LOAD TRANSFER FROM HIGH STRENGTH CONCRETE INTERIOR COLUMNS THROUGH ORDINARY STRENGTH CONCRETE SLABS

Thesis

Of

Master of Science

By

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This is to certify that the

Thesis entitled

LOAD TRANSFER FROM HIGH STRENGTH CONCRETE INTERIOR COLUMNS THROUGH ORDINARY STRENGTH CONCRETE SLABS

Submitted By

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Has been accepted towards the partial fulfillment

of the requirements

For

Master of Science in Civil Engineering (Structures)

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National University of Sciences and Technology, Pakistan.

To

My Wife

And

Ali & Zainab

ACKNOWLEDGMENT

The author is extremely grateful to all those who offered their willing support during the extremely laborious work of casting. Thanks are due to administration of Military College of Engineering, for extended full cooperation.

I am also grateful to my respectable teachers Col (Dr) Khaliq-ur-Rehman Shad and Col (Dr) Allah Nawaz Qaisrani whose high quality of teaching made me able to explore different areas of the subjects taught by them during the course work.

Special thanks to my class fellows Major Ansar Iqbal and Major Ehsan Ullah Khan, without their help I was unable to complete my tenure at MCE.

Thanks cannot be expressed in words for, the amount of patience displayed and guidance extended, by Lt Col (Dr) M. Khaliq-Ur-Rashid Kayani and Lt Col (Dr) Gulfam Alam throughout the period.

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b	=	Width of column section
d	=	Depth of column section
h	=	Slab thickness
b'	=	Outer to outer width of lateral tie
d'	=	Outer to outer depth of lateral tie
L	=	Length of overhanging slab portion
W	=	Width of overhanging slab portion
A_c	=	Net area of concrete in column section
A_g	=	Gross area of concrete in column section
A_{st}	=	Area of column reinforcement
f'_c	=	28-day standard cylinder compressive strength
f'_{cc}	=	28-day compressive strength of column concrete control cylinders
f_y	=	Yield point stress of column reinforcement
f'_{cf}	=	28-day compressive strength of floor concrete control cylinders
f'_{cp}	=	Apparent strength of floor concrete
P	=	Ultimate strength of a concentrically loaded columns
P_{test}	=	Maximum test load on Specimen

ABSTRACT

Nine interior slab-column specimens were tested to investigate the effects on high strength concrete columns, due to the presence of ordinary strength floor concrete layer in between the columns. The 28 and 30 inches long columns had a floor slab of varying thickness i.e. 3.0, 4.5 and 6 inches. The slab extended beyond the column faces in four directions. These column specimens were designed to study the influence of longitudinal as well as lateral steel on the strength characteristics and behavior of the floor concrete. The data from these tests combined with the previously reported similar studies was analyzed to find the appropriate parameters for the estimation of the apparent strength of the floor concrete to be used in calculation of load carrying capacity of columns. Mechanics of materials approach used for the analysis of the composite materials, as proposed by Kayani in his research, was applied for the theoretical analysis of the problem. This approach with the use of the available test data lead to an expression for the calculation of the apparent floor concrete strength.

$$f'_{cp} = 2.0 \times \lambda_G \frac{f'_{cc} \times f'_{cf}}{f'_{cc} + f'_{cf}}$$

Chapter 1

INTRODUCTION

1.1 GENERAL

In modern day reinforced concrete construction, substantial economy can be achieved by using high strength concrete. High strength concrete (f'_c greater than 60 Mpa) is a relatively new construction material with enormous potential. Structures using high strength concrete are becoming increasingly common. Use of high strength concrete to construct the columns in tall structures is extremely advantageous as considerable savings in material quantities can be made. In addition free floor space is achieved due to the resulting reduction of the column cross section. It is now a common practice to design the floors of high rise buildings with slabs or slabs and beams of ordinary strength concrete and columns of high strength concrete. In the resulting structure, layers of floor concrete intersect the columns at each floor level. As these layers are made of lower strength concrete than the column, it is obvious that under some circumstances, such layers may decrease the load carrying capacity of the columns.

The nominal strength of an axially loaded column is generally determined by combining the load carrying capacities of longitudinal reinforcement of the column and the concrete section as in the equation:

$$P_o = A_{st} f_y + \left(A_g - A_{st}\right) 0.85 f_c' \text{ ----------------- } (1.1)$$

It is obvious that there is a problem in selecting the value of f'_{cc} for calculating the column capacity when there is a difference in the concrete strengths of column and the intervening slab. It has been demonstrated experimentally that the column strength is not limited to the strength of the intervening floor slab, but that on the other hand the differential between the two concretes cannot be too large. The current ACI code used these tests as the background for Sec. 10.15, which permits the column concrete to be up to 1.4 times the slab concrete strength before other measures must be taken. For interior columns, where the joint-region concrete in the slab between the ends of the columns is confined by the continuing slab concrete on all four sides, a partial remedy is suggested, which is commonly known as “Puddling’’. Puddling is the addition of high strength concrete in the region of column slab joint and demands very resolute supervision and extensive planning, especially at higher story levels. There are no reported studies where effects of “Puddling” and integration of concrete have been looked into. Moreover no guidance is provided for edge or corner columns other than adding dowels and spirals.

1.2 BACKGROUND

Bianchini, Woods, and Kesler conducted the first experimental study on the subject at the University of Illinois. Forty-five specimens representing portions of the corner, edge, and interior columns having floor systems of flat plate and two-way slab types were tested under compressive axial loads. The ratio of column and floor concrete strengths ranged from 1.5 to 3.0 with a maximum column concrete strength of about 8000-psi. From test data, Bianchini plotted the ratio of apparent column concrete strength to the floor concrete strength f'_{cp}/f'_{cf} against, the ratio of column concrete strength to the floor concrete strength f'_{cc}/f'_{cf}. The apparent concrete strength can be defined as the concrete strength to be used in calculation of load carrying capacity of the column based on test results.

Bianchini observed that there was no reduction in column concrete strength due to intersecting floor concrete up to some maximum critical value of (f'_{cc}/f'_{cf}). Values of (f'_{cc}/f'_{cf}) greater than the critical value cause a reduction in the column load carrying capacity. Following recommendations were made:

a. No reduction in column strength occurred for ratios of column concrete strength to floor concrete strength up to 1.4 for corner and edge columns and 1.5 for interior columns.

b. For corner and edge columns, no substantial benefits are obtained by increasing the column concrete beyond 1.4 times the floor concrete strength.

c. For interior columns, it was recommended that 75percent of column concrete strength above 1.5 times the floor concrete strength might be effective in sustaining the load.

In this study, column concrete strength used in specimens ranged up to 8000 psi whereas concrete having compressive strength of 14000 psi to 20000 psi is being commonly used in buildings. The slab thickness and column cross- sectional dimensions were kept constant. The column location and ratio of the column and floor concrete strengths were the only parameters investigated.

The second study on the subject was also conducted at the University of Illinois by Gamble and Klinar. A similar series of tests with column concrete strength of about 14000 psi and floor concrete strength ranging from 2300 psi to 6600 psi was carried out. In this study, interior and edge column sections from flat plate floor systems were tested under axial compressive loads. Thirteen specimens were designed to extend the range of strength of the column concrete, and to extend the range of the ratio of column strength relative to slab concrete strength. Six specimens modeled interior column-slab connections, six-modeled edge column-slab connections and one had no slab. Using 5 inch thick slabs for two edge and one interior column specimens and 7 inch slab for all other specimens. All columns were 10 inch square and were reinforced with 4#6 bars and had 1/4 in. ties at 10 inch spaces. The slabs had top reinforcement in both directions. Most of the slabs also had bottom reinforcement. One specimen had a spiral consisting of four turns of #3 bar placed round the column bars and between upper and lower layers of slab reinforcement. The tests greatly extended the ranges of maximum strengths as well as ratio of column concrete strength to the slab concrete strength.

Test results of this study confirmed the current ACI code provisions, that there is no problem so long as the column concrete strength does not exceed 1.4 times the slab concrete strength. However, they observed that the code appears to overestimate the strength of members with large ratios of f'_{cc}/f'_{cs}. Another important conclusion was that the apparent strength relationship based on f'_{cc}/f'_{cs} appears to be general across the full range of concrete strengths considered.

Another experimental study was conducted at the University of Illinois by Kayani in which six specimens were tested. These specimens consisted of four high strength sandwich columns with a 7 inch layer of ordinary strength floor concrete and two edge columns. The edge column specimens had a 7 inch slab protruding out of the column faces on three sides in between the column longitudinal steel sections. It concludes that the apparent concrete strength may not have a linear relationship with the ratio of f'_{cc}/f'_{cf}. The study led to the following observations:

a. The increase in strength of floor concrete is not entirely dependent upon the restraint or confinement provided to the sandwiched concrete by the slab on all or some of its sides although these restraints do enhance the strength gain. The presence or the absence of the lateral or longitudinal steel in columns did not affect the limiting gain in strength of the floor concrete.

b. The test data also indicated the overestimation of the floor concrete strength in case of interior columns if done in accordance with Sec. 10.15.3 of the ACI building code when there is substantial difference in strength of the two concretes.

c. The gain in strength of floor concrete is proportional to the ratio of the product and sum of the two concrete strengths, which can be, explained by the application of the principles of the mechanics of composite materials.

1.3 CONFINEMENT OF CONCRETE COLUMNS

Transverse reinforcement is provided to increase the lateral confinement of the core concrete so that the axial compressive strength of the concrete is enhanced and the ductility is improved. The beneficial effects of transverse reinforcement, on the strength and deformation characteristics of concrete have been recognized since the early days of reinforced concrete construction. In 1874, Thaddeus Hyatt patented reinforced concrete members, which contained helical wound flat bars with encased longitudinal rods. Consider investigated the concept, which lead him to confine his research to the circular hoops and helices because of their effectiveness over rectilinear transverse reinforcement.

A number of studies involving a number of tests on nearly full-size specimens have been carried out. These studies demonstrate that the confinement is greatly improved if: -

a. The transverse reinforcement is placed at a relatively close spacing and is well anchored by hooks, etc.
b. Additional supplementary overlapping hoops or cross ties with several legs crossing the section is included.
c. The longitudinal bars are well distributed within the section.
d. The ratio of volume of transverse reinforcement to volume of concrete core or the yield strength of transverse reinforcement is increased.
e. Spirals or circular hoops should be used instead of rectangular and supplementary cross ties.

1.4 EXPERIMENTAL PROGRAM

The conclusions drawn from previous studies are based on a limited number of test data. The available test data is inadequate to understand the load transfer mechanism. There is a definite need for further research on various aspects of the subject like: -

a. The affect of thickness of floor concrete on the strength characteristics of the column concrete.
b. The behaviour of lower strength floor concrete layer with varying confining pressures.

c. The performance of the high strength concrete columns with different floor concrete in flexure with axial loads.

d. The length effects on the proposed relationships.

1.5 OBJECTIVES AND SCOPE

Nine interior slab-column specimens were tested in this research program. Each of the test specimens consisted of two tied columns with an intersecting floor between the two columns. The typical layout of the test specimen is shown in Fig. 1.4. The specimens were tested in the laboratory at UET Peshawar till failure under the concentric axial loads. The testing program was carried out in order to study and investigate, following aspects of the subject: -

a. Behaviour of the interior slab-column specimens under axial compressive loads with varying ratios of slab thickness and slab confinement.

b. Development of the relationship for estimating the load carrying capacity of the column.

Chapter 2

SPECIMENS, MATERIALS, AND FABRICATION

2.1 DESCRIPTION OF SPECIMENS

In the present study the interior slab-column specimens with 6 inch square columns having a total length of 33, 34 and 34.5-inches, including a slab concrete layer of varying thickness were tested. The specimens generally consisted of three distinct portions referred to as Bottom Column, Slab portion, and a Top Column as shown in the Fig. 1.1. Longitudinal bars were welded to a 6 inch square and ½ inch thick steel plate at the bottom of the specimen in order to ensure that the steel bars are vertically aligned. In bottom column the total height of the column was exclusive of steel plate thickness. In top column, after pouring of concrete a steel plate of similar dimensions was placed at the top end and properly flushed with its outer edges. The total height of top column was also exclusive of steel plate thickness. The purpose of adding steel plates was to ensure even distribution of load at the time of testing. This practice resulted in different end conditions for top and bottom columns. Longitudinal column reinforcement consisted of # 5 bars whereas ties, wherever provided, were of # 3 bars.

Reinforcement for slab portion consisted of # 3 bars. Slab reinforcement was provided at the top only. Slab thickness was varied from 3 inch to 6 inch depending upon the category of specimen as listed in paras below.

2.2 CATEGORIES OF SPECIMENS

Based on configuration and desired objective of research, the specimens were divided into three different categories. Three specimens were cast in each category

2.2.1 Category IA

The intersecting overhanging slab of ordinary strength concrete, protruding on all sides of the column was 6 inch thick and column height was kept 14 inch each (both top and bottom columns). These specimens contained 6 #5 longitudinal steel bars. Two rectangular ties, each at 6 inch c/c spacing starting at 3 inch from end of the column, were provided in both the columns. No tie was provided at the centre of the slab. Total height of the specimen was 34 inch. Details of this category specimen are shown in Fig. 2.1.

2.2.2 Category IB

These specimens are similar to category IA specimens, as shown in Fig. 2.2. The only difference between the two categories is the slab thickness and column height, which was kept as 3 inch and 15 inch respectively, thus decreasing the total height of the specimens to 33 inch.

2.2.3 Category IC

These specimens are also similar as above except the slab thickness which was kept as 4.5 inch, thus increasing the total height of the specimens to 34.5 inch. The details of the three specimens in this category can be seen in Fig. 2.3.

2.3 CONCRETE MIX DESIGN

2.3.1 Column Concrete Mix

The mix design selected after extensive trials is as under:

a. Cement = 19.31 KGs

b. Sand = 9.65 KGs

c. Aggregate = 24.14 KGs

d. Water = 4.44 KGs

e. Superplasticizer = .772 KGs

The quantities of different materials in actual mix were as follows: -

a. Cement = 33.10 percent

b. Sand = 16.54 percent

c. Aggregate = 41.40 percent

d. Water = 7.61 percent

e. Superplasticizer = 1.32 percent

All specimens of above categories were cast with above mix design, with constant w/c ratio as .23. 4percent superplasticizer was used for casting of all the specimens. Six cylinders were cast from each batch of concrete. Three cylinders were tested for compressive strength at 28 days and three were tested on the date of testing of specimens.

2.3.2 **Floor Concrete Mix**

a. Cement = 10 percent

b. Sand = 30 percent

c. Aggregate = 60 percent

d. W/C Ratio = 8 percent

The entire casting of slab portions with six cylinders was completed from four batches of concrete mix.

2.4 DESIGNATION OF THE TEST SPECIMENS

The specimens are designated as IA, IB, IC . The first alphabet indicates the type of specimen as interior where as the second alphabet is for category to which it belongs.

2.5 MATERIALS

2.5.1 Column Concrete Materials

The various properties of the materials used were:

a. **Cement.** Standard Portland Cement Type-I.

b. **Coarse Aggregate**. The coarse aggregate consisted of 3/8” maximum size limestone chips. These chips had bulk specific gravity (ssd) of 2.81, crushing value of 11.15, and abrasion value of 9.10.

c. **Fine Aggregate**. The fines consisted of sand with a fineness modulus of 2.875.

d. **Admixtures**. Naphthalene based superplasticizer was used as admixture.

e. **Water**. Normal potable water was used in the mix.

2.5.2 Floor Concrete Materials

a. **Cement**. Standard Portland Cement Type-I.

b. **Coarse Aggregate**. Aggregate with a bulk specific gravity (ssd) of 2.65 was used.

c. **Fine Aggregate**. Coarse sand having fineness modulus of 1.81 was used.

d. **Water**. Normal potable water was used.

2.5.3 **Reinforcing Steel.**

The reinforcement mainly consisted of deformed # 5 and # 3 bars. Grade 6o # 5 bars were used for longitudinal reinforcement of all the columns whereas grade 60 # 3 bars laterally tied the longitudinal bars. The stress-strain curves for # 5 and # 3 steel bars are shown in Fig. 2.4 and 2.5.

2.6 **FABRICATION OF SPECIMENS**

All the specimens were cast in waterproof wooden forms in the Military College of Engineering Concrete Laboratory at Risalpur. The formwork was specially designed and fabricated locally. 1 inch thick properly seasoned deodar wooden planks were used for this purpose. The specimens were cast in upright position. The steel reinforcement was tied with sixteen-gage wire. The cage was then welded to the ½ inch steel plate. This was basically done to keep the proper vertical alignment of the longitudinal bars.

The concrete for the columns was mixed in a drum mixer as one batch was required for each set of nine 14 inch or 15 inch tall columns (upper or lower) and the associated 6 inch control cylinders. The concrete in the forms was consolidated with a high frequency, internal rod vibrator. The slab concrete for all the category specimens was also mixed in the drum mixer. The consolidation procedure remained the same, as it was for the column concrete.

Each specimen was cast in three distinct stages. In first stage, the forms for the bottom columns were set in place and the concrete was poured into the forms and vibrated. Six control cylinders were also cast from the batch in the standard steel forms and vibrated by the internal rod vibrator. The forms for the bottom columns were removed after at least 20 to 24 hours. The second stage consisted of the setting up of the formwork, placement of the slab reinforcement, and pouring of the slab concrete. Six control cylinders were also cast from each batch of the slab concrete. The third stage mainly consisted of the casting of the top columns. A steel plate of 6x6x1/2 inch size was placed at the top of the top column and properly flushed with the outer edges of the column. This was done to ensure the equal distribution of the load on the specimen.

Each stage in the fabrication of the specimens had a time difference of at least one-day from the preceding one. The parts of the specimens already cast along with the corresponding cylinders were covered with wet hessian cloth during the preparations for the next stages. The specimens and the cylinders were placed under moist conditions, after they were completely cast, for 28 days. The specimens, once removed from moist conditions, were kept in the lab under normal conditions.

Strain gages were applied to the column reinforcing bars in general in all the specimens. All the specimens had the strain gages on the longitudinal bars both in the columns and slab portions. The details of the location of the strain gages in specimens are given in Fig. 2.4.

The deformations were recorded from the reinforcing bars where the gages were to be applied. After soldering the lead wires with the gages, they were water proofed so that, no damage is caused to them due to casting concrete around them.

The specific information about the fabrication of the specimens is given in table 2.1.

Chapter 3

INSTRUMENTATION, TEST SETUP, AND TESTING PROCEDURE

3.1 INSTRUMENTATION

Strain gages were applied to the column reinforcing bars in all specimens. Six strain gages were applied in each specimen. Two each in the top column, bottom column, and slab portion on diagonally opposite reinforcing bars. The location of strain gages is shown in Fig. 2.4.

The electrical strain gages were of the foil type EA series gages. This series is a general-purpose family of constant an alloy strain gages widely used in experimental stress analysis. EA gages are of open faced constructed with a 0.001 inch (0.03mm) tough, flexible polyamide film backing. They work at temperature range of –100 to +350 F^o with an approximate range of 5 percent strain for 0.240"(6mm)-gage length. These gages had a resistance of 120.0 ± 0.3 percent ohms at 24 ^{o} C and gage factor of 2.060± 0.5 percent at 24 ^{o} C.

3.2 TEST SETUP

The specimens were subjected to the axial compression in a 200 tons compression-testing machine, in Concrete testing Laboratory at the University of Engineering and Technology (UET) Peshawar. The specimens were placed between the two bearing plates, mounted on moveable heads making the alignment and uniform application of load on specimens possible by moving the plates in the required directions. The specimens were carefully transported to the Testing Lab at UET Peshawar, ensuring that no damage is caused to the specimens and the wires connected to the strain gages.

The specimens were carefully centred in the testing machine between the two bearing plates for the application of the load. After centring the specimens, it was ensured that the specimens are aligned vertically on the bearing plates.

3.3 TESTING PROCEDURE

The objective was to observe the behaviour of the specimens under axial load. The load was applied after centring and alignment of the specimens in the testing machine and making the necessary connections for reading the strain measurements. The load was applied with medium rate of loading. On appearance of cracks or any unusual change the dial reading of load was recorded. The cracks were marked along with the load readings on the specimens. Strain gage readings were also recorded after each interval of 5 tons of load for first 50 tons and after that till failure the interval was increased to 10 tons. After failure of the specimens, the loose concrete around the failure area was removed to look at the condition of the reinforcing bars and the gages.

The testing procedure described above generally took 10-20 minutes to its completion.

Chapter 4

EXPERIMENTAL RESULTS

4.1 BEHAVIOUR OF SPECIMENS

A tied column with appropriate amount and detailing of reinforcement fails in compression at the load given in Eq 1.1. At this load the concrete fails by crushing and shearing outward along inclined planes, and the longitudinal steel by buckling outward between the ties.

The details of behaviour of each specimen during the test are given in subsequent paragraphs.

4.1.1 Specimen IA-1

The initial application of the load to the specimen, of 6” thick slab over hanging on all four sides, started on 13 Mar 2001. All the strain gages were working properly and recording of the strains from these gages was possible. At 102.6 tons of load formation of vertical cracks started in top column. At106.8 tons diagonal cracks appeared in the slab and propagated towards corners of the slab. Specimen failed at ultimate load of 114.2 tons from top column. Fig 4.1 and 4.10 shows the exact behaviour of all three portions of the specimen.

4.1.2 Specimen IA-2

This specimen was tested on 13 Mar 2001. All the strain gages were working properly. At 87.81 tons of load vertical cracks started from upper end of top column. At100.7 tons two diagonal cracks appeared at the upper surface of slab. These cracks progressed towards cover of slab with further increase in load. These cracks were also prominent at lower side of the slab. Specimen failed at ultimate load of 102.7 tons from top column. In fig 4.2 and 4.11 it can be seen that the load vs strain curve of bottom column shows that it did not fail at all. The strains in slab portion indicate its failure, but the ultimate failure of specimen was due to crushing of top column.

4.1.3 **Specimen IA-3**

This specimen was tested on 13 Mar 2001. All the strain gages were working properly. At 69.5 tons of load vertical cracks started from upper end of the top column. At 91.3 tons cracks appeared at the joint of top column and slab and propagated towards middle of slab. No cracks were observed at corners of the slab. At 98 tons of load top column cracks propagated towards the joint. Specimen failed at ultimate load of 103.1 tons from top column. Nothing happened to bottom column. Fig 4.3 and 4.12 clearly shows the top column failure.

4.1.4 **Specimen IB-1**

This specimen had 3 inch thick slab overhanging on all the four sides of the column. All the six strain gages were working properly. The specimen was subjected to loading on 13 Mar 2001. At 46.5 tons of load minor cracks appeared in top column. Horizontal cracks in top column at the location of rectangular ties at 3 inch & 9 inch from top appeared at 80.4 tons of load. At 83.6 tons the intervening slab started cracking from the corners of top column- slab joint and propagated towards the corners of the slab. Vertical cracks in the bottom column appeared at 87.3 tons of load. Specimen failed at ultimate load of 91 tons from top column – slab joint. Also ref. Fig 4.4 and 4.13.

4.1.5 Specimen IB-2

This specimen was tested on 13 Mar 2001. All the strain gages were working properly. At 112 tons of load cracks in upper portion of slab appeared and propagated towards corners of slab. Vertical cracks in bottom column at slab-column joint were observed at 115 tons. Specimen failed at ultimate load of 119.9 tons, from lower slab-column joint. Nothing happened to top column. Ref. fig 4.5 and 4.14.

4.1.6 Specimen IB-3

This specimen was tested on 15 Mar 2001. All the strain gages were working properly. At 82.3 tons of load crack in slab originated from middle of column face. Immediately cracks also appeared in outer face of slab. Other crack in slab at 94 tons appeared at opposite face of column. At 100 tons of load further cracks in slab appeared on the corners. All these cracks were also prominent at lower side of slab. Bottom column started cracking from slab-column joint at 109.8 tons of load. At 112.8 tons bottom column cracks propagated and minor cracks in top column also appeared. Specimen failed at ultimate load of 119.1 tons from lower slab-column joint. Ref fig 4.6 and 4.15.

4.1.7 Specimen IC-1

The specimen was tested on 14 Mar 2001. All the strain gages were working properly. At 105 tons of load vertical cracks at upper end of top column appeared. At 112 tons cracks in upper side of slab appeared from joint and propagated towards corners. The cracks in slab were more prominent at lower side. Specimen failed at ultimate load of 117.8 tons from top column. Ref fig 4.7 and 4.16.

4.1.8 **Specimen IC-2**

This specimen was tested on 14Mar 2001. All the strain gages were working properly. At 103 tons of load vertical cracks in top column appeared. The cracks in slab appeared at 106 tons. These cracks initiated from the face of the column and propagated straight towards end of the slab. The diagonal cracks in slab appeared at 114 tons, which propagated towards the corners of the slab. Specimen failed at 117.6 tons from top column. Ref fig 4.8 and 4.17.

4.1.9 **Specimen IC-3**

This specimen was tested on 15 March 2001. All the strain gags were working properly. Vertical cracks at upper end of top column appeared at 113 tons of load. At 127 tons diagonal cracks in slab originated from corners of top column and propagated outwards. At 140 tons cracks in slabs reached to corners, and cracks in bottom columns also originated from upper end. At 143.8 tons of load cracks in bottom column opened up and specimen failed. Ref fig 4.9 and 4.18.

4.2 **CONCRETE STRENGTHS**

Six concrete cylinders were cast from each of the concrete batch, prepared for different sections of the specimens, while fabricating the specimens. Three of these cylinders were tested after 28 days of the casting of that particular concrete. The remaining three cylinders were tested at the final day of testing of that particular specimen. These strength values are tabulated in Tables 4.1 and 4.2 respectively.

From the results of testing of these cylinders it was observed that the cylinder strengths varied from one another. The average of the six cylinders has been used throughout the course of this presentation for analysis and other purposes (Table 4.3). The cylinders were capped with the melted mixture of sulphur well before the actual testing of the cylinders. The thickness of the capping material on the cylinder apparently influenced the strengths appreciably. An effort was made to minimize this variation by careful preparation of the capping mixture and its application on the cylinders. Cylinder strengths also varied because of the variations in the water content of the concrete batches. The water content could not be kept constant due to the variations in the moisture contents of the coarse and fine aggregates.

4.3 STRAIN MEASUREMENTS

The strains were measured by using the strain gages on the longitudinal as well as lateral steel. The strain values at different loads during the tests have been tabulated separately for each of the specimen in the tables 4.4 – 4.12. The strain values for each gage have been plotted against the load in Figs. 4.4 – 4.12. Strain values acquired during the tests give the axial strains in columns.

The strains in all categories are quite similar and normal under different load conditions. All the strain gages remained in proper working condition. The general trend of the behaviour of the specimens under loads is similar.

Chapter 5

ANALYSIS OF EXPERIMENTAL RESULTS

5.1 GENERAL

The interpretation of the test results is required to reach at the conclusions for understanding the data and to analyze the behaviour of the structures. In order to evaluate the test data, column concrete compressive strength (f'_{cc}), floor concrete compressive strength (f'_{cf}) and the apparent concrete strength of the column structures, (f'_{cp}), were used. The apparent concrete strength can be defined as the concrete strength to be used in calculation of load carrying capacity of the column based on test results. This value is calculated and given interpolation by Eq. 1-1 as:

$$f_{cp} = \frac{P_t - A_{st}\, f_y}{0.85\left(A_g - A_{st}\right)}$$

Where A_g and A_{st} are the gross concrete and longitudinal steel areas respectively.

These parameters have been looked at in different forms and combinations to have some indication about the behaviour of the specimens. The present experimental program consisted of nine specimens. In addition to the data obtained from the present experimental program, the test data from previous experimental programs has also been included for the evaluation of the behaviour of columns in presence of a weaker floor concrete layer.

5.2 ANALYSIS

Kayani in his research established that sec 10.15 of ACI code is not based on appropriate parameters and behaviour of columns with a lower strength floor concrete layer in between can be compared with the behaviour of composite materials. Applying mechanics of material approach, a relation for the estimation of apparent concrete strength, applicable to all kinds of columns in a structure, has been proposed as under:

$$f'_{cp} = 2.0 \times \lambda_G \frac{f'_{cc} \times f'_{cf}}{f'_{cc} + f'_{cf}} \text{-------------------} \qquad (5-1)$$

or

$$\sqrt{f'_{cp}} = 2.0\lambda_G \frac{\sqrt{f'_{cc}} \times \sqrt{f'_{cf}}}{\sqrt{f'_{cc}} + \sqrt{f'_{cf}}} \text{-----------} \qquad (5-2)$$

Where λ_G is constant whose value depends upon the location of the column in the structure. The value assigned to this constant for interior column is:

λ_G = 1.25 (for Eq. 5-1)

λ_G = 1.10 (for Eq. 5-2)

The apparent concrete strength has been plotted to verify equations 5-1 and 5-2 and shown in Fig. 5.1 and 5.11. The plots of these values indicate that the current specimens matches the trends observed from earlier studies.

The current specimens and their test results must be evaluated based on following perspectives:

a. Failure of most of the specimens, tested in this program was due to failure of top columns. Four specimens failed from the slab column joint.

b. This was evident from the test results that cover concrete was effective in load resistance. It was therefore required to analyse the test results using gross area for load carrying capacity of columns.

c. Increased (h/b) ratio has reduced the load carrying capacity of the specimen

5.3 INTERPRETATION OF TEST RESULTS

The values of f'_{cp} , based on gross area of the section, along with previous test data, were plotted in Fig. 5.1 and 5.11, using equations 5-1 and 5-2 .

5.3.1 Apparent Concrete Strength Vs Ratio Of Product And Sum Of Concrete Strengths.

Detailed calculations of previous and present test results are summarized in table 5.2 -5.4. This data for the above parameters has been plotted in Fig 5.1 and compared with Eq. 5-1:

$$f'_{cp} = 2\lambda_G \frac{f'_{cc} \times f'_{cf}}{f'_{cc} + f'_{cf}}$$

From the plot, it can be seen that Eq. 5-1 is also valid for the current test data.

5.3.2 **Square Root of Apparent Concrete Strength VS Ratio of Square Roots of Product and Sum of Concrete Strengths**

The plot for above parameters is given in Fig 5.11 with Eq. 5-2:

$$\sqrt{f'_{cp}} = 2.0\lambda_G \frac{\sqrt{f'_{cc}} * \sqrt{f'_{cf}}}{\sqrt{f'_{cc}} + \sqrt{f'_{cf}}} \qquad (5-2)$$

Again, it is evident that the above equation is applicable and remains valid for the current test results.

Where, λ_G, is a constant whose value assigned for this relationship is 1.18

5.3.3 **Comparison Of Apparent Concrete Strengths Found From ACI, Test Results And Calculated By Using Eqs. 5.1 & 5.2**

A comparison of apparent concrete strength was carried out as shown in fig 5.13 & 5.14 from which it is again evident that Eqs. 5-1 & 5-2 are valid for current test results.

5.4 **EFFECT OF WIDTH OF COLUMN SECTION (b) AND SLAB THICKNESS (h) IN BEHAVIOUR OF SLAB – COLUMN JOINT.**

Effect of h &b have been plotted in fig. 5.3 to 5.10, using following parameters:

a. Apparent concrete strength Vs Ratio of product and sum of concrete strengths multiplied by slab thickness.

b. Apparent concrete strength Vs Ratio of product and sum of concrete strengths multiplied by square root of slab thickness.

c. Apparent concrete strength Vs square root of Ratio of product and sum of concrete strengths multiplied by slab thickness.

d. Apparent concrete strength Vs Square Root of Ratio of product and sum of concrete strengths multiplied by square root of slab thickness.

e. Apparent concrete strength Vs Ratio of product and sum of concrete strengths multiplied by ratio of slab and column thickness.

f. Apparent concrete strength Vs Ratio of product and sum of concrete strengths multiplied by square root of ratio of slab and column thickness.

g. Apparent concrete strength Vs square root of Ratio of product and sum of concrete strengths multiplied by ratio of slab and column thickness.

h. Apparent concrete strength Vs square root of ratio of product and sum of concrete strengths multiplied by square root of ratio of slab and column thickness.

In the present programme it is seen that almost every specimen exhibited a reduction in the ultimate load with increase in the slab thickness i.e. increased (h/b) ratio.

Steel proportions can be used to improve the behaviour of slab column joint as far as the axial load carrying capacity of columns is concerned. However, the adequacy of slab should be ensured to resist excessive stresses emanating from the joint at high axial load.

5.5 (h/b) ratio also seems to be a major factor in behaviour of slab – column joint which should be investigated further. In the present program almost every specimen exhibited a reduction in the ultimate load with increase in the slab thickness i.e.

increased h/b ratio. Behaviour of two specimens was not as envisaged therefore there behaviour can be discarded.

5.6 Failure of top column is mainly due to the reduction in the strength of top column which lead to such an abnormal behaviour.

5.7 Amount of longitudinal reinforcement affects the behaviour of columns. Excessive reinforcement may reduce the load carrying capacity of columns due to possibility of a very weak plane which may not allow cover concrete to play any role in load resistance.

Chapter 6

CONCLUSIONS AND RECOMMENDATIONS

6.1 GENERAL

Test data from the present and the previously reported experimental programs has been analyzed in the previous chapter. This analysis amplifies many aspects of the effects of the floor slab layer present in the columns when there is a difference in the strength of the two concretes. This difference is quite normal in case of high-rise buildings where the high loads and the size considerations force the engineers to design the high strength concrete columns. These along with the economic factors force the use of ordinary strength concrete floor systems. The process results in a dilemma of what concrete strength should be used for determination of the structural properties of such columns.

Major conclusions drawn from this study are discussed in subsequent paragraphs.

6.2 CONCLUSIONS

a. ACI Sec. 10.15 should be changed for calculation of load carrying capacity of columns as under:

$$P_n = 0.85 f'_{CP}\left(Ag - A_{st}\right) + A_{st}\, f_Y$$

Where

$$\sqrt{f'_{CP}} = \frac{2\lambda g \sqrt{f'_{CC}} \times \sqrt{f'_{Cf}}}{\sqrt{f'_{CC}} \times \sqrt{f'_{Cf}}}$$

or

$$f'_{CP} = \frac{2\lambda G\ f'_{CC} \times f'_{Cf}}{f'_{CC} + f'_{Cf}}$$

Where

$\lambda g = 1.18$ for interior columns

$\lambda G = 1.48$ for interior columns

b. Amount and detailing of lateral and longitudinal reinforcement can be used to improve the carrying capacity of slab concrete in the slab – column joint.

c. Ratio of least column dimension and thickness of the slab affects the behaviour of the joint, which may be verified.

d. Modulus of Elasticity of concrete may be a function of simple (f'_c). This needs further validity.

e. Mechanics of composite materials has worked well to predict the response of slab – column joints to axial loads. The approach may be used to understand the unpredictable and varying behaviour of reinforced concrete members.

6.3 RECOMMENDATIONS

Experimental program should be expanded to include:

a. Effect of (h/b) ratio on load carrying capacity of columns.

b. Enhanced ranges of concrete strengths should be used in test Specimens.

c. Confinement of high strength concrete should be studied in detail to propose a theoretical model.

d. Size of the specimens should also be increased to represent the physical structures. For the purpose, 500 tons axial load testing machine may be procured and installed at MCE.

e. Amount and detailing of longitudinal as well as lateral reinforcement in load carrying capacity of columns and slab column joints should be investigated.

f. Behaviour of slab – column joint in presence of moments in addition to the axial loads should also be studied.

g. Behaviour of slab–column joint by providing spiral reinforcement should also be investigated.

LIST OF REFERENCES

1. Park,Robert and William L. Gamble, “ Reinforced Concrete Slabs,’ John Wily & Sons, 1980,618p.

2. Bianchini, A.C.R.E. Wood ,and C.E. Kesler, “Effect of Floor Concrete Strength on Column Strength ,” Journal ACI. Proc. Vol. 56 No. 11 May 1960pp 1149-1169.

3. “Building Code Requirement for reinforced concrete (ACI) 318-95) and commentary- ACI 318-95,” American concrete institute, Detroit,1995-353p

4. Gamble, W.L. and J.D. Klinar, Tests of High Strength Concrete columns with intervening floor slabs,” Journal of structural Division, ASCE, Vol. 117 No. 5, May 1991.

5. Kayani, M.K.R. “Load transfer from high strength concrete column through lower strength concrete slabs,” PHD. Dissertation, University of Illinois, Urbana, Illinois, 1992.

6. Sheikh, Shamim A, and S. M. Uzumeri, “strength and ductility tide concrete column.” Journal of structural Division, ASCE. Vol. 106 No. ST5, May, 1980 pp 1079-1102.

7. Park, Robert, Priestly, M. J. N. and Wayne D. Gill, “Ductility of squire-confined concrete column,” Journal of structural Division, ASCE, Vol. 108 No. ST4 Apr, 1982, pp 929-950.

8. Szulczynski, Tadeusz and M.A Sozen. Load-Deformation Characteristics of concrete prisms with rectilinear transverse reinforcement,” Civil

Engineering Studies, Structural Research Series, No. 224 , University of Illinois, Urbana, Sept. 1961,54p.

9. vellenas, J.V.V. Bertero, and E.P. Popov. "Concrete confined by rectangular hoops and subjected to Axial load," Report No UCB/ EERC-77/13, Earthquake Engineering research centre University of California Berkeley Aug. 1977, 105p.

10. Scott, B. D. R. Park and M.J. N. Priestly, "stress-strain Behaviour of concrete confined by overlapping hoops al low and high strain rates ," Journal ACI, Vol. 117, No. 5 may 1991.

11. Mander, B. j. "Seismic design of bridge piers," Research Report No. 84-2 University of cantervury, Christchurch, Feb. 1984, 442p.

12. Sheikh, Shamim A, and S. M. Uzmeri, Analytical Model for concrete confinement in Tied column," Journal of structural Division, ASCE, Vol.108, No. St12, Dec. 1982, pp 2703-2722.

13. Kent, D.C. and R. Park, "Flexural Member with confined concrete," Journal of structural Division, ASCE Vol.97, NO. St7, Jul, 1971, pp 1969-1990.

14. Mander, B.J. M.J.N. Priestly and R. Park, "Theoretical Stress-strain model for confined concrete," Journal of structural Division, ASCE, Vol. 114, No. 8, AUG, 1988, pp.1804-1826.