13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 3402

SEISMIC PERFORMANCES OF REINFORCED CONCRETE FRAMES UNDER LOW INTENSITY EARTHQUAKE EFFECTS

Bing Li[1] and Tso-Chien Pan[2]

SUMMARY

In a region of low to moderate seismic risk and low wind speed, such as Singapore and Malaysia, buildings with relatively weak lateral structural resisting system are likely to represent a large portion of the building inventory. Many buildings appear to be of soft story structures. Although ground motions, due to long distance earthquakes centred in Sumatra, have occurred in Singapore and Malaysia there has been no record of earthquake damage in this region. However the reinforced concrete design code, BS 8110 [1], used in Singapore and Malaysia does not specify any requirement for seismic design or detailing of reinforced concrete structures. The main objective of this paper is to strengthen the need to look into the seismic performance of some typical existing and prospective reinforced concrete frame structures designed to BS 8110 [1] in Singapore under low seismic loading. The performance of the structures is checked through a non-linear dynamic analysis.

INTRODUCTION

Reinforced concrete frame structures are very common in a region of low to moderate seismicity, and are the predominant structural system in Singapore and Malaysia. They are usually quite stiff in one direction and flexible in the other. Buildings in this region are usually designed without consideration of seismic loading. The lack of seismic considerations resulted in non-seismic reinforcing details that are in sharp contrast to those used in modern seismic design. Therefore, it is of great concern that the strength, ductility, and energy dissipation capacity of these frame structures may not be adequate to sustain earthquake-induced loads due to the lack of reinforcement details in this type of structures. This paper presents results from the seismic assessment of a six-storey reinforced concrete moment resisting frame, which was designed based on BS 8110 [1], and the test results of the interior joints. An analytical and an experimental investigative program was undertaken to determine whether the structures, as built in Singapore, could comply with the stated (performance) design objectives. The experimental program tested the post-yield behavior of a representative subassembly of the frame designed to assess not only the post-yield behavioral characteristics of the subassembly bur also the ability of noncompliant (from a code perspective) aspects of the frame to perform acceptably when the frame was subjected to significant post-yield story drifts.

[1] Associate Professor , Nanyang Technological University, Singapore 639798
[2] Professor , Nanyang Technological University, Singapore 639798

EXPERIMENTAL PROGRAM

A subassembly (beam-column) test program was developed to assess the significance of the perceived concerns on the post-yield behavior of buildings designed to BS 8110 [1]. Two full-scale models of the prototypical subassembly were constructed and tested by Li et al [2, 3] in the heavy structure laboratory of the Nanyang Technological University. The details of the beam-column specimens are shown in Figure 1. To ensure that critical stresses would be the same in the both the prototype and model subassembly, model reinforcement was sized so that beam shear stress, bond stress in the lap splice. And joint shear stress levels would be the same. The beam-column joints were tested subjected to quasi-static load reversals that simulated earthquake loading.

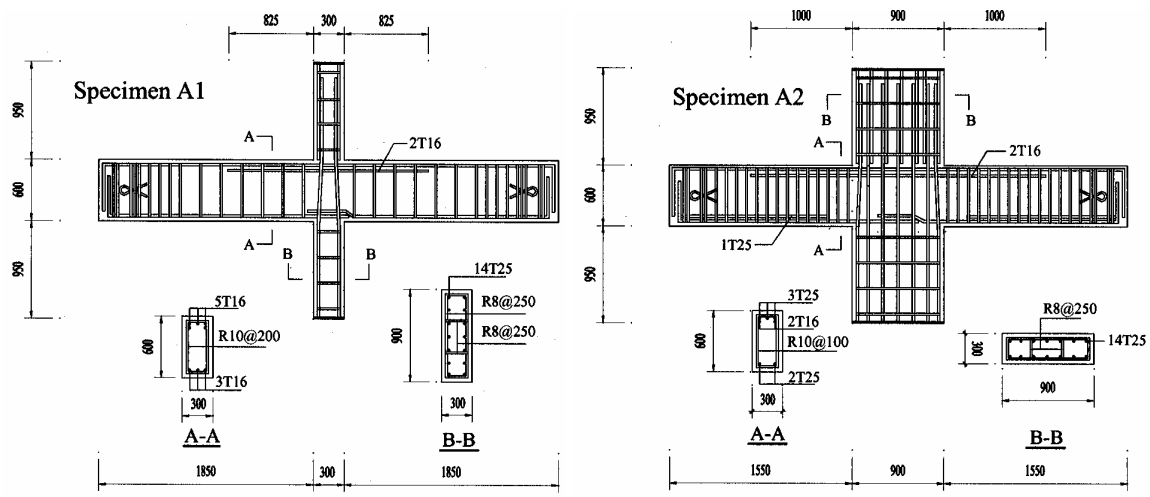


Figure 1 Beam-column joints detail

Specimen behaviors

The measured horizontal story shear force versus horizontal displacement hysteresis loop of Specimen A1 is shown in Figure 2. In addition, the theoretical ideal story horizontal load strength P_i when the beam plastic hinges were developed and the theoretical stiffness $K_{theoretical}$ are also shown in Figure 2. The theoretical initial stiffness $K_{theoretical}$ of Specimen A1 is 12.7 kN/mm, assuming that the effective moment of inertia of the beams and the columns is $0.5I_g$, and that the deformation due to joint shear distortion contributes 20% of the total horizontal displacement, where I_g is the moment of inertia based on the uncracked gross concrete area. In the loading to $\pm 0.5P_i$, flexural cracks were initiated in both the columns and the beams. No crack was observed within the joint core region. In the loading to $\pm 0.75P_i$, diagonal cracks were initiated within the joint core region. In the column side face, a few flexural cracks were initiated accompanied by a few diagonal cracks. Some pinching was observed in the hysteresis loop (see Figure 2). In the loading to a displacement ductility factor of 1, diagonal tension cracks in the joint core region extended and the number of those cracks increased rapidly. At this stage, more pinching of the hysteresis loop was observed due to the formation of the diagonal tension cracks within the joint core region, and the formation of splitting cracks along the column longitudinal bars, plus bond deterioration

along the beam bars and the column bars. In the first positive cycle of loading to a ductility factor of 2, the maximum horizontal load strength of 162.4 kN, which was equal to the ideal story horizontal load strength of Specimen A1, was reached at a corresponding story drift angle of about 2%. A maximum nominal horizontal shear stress in the joint core of $0.84\sqrt{f_c^{'}}$ or $0.15\ f_c^{'}$ was obtained in the first positive cycle of loading to a ductility factor of 2, where $f_c^{'}$ is the measured compressive cylinder strength of concrete. Within the joint core region, a few diagonal cracks opened widely. Bond splitting cracks along the column longitudinal bars extended opened wide and connected with the joint diagonal tension cracks. In the negative loading cycle, the measured maximum horizontal load strength is 149.5 kN, which did not reach the ideal load strength of Specimen A1. The hysteresis loops were significantly pinched due to severe bond deterioration along the beam and column bars and the joint diagonal tension cracking. In the second positive cycle of loading to a ductility factor of 2, severe strength and stiffness degradation due to joint diagonal tension cracking and bond deterioration along both the beam bars and column bars were observed. The measured maximum horizontal load strength was 123 kN, which is only equal to 76% of that measured in the first positive cycle.

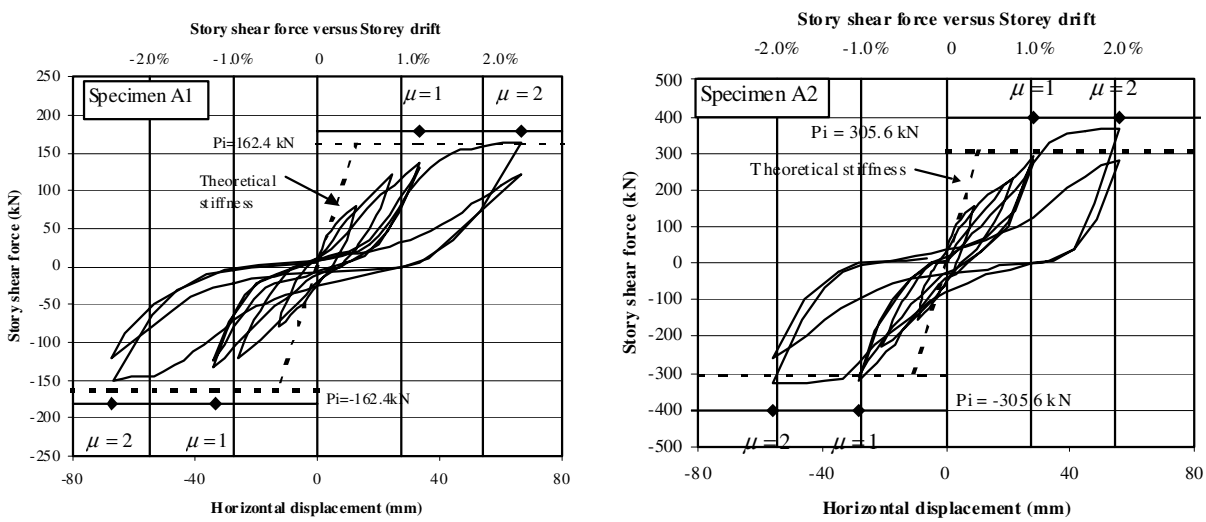


Figure 2 The story shear force versus the horizontal displacement relationship for Specimens A1 and A2

The measured horizontal story shear force versus the horizontal displacement hysteresis loop of Specimen A2 is shown in Figure 2. Also, the theoretical ideal story horizontal load strength P_i when the beam plastic hinges were developed and the theoretical stiffness $K_{theoretical}$ are also shown. Firstly, in the loading to $\pm 0.5P_i$, flexural cracks were initiated in both the columns and the beams. A few diagonal tension cracks were observed within the joint core region, and there was a small joint shear distortion and expansion observed. In the loading to $\pm 0.75P_i$, a large number of diagonal cracks were initiated within the joint core region, while the joint shear distortion and expansion increased rapidly. No obvious pinching was observed in the hysteresis loop. As the loading reached a displacement ductility factor of 1, with the opening of diagonal tension cracks in the joint core region, the joint distortion and expansion continued to increase. The flexural cracks in the beams opened widely and were accompanied by wide flexure-shear cracks, and the concrete in the beam compression zone began to be crushed. In the column, although no flexural cracks appeared, there were a few diagonal tension cracks extending from the joint core region

into mainly the upper part of the column. In the first cycle with a ductility factor of 1, the maximum horizontal load strength of 322.1 kN, which was larger than the ideal story horizontal load strength of Specimen A2, was reached at a corresponding story drift angle of 1.02%. At this stage, some pinching of the hysteresis loop was observed due to the formation of the diagonal tension cracks within the joint core region and the large flexural cracks at the column faces. In the loading to a ductility factor of 2, within the joint core region diagonal cracks increased rapidly, and opened wider. These cracks finally connected with the bond splitting cracks along the column main bars, and at the same time the joint distortion and expansion increased rapidly. The maximum joint distortion observed was 0.595%, and the maximum joint expansion was 4.34 mm. Beam flexural cracks opened wider especially at the column faces, and in the beam compression zones much more concrete was crushed and spalled. A maximum nominal horizontal shear stress in the joint core of $0.61\sqrt{f_c^{'}}$ or $0.11\ f_c^{'}$ was obtained. In the first positive cycle of loading to ductility factor of 2, the maximum horizontal load strength of 367.6 kN, which was greater than the ideal story horizontal load strength of Specimen A2, was reached at a corresponding story drift angle of about 2.04%. Bond deterioration was obvious in the bottom beam bars where bond stresses decreased rapidly in the loading to ductility factor of 2. No bond deterioration was observed in the columns and this was due mainly to the low stress in the column main bars. In the second cycle of loading to a ductility factor of 2, the maximum horizontal load strength measured in the positive loading cycle and negative loading cycle were 281.0 kN and 256.5 kN, respectively. These were about 76% and 78% of those measured in the first loading cycle.

RESPONSE OF A SIX-STOREY BUILDINGS

A typical six-storey reinforced concrete moment resisting frame was considered for the present study. The elevation and the plan of frame are shown in Figure 3. The effects of seismic action were considered in both the strong and weak directions. The frame was designed for combined gravity and lateral loads in accordance with the Singapore Loading Code, and their structural members were proportioned and detailed according to BS 8110 [1]. The typical beam-column joints in the strong and weak directions of the frame are already shown in Figure 2, representing the joint regions in the frame. Torsional and P-delta effects were not considered in the design. In non-linear dynamic analysis two ground motions are selected from the earthquake database system as the input ground motions for the frame. Since in Singapore, not until recently has attention been drawn to the safety of buildings during an earthquake because of the increased numbers of tremors generated by the long distance Sumatra earthquake. Up to now, few ground motions have been recorded; therefore two extensively used earthquake time-history records have been selected for this study. One is the 1940 E*l*-Centro record (NS component) and the other is the 1977 Bucharest record. According to the studies conducted by Pan and Sun [4], 0.1g may be taken as a creditable peak ground acceleration, which may occur in Singapore due to the long distance Sumatra earthquake, thus the selected two records were both scaled down to 0.1g to represent the creditable earthquake attacking in Singapore.

A computer program commonly referred to as *RUAUMOKO* [5] was used to study the analytical behavior of this building in the inelastic range. This program is being developed at the University of Canterbury, New Zealand, and represents the state of the art in this area. Many hysterisis models have been proposed in the previous studies for reinforced concrete structural members. However, the choice of a particularly hysterisis model in the analysis depends on the actual design and detailing of the members. In this study, it is assumed that insufficient transverse reinforcement has been provided for the structural members so that the stiffness and strength deterioration due to shear or bond loss are more significant. A bi-linear hysteresis model is therefore used to express the moment-curvature hysteresis loops of the columns. For beams, the pinching model is chosen. Factors controlling the unloading and reloading stiffness were

selected to make the hysteresis loop as thin as possible. The damping was represented using the Rayleigh damping model, and it is expressed as a linear combination of the mass and stiffness matrices. The combination coefficients are selected to give 5% of critical damping in the first two modes of vibration. The lumped nodal weights are determined assuming the average weight of floor to be 11KPa.

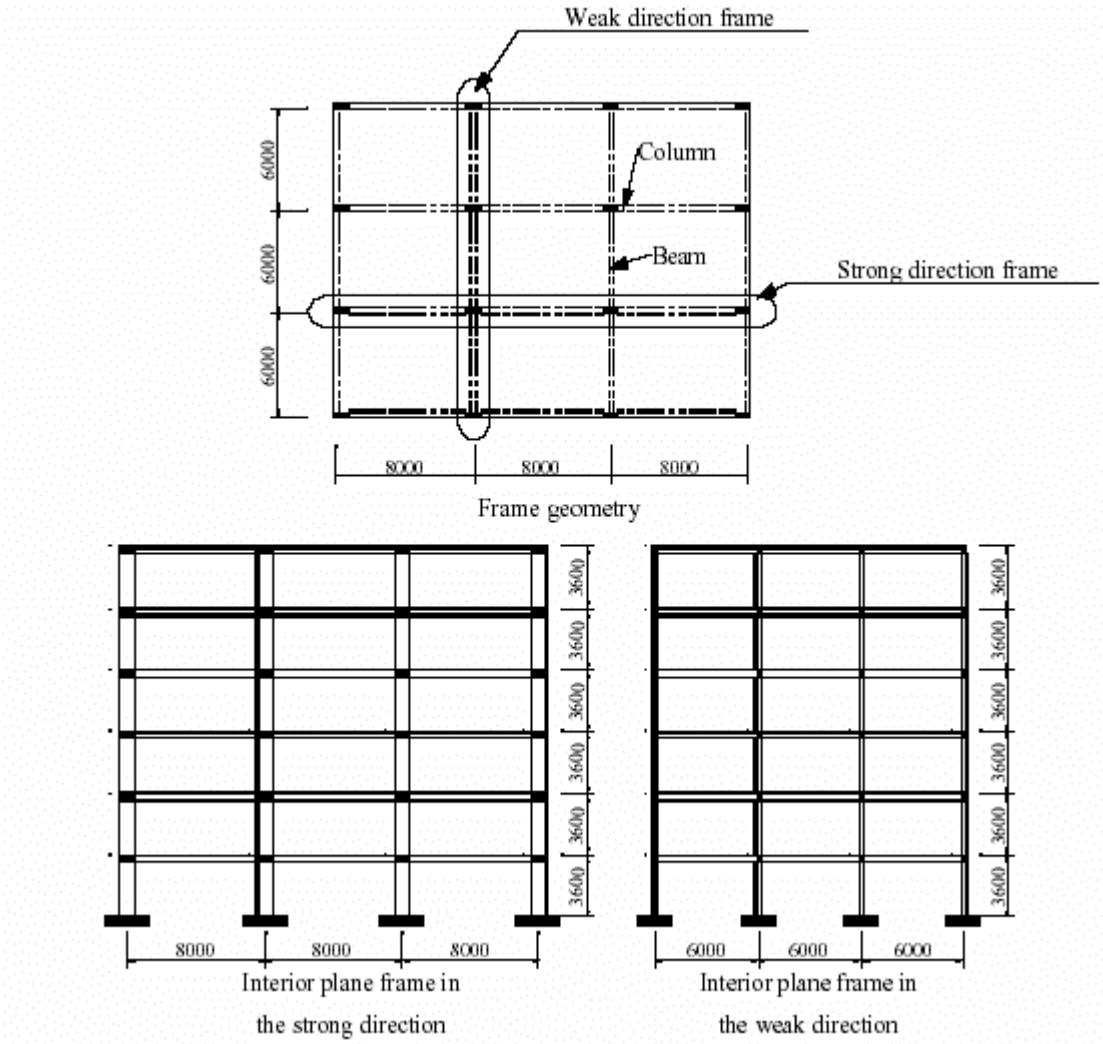


Figure 3 Reinforced concrete frame

RESULTS OF NON-LINEAR DYNAMIC ANALYSIS

Maximum Interstorey Drift Ratio

Interstorey drift ratio is considered as the primary global performance parameter. Figure 4 shows the maximum interstorey drift ratios observed in the strong and weak direction of the frame, respectively. A relationship between the desired overall seismic performance and the maximum transient drift specified by SEAOC-1995 [6] is also incorporated into the figure to better understanding of the building performance. It can be seen that for the strong direction frame only moderate damage may be caused while for the weak direction frame severe damage may occur.

Rotational Ductility Demands at Member Levels

Rotational ductility is defined by the ratio of the maximum rotation at the end of a member to the yield rotation as follows:

$$\mu_\theta = \frac{\theta_{\max}}{\theta_y} = 1 + \frac{\theta_{plastic}}{\theta_y} \tag{1}$$

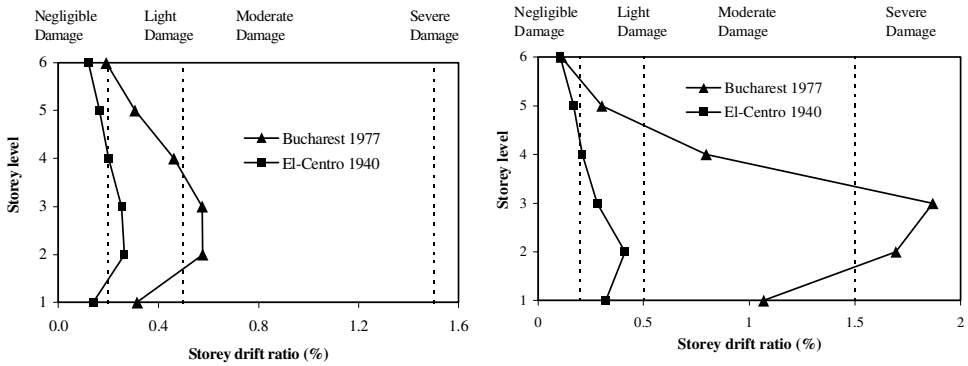


Figure 4 Inter-storey drift

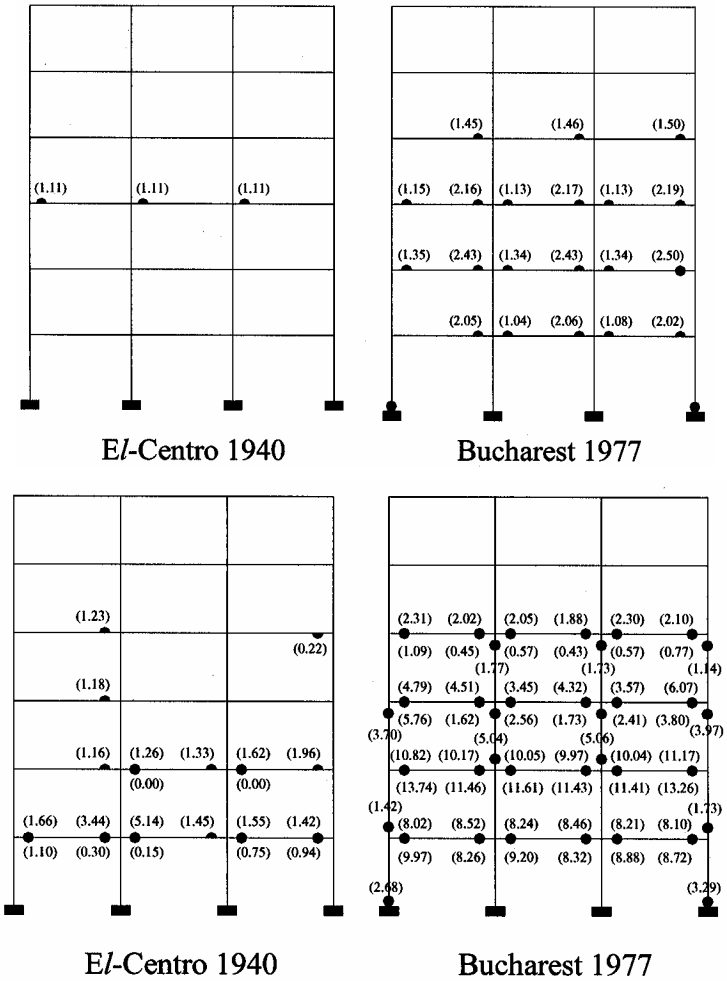


Figure 5 Rotational ductility

In which $\theta_{\max}$ and θ_y represent the maximum and the yield rotations at the end of a member. Because the inelastic flexural deformation of a beam is assumed to be concentrated at the ends, the plastic rotations occur at the plastic hinges.

The rotational ductility demands at each end of the elements are evaluated for two direction frames and two input ground accelerations described. Figure 5 shows the distributions of rotational ductility demands in two direction frames, respectively. It can be observed that the inelastic deformations of frames subjected to the scaled Bucharest earthquake are widespread at all storey levels. However, in the case of the scaled E*l*-Centro earthquake, it can be seen, that the ductility demand was much less.

Joints' behavior

Beam-column joints are often the weakest links in a structural system. For lightly reinforced beams, or with columns with high axial force levels, joint cracking may not develop and the joint failure may be judged according to the principal tension stress [7] as follows:

$$f_{dt} = \frac{f_c}{2} - \sqrt{\left(\frac{f_c}{2}\right)^2 + v_{jh}^2} \le 0.29\sqrt{f_c^{'}} \qquad (2)$$

For beam-column joints with high shear stress levels, premature failure of diagonal compression strut tends to occur and the joint failure may be judged according to the principal compression stress (Comite Euro-International 1997 [8]) as follows:

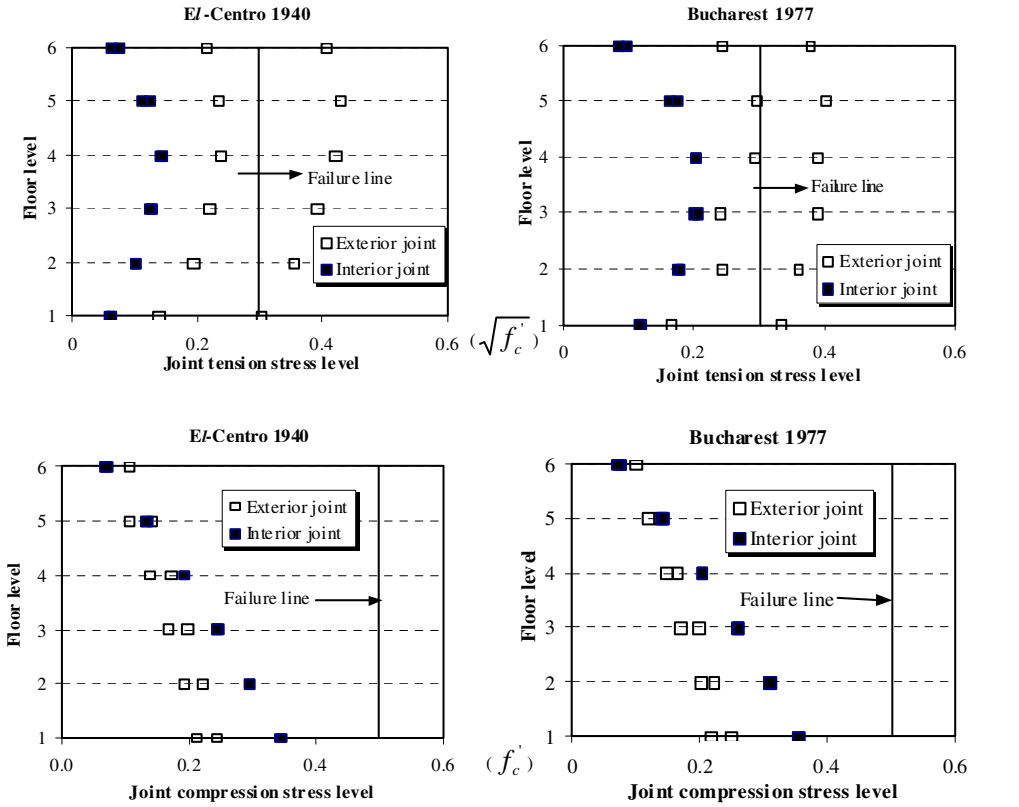
$$f_{dc} = \frac{f_c}{2} + \sqrt{\left(\frac{f_c}{2}\right)^2 + v_{jh}^2} \le 0.5 f_c^{'} \qquad (3)$$

For joints with principal tension stress greater than $0.29\sqrt{f_c^{'}}$ and principal compression stress less than $0.5\, f_c^{'}$, failure may due to joint shear, bond slip, and flexural ductility (Comite Euro-International 1997).

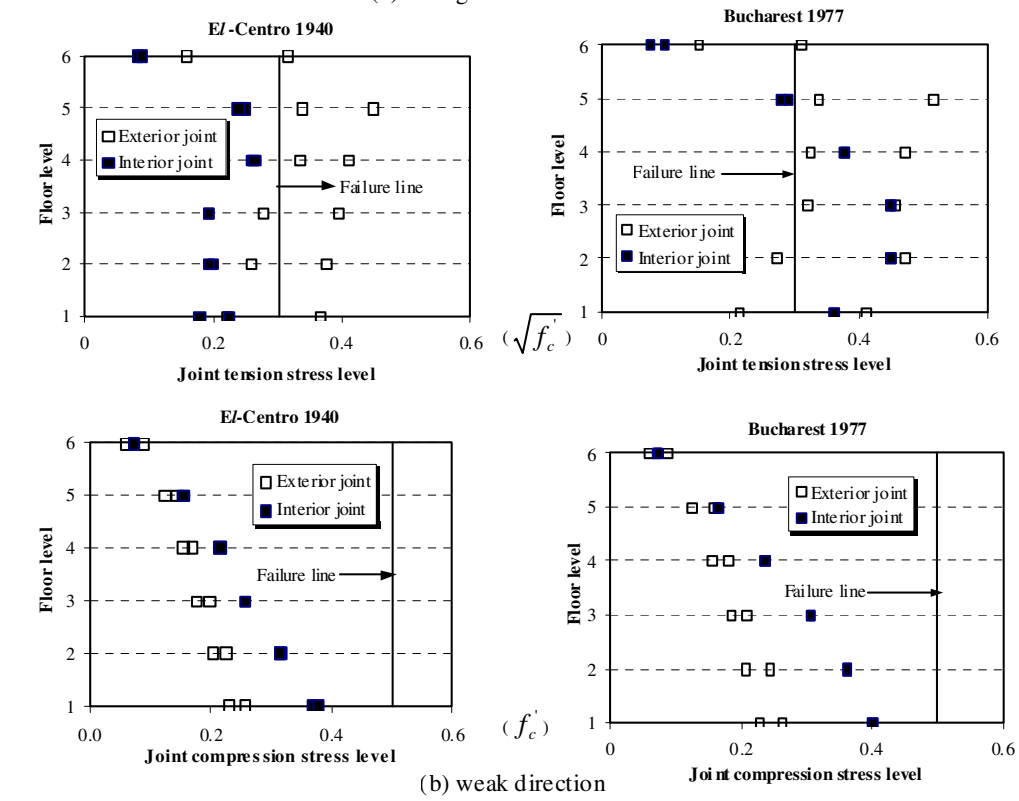
The maximum principal tension and compression stresses of interior and exterior joints in two direction frames are shown in Figure 6. According to the results, in the strong direction frame exterior joints are critical, and joint failure was predicted to occur. In the weak direction frame both interior and exterior joints are critical, the principal tension stresses of which are far beyond the failure line.

Base shear

The push-over analysis shows that, for the strong direction, the structure collapsed when the base shear attained 10.9% of the total weight of the frame, where the global ductility of the frame was 2.14. While for the weak direction, the structure collapsed when the base shear attained 5.44% of the total weight of the frame, where the global ductility of the frame was 3.91.

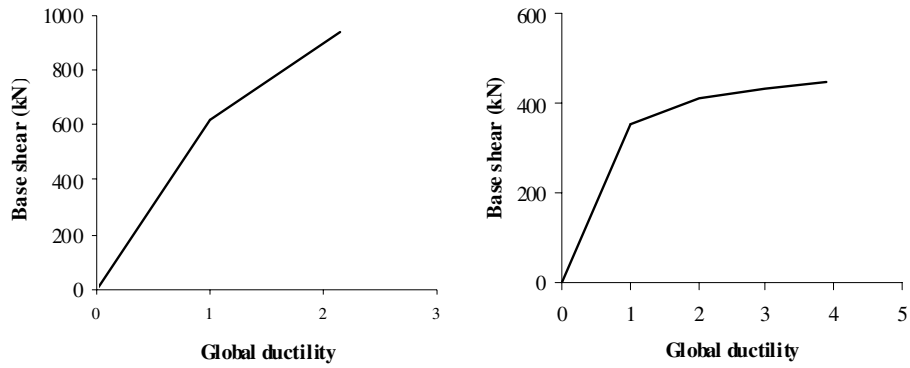


(a) strong direction



(b) weak direction

Figure 6 Joint stresses



(a) Strong direction (b) Weak direction

Figure 7 Base shear

DISCUSSION

Type text immediately below subheadings For the strong direction frame the maximum joint shear stresses in interior joints obtained from time history analysis were 2.25 MPa (0.41 $\sqrt{f_c^{'}}$ or $0.08 f_c^{'}$) and 3.01 MPa (0.55 $\sqrt{f_c^{'}}$ or $0.10 f_c^{'}$) for the E*l*-Centro and Bucharest earthquake, respectively. For the weak direction frame the maximum joint shear stresses in interior joints obtained from time history analysis were 3.72 MPa (0.68 $\sqrt{f_c^{'}}$ or $0.12 f_c^{'}$) and 5.17 MPa (0.94 $\sqrt{f_c^{'}}$ or $0.17 f_c^{'}$) for the E*l*-Centro and the Bucharest earthquake, respectively. So compared to the experimental data, during a maximum credible earthquake, which may occur in Singapore, for interior beam-wide column joints located in the lower part of the weak direction wide-column frame, beam bar bond slip and joint shear failure may occur. The joint may undergo severe strength degradation where maximum inter-storey drift ratio is attained. While for the joints in strong direction, there will not be any damage.

CONCLUSIONS

Based on the experimental results on two prototypical beam-column joint subassemblies, the joints experienced significant strength and stiffness degradation due to cyclic loading. Such behavior could have deleterious effects on the drift of moment-resisting frames designed according to BS 8110.

Nonlinear dynamic analysis has shown that a low intensity earthquake might cause the frames to generate a maximum inter-storey drift ratio of about 2%. The critical failure mechanism of the frame is a hybrid mechanism including beam and column side-sway mechanism and beam-column joint failure. A more critical aspect in shear was found in the beam-column joints. Relatively large joint shear input during the low to moderate earthquakes indicate that the joints of the structure could suffer severe diagonal tension cracking and the strength of the structure is likely to be governed by the joint shear failure mode.

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