التصرف الزلزالى لوصلات الكمرات والأعمدة فى إطارات المبانى الخرسانية عالية المقاومة

ابراهيم جلال شعبان[١] **ماهر عبد الرحمن آدم**[٢]

[١] أستاذ مساعد، قسم الهندسة المدنية–جامعة الزقازيق "فرع بنها"
[٢] مدرس، قسم الهندسة المدنية – جامعة الزقازيق "فرع بنها"

ملخص: يقدم هذا البحث دراسة نظرية للإطارات الخرسانية المسلحة عالية المقاومة (خ.ع.م) فى المبانى وذلك باستخدام برنامج تحليل ديناميكى لاخطى تم استخدامه من قبل فى منشآت خرسانية تقليدية ثم تم تعديله لتطبيقه على (خ.ع.م). على غير المعهود فى مثل هذه الدراسات فإن دراستنا هذه تعتبر الوصلات جزء من الإطار محل الدراسة والمكون من عشرة أدوار من (خ.ع.م). وجد من دراسة السلوك الزلزالى للإطار وكذلك وصلة الكمرة والعمود بالدور الأول أن استخدام (خ.ع.م) يؤدى لزيادة قوة تحمل الأعمدة وزيادة جساءة الوصلات، وتقليل تأثير حديد الكانات فى الأعمدة والكمرات وكذلك تقليل الفترة الطبيعية الأساسية للإطار. بالإضافة إلى ذلك فأن نوع الركائز للمنشأ له تأثير كبير على سلوكه الزلزالى.

الكلمات الدالة: خرسانة عالية المقاومة، وصلة الكمرة بالعمود، التصرف الغير مرن، حمل الزلازل، المنشآت المقاومة للزلازل، الإطارات الخرسانية المسلحة ذات الممطولية.

SEISMIC BEHAVIOR OF BEAM-COLUMN CONNECTIONS IN HIGH STRENGTH CONCRETE BUILDING FRAMES

Ibrahim G. SHAABAN[1] **Maher A. ADAM**[2]

[1] Associate Professor, Civil Eng. Dept., Zagazig University, Banha Branch.
[2] Assistant Professor, Civil Eng. Dept., Zagazig University, Banha Branch.

Abstract: This paper presents, an analytical study carried out on High Strength Concrete (HSC) building frames using a nonlinear dynamic analysis computer program (IDARC-M). The program was originally developed for the analysis of normal R.C. frames and it was modified to predict the response of HSC frame structures. Unlike most of conventional investigations into HSC beam-column connections, this work considers such connections as integral part of the studied ten-story HSC frame. The inelastic behavior of an interior beam-column connection in the first floor was studied. It was found that the use of HSC improves column capacity, enlarges rigidity of beam-column joints, reduces the effect of lateral reinforcement distribution in beams and columns, and decreases the fundamental natural period of the frame. The type of column support at foundation level has a great effect on the drift of the studied building frame.

Keywords: High Strength Concrete, Beam-Column Connection, Inelastic response, Seismic Loading, Earthquake Resistant structures, Ductile RC frames.

INTRODUCTION:

Recently, the advancement of material technology and production has led to higher grades of concrete strengths. The use of High Strength Concrete (HSC) elements for concrete structures has proven very popular, with strengths of concrete up to about 1500 kg/cm^2 used around the world [1]. The main advantages of HSC include higher strength and higher stiffness, improved durability, cost efficiency, reduced creep and drying shrinkage, better impact resistance and better resistance to abrasion [2]. However, due to the variations in fracture modes, microstructure and the differences brought about by various additives, the empirical design rules originally intended for normal strength concrete (strengths less than 500 kg/cm^2), need to be re-evaluated [3]. Earlier experimental investigations have shown that HSC columns may behave in a ductile manner when subjected to moderate axial compression and reversed cyclic bending [3-7]. For high axial compression, ductility is achieved with the use of a greater amount of confinement steel. In this case, high-yield-strength steel (HYSS) may be used to decrease the lateral steel content [2].

Concrete structures are inherently heavy and hence have the potential to induce substantial inertial forces. HSC members, however, have the distinct advantage of reducing these inertial loads following the reduction in member sizes and reduction of the drift due to lateral loads, especially seismic ones. After more than thirty years of the first seismic loading test carried out by Hanson and Conner [8] on a beam-column joint of a reinforced concrete moment resisting frame, and after a great deal of experimental work, it is interesting to note that recommendations for the design of beam-column joints in different design standards still have many discrepancies. These discrepancies can be partly attributed to the difficulties in identifying the main parameters that affect the behavior of joints. Moreover, recent earthquakes have strengthened the need for proper reinforcement of beam-column joints to avoid structural collapse in large events and to avoid irreparable damage in moderate events. Reinforced concrete beam-column joints should not be studied in isolation, but must be considered as an integral part of the building frame structure [1 and 9]. It is preferable, however, that beam-column joints remain strong so that energy will be dissipated in the adjacent members rather than in such joints.

This paper aims to study the seismic behavior of HSC beam-column connections in building frames. Seismic analysis was carried out for a ten-story HSC frame using the nonlinear dynamic analysis computer program “IDARC-M” [10]. Unlike most of conventional researches into HSC beam-column connections, this investigation considers such connections as integral part of the studied ten-story HSC frame. The maximum structure responses and the inelastic behavior of an interior beam-column connection in the first floor were investigated. The studied parameters were the concrete strength, distribution of lateral reinforcements, slight segregation of concrete in columns during construction, and the end conditions of the supporting columns.

COMPUTER PROGRAM “IDARC-M”:

The enhanced computer program IDARC3 [11] was originally developed for the nonlinear dynamic analysis of R.C building frames and shear walls. The program idealizes the building as a set of frames parallel to the loading direction and inter-connected by transverse elements to permit flexural-torsional coupling. The structure is modeled using end node degrees of freedom (DOF) to simplify the problem within acceptable accuracy. All elements are assumed to move with the same lateral displacement within the same frame to reduce the total DOF of the structure. The modified version of the program “IDARC-M” was developed by Shaaban and Torkey [10] in order to be capable for the analysis of HSC structures. The moment-curvature envelopes, material modeling and hysteric response modeling are detailed in Ref. [10]. The program was verified by predicting the response of different structural elements to a reasonable accuracy [10, 11]. In the current investigation, the program was used to study the seismic behavior of HSC beam-column connections in building frames.

DETAILS OF THE STUDIED HSC BUILDING FRAME:

The configuration and dimensions of the building under study are shown in Fig. (1). A gravity load of 1.0 t/m^2 was used for all floors. A typical lateral spacing between frames of 4.5 m was chosen. The yield strength of longitudinal reinforcement bars was considered to be 3600 kg/cm^2 while the lateral reinforcement was normal mild steel with yield strength of 2400 kg/cm^2. A total of ten combination cases between the different studied parameters have been considered in the analysis as shown in Table (1). Studied Cases (1 to 4) comprise different values of concrete cylinder strength. The lateral reinforcement for beams and columns in the general case was $5\varphi8$ mm/m. This reinforcement was increased to $10\varphi8$ mm/m in certain zones as shown in Fig. (1) to study the effect of the lateral reinforcement distribution in the study Cases (5 to 7) in Table (1). The slight segregation of concrete, which usually occurs during construction, was simulated in the studied Cases (8 and 9) by reducing f'_c at zones 1 and 2 (see Table 1 and Fig. 1). The frame supporting columns were assumed to be fixed at the base in all studied cases except for Case (10) where the columns were hinged. The El-Centro earthquake record (1940) with Peak Ground Acceleration of 0.30g (g is the gravity acceleration) was selected to represent a major earthquake input motion during the nonlinear dynamic analysis.

ANALYTICAL RESULTS AND DISSCUSION:

Effect of Concrete strength

Table (2) summarizes the maximum responses of the structure for different values of concrete strength. The results show that the fundamental natural period of the structure decreases as the concrete strength increases. This may be due to the increase in the structure stiffness with no changes in the mass, since the element dimensions are not altered. In addition, the top floor's maximum lateral displacement was reduced with the increase of the concrete strength. For example, Case (4, f'_c = 1200 kg/cm^2) achieved a reduction of displacement of 17% less than that of Case (1, f'_c = 500 kg/cm^2). In Cases (1 to 4), the maximum displacement are less than 0.5% of the total building height (0.005 H) (see Table 2). Concerning the first floor response, little reduction in the story displacement was remarked but smaller ratios of the story height were recorded in all the four cases (much less than 0.005 h). De Stefano et al. [12] suggested that the onset of severe structural damage occurs approximately at an overall (roof) displacement of 0.01H. Hence, the damage level in the studied frame cases is expected to be moderate. This can be seen in the present analysis from the resulted overall structural damage index given in Table (2). In the definition, the index values between 0.0 and 0.4 indicate light damages, the values higher than 0.4 up to 1.0 indicate heavy damages while the index value more than 1.0 means total collapse [11]. In the analyzed four cases, the index value lies between 0.401 and 0.582 (see Table 2) which may be considered as moderate damages.

Although the ultimate moment capacity of beams has not significantly increased by using higher strength concrete as shown in Table (2), the higher stiffness resulted in a smaller deflection and this, in turn, allow to design of longer spans. Despite HSC is considered as a brittle material, flexural member cast with HSC exhibit greater rotational ductility since it has a lower depth of the neutral axis [13]. The moment-curvature hystereses for the first floor beam are shown in Fig. (2) for different values of concrete strength (Cases 1 to 4). The responses are depicted at the left-hand side of the middle joint in the first floor (connection 1, Fig. 1). The higher stiffness in the elastic zone and the higher curvature in the post yield zone are observed for higher concrete strength (Fig. 2). The values of beam curvature ductility summarized in Table (2) show the increase in ductility with higher concrete strength. The value of curvature ductility was obtained as the ratio of curvature at 80% of the ultimate moment in the post peak region to the yield curvature [1, 14]. In seismic design, a beam mechanism is preferred to a column mechanism and the formation of beam hinges is assured by having a strong column-weak beam system. Therefore, higher rotational ductility of HSC is considered as an advantage in seismic resistant frame design.

The story shear-displacement hystereses are shown in Fig. (3) for the first floor. It can be seen from the figure that there is no brittle shear failure occurred in all cases despite that the story shear increases with the increase of concrete strength. The moment-curvature hystereses for central column at the first floor level are depicted in Fig. (4). The column stiffness increases as the concrete strength increases and for the four studied values of concrete strength, the column does not exceed the elastic limit. This behavior forces the plastic hinges to be formed in the beam, which is preferable. Priestley [15] suggested that a brittle shear failure may ensue, even if there has been some ductile response, if there is inadequate lateral reinforcement at the critical section due to the degradation in the concrete shear strength as the curvature ductility of the section increases. In such case, the axial compression can play a significant role in the closing of both flexural and shear cracks. Park [16] reported that HSC columns carrying high axial loads can have a marked reduction in cross section size and the amount of longitudinal steel reinforcement can be substantially reduced.

In the present work, the column cross section and its reinforcement were not altered with the increase of the concrete strength. As a result, a reduction of the applied axial force to the strength ratio was achieved and, in turn, higher moment and ductility capacity of the column could be obtained. Hence, it can be considered that the strong column-weak beam theory is achieved in the analyzed frame.

Effect of Lateral Reinforcement

The studied HSC building frame was analyzed using three different arrangements for the lateral reinforcements for beams and columns in the connection zones as shown in Fig. (1) and given in Table (1). In Case (5), the column stirrups only were increased to $10\varphi8$ mm/m at zones 1 & 2. Similarly, the beam stirrups only were increased to $10\varphi8$ mm/m at zones 3 & 4 in Case (6). Both beam and column stirrups were increased to $10\varphi8$ mm/m at zones 1, 2 ,3 and 4 in Case (7). The concrete strength in all cases was set to be 1200 kg/cm^2. The responses of the studied frame cases were identical in the three cases as given in Table (2). These responses were similar to the response of Case (4) using $5\varphi8$ mm/m stirrups in the beam and the column. Hence, there is no effect due to the increase of lateral reinforcement.

Kovacic [17] carried out experimental and analytical studies on heavily loaded HSC compression members and he found that ductility demands can be satisfied by providing additional ties. Mendis and Kovacic [18] proposed a new formula to calculate the spacing of lateral reinforcement by modifying the present requirements in AS3600 [19]. Priestley [15] concluded that with lightly reinforced beams joint cracking may develop if the principal tension stress in the joint is more than $0.29f'_c$ and the beam-column joints with high shear stress levels tend to fail in shear regardless of the amount of transverse reinforcement. The reason for failure is the principal compression stress, and it is thus more logical to limit this directly, rather than through the shear stress, which does not recognize the influence of axial compression. Priestley [15] suggested a limit of 0.5f'_c for the principal compression stress. Then, HSC can significantly reduce the chance of joint cracking or the failure due to the principal compression stress.

Design recommendation in different standards can be broadly classified in two main groups. One group such as Eurocode8 [20] and NZ3101 [21] base their recommendations on the behavioral parallel angle steel truss and diagonal concrete strut transfer mechanisms proposed by Park and Paulay [22]. The second group including ACI318-1995 [23] and AIJ-1994 [24] base their design recommendations on a confinement criterion and tacitly recognizes that transverse reinforcement does not enhance the joint shear strength.

The results of the present analysis suggest that the use of HSC and the low level of the axial force to strength ratio increased the joint strength and reduced the compression stresses. This prevents the joint cracking and shear failure forcing the failure in this frame to occur due to beam flexural ductility. Therefore, no more enhancements could be obtained by the increased lateral reinforcement, which agrees with the second group of design standards [23 and 24].

Effect of Slight Segregation of Concrete

Although the construction of HSC involves strict quality control programs to ensure perfect compacting and placing, the effect of slight segregation of concrete, which may occur during the construction of columns, was studied for interest. Two cases for analysis were performed with reduced concrete strength in the bottom and top 0.60 m of each column (zones 1 and 2 in Fig. 1). In Case (8), the reduced strength was assumed to be as low as 40% of the original strength, while in Case (9), it was assumed to be 60% as given in Table (1). The resulted fundamental period was increased in both cases as given in Table (2). This indicates a reduction of the structure stiffness that led to an increase in the maximum lateral displacement of top floor of approximately 8% in Case (8) and 5% in Case (9) compared with Case (4). The maximum moments transmitted to the columns were increased leading to some cracks in the upper floor columns, which carry lighter axial forces as reported in the computer output and not shown here. As the original strength was considered as 1200 kg/cm^2, and due to the large capacity of the cross section, no yielding occurred in the column as shown in Fig. (5) and the plastic hinges were formed in the beams. Moreover, the overall damage index was increased to be 0.48 in Case (8) and 0.411 in Case (9) instead of 0.401 in Case (4) as given in Table (2). This slight effect of the segregation may be owed to the very high value of concrete strength, so that 40% of this value is 480 kg/cm^2, which is very close to the upper limit of normal strength concrete. Therefore, it can be stated that the use of HSC reduces the risks of unfavorable effects of segregation.

Effect of Supporting Conditions

To check the effect of supporting conditions, the HSC frame was analyzed assuming that all columns are hinged at their bases with concrete strength of 1200 kg/cm^2 (Case 10, Table 1). The results are reported in Table (2) showing a significant increase in the fundamental period to be 0.967 (sec) which is very close to that resulted in Case (2) with concrete strength of 700 kg/cm^2. In addition, the first story displacement was drastically increased from 9.47mm in Case (4) with fixed base to be 34.73 mm which represents 1.16% of the story height (0.0116h). Although the top floor displacement was also increased to be 127.90 mm, it is still less than 0.5% of the total height (0.005 H). The overall damage index became 0.645, which indicates heavy damage level. The maximum moments transmitted to the column top at first floor level was drastically increased, because of the zero moment at the hinged base (see Table 2), but the column did not exceed the elastic limit as shown in Fig. (6-a). The beam moment-curvature hystereses are given in Fig. (6-b) where the very large curvature was observed while the curvature ductility was slightly increased (see Table 2). The shear-displacement hystereses for the first story are depicted in Fig. (7). It is interesting to note that a softer story response was observed compared with that of Case (4, 1200 kg/cm^2) shown in Fig. (3-c).

CONCLUSIONS:

This paper described the nonlinear dynamic analysis of a ten-story high strength concrete frame performed using the modified computer program IDARC-M [10]. The following conclusions can be drawn:

1. The computer program is capable of predicting the seismic response of HSC building frames including the inelastic behavior of beam-column connections in such frames.
2. As the concrete strength increases, the stiffness of the structure increases and its fundamental period decrease. As a result, the maximum lateral displacement of the frame structure is reasonably decreased.
3. Flexural members cast with HSC exhibit greater rotational ductility resulting in higher curvature ductility for beams. In addition, HSC improves column capacity and enlarge rigidity of beam-column joints. As a result, no more enhancements could be achieved by increasing the lateral reinforcement in the present study case.
4. Segregation of concrete reduces the column stiffness and increases the lateral displacement but the use of HSC reduces its harmful effects.
5. The hinged base of the supporting column drastically increases the first floor drift and increases the damage level.

REFERENCES:

[1] **Mendis, P.A. and Panagopoulos, C.,** “Applications of High Strength Concrete in Seismic Regions”, 12th World Conference on Earthquake Engineering (12WCEE), Auckland, New Zealand, Jan.-Feb. 2000, Paper No. 809.

[2] **Paultre, P., Légeron, F., Savard, C.,** “Use of High-Yield Strength Materials in Seismic Zones: A Strategic Approach”, 12th World Conference on Earthquake Engineering (12WCEE), Auckland, New Zealand, Jan.-Feb. 2000, Paper No. 816.

[3] **Azizinamani, A. et al**, “Seismic Bhavior of Square High Strength Concrete Columns”, ACI Structural Journal, V91 N3, 1994, pp 336-345.

[4] **Tanaka, H. et al,** “High Strength Concrete Columns with Longitudinal Reinforcement of Mixed Steel grades”, ACI International Conference, High Performance Concrete, Singapore, 1994, pp 391-411.

[5] **Cusson, D., Paultre, P.,** “Stress-Strain Model for Confined High Strength Concrete”, ASCE Journal of Structural Engineering, V121 N3, 1995, 468-477.

[6] **Kimura, H. et al.** “Seismic Behavior of Reinforced Concrete Using Ultra-High-Strength Concrete Under High axial Load”, Fourth International symposium on Utilization of High Strength/High Performance Concrete, Paris, France, 1996, pp 893-904.

[7] **Beolchini, D. et al,** “Experimental and Analytical Studies on the Hysteretic Behavior of High Strength RC Columns”, 11th European Conference on Earthquake Engineering, Balkema, Rotterdam, 1998, CD Rom.

[8] **Hanson, N.W. and Conner, H.W.,** “Seismic Resistance of Reinforced Concrete Beam-Column Joints”, Journal of the Structural Division ASCE, V93 ST5 1967, pp 533-560.

[9] **Kashiwazaki, T., Noguchi, H.,** “Three-Dimensional Nonlinear Finite Element Analysis on The Shear Strength of RC Interior Beam-Column Joints with Ultra High Strength Materials”, 11th World Conference on Earthquake Engineering (11WCEE), Acapulco, Mexico, 1996, Paper No. 476.

[10] **Shaaban, I.G. and Torkey, A.A.,** “Inelastic Damage Analysis of High Strength Concrete Structures”, Journal of Engineering and Applied Science, Cairo University, “In Press”.

[11] **Kunnath, S.K, Reinhorn, A.M and Lobo, R.F.,** “IDARC Version 3.0: A Program for the Inelastic Damage Analysis of Reinforced Concrete Structures”, Technical Report NCEER-92-0022, National Center for Earthquake Engineering Research. State University of New York at Buffalo, N.Y 14261, 1992.

[12] **De Stefano, M., Faella, G. and Realfonzo, R.,** “Seismic Response of 3D RC Frames: Effect on Plan Irregularity”, European Seismic Design Practice, ed. Elnashai, A., 1995, pp 219-226.

[13] **Pendyala, R., Mendis, P.A. and Patnaikuni, I.,** “Full Range Behavior of High-Strength Concrete Flexural Members, Part 1”, ACI Structural Journal, V93, N1, 1996, pp 30-35.

[14] **Paulay, T. and Priestley, M.,** “Seismic design of reinforced concrete and masonry buildings”, John Wiley & Sons, New York, 1992.

[15] **Priestley, M.,** “Displacement-Based Seismic Assessment of Existing Reinforced Concrete Buildings”, Proceedings of Pacific Conference on Earthquake Engineering, Australia, 1995, pp 225-244.

[16] **Park, R.,** “Some Current and Future Aspects of Design and Construction of Structural Concrete for Earthquake Resistance”, Australian Structural Engineering Conference, Auckland, 1998, pp 1-16.

[17] **Kovacic, D.,** “Flexural Ductility of High Strength Concrete Columns”, MSc Thesis, University of Melbourne, Australia, 1995.

[18] **Mendis, P. and Kovacic, D.A.,** “Lateral Reinforcement Spacing For High-Strength Concrete Columns in Ordinary Moment Resisting Frames”, Accepted for publication in Journal of Structural Engineering, Institution of Engineers, Australia, 1999.

[19] **AS3600**, “Concrete Structures”, Standards Association of Australia, Sydney, Australia, 1994.

[20] **Eurocode8,** “Design Provisions for Earthquake Resistance of Structures”, ENV 1998 1-3: 1994

[21] **NZ3101,** “Design of Concrete Structures”, Standards Association of New Zealand, Wellington. NZS 3101:1995.

[22] **Park, R. and Paulay,T.,** “Reinforced Concrete Structures”, John Wiley & Sons Inc., New York, 1975.

[23] **ACI318-95,** “Building Code Requirements for Structural Concrete and Commentary”, American Concrete Institute, Detroit, Michigan, USA, 1995.

[24] **AIJ-1994,** “Structural Design Guidelines for Reinforced Concrete Buildings”, Architectural Institute of Japan, Tokyo, Japan, 1994.

Table (1): Description of the Studied Parameters and Case Studies

Case Number	Studied Parameters			
	Cylinder strength, f'_c, kg/cm^2	Slight segregation f^*_c, kg/cm^2	Lateral reinforcement	End condition
1	500	--------	5 φ 8/m	Fixed
2	700	--------	5 φ 8/m	Fixed
3	1000	--------	5 φ 8/m	Fixed
4	1200	--------	5 φ 8/m	Fixed
5	1200	--------	10 φ 8/m (zones 1 & 2)	Fixed
6	1200	--------	10 φ 8/m (zones 3 & 4)	Fixed
7	1200	--------	10 φ 8/m (zones 1,2,3 &4)	Fixed
8	1200	0.4 x 1200 = 480	5 φ 8/m	Fixed
9	1200	0.6 x 1200 = 720	5 φ 8/m	Fixed
10	1200	--------	5 φ 8/m	Hinged

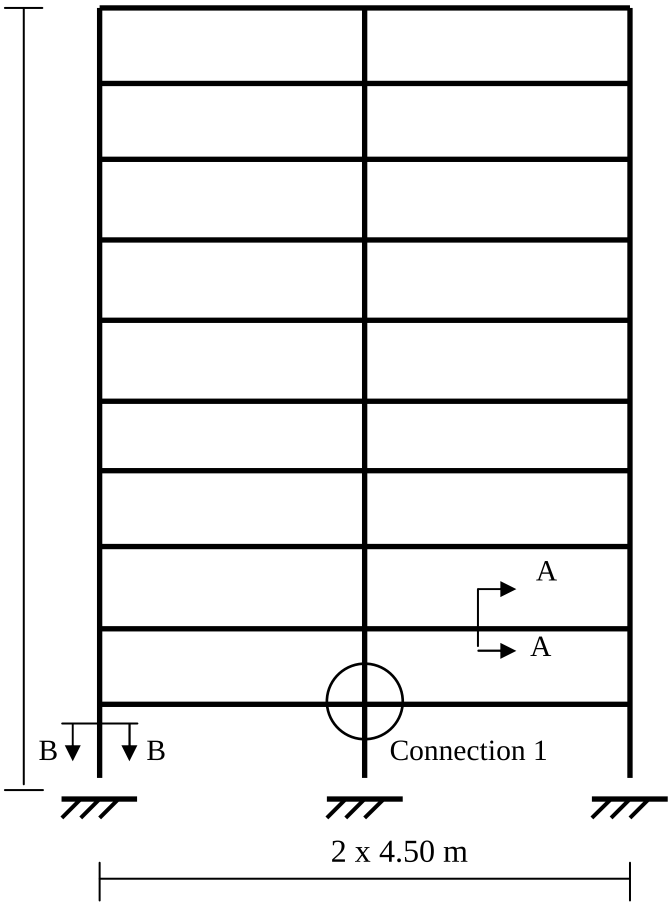
f^*_c is the reduced value of f'_c applied at zones 1 and 2 (see Figure 1)

Table (2): Maximum Responses for Different Case Studies

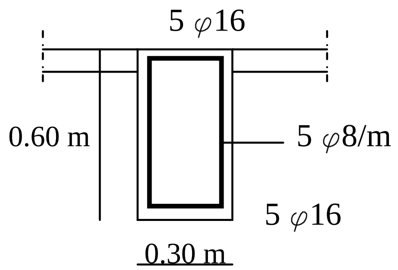
Case Number	T_o (sec)	Δ_t (mm)	Δ_f (mm)	Damage index	M_b (t.m)	μ_b	M_c (t.m)
1	1.051	122.40	10.33	0.582	19.74	2.90	12.58
2	0.971	114.68	11.44	0.437	19.65	7.90	14.57
3	0.880	112.57	10.41	0.420	20.09	8.47	16.20
4	0.836	102.68	9.47	0.401	20.59	10.78	16.59
5	0.836	102.68	9.47	0.401	20.59	10.78	16.59
6	0.836	102.68	9.47	0.401	20.59	10.78	16.59
7	0.836	102.68	9.47	0.401	20.59	10.78	16.59
8	0.896	110.58	9.77	0.481	20.59	10.78	18.57
9	0.867	107.02	9.28	0.411	20.21	10.50	18.34
10	0.967	127.95	34.72	0.645	20.81	11.68	36.83

Note:
T_o = Fundamental period (sec)
Δ_t = Top floor Max. displacement (mm)
Δ_f = First floor Max. displacement (mm)
M_b = Ultimate moment in beam (t.m)
μ_b = Curvature ductility for beam
= 80% of curvature at ultimate moment/ curvature at yield

M_c = Maximum moment in column (t.m)

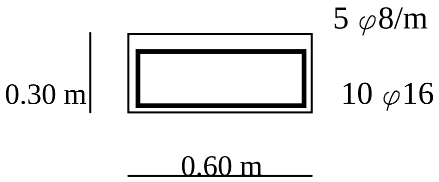


Elevation



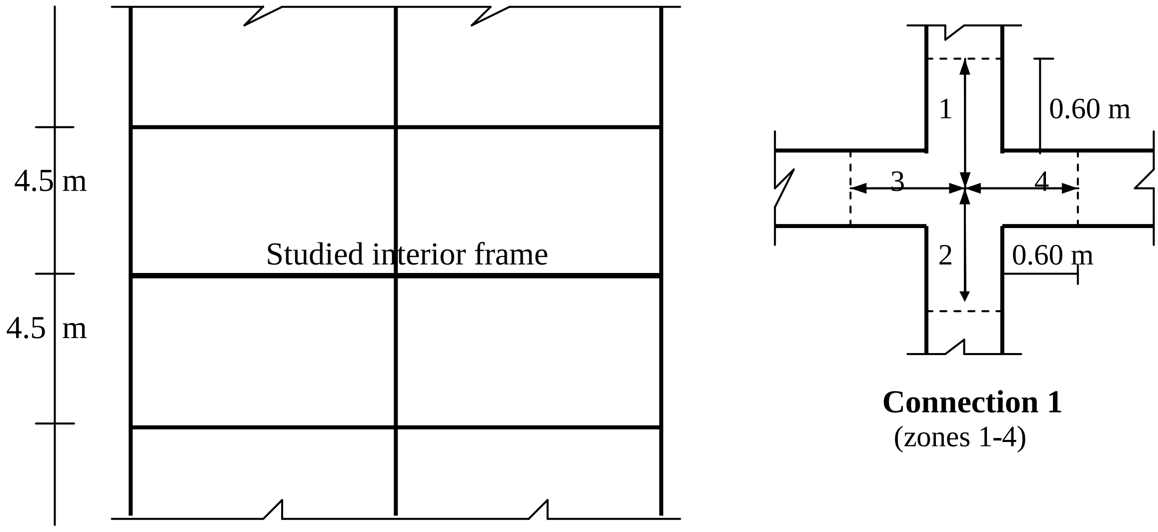
(Beam Rft.)

Sec. A-A



(Column Rft.)

Sec. B-B



Plan

Connection 1
(zones 1-4)

Fig. (1): Dimensions and Configurations of the Analyzed Ten-story Building Frame