CHARAKTERISTIKA SEISMICKÉHO CHOVÁNÍ ŽELEZOBETONOVÝCH BUDOV

SEISMIC BEHAVIOUR CHARACTERIZATION OF REINFORCED CONCRETE BUILDINGS

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Anotace: Stanovení seismické zranitelnosti železobetonových budov postavených do konce sedmdesátých let v mnoha evropských zemích s mírným až vysokým seismickým nebezpečím, projektovaných a postavených bez přiměřených protiseismických opatření, má extrémní význam. Tyto budovy představují významný zdroj rizika pro naše města. V článku se uvádějí hlavní výsledky výzkumu na úseku stanovení seismické zranitelnosti a modernizace existujících železobetonových budov.

Abstract: The seismic vulnerability assessment associated to existing reinforced concrete buildings, constructed until the late 70’s in many European countries with moderate to high seismic hazard, designed and constructed without considering adequately earthquake provisions, is of extreme importance. They constitute a significant source of risk for our cities. In this paper are presented the main results of research developed in the field of seismic assessment and retrofitting of existing RC buildings.

1. INTRODUCTION

Recent major earthquakes around the world have evidenced that this type of existing buildings lacking appropriate seismic resisting characteristics are very vulnerable. 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 development and calibration of simplified numerical models to represent the non-linear behaviour of structures is of extreme importance. The objectives of recent research developed at the University of Aveiro on the influence of infill masonry panels in the response of framed structures, on the biaxial bending behaviour of RC columns, and on the influence of bond characteristics in the response of RC elements with plain smooth reinforcement bars are presented in this paper.

2. TESTS ON FULL-SCALE STRUCTURES (BARE AND INFILLED)

In the framework of the ICONS Topic 2 - Assessment, Strengthening and Repair - research programme [Pinto *et al.*, 2002-a], 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, have been constructed and tested in order to assess the vulnerability of bare and infilled structures. The general layout of the building frame model is shown in Figure 1. It is a reinforced concrete 4 storeys full-scale frame with three bays. A comprehensive description of the frames, tests on material samples used in the construction, reinforcement detailing, loads, and PsD test results can be found in [Carvalho *et al.*, 1999; Pinto *et al.*, 2002-b; Varum, 2003]. The input seismic motions were defined in order to be representative of a moderate-high European seismic hazard scenario [Campos-Costa and Pinto, 1999]. Acceleration time histories for 475, 975 and 2000 years return periods (yrp) were used in the tests (PGA of 218, 288 and 373cm/s^2, respectively).

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.

The general layout of the structure including infill panels (IN) and the type and location of the openings are presented in Figure 1. The 150 mm thick infill-walls (non-load bearing) were constructed after the reinforced concrete frame. 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 3 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 3). 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.

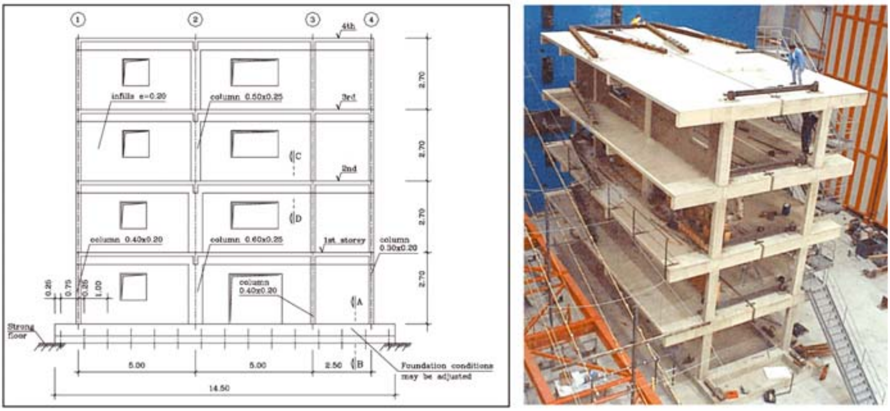


Figure 1. Tested frames:
a) elevation views of the frames; b) models in the ELSA laboratory

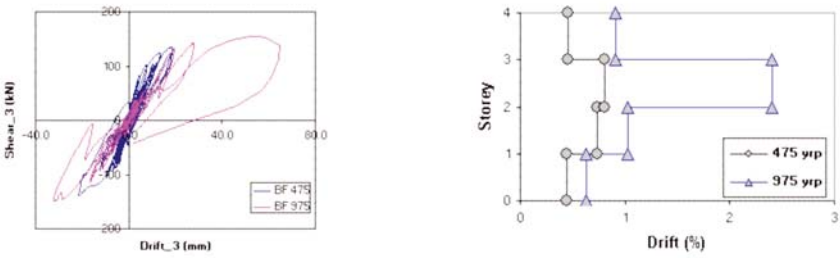


Figure 2. BF test results:
a) shear-drift diagram at the 3rd storey; b) maximum inter-storey drift profiles

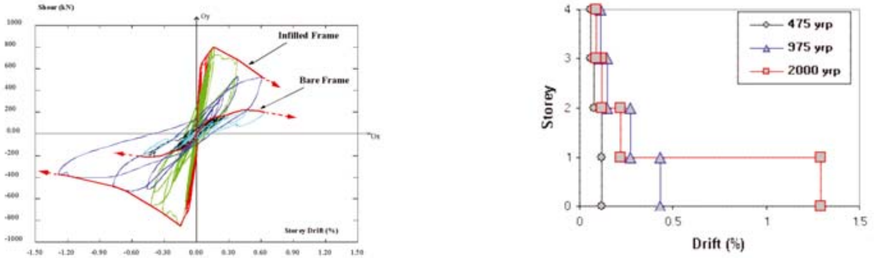


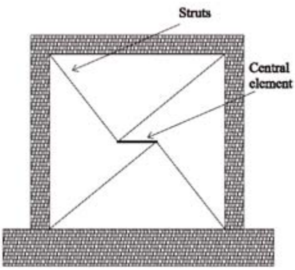
Figure 3. IN test results:
a) 1st storey shear-drift diagrams and envelope curves (comparison with the BF);
b) maximum inter-storey drift profiles

3. INFILL MASONRY MODELLING

It is inadequate to assume that masonry infill panels are always beneficial in terms of structural response. The contributions of infills to the building's seismic response can be positive or negative, depending on a series of phenomena and parameters such as, for example, relative stiffness and strength between the frames and the masonry walls. In recent earthquakes, numerous buildings were severely damaged or even collapsed as a result of the structural modifications to the basic structural system induced by the non-structural masonry partitions. Masonry infill panels can increase substantially the global stiffness of the structure. Consequently, its natural period will decrease and, depending on the seismic spectrum values at the vicinity of the bare structure natural period, the seismic forces can increase. There are many different techniques proposed in the literature for the simulation of the infilled frames, which can be basically divided in two groups, namely the micro-models and the simplified macro-models. The micro-models considers a high level of discretization of the infill masonry panel, in which the panel is divided into numerous elements to take into account the local effects in detail, the simplified models are supported in simplifications with the objective of representing the global behaviour of the infill panel with a few structural elements. Micro-models can simulate the structural behaviour with notable detail, requiring different types of elements to represent respectively the bricks, mortar, interface brick-mortar, interface masonry-frame and the frame elements. However, they are computational intensive and difficult to apply in the analysis of large structures. Relatively to the simplified models, the most commonly used technique to model infill panels is based on a single or multiple compressive equivalent diagonal struts strategy.

Being aware of the importance of infill masonry elements in the behaviour of RC buildings in the last few years the new codes have included some provisions regarding the consideration of the infills and their influence on the structural response. For example, the European code, Eurocode 8 (EC8) [2003], include provisions for the design of infilled RC frames. EC8 specifies that the period of a structure to be used to evaluate the seismic base shear shall be the average of that for the bare frame and for the elastic infilled frame. Frame member demands are then determined by modelling the frame structure without the infills. The influence of irregular infill distributions, in plan and in elevation, is addressed in EC8. The American guideline FEMA-273 [1997] and FEMA 356 [2000] includes a procedure to assess the structural response of buildings, considering the infills panels. According to this document, masonry infill panels shall be represented by the equivalent diagonal struts. The struts may be placed concentrically across the diagonals, or eccentrically to directly evaluate the infill effects on the columns. The shear behaviour of masonry infill panels is considered as a deformation controlled action. FEMA-273 provides deformation acceptance criteria. The linear procedure involves a comparison between the design elastic shear force in an infill panel with the factored expected shear strength of the panel.

A macro-model was recently proposed [Rodrigues, 2005], based on the equivalent bi-diagonal-strut model. The proposed model considers the interaction of masonry panel behaviour in both directions; damage to the panel in one direction affects its behaviour in the other direction. In the proposed infill panel model, each masonry panel is structurally defined by considering four support strut-elements, with rigid behaviour, and a central strut element, where the non-linear hysteretic behaviour is concentrated (Figure 4-a).



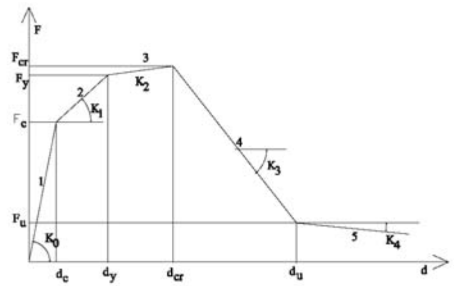


Figure 4 – Infill masonry model proposed:
a) macro-model; b) force-displacement monotonic behaviour curve

The non-linear behaviour is characterized by a multi-linear envelope curve, defined by nine parameters (Figure 4-b), representing: i) cracking (cracking force, F_c; cracking displacement, d_c); ii) yielding (yielding force, F_y; yielding displacement, d_y); iii) maximum strength, corresponding to the beginning of crushing (F_{cr}; and corresponding displacement, d_{cr}); iv) residual strength (F_u) and corresponding displacement (d_u); the fifth branch of the behaviour curve is defined by its stiffness (K_4). The hysteretic rules calibrated for masonry models are controlled by three additional parameters, namely: α - stiffness degradation; β - "pinching" effect; and, γ - strength degradation. The hysteretic rules are briefly exemplified in Figure 5.

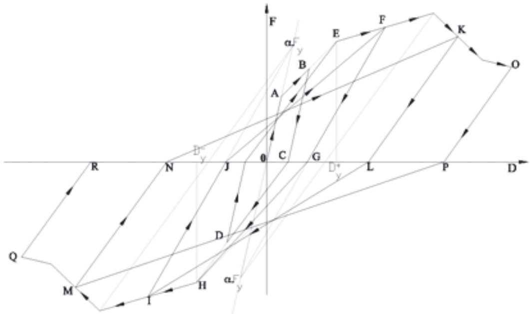


Figure 5 – Hysteretic rules for the implemented model

An existing building, representative of the Modern Architecture in Portugal (see Figure 6) was studied. The block plan is rectangular with 11.10m width and 47.40m length. The building has the height of 8 habitation storeys plus the pilotis height at the ground floor. The building has nine storeys and the structure is mainly composed by twelve plane frames oriented in the transversal direction (direction Y). The building was analysed with a simplified plane model for each direction (X - longitudinal direction, Y - transversal direction). A peculiar structural characteristic of the type of buildings, with direct influence in the global structural behaviour, is the ground storey without infill masonry walls. Furthermore, at the ground storey the columns are 5.5 m height. All the upper storeys have an inter-storey height of 3.0 m. A detailed definition of the existing infill wall panels was considered in the global structural models, namely, openings dimensions and position in the panel, interface conditions between panel and surrounding RC elements, masonry material's properties. Three artificial earthquake input series were adopted for the seismic vulnerability analysis of the building.

Figure 6 – General views of the building block under analysis

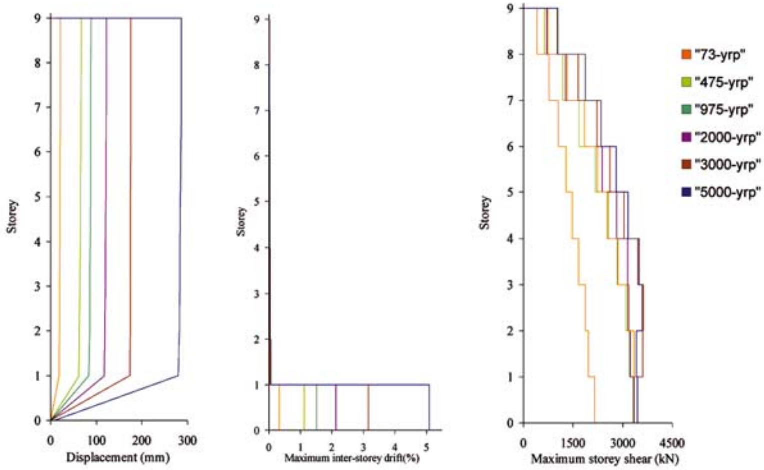


Figure 7 – Results for the longitudinal direction (X) and earthquakes of the series A: a) envelop deformed shape; b) maximum inter-storey drift profile; c) maximum storey shear profile

In Figure 7 are illustrated, for the longitudinal direction, the numerical results in terms of envelop deformed shape, maximum inter-storey drift, and maximum storey shear, for each earthquake input motion of the series A (73, 475, 975, 2000, 3000, 5000 years return period). From the analysis of the results in terms of building envelop deformed shape and inter-storey drift profile, it can be concluded that the deformation demands are concentrated at the first storey level. In fact, the absence of infill masonry walls at the ground storey and the larger storey height induces an important vertical structural irregularity, in terms of stiffness and strength. From the numerical analyses performed, it was verified for the earthquake input motions that the infill masonry panels basically do not reveals any damage. In fact, due to the building structural system and its behaviour, on one hand, and to the absence of masonry panels at the ground floor, on the other hand, the global deformation demand is concentrated at the ground floor.

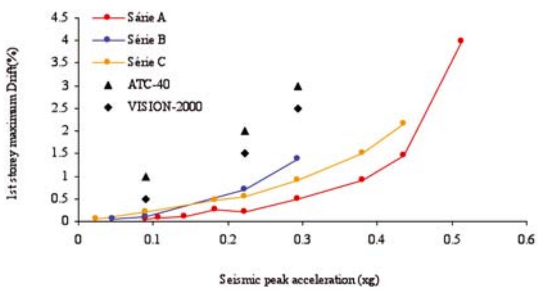


Figure 8 – 1st storey drift vs. peak acceleration and safety limits (transversal direction - Y)

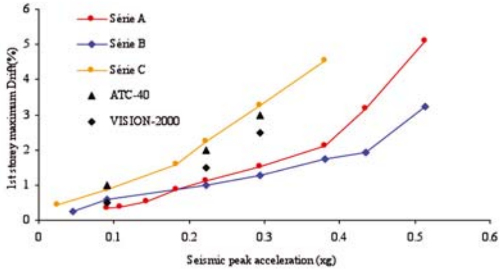


Figure 9 – 1st storey drift vs. peak acceleration and safety limits (longitudinal direction - X)

VISION-2000 [SEAOC, 1995] proposes performance objectives for buildings for three performance levels (called: Basic, Essential Hazardous, and Safety Critical). For the building under study, and due to the nature of its use, the structural safety was investigated for the Basic Performance Objectives. In Figures 8 and 9 are presented the vulnerability functions in terms of maximum 1st storey drift, comparing with the safety limits proposed at the ATC-40 [1996] and VISION-2000 recommendations. It can be concluded that the building safety is guaranteed in the transversal direction (Y), for the three earthquake input series considered. For the longitudinal direction (X), the safety is guaranteed for series A and B, but not for C series.

Although the first numerical results generally indicate the building safety for the Basic Safety Objectives, according to the international seismic recommendations (ATC-40 and VISION-2000), it should be pointed out that additional analyses have to be performed to validate this first conclusions. The model adopted for these analyses does not consider the geometric non-linearity, which can increase significantly the moments in columns and global storey lateral deformations. The high seismic risk associated to these buildings can be significantly reduced with the adoption of adequate retrofitting solutions. The seismic retrofitting of these buildings can be performed adopting economic solutions, since intervention can be resumed at the ground storey, usually without infill masonry walls, in this typology, for example with bracing systems and, eventually, combined with energy dissipation devices.

4. WORK UNDER DEVELOPEMENT

The sudden loss of concrete-steel bond is one of the sources of brittle failure in RC elements, and is reported to have been the cause of severe local damage and even collapse of many structures during earthquakes. In general terms, the work strategy planned for the development of this subject consists in three main parts: experimental tests on RC elements built with smooth reinforcement bars; development and calibration of numerical models; application of the numerical models in the vulnerability assessment of existing RC buildings built with smooth bars. The test campaign has been initiated with the cyclic testing of a RC beam from an ancient building (Figure 10-a) and will continue with the testing of a set of RC beam-column joints built with smooth bars according to the old design and construction practice (Figure 10-b).

The behaviour of axially loaded RC members under biaxial bending moment reversals is generally recognized as a very important topic. On the one hand, the actual response of RC frame columns to horizontal actions is in general three-dimensional (3D); on the other hand, the 2D features of bending moments histories applied to a given RC column section tends to reduce its actual capacity and to accelerate the strength and stiffness deterioration process during cyclic loading. Within this research is been developed an experimental program for testing RC columns bi-axially and also are been developed and calibrated numerical models for slender columns. Simplified design methods for columns subjected to biaxial bending, proposed in the standards will be validated. The present experimental study is included in a large testing campaign promoted by the Laboratory of Earthquake and Structural Engineering (LESE), of FEUP for the experimental study of RC columns (buildings and bridges) under cyclic loadings (Figure 10-c-d).

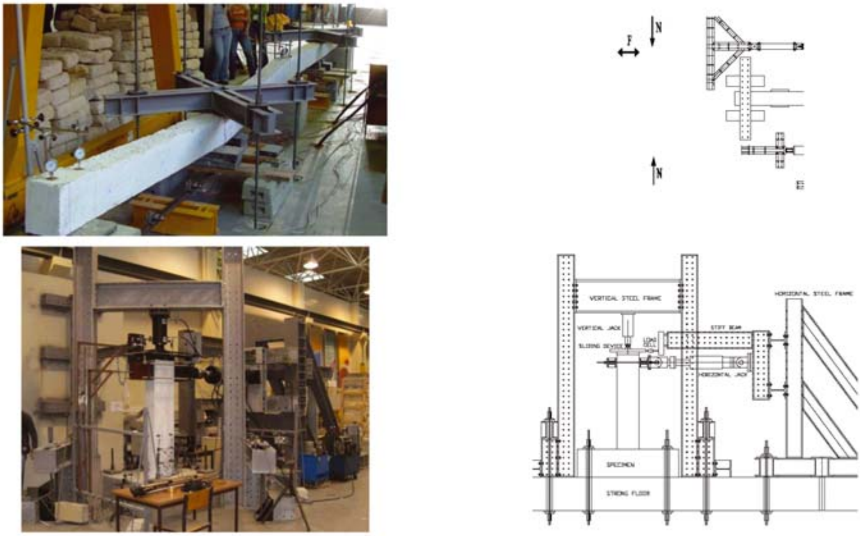


Figure 10 – Experimental work under development:
a) RC beam testing; b) RC beam-column joints testing setup;
c) RC column testing; d) RC column testing setup

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