TMR-Network research programme (contract No. FMRX-CT96-0022 (DG 12 - RSRF)) and the tests carried out at the ELSA Laboratory were financed by the European Commission under the Training and Mobility of Researchers Programme, TMR - Large-Scale Facilities (contract N. FMGE-CT95-0027).

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