EFFECT OF USING DIFFERENT CONCRETE STRENGTHS FOR COLUMNS AND BEAMS ON THE BEHAVIOR OF BUILDING FRAMES

M.A. ADAM*  M.A.K. EL-MOHR*

ABSTRACT

In high-rise buildings and heavy loaded structures where RC columns are subjected to heavy loads, the use of High Strength Concrete (HSC) in columns construction is essential for the purpose of reducing column size and increasing column capacity. However, from the economical standpoint, combination of high and normal strength concrete (NSC) in building construction is becoming common practice, where HSC is used for columns and NSC is used for the surrounding beams/slabs floor system. This creates a situation where concrete strength of the column portion at the beam/slab floor level is lower than concrete strength used for rest of the column. Previous studies indicated that such variation in concrete strength affects the load carrying capacity of the RC columns. This paper presents a theoretical study on the effect of using concrete with different strengths for columns and floor beams on the structural behavior of integrated RC building frames under static, lateral pushover and earthquake loading cases. Four-story frame was analyzed employing a ready package program for the inelastic structural analysis of buildings (IDARC-5). The obtained results indicated that under static loading, variation in the concrete strength of the transition zone has a negligible effect on the behavior of the studied frame. However, under lateral pushover and earthquake loading the behavior of the studied frame is influenced by the ratio of column concrete strength to the transition zone concrete strength. For a ratio of 1.4 or less, no real influence was noted. On exceeding this ratio, the frame response was adversely influenced. This agrees with the recommendations of the ACI 318 Building Code.

KEY WORDS

High Strength Concrete, Column-Beam Transition Zone, Seismic Loads, Pushover Lateral Loads, Non-linear Structural Analysis, Frame Stiffness.

* Assistant professor, Dpt. of Civil Eng., Zagazig University, Banha Branch, Cairo, Egypt.
INTRODUCTION

Due to the application of advanced material technology, concrete with compressive strength in the range of 500 to 1500 kg/cm$^2$ is currently produced and used in many countries such as Norway, United States, Japan, Hong Kong [1] and in major projects in Egypt. This type of concrete can now be produced by employing micro-silica and superplasticizers as well as applying good quality control procedure [2]. The use of high strength concrete (HSC) in building construction is becoming popular due to many advantages such as increased strength and stiffness, reduced size of concrete sections, improved resistance to creep and drying shrinkage and enhanced material durability [3]. This was confirmed in previous studies [4-6] where improvements in the behavior of reinforced concrete columns and building frames under seismic loads as a result of using HSC were noted. In these studies, HSC was used in both columns and floor slabs/beams.

In high-rise buildings and heavy loaded structures such as stores, warehouses and garages where the columns are subjected to heavy loads, the use of HSC in RC columns construction is becoming more essential for the purpose of having slender columns, which is recommended from the architectural point of view. While, from the economical standpoint, a combination of high and normal strength concrete (NSC) is becoming common practice in building construction [7]. In the latter case, HSC is used for columns construction and NSC is used for the construction of the surrounding beams/slabs floor system. This creates a situation where concrete strength of the column portion at the beam/slab floor level is lower than concrete strength used for rest of the column and thereby the load carrying capacity of the column may be reduced [8]. The reduction in the column capacity in this case was found to be a function of the ratio of the column concrete strength to the floor concrete strength and the number of restrained edges around the column. Based on the results presented in Ref. [8], ACI 318 Building Code [9] introduced recommendations that considered the aforementioned variation in the concrete strength while estimating the load carrying capacity of columns. Accordingly, no special precautions need to be taken when the column concrete strength is less than 1.4 times that of the floor system regardless of restraint provided. On exceeding this ratio, one of the following guidelines should be used:

1. Concrete of strength specified for the column shall be placed in the floor slab at the column location and to be extended 60 cm into the concrete slab from column face.

2. Load carrying capacity of the column through beam/slab system shall be based on the lower value of concrete strength with vertical dowels and spiral as required.

3. For column laterally supported on four sides by beams or slabs of equal depth, load carrying capacity of the column at beams/slab system shall be estimated using an assumed concrete strength of 75 percent of the column strength and 35 percent of the floor concrete strength.

Siao[7] suggested that reduction in column capacity due to using lower concrete strength in the floor system can be avoided by the virtue of providing confinement to the considered column by the surrounding beam/slab floor system. Such confinement was found to be
dependent on the amount of steel reinforcement continuous through the column and within the adjacent slab/beams.

Although the above findings are based on experimental results, floor/beam–column joint in these studies was considered as an individual part and not as an integral part of the frame structure. This paper aims to study the effect of using different concrete strengths for column and beams/slabs floor system on the structural behavior of integrated RC building frames under incremental static loads, lateral monotonic pushover loading and seismic induced vibrations. ACI 318 Building Code [9] recommendations were used as guidelines in this study.

**NUMERICIAL MODEL AND STUDIES PARAMETERS**

In this research, four-story RC building frame was used as a simple example for studying the characteristic structural behavior of RC frames under different types of loads. The frame as presented in Fig.1-a consists of two bays and it has been analyzed employing a ready package program for the inelastic structural analysis of buildings (IDARC-5) [10]. The frame is divided into three regions as shown in Fig.1-b. First region represents the transition zone comprising the column portion at the beam/slab level and 60 cm of the beam at both sides of the column as per the ACI 318 Building Code [9] recommendations. The second region is the column part above and below the concrete floor. While, remaining part of the beam represents the third region. Columns and beams cross-section and reinforcements are shown in Fig. 1-c. High-grade steel of 3600 kg/cm² yield stress was used for longitudinal bars while normal mild steel of 2400 kg/cm² yield stress was used for stirrups.

Eight cases of study presented in Table 1 were considered in this research. Uniform concrete strength of 300 kg/cm² and 900 kg/cm² were used in cases 1 and 2 respectively for all frame elements. In remaining cases, columns are assumed to have HSC of 900 kg/cm² and NSC of 300 kg/cm² was considered in the floor beams. For the purpose of studying the influence of concrete strength of the transition zone on the overall structural behavior of the integrated RC building frames, concrete strength of this transition zone has been varied from beam concrete strength of 300 kg/cm² to columns concrete strength of 900 kg/cm².

**Cases of Loading**

Three different types of loading were considered in this study. These cases are as follows:

1. Incremental Static Loading: For the eight cases of study considered in Table 1, floor beams were loaded with uniformly distributed vertical load ranged from zero to failure load on 500 incremental steps. At failure, axial loads and the associated bending moment were calculated for side and central columns. Bending moments were also obtained at sections I and II of the first floor beam and at its mid-span.

2. Monotonic Pushover Loading: The purpose of this type of loading is to predict the response of the frame to lateral load which is recommended in the Egyptian Code of practice [11] as equivalent to earthquake loading cases. The frame is subjected to an
incremental lateral force as a percentage of the structure weight. This force varied with
the height of the frame in the form of modal adaptive distribution, in which the load is
automatically adapted according to the current stiffness of the frame elements. More
details are given Ref. [10]. In additions to straining actions considered in the above
case, lateral drift at top and first floor, axial force ratio (N/N_u) for columns and bending
moment ratio (M/M_u) for both beams and columns at failure along with the overall
damage index of the frame structure were also obtained.

3. Earthquake Loading: The 1940 El Centro earthquake accelerogram was selected to
represent a major earthquake loading case. The peak base acceleration is scaled to be
0.2g. Top and first floor displacements along with element internal forces-deformation
hystereses were presented to illustrate the behavior of the frame under dynamic
earthquake loading. The overall damage index of the structure and the story shear were
also presented.

The computation of the overall damage index (DI_{\text{overall}}) is detailed in Ref. [10]. However, a
brief description will be presented in this study for sake of completeness. Element damage
index is calculated first for each element in each story, then for each story in the frame
and finally for the entire frame structure as follows:

\[
\text{DI}_{\text{element}} = \frac{\theta_m - \theta_r}{\theta_u - \theta_r} + \frac{\beta}{M_y \theta_u} E_h
\]  

(1)

Where \( \theta_m \) is the maximum rotation attained during the loading history; \( \theta_u \) is the ultimate
rotation capacity of the section; \( \theta_r \) is the recoverable rotation when unloading; \( M_y \) is the
yield moment; and \( E_h \) is the dissipated energy in the section.

Story and overall damage indices are then computed using weighting factors depending on the
dissipated hysteretic energy at each element and story levels respectively:

\[
\text{DI}_{\text{story}} = \sum (\lambda_i)_{\text{element}} (\text{DI}_i)_{\text{element}} ; \quad (\lambda_i)_{\text{element}} = \frac{E_i}{\sum E_i} \\

\text{DI}_{\text{overall}} = \sum (\lambda_i)_{\text{story}} (\text{DI}_i)_{\text{story}} ; \quad (\lambda_i)_{\text{story}} = \frac{E_i}{\sum E_i}
\]  

(2a)

(2b)

Where \( (\lambda_i) \) is the energy weighting factor; and \( (E_i) \) is the total absorbed energy by the
element or story “i”.
RESULTS ANALYSIS AND DISCUSSIONS

Frame Response to Incremental Static Loading

The results of incremental static loading are presented in Table 2. As a result of increasing the concrete strength of the transition zone from beam concrete strength of 300 kg/cm² to column concrete strength of 900 kg/cm², the axial force on the side and interior columns increased by about 6% and 8% respectively. Bending moments produced on the side column and on section I of the first floor beam have also been increased by about 5% and 8% respectively. It is evident that, variation in the concrete strength of the transition zone has a very slight effect on the behavior of the studied frame and for practical reasons it may be neglected. Consequently floor concrete of lower strength can be used at the column location in the floor slab without having major effect on the structural response of the integrated RC frames.

Frame Response to Monotonic Pushover Loading

Table 3 summarizes the maximum responses of the frame structure for the considered cases of study. As presented in this Table, with increasing the frame concrete strength from 300 kg/cm² in Case 1 to 900 kg/cm² in Case 2, no real change was noted in the M/Mu ratio, while N/Nu ratio decreased to a nearly one-third. This could be explained by the fact that the ultimate axial load (Nu) in Case 2 (f′c1=f′c2=f′c3 = 900 kg/cm²) is approximately three times that of Case 1 (f′c1=f′c2=f′c3 = 300 kg/cm²), and the absolute values of the axial force (N) in both cases are approximately equal. For Cases 3 to 8, N/Nu decreased with increasing the concrete strength of the transition zone. This is probably due to the improvements achieved in the ultimate axial capacity of the column as a result of increasing the concrete strength of the transition zone.

The behavior of the frame under lateral monotonic pushover loading is presented in Figs. 2 to 5. As shown in Fig. 2, the frame exhibited long post-yield range and consequently large amount of displacements for top floor in Case 1 (f′c1=f′c2=f′c3 = 300 kg/cm²) compared with that of Case 2 (f′c1=f′c2=f′c3 = 900 kg/cm²). The percentages of maximum top floor displacement to the total height of the frame (Ut/H) were 2.05% and 0.78%, for Cases 1 and 2, respectively, as shown in Table 3. Moreover, Fig. 2 reveals that the slope of the curve relating the base shear to the top floor displacement of the structure is the lowest in Case 1 and is the steepest in Case 2. This gave higher value of displacement in Case 1 compared with that of Case 2, for same percentage of lateral load. For example, the resulted top floor displacement in Case 2 is approximately 50% of that associated with Case 1 for a lateral load percentage of 15%. This could be attributed to the reduced structure stiffness in Case 1 as a result of using NSC compared with the high stiffness of the frame structure in Case 2. In both Cases, there was no change in the mass of the frame structure and consequently no change in the value of the lateral load.

Increasing the column concrete strength (f′c2) from 300 kg/cm² in Case 1 to 900 kg/cm² in Case 3, while keeping other parameters constant caused a reduction of about 15% in the top floor displacement at a lateral monotonic load of 15% as shown in Fig. 2. This could be attributed to the improvement in the structure stiffness which has led to a reduction in the
resulting lateral displacement. With increasing the concrete strength of the transition zone \((f'c_1)\), the structure stiffness increased and the resulting displacement decreased. For \(f'c_2/f'c_1=1.4\) (Case 6), the structural response of the frame is very close to that of Case 2 and Case 8. No real improvement was noted with increasing the value of \(f'c_1\) beyond the ratio of 1.4 as obtained in Case 7 (which is omitted from Fig. 2 for clarity). This agrees with the previous findings [8,9].

On comparing the behavior of the structure in terms of slopes and displacements for Case 2 \((f'c_3 = 900 \text{ kg/cm}^2)\) and Case 8 \((f'c_3 = 900 \text{ kg/cm}^2)\), no real difference was noted. This means that, away from the 60 cm of the transition zone, the concrete strength of the frame beams \((f'c_3)\) has a negligible effect of the structure behavior under the studied cases of loading. Similar behavior was noted with regard to first story displacement against story shear as presented in Fig. 3 and bending moments against curvature of the central column shown in Fig. 4. The relation between bending moments and curvature of section I of the first floor beam was greatly influenced by the value of the concrete strength of the transition zone as shown in Fig.5.

In all studied cases of monotonic pushover loading the failure occurred in the frame columns as indicated in Fig. 4. On the other hand, the frame beams were still in the elastic stage, since a linear relationship between bending moments and curvature was noted as shown in Fig 5. Such trend was confirmed by the layout of plastic hinge locations presented in Fig. 6. As presented in Ref. [10], damage index in the range of 0.0 to 0.40 indicates moderate damage, which could be repaired. Index values higher than 0.40 and up to 1.00 indicate severe damage, which is beyond repair while index values more than 1.0, means total collapse. Accordingly, the damage of the present frame is from moderate to severe, since the overall damage index values varied from 0.379 to 0.691 as indicated in Table 3. De Stefano et al [12] suggested that the onset of the severe structural damage occurs approximately at an overall displacement of 0.01H. Accordingly, damage of the studied frame is severe in Cases 1 and 3 and moderate in the remaining cases. This agrees with the content of Fig. 6, where number of formed plastic hinges was 26 in Case 1, 15 in Case 2 and 17, 18 in Cases 6 and 8, respectively. Although, number of plastic hinges in Cases 2 and 8 is less than that of Case 1, the damage index in Case 1 is the smallest. This could be attributed to the formation of local widened plastic hinges in the side columns of the first and second floors of the studied frame as revealed from the output files. With these widened hinges more energy has been dissipated and more energy weighting factors are obtained and accordingly greater values of damage indices are produced for these columns. This in turn increased the overall damage index of the frame structure in Cases 2 and 8.

**Frame Response to Earthquake Loading**

Tables 4 and 5 summarize the maximum dynamic response of the frame structure under earthquake loading cases. As shown in Table 4, the fundamental natural period of the structure decreased with increasing the concrete strength of the frame structure. Increasing the concrete strength from 300 kg/cm\(^2\) in Case 1 to 900 kg/cm\(^2\) in Case 2 caused a reduction in the fundamental natural period of 28%. This could be attributed to the improvements in the structure stiffness as a result of increasing the concrete strength without any change in the
structure mass, since the elements sizes are not changed [4]. With increasing the concrete strength of the transition zone from 300 kg/cm² in Case 3 to 900 kg/cm² in Case 8, the fundamental natural period decreased from 0.70 to 0.58 seconds. This reflects the effect of the concrete strength of the transition on the overall stiffness of the frame structure and the related fundamental natural period.

Although the columns had higher concrete strength, the resulted bending moments in the columns exceeded their ultimate flexural capacity while the moment in the beams were still less than the ultimate capacity in all cases as presented in Table 5. The beam stiffens increased with the increase of the transition zone concrete strength (f’c₁) and did not exceed the linear elastic limit as shown in Fig. 7. The relationship between story displacement and story shear for first and top stories under earthquake loading is presented in Figs. 8, 9 and 10 for Cases 1, 2 and 6, respectively. The bending moment -curvature hystereses of the central column are also shown in same Figures. As indicated in these figures and in Table 4, first story maximum displacement was 26.86, 75.27 and 38.72 mm for Cases 1, 2 and 6 respectively. This response was not expected, however, reviewing the output files made clear that in Cases 2 and 6 local sliding has occurred in the first floor due to local failure in the first floor columns which in turn caused such large values of displacements. This is supported by the resulted values of local damage indices of these columns. The indices were 0.229, 1.53 and 0.274 for side column and 0.343, 1.19 and 0.364 for central columns in Cases 1, 2 and 6, respectively. The values of overall damage indices of the frame structure presented in Table 4 (0.236, 0.829 and 0.507 for Cases 1, 2 and 6 respectively) confirmed this response. The rotations of central column with respect to applied bending moment were found reasonable in Cases 1 and 6. However severe rotation has occurred in Case 2 which was indicated by a kink in Fig.9-a. This is probably due to an initiation of plastic hinge in this column at that time and then uniform hystresses were obtained. This is supported by the results of first story displacements.

CONCLUSIONS

A theoretical study on the effect of using concrete with different strengths for columns and floor beams on the structural behavior of integrated RC building frames under static, lateral pushover and earthquake loading cases is presented. Four-story frame was analyzed employing a ready package program for the inelastic structural analysis (IDARC-5). The following conclusions could be stated:

1. Under static loading, variation in the concrete strength of the column-beam transition zone has a negligible effect on the behavior of the studied frame.

2. Using High strength concrete for column increased the overall stiffness of the frame structures and decreased the tope floor displacement under monotonic pushover loading.

3. The ratio of column concrete strength to floor concrete strength (f’c₂/f’c₁) has a major influence on the performance of the RC frame under monotonic pushover loading. For
\( \frac{f'c_2}{f'c_1} > 1.4, \) increasing the concrete strength of the transition zone \((f'c_1)\), increased structure stiffness and the resulting displacement decreased. For \( \frac{f'c_2}{f'c_1} \) equal to or less than 1.4, no real effect was noted with increasing \((f'c_1)\). This agrees with the recommendations of the ACI 318 Building Code [9].

4. Away from the 60 cm of the transition at both sides of column face, beams concrete strength \((f'c_3)\) has a negligible effect on the overall structural behavior of the RC frame under the considered cases of loading.

5. Concrete strength of the transition zone has a major impact on the fundamental natural period of the RC frame.

6. The stiffness of the floor beam increased with the increase of the value of the concrete strength of the transition zone and the beams did not exceed the elastic limit during earthquake excitation.

7. In all cases under earthquake loading, the failure occurred in the column due to bending moments and formation of plastic hinges. The locations of these plastic hinges affected the overall displacement of and the overall damage index of the frame structure.
REFERENCES


[9] ACI Committee 318, “Building Code Requirements for Reinforced Concrete, ACI 318-95”, American Concrete Institute, Detroit 1995, Sec. 10.15.

