SEISMIC BEHAVIOUR EVALUATION AND STRENGTHENING OF RC BUILDINGS

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SUMMARY

Reinforced concrete buildings constructed until the late 1970's in most southern European countries 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 or replacement should be made in order to reduce vulnerability, and consequent risk, to currently accepted levels. The development of retrofitting techniques represents a key issue in order to avoid both human casualties and economic losses in future events. Despite the advantages of a refined non-linear dynamic structural fibre modelling, it must be admitted that this approach can frequently become elaborated and costly. This fact sustains the development of less complicated structural models without debasing the essential features of dynamic response. Thus, it is proposed a simplified methodology for non-linear dynamic analysis of buildings based on the multi-modal spectral seismic response. It is also proposed an optimisation tool for the strengthening design of existing buildings.

INTRODUCTION

In Europe, many structures are potentially seismically vulnerable due to the late introduction of seismic loading 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. 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 tests have shown that the vulnerability of existing reinforced concrete structures constitute a source of high risk 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.

BRIEF DESCRIPTION OF THE TESTING CAMPAIGN

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].

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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).

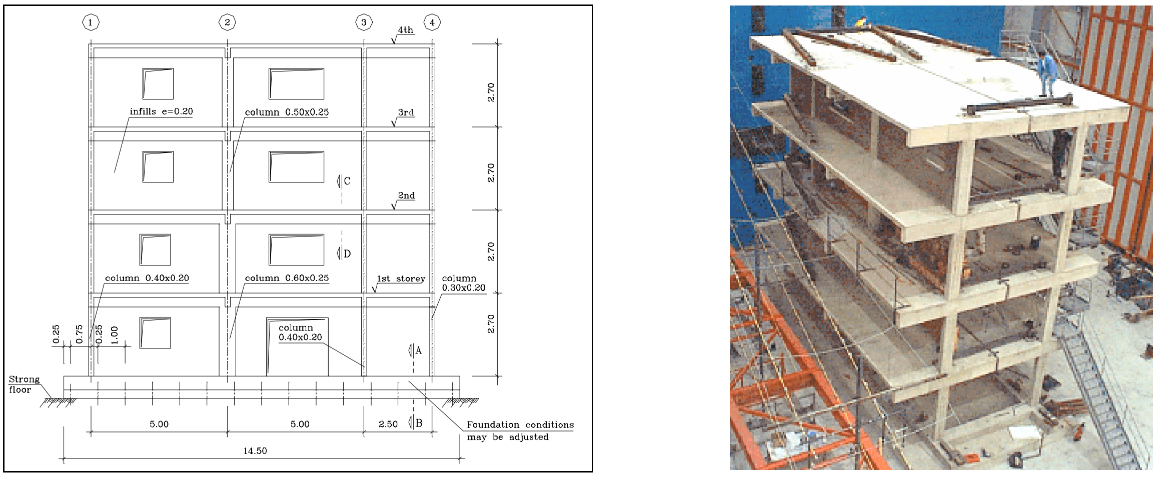


Figure 1: Tested frames: a) elevation views of the frames; b) models in the ELSA laboratory

TESTS ON THE BARE FRAME

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. 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.

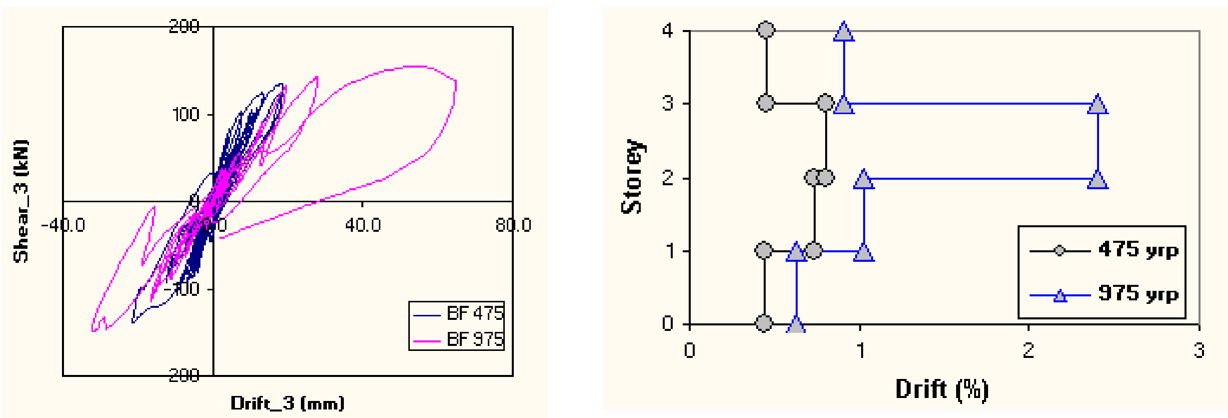


Figure 2: BF test results: a) shear-drift diagram at the 3rd storey; b) maximum inter-storey drift profiles

TESTS ON THE RETROFITTED FRAME

Following the two earthquake tests on the bare frame, the damaged parts of the structure were repaired: the 'spalled' concrete was removed and the cracks were injected with resin epoxy. 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.

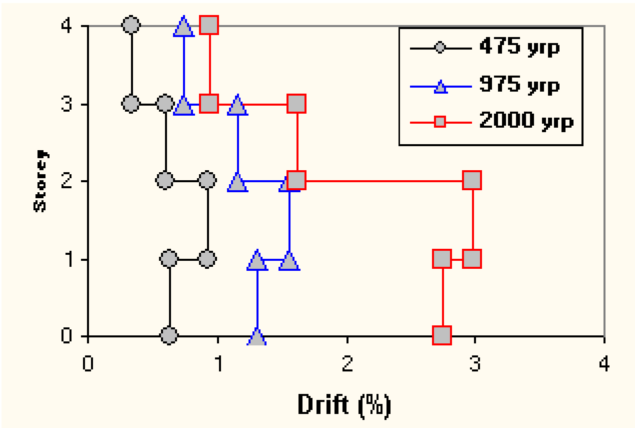


Figure 3: SR test results: maximum inter-storey drift profile

MASONRY INFILLED FRAME

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.

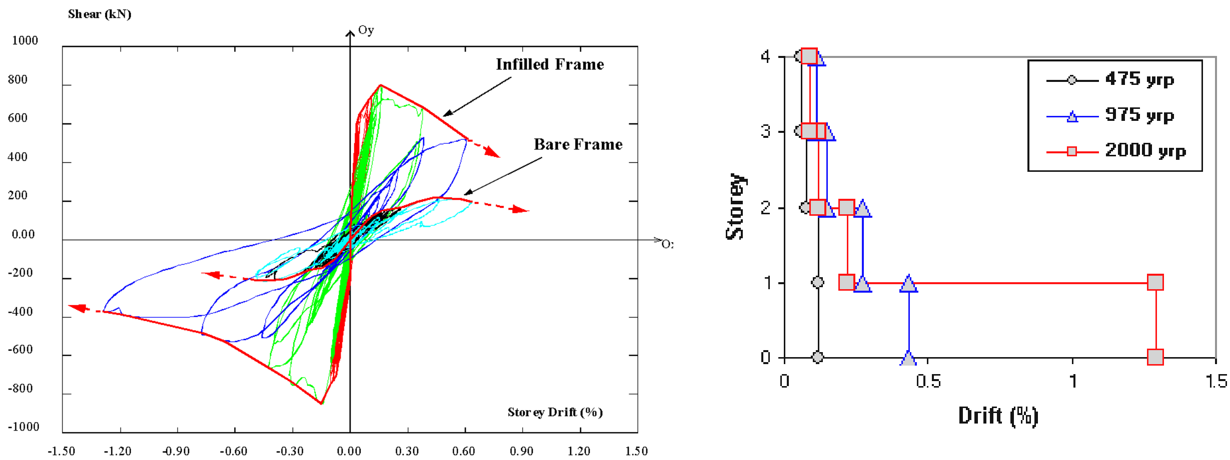


Figure 4: IN test results: a) 1st storey shear-drift diagrams and envelope curves (comparison with the BF); b) maximum inter-storey drift profiles

IMPROVED MDOF NON-LINEAR DYNAMIC MODEL FOR STRUCTURAL ASSESSMENT

Despite the advantages of refined non-linear dynamic structural analysis models, as fibre modelling, it must be recognised that this approach can frequently become elaborated and costly. This fact sustains the development of less complicated structural models without debasing the essential features of dynamic response. Simplified non-linear static models considering just one DOF (such the Capacity Spectrum Method) are frequently not able to assess accurately irregular structural systems. Thus, it is proposed a simplified methodology for non-linear dynamic analysis of buildings based on the multi-modal spectral seismic response.

Description of the model

A simplified non-linear MDOF dynamic procedure, for structural assessment was developed. 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.

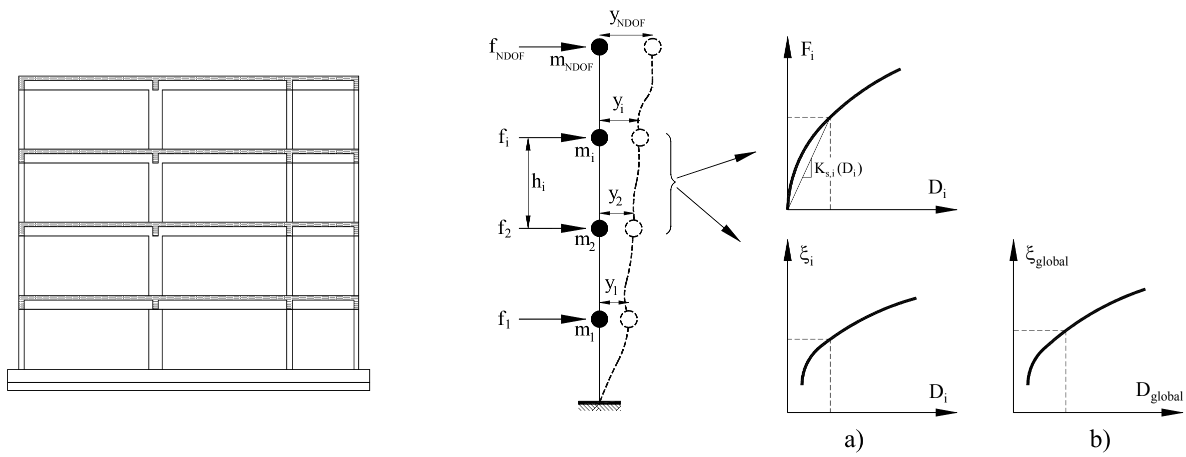


Figure 5: MDOF simplified model with concentrated masses at storey levels being connected by shear beam elements: a) damping defined for each storey, b) global first mode damping

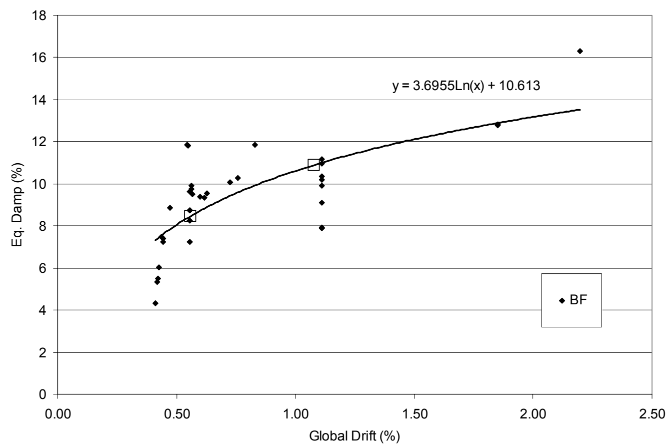
The non-linear damping relationships can be modelled in two different ways, namely: a) variable (with damping functions defined for different structural components, e.g. for each DOF, storey); and, b) modal (global structural level). It was included the possibility of participation of several natural modes (multi-mode) 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). Therefore, the i-th storey mass (m_i) concentrates the total storey mass at node (storey) i, and these nodes are connected by shear-beam elements. The storey damping is labelled ξ_i. The force vector {F} is expressed in terms of the shear forces acting on the beam elements (storey shear), and the relative inter-node displacement vector {D} is expressed in terms of lateral deformation of the beam element (inter-storey drift). The storey shear force (F_i) acting on a beam element and the inter-storey drift (D_i) are related by the non-linear F_i-D_i 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.

Validation of the model with earthquake test results

The proposed MDOF non-linear dynamic model was tested 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/s^2 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 (m_1, m_2 and m_3) 44.6 ton, and for the fourth storey (m_4) 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.



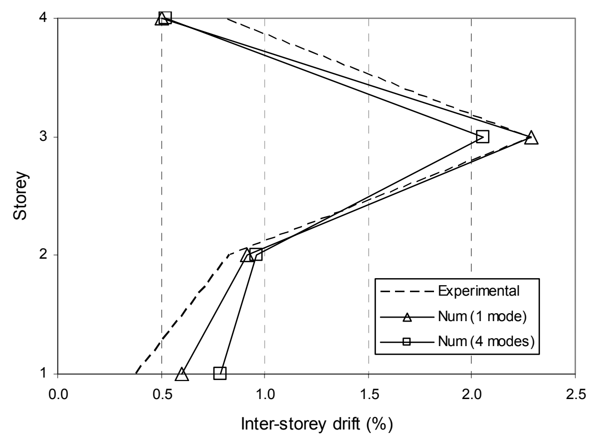


Figure 6: MDOF simplified model: a) equivalent global damping versus global drift for the BF structure; b) maximum inter-storey drift profile computed and test results for BF structure (975-yrp earthquake)

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.

STRUCTURAL OPTIMIZATION PROBLEMS

Structural optimization problems consist on determining the configurations of structures that obey assigned constraints, and produce an extremum for a chosen objective function. In order to solve them, they are normally transformed into a mathematical form that can be solved by general optimization tools. 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 have 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.

Structural strengthening optimization problems' formulation

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).

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, ΔF_y), see Figure 7-b; ii) the additional pre-yielding stiffness (ΔK_y); and, iii) the yielding strength of the energy dissipation devices (F_{ydev}).

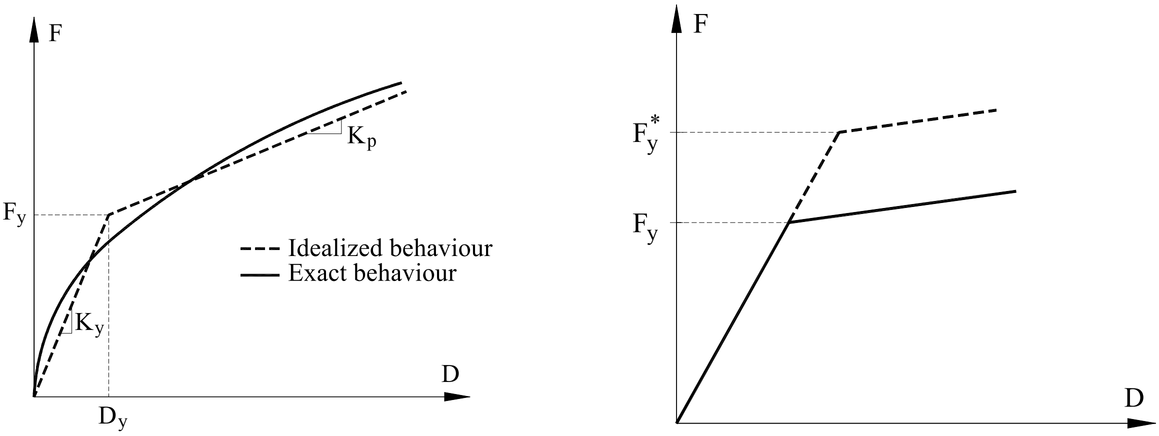


Figure 7: Strengthening optimisation: a) lateral storey shear versus inter-storey drift behaviour (exact and idealized bilinear behaviour); b) additional strength strategy

Illustrative example: optimum redesign of the existing structure

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 (non-seismically designed), 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.

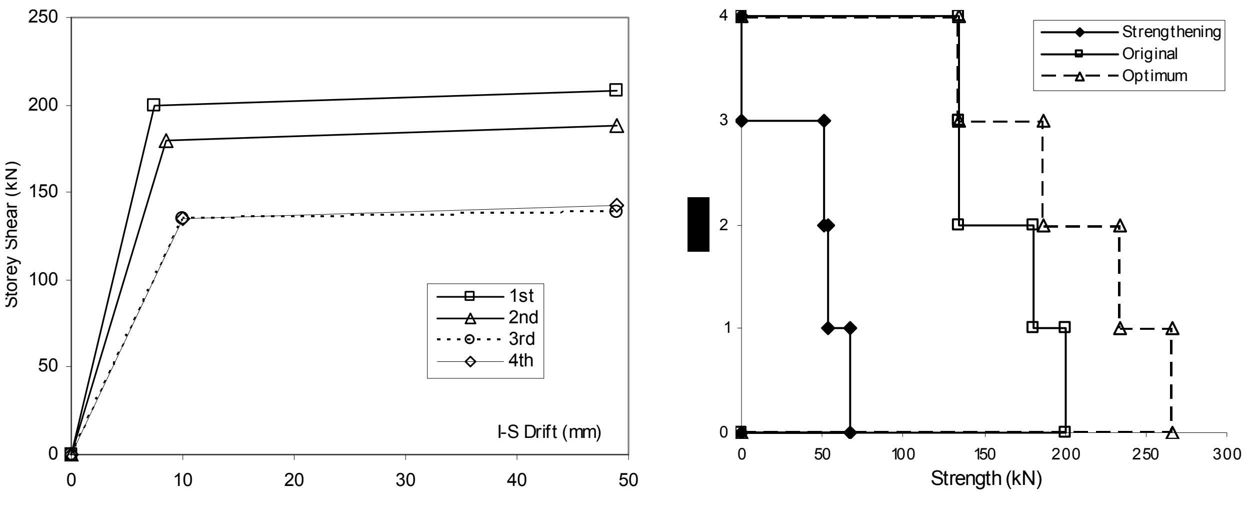


Figure 8: Example: a) storey shear-drift curves adopted from the experimental tests; b) storey yielding strength of the existing structure and optimum strengthening distribution

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: 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.

FINAL REMARKS

A series of pseudo-dynamic tests on two full-scale models of a 4-storey RC frame representative of existing structures designed without specific seismic resisting characteristics (common practice of 50~60 years ago) were carried out at the ELSA Laboratory. Analysis of the test results and comparison between the behaviour and earthquake vulnerability and performance of the different structures were briefly reviewed. Detailed results from the tests and corresponding analysis can be found elsewhere [Pinto *et al.*, 2002; Varum, 2003].

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.

ACKNOWLEDGMENTS

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