SEISMIC ANALYSES OF A R/C BUILDING - STUDY OF A RETROFITTING SOLUTION

Seismic retrofitting of R/C buildings

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Abstract

The preliminary experimental results from the tests on a 4-storey R/C frame structure are presented and discussed. The full-scale model is representative of the common practice of 40~50 years ago in most south European countries. Special attention is devoted to the study of a retrofitting solution based on bracing and rubber dissipaters, which intends to increase stiffness and damping reducing consequently the earthquake deformation demands.

Keywords: Reinforced concrete buildings, seismic retrofitting, dissipating devices, earthquake tests, global modelling.

1 Introduction

The recent earthquakes have dramatically demonstrated that research in earthquake engineering must be directed to the assessment and strengthening of existing constructions lacking of appropriate seismic resisting characteristics. The very recent 'European earthquakes' (e.g. Italy-1997, Turkey - August 1999, Greece - September 1999) confirm and highlight that also Europe may suffer from the vulnerability of the existing building stock.

There is an increasing effort devoted to the issue; however, it is also recognised the great difficulties of the problem. In fact, it involves several actors namely the earthquake engineering (EE) community, policy makers and building owners who must work together for a successful end. To the EE community should be assigned the following tasks: development of effective retrofitting solutions and techniques and development of codified re-design methods and rules allowing their widespread application by the technical community.

Along these lines, a European project, the ICONS project, financed by the TMR programme of the Commission, was recently set-up. Under the ICONS-Topic 2 - Assessment, Strengthening and Repair research programme it is foreseen to test pseudo-dynamically two full-scale reinforced concrete frames [1], which are supposed to be representative of the design and construction practice of 40~50 years ago in most of south European, Mediterranean countries. Design of these frames was performed at LNEC [2] under the framework of the ICONS project and the tests will be carried out at the ELSA laboratory of the Joint Research Centre financed by the TMR-Programme, Access to Large-scale Facilities.

Aiming at a preliminary assessment of the structure and to evaluate the effectiveness of different retrofitting solutions several ICONS participants are performing non-linear analyses and are also studying different retrofitting solutions. In addition to the non-linear analysis of the frames, it is assessed the effectiveness of a retrofitting solution based on bracing with rubber dissipaters. A preliminary analytical assessment of the frame capacity was made by Griffith [3] who made also a simplified design of the bracing system. The final design of the bracing system was made by Taucer [4] without taking into account the infill panels.

This paper summarises the results from the non-linear analyses of the structure considering several cases, namely: the bare frame (Frame), the infilled frame (Frame + Inf) and the retrofitted frame (Frame + Inf + Ret). Also included is the analysis of the retrofitted frame without infill panels because the design of the retrofitting system was performed ignoring the infills. These numerical results were labelled (Frame + Ret).

Section 2 gives details on the structure, materials, loads and retrofitting solution. The modelling aspects (models, assumptions, etc.) and corresponding parameters are presented in section 3. Section 4 focuses in the non-linear analyses and corresponding results. Section 5 briefly presents and discusses the experimental results from the bare frame tests, discusses the issue of modelling (refinement, parameters, etc.) and compares the numerical results with the available experimental ones. Finally, section 6 summarises the main conclusions of the study.

2 Structure, materials, loads and retrofitting solution

2.1 Structure Geometry and Material Properties

The dimensions of the building and section details are shown in Fig. 1. It can be seen in the elevation and plan drawings that the storey heights are 2.7 m and there are two 5 m span bays and one 2.5 m span bay. Brick masonry infill (200 mm thick) is contained within each bay. The left-hand bay infill contains a window (1.2 x 1.1 m) at each of the 4 levels. The central bay contains a doorway (2.0 x 1.9 m) at ground level and window openings (2.0 x 1.1 m) in each of the upper 3 levels of the building. The right-hand (2.5 m span) bay contains solid infill (i.e., without openings). It should be noted that the longitudinal reinforcing steel was smooth round bars, not the deformed steel bars used for reinforcement today. All beams in the direction of loading are 250 mm wide and 500 mm deep. The transverse beams are 200 mm wide and 500 mm deep. The concrete slab thickness is 150 mm. The column splice joint detail and the column stirrup detail should be noted in particular. Their likely poor seismic performance will be discussed later.

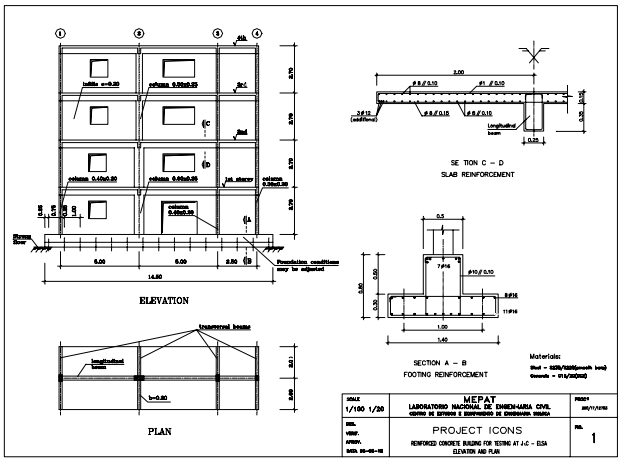

Fig. 1. Plan and elevation views of concrete frame plus masonry infill building

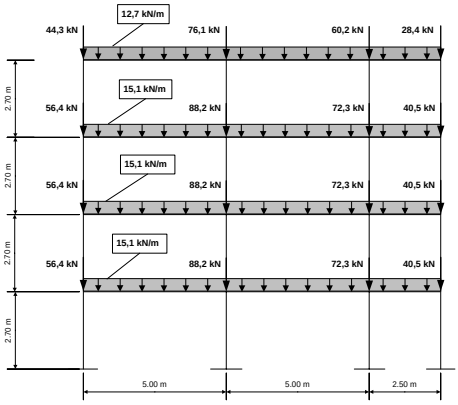

Fig. 2. Scheme of vertical loads for non-linear analyses

Preliminary calculations have been carried out in order to establish which failure mechanisms are most likely to occur under seismic loading. In order to do this, the *mean values* for the respective material strengths shown in Table 1 have been used.

Table 1. Material properties

Material	Relevant Properties (*mean values*)
Steel (FeB22k)	$f_{sy} = 235MPa \quad f_{su} = 365MPa \quad \varepsilon_{su} = 29.9\% \quad E_s = 200x10^3\,MPa$
Steel (tests results)	$f_{sy} = 337MPa \quad f_{su} = 455MPa \quad \varepsilon_{su} = 25.0\%$
Concrete (C16/20)	$f_{cu} = 24\,MPa \quad \varepsilon_{cu} = 0.2\% \quad f_{tu} = 1.9\,MPa \quad E_c = 20x10^3\,MPa$

2.2 Loads, masses and input motions

Vertical loads - For the analyses, 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 (live-load, weight of partitions, finishing). Fig. 2 gives the details of the loads considered [2]. The accelerograms considered in the non-linear analysis were derived from hazard consistent response spectra corresponding to several return periods. Accelerograms with 15 seconds were assumed. The storey masses considered were: 40.0 tons for the last floor and 44.6 tons for the others. A Rayleigh damping of 2% for the first and second modes was considered.

2.3 Retrofitting solution

It is expected that the 4-storey RC frame under analysis will perform not satisfactorily for the earthquake motions corresponding to the ones assumed in the present design codes. Several deficiencies were identified in the structure, such as, inadequate dissipation/collapse mechanisms, inadequate detailing of members and joints. In order to improve the seismic performance of such a structure, a retrofitting intervention is required. There are three basic solutions to increase seismic performance of the

structure, namely: to isolate the structure, to increase its deformation capacity and to increase its stiffness, strength and damping characteristics.

The retrofitting herein studied is based on the last solution. It is a bracing system with rubber dissipation devices, which will increase stiffness and damping of the system, reducing consequently the deformation demands.

Two alternative layouts were studied for the bracing: one located in the central bay (K-bracing) (see Fig. 3), which leads to better distribution of the storey forces but interferes with the openings (door and windows) and the other (X-bracing) located in the shorter external bay. This two alternative solution led two similar results.

The design of the bracing system, including the dissipation devices, was performed assuming [3] that 1% drift (27 mm inter-storey drift) corresponds to the ultimate limit state for the frame under analysis. Furthermore, it was assumed that, for these deformation levels, the effects of the infill panels are negligible. Further, it was assumed that the peak base shear strength of the frame, for the 1% drift, is 150kN and the effective stiffness (secant stiffness) of the equivalent SDOF system with the mass located at 2/3rd of the total height of the building leads to a Period (Ts = 1.8 sec).

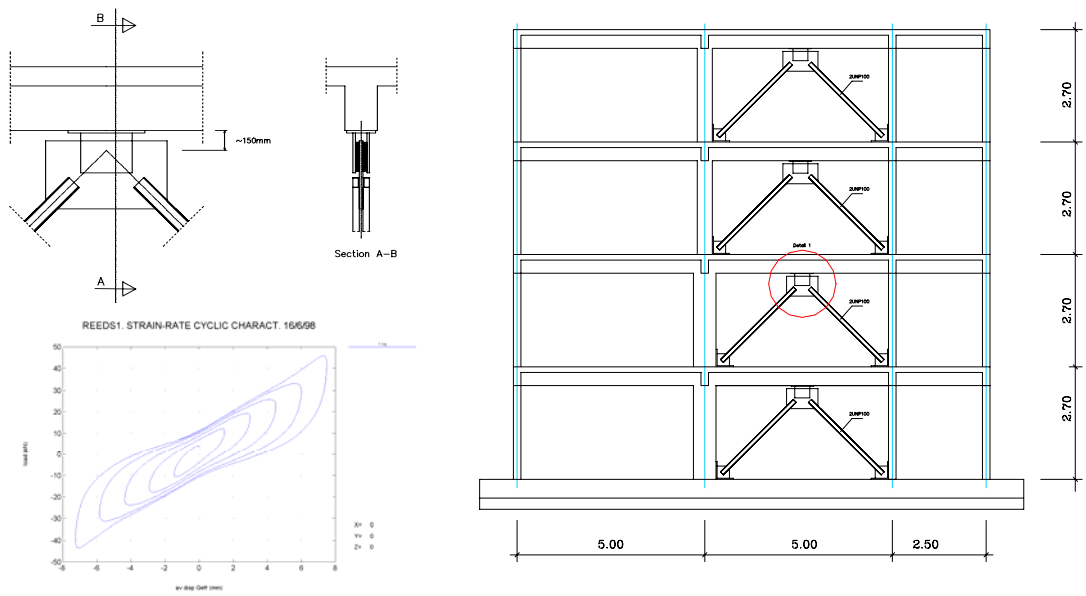

Fig. 3. Bracing system: Device details and general layout and typical diagram for a device tested at ELSA under the project REEDS

Table 2. Characteristics of the Energy dissipation devices (one device)

1% Int.-Storey Drift	Location	DLF	F_u (kN)	D_u (mm)	F_y	K_1	Obs.
10% Non-exceeding probability	Storey 0-2	0.35	80	25	$F_u/3$	$K_0/10$	1 device per storey (Fig. 3)
	Storey 2-4		50	25			

Energy dissipation device loss factor - DLF; DLF = tang δ; $\delta = \sin^{-1}(2W/(\pi\Delta W))$; W - area surrounded by the hysteresis loop; ΔW - half of the area of the rectangle that inscribes the hysteresis loop ($= 2F_{max}.D_{max}$)

Note: Devices are able to accommodate displacements and forces up to 140% of their nominal capacity ***(F_u,D_u)***

The design displacement spectra for the different damping ratios were derived from a basic one for 5% damping (assumed to increase linearly from 0, for T=0 seconds, to

200 mm for T=2 seconds, and being constant for higher periods) using the following 'correction factors' (SQRT(5/ζ)).
For a 50-years non-exceeding probability of 10% a device is required at each storey with the characteristics given in Table 2.

3 Modelling

The structure (reinforced concrete frame) has been modelled by beam elements with non-linear behaviour at the potential hinge zones (vicinity of the frame joints) and linear elements in the internal parts of the structural elements. Furthermore, an elastic element was also considered to simulate the joint thickness. The non-linear elements are represented by a fiber model with uniaxial constitutive laws for concrete and steel. To simulate the slab contribution, 1.0 m was considered for the effective flange width.

The infill panels were simulated with bidiagonal struts and the bracing system with dissipaters were represented with bar elements (bracing) and a non-linear spring element for the dissipater.

The length of the non-linear fiber element was estimated on the basis of empirical formulae and taking into account that this element is a Timoshenko element with constant curvature (one integration point only). Assuming a common empirical expression for the effective plastic hinge length and that the curvature in the plastic hinge zone has a parabolic distribution, the equivalent length hinge-element, L_p^*, calculated for the same chord rotation, depends on the ductility. However, it tends asymptotically to half of the empirical value of the plastic hinge length.

Concrete model - In compression, a parabolic curve is assumed from the initial unloaded stage up to the peak stress values, with initial tangent modulus equal to the concrete Young modulus. The softening branch is described by a straight line, whose slope depends on the confinement degree. Under tensile stresses, the behaviour is described by a linear elastic branch with a subsequent softening branch, which accounts for tension stiffening effects. The cyclic behaviour of concrete as been firstly described by a crude model representing the main feature of the concrete behaviour under cyclic loading and in a second stage the model has been improved in order to account for secondary effects such as crack closing and to avoid eventual numerical difficulties in the algorithms. Analytical formulae and detailed description of this model can be found elsewhere [5].

Steel model - The steel model includes typical curves for monotonic and cyclic loading. The monotonic curve is characterised by an initial linear branch followed by a plateau and a hardening branch up to failure. The cyclic behaviour is described by an explicit formulation proposed by Giuffré and Pinto and implemented by Menegotto and Pinto [5].

Masonry (infill) model - The model for infill panels is the strut model proposed by [6]. It is a general multi-linear model which accounts for cracking, compression failure and strength degradation due to either monotonic or cyclic loading as well as for the pinching effects due the crack closing. The model assumes no tensile

resistance and the behaviour in monotonic compression is described by a multi-linear curve including a primary linear elastic behaviour, a second branch approximating the cracking process and two final branches representing two phases of the masonry behaviour, which can be considered as a plastic behaviour (crushing of the masonry panel) with positive and subsequently negative strain hardening. Cyclic behaviour is characterized by a linear unloading-reloading law without plastic displacement in the primary branches. The hysteretic behaviour, after having reached the plastic point, is also governed by a multi-linear curve with specific rules to account for plastic deformations, crack closing strength degradation.

Identification of the strut model parameters was performed by empirical expressions suggested in [7]. The values showed in Table 3 were considered in the analyses, where: E_p - Young modulus; G_p - Shear modulus; f_{tp} - reference tensile strength; C_R - factor of quality of masonry work; ν - post-yield slope of envelope curve; and, μ - ductility factor.

Table 3. Mechanical properties - Mean values used in the analyses

E_p (GPa)	G_p (GPa)	f_{tp} (kPa)	C_R	ν	μ
1.28	0.24	200	0.9	0.05	2.5

Dissipater model - The dissipaters were simulated by a bilinear model (see Fig. 3). The steel model introduced above was used to represent the constitutive uni-axial law of the dissipater setting the model parameters according to the relevant requirements, namely a sharp transition between the linear and the 'post-yielding curves and the tangent of the asymptotic curve defining the post-yielding range.

4 Non-linear analyses

Static pushover analyses were initially performed, in order to identify the global behaviour of the structure and to compare relative strengths (frame and frame+infills) and corresponding evolution with the imposed deformations. Non-linear analyses were performed for several earthquake intensities. Some results (drift profiles and vulnerability functions) from the non-linear analyses are hereafter illustrated.

Table 4. Frequencies calculated for bare and infilled frames

Frequencies (Hz)	1st Mode	2nd Mode	3rd Mode	4th Mode
Bare frame	1.47	4.32	7.04	9.55
Infilled frame	3.85	10.90	14.13	16.78

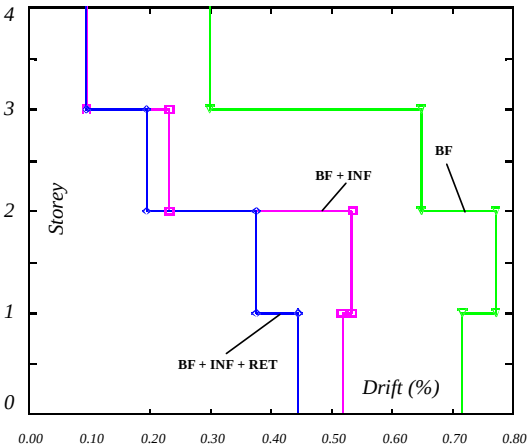

Fig. 4. Maximum drift profiles for bare, infilled and retrofitted frames (975yrp)

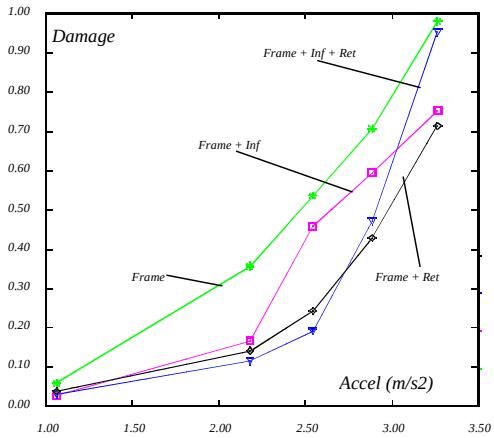

Fig. 5. Global damage on the frame structure (evolution with input intensity)

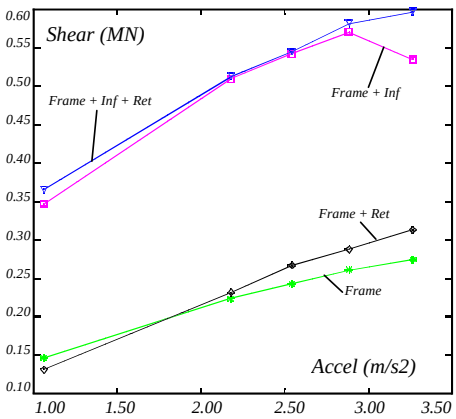

Fig. 6. Maximum base-shear (evolution with input intensity)

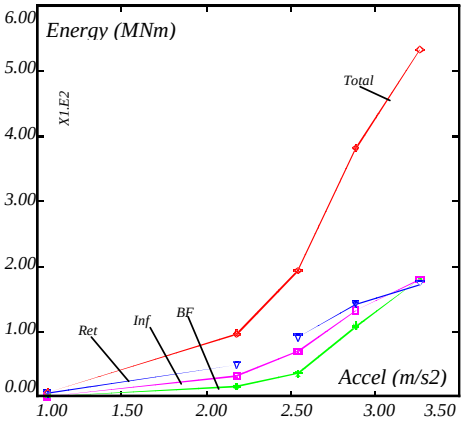

Fig. 7. Energy dissipation (evolution with input intensity)

5 First test results against numerical results

5.1 Results from the tests on the bare frame specimen

A series of pseudo-dynamic (PsD) tests on the reinforced concrete full-scale frame model is currently being carried out at the ELSA laboratory [1]. It is programmed to test both the infilled and the bare frame and to assess experimentally the effectiveness of various retrofitting solutions and techniques. The tests on the bare frame were just performed and a few results are hereafter presented.

The bare frame specimen (full-scale 4 storey R/C frame - without masonry infill) was subjected to one earthquake corresponding to 475 years return period (475-yrp) and subsequently a second PsD test with a 975-yrp was carried out. The results from these tests are given in Fig. 8-9, in terms of maximum inter-storey drift profiles for positive and negative deformations and shear-drift diagrams for the 3rd storey.

It is apparent that the deformation demands concentrate in the 3rd and 4th storeys for the 475-yrp earthquake test and collapse of the 3rd storey was almost reached for the

975-yrp test. This test was stopped at 7.5 seconds in order to allow repair and retrofitting and to assess their effectiveness in the subsequent tests.

From these tests on the bare frame it is possible to confirm the storey mechanism, which was expected to develop during the earthquake response. In fact, the structure represents design common practice of ~40 years ago when seismic loading was roughly considered or not even taken into account. From the shear-drift diagrams for the 475-yrp test it is apparent that a rather limited non-linear behaviour (storey ductility of about 2 at the 3rd storey) and quite limited damage occurred during the test. Slight cracking at column extremities, as well as in the girders (at the slabs - for negative moments) could be observed and no spalling of cover concrete occurred. The 975-yrp test was subsequently performed and was stopped at 7.5 seconds because failure of the 3rd storey was imminent. In fact, clear hinging of the strong column of the 3rd storey at the base, top and also at the bars termination zone (700 mm from the base of the column) developed with severe damage (yielding, spalling and yielding of the stirrups at the bars termination one). Disclosure of the 90 degrees bent stirrups was not observed but it would certainly have occurred if the test had been continued.

The results have only been recently available and a more detailed analysis is required. However, it is already possible to confirm the high vulnerability of these structures. In fact, it was demonstrated that, in spite of the very limited damages for the 475-yrp earthquake, the demands for a slightly higher intensity earthquake (1.3 times the reference earthquake, in terms of peak acceleration) led to imminent storey failure and consequent collapse of the structure. Development and validation of effective (also economical) retrofitting solutions and techniques for this type of structures is therefore urged. The second part of the testing campaign will devoted to these issues.

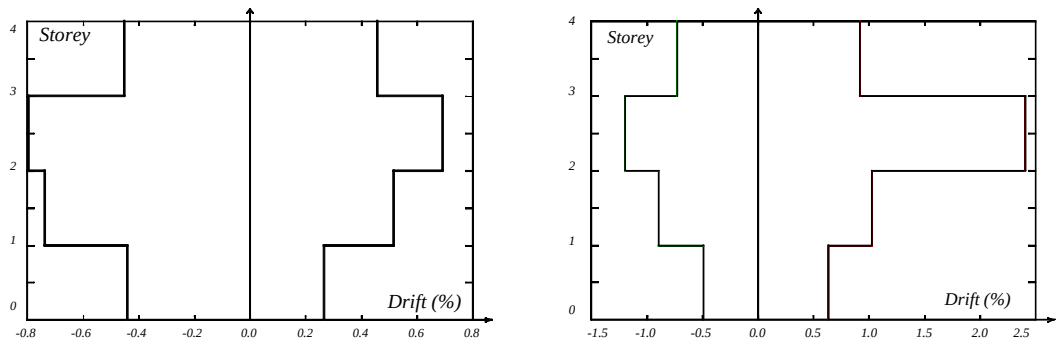

Fig. 8. Maximum Drift Profiles for 475 yrp (left) and 975 yrp (right)

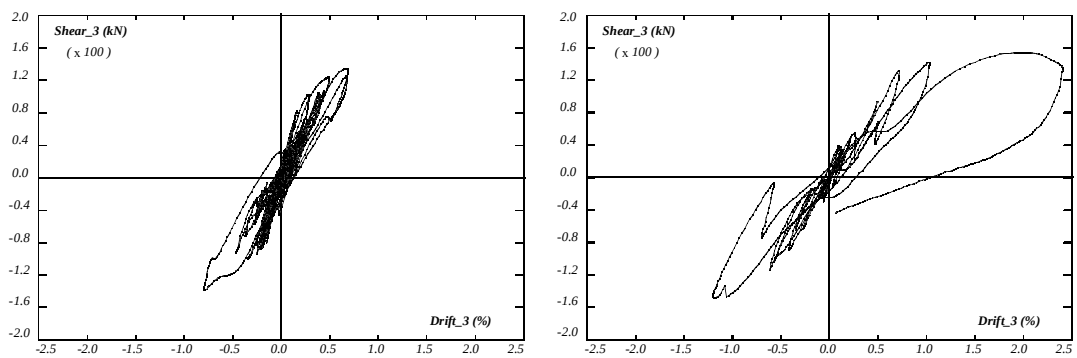

Fig. 9. 3rd Storey Shear/Drift diagram for 475 yrp (left) and 975 yrp (right)

5.2 Numerical modelling - refinement and model parameters

One of the important objectives of the numerical benchmark on the response of the structure is to find out the most suitable numerical models to predict the seismic response of this kind of structures and to identify the sensivity of the models to their characteristic parameters. It is also expected that such a type of structures will experience shear failure, failure at the beam column joints and phenomena like slippage of rebars (steel rounded bars) and strain penetration. Therefore, one should use appropriate models to take into account most of the above mentioned phenomena.

The JRC used a fibre model [5] considering a rectangular cross-section for the columns and a T-beam to represent the girders because such a model allows to consider both bending and shear, which is likely to control failure in the central stocky column. However, the following aspects were not taken appropriately into account: the inter-storey high was uniformly considered with 2.7 m but, as the beam element supporting the cross-section should be located at the cross-section centroid, the first story high must be shorten. Therefore, the first storey stiffness and strength were underestimated. Additionally, the slab participation was also almost neglected. This point is particularly relevant for the refined modelling considered because the effects of the slab reinforcement can be significant. In fact, as the equal displacement condition for the storey nodes is not imposed, the girder is allowed to deform axially and the section stiffness drops suddenly after cracking. On the contrary, this drop does not happen in the columns. Consequently the relative strength of the columns and girders may differ strongly from the reality.

The results from post test non-linear analyses (maximum drift profiles) taking into account the aspects discussed above are shown in Fig. 10 together with the results from the experimental tests and the numerical results obtained at LNEC. These non-linear analyses were performed with a Takeda-type model and the parameters for the multi-linear constitutive laws were obtained assuming full-cracked sections. Furthermore, bilinear models were considered for the envelope curve (pointing directly from origin to yielding). Therefore, the 2% damping considered by LNEC seem to be insufficient to take into account the cracking affects. The higher flexibility of the LNEC model is apparent in Fig. 10. However, drift profiles (pattern) are rather well in agreement with test results.

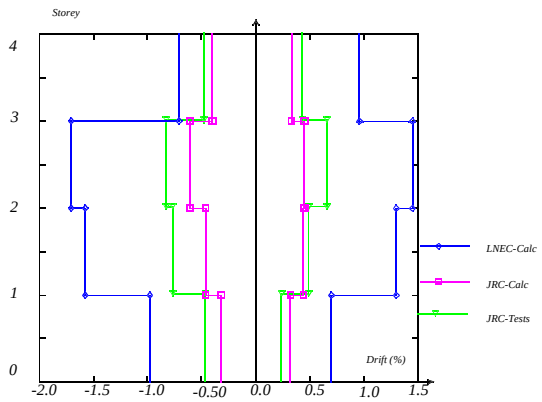

Fig. 10. Drift profiles (numerical and experimental)

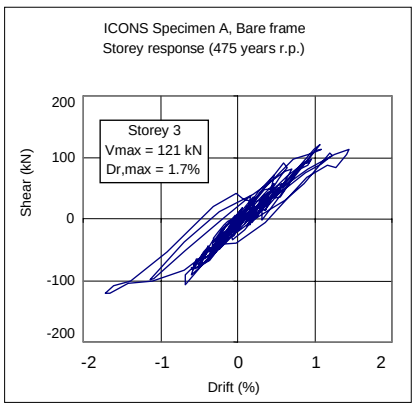

Fig. 11. LNEC 3rd Storey Shear/Drift diagram

Therefore, it is concluded that much care should be taken in the modelling of these structures. Furthermore, the use of refined models may lead to unrealistic results if the model parameters are not correctly chosen. It is also clear that the sensitivity of the response to such model parameters increases with the complexity of the models.

6 Conclusions

The results from the analyses show that the infill panels considerably protect the reinforced concrete frame. The numerical analyses for the retrofitted frame case allow to conclude that:

- The proposed light retrofitting solution is effective for low, medium and high intensities but not particularly effective for very high intensities, when infill panels exist. This retrofitting system was designed for the bare frame and it is very effective for this case. However, a more accurate design shall take the infill panels into account.
- The system leads only to a small increase of storey shear forces.
- Additional energy dissipation - the energy dissipation is equally shared by the RC frame, the infill panels and the retrofitting devices.

The preliminary results from the bare frame tests demonstrate how vulnerable is this type of structures. In spite of a 'satisfactory performance' for the nominal input motion, the structure exhibits a premature storey collapse mechanism (column hinging at the 3rd storey) for an input motion slightly higher than the nominal one.

7 References

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