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

BY

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This Is To Certify

That

LOAD TRANSFER FROM HIGH STRENGTH EDGE COLUMNS

THROUGH ORDINARY STRENGTH CONCRETE SLABS

Submitted by

MAJOR ANSAR IQBAL

Has been accepted towards partial fulfillment

Of

The requirements

For

Master of Science in Civil Engineering (Structures)

Lieutenant Colonel Dr Gulfam Alam

National Institute of Transportation, Risalpur

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To

My School Teacher

Qureshi Ghulam Sarwer

ABSTRACT

Nine edge slab-column specimens were tested to investigate the effects on high strength concrete columns, due to presence of ordinary strength floor concrete layer in between the columns. The 30'' long columns had a floor slab of varying thickness i.e. from 3-in to 4.5-in. The slab extended beyond the column face in three 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 previously reported similar studies was analyzed to find the appropriate parameters for the estimation of apparent strength of floor concrete to be used in calculation of load carrying capacity of columns. Mechanics of material approach used for 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 available test data, lead to an expression for the calculation of apparent floor concrete strength as under:

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

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b = Width of column section

d = Depth of column section

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-compressive strength of column concrete control cylinders

f_y = Yield 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

Chapter 1

INTRODUCTION

1.1 GENERAL

In modern day reinforced concrete construction, substantial economies 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:

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

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 can not be too large (2). The current ACI code (3) 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

1.2.1 Bianchini, Woods, and Kesler (2) 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 75% 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.

1.2.2 The second study on the subject was also conducted at the University of Illinois by Gamble and Klinar (4). 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" thick slabs for two edge and one interior column specimens and 7 in. slab for all other specimens. All columns were 10" square and were reinforced with 4#6 bars and had 1/4 in. ties at 10" 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 extend 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.

1.2.3 Another experimental study was conducted at the University of Illinois by Kayani (5) in which six specimens were tested. These specimens consisted of four high strength sandwich columns with a 7" layer of ordinary strength floor concrete and two edge columns. The edge column specimens had a 7” 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. Considere investigated the concept, which lead him to confine his research to the circular hoops and helices because of their effectiveness over rectilinear transverse reinforcement (8).

A number of studies (6, 9-11) 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 behavior 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 edge 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. Behavior of the edge 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.

SPECIMENS, MATERIALS, AND FABRICATION

2.1 DESCRIPTION OF SPECIMENS

In the present study the edge slab-column specimens with 6" square columns having a total length of 33 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" square and 1/2" 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“ to 6” 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 EA

The intersecting overhanging slab of ordinary strength concrete, protruding on three sides of the column was 3" thick and column height was kept 15”each (both top and bottom columns). These specimens contained 4 #5 longitudinal steel bars. Two rectangular ties, each at 6”c/c spacing starting at 3” from end of the column, were provided in both the columns. No tie was provided at the center of the slab. Total height of the specimen was 33“. Details of this category specimen are shown in Fig. 2.1.

2.2.2 Category EB

In this category slab thickness was increased to 4.5”. Thus total height of the specimen became 34.5”. An additional #5 steel bar was provided on the outer side of column. The details of this category specimen are shown in Fig. 2.2.

2.2.3 Category EC

In this category slab thickness was 4.5”. Thus total height of the specimen remained 34.5”. An additional tie was provided in the slab region. The details of this category specimen are shown 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 %

b. Sand = 16.54 %

c. Aggregate = 41.40 %

d. Water = 7.61 %

e. Superplasticizer = 1.32 %

2.3.1.1 The mix design used for column concrete is as under:-

a. W/C ratio =0.23

b. Superplasticizer =4% by weight of cement

All bottom and top columns were cast with the above mix. One batch was prepared to cast nine columns and six cylinders. Three cylinders were tested at 28 days and three cylinders were tested on the test date.

2.3.2 Floor Concrete Mix

a. Cement = 10 %

b. Sand = 30 %

c. Aggregate = 60 %

d. W/C Ratio = 8 %

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

2.4 Designation of the Test Specimens

The specimens are designated as EA, EB, and EC. 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.

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 3.28.

d. **Admixtures.** Nephthalene 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.

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 71 # 5 bars were used for longitudinal reinforcement of all the columns whereas grade 60 # 3 bars laterally tied the longitudinal bars.

2.6 Fabrication of Specimens

2.6.1 All the specimens were cast in wooden forms in the Military College of Engineering Concrete Laboratory at Risalpur. The formwork was specially designed and fabricated locally. 1″ 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 1/2″ steel plate as shown in Fig 2.6. This was basically done to keep the proper vertical alignment of the longitudinal bars.

2.6.2 The concrete for the columns was mixed in a drum mixer as one batch was required for each set of nine 15” tall columns (upper or lower) and the associated 6” 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.

2.6.3 Each specimen was cast in three distinct stages. In first stage, the forms for the bottom columns were set in place and the concrete 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 6″x 6″x 1/2″ 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.

2.6.4 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.

2.6.5 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. 2.7, and 2.8.

2.6.6 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.

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

INSTRUMENTATION, TEST SETUP, AND TESTING PROCEDURE

3.1 Instrumentation

Strain gages were applied to the column reinforcing bars in all specimens. The column reinforcement in category EA and EB specimens had generally six strain gages (two in top column, two in bottom column and two in slab portion) each mounted on the longitudinal diagonally opposite bars. The columns of category EC specimens had eight strain gages out of which six strain gages were applied at the same locations as in case of category EA and EB whereas two additional gages were provided on a rectangular tie provided in the slab region. The locations of strain gages on different categories are shown in Fig. 2.7 and 2.8.

The electrical strain gages were of the foil type EA series gages. This series is a general-purpose family of constantan 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 with an approximate range of 5 % strain for 0.240"*(6mm)-gage* length. These gages had a resistance of 120.0 ± 0.3 % ohms at 24 °C and gage factor of 2.060± 0.5 % at 24° C.

3.2 Test Setup

The specimens were subjected to continuous axial compression, in a 200 tons compression-testing machine, in Strength of Material Laboratory at the University of Engineering and Technology (UET) Peshawar. 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 centered in the testing machine between the two bearing plates for the application of the load. After centering the specimens, it was ensured that the specimens are aligned vertically on the bearing plates. The test set-up is shown in Fig. 3.1.

3.3 Testing Procedure

The objective was to observe the behavior of the specimens under axial load. The load was applied after centering and alignment of the specimens in the testing machine and making the necessary connections for reading the strain measurements as shown in Fig. 3.2. The load was applied continuously. The cracks were marked as they appeared, along with the load readings on the specimens and updated thereafter till the complete failure of specimen. Till fifty tons load the strain gage readings were taken at the five tons load intervals and thereafter the load intervals for strain gage readings were increased to ten ton till ultimate load. After the ultimate load failure has been reached the load intervals were again reduced to five ton. 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 Behavior Of Specimens

Tied column with substantial lateral reinforcement and appropriate detailing fails in compression in two distinct stages. In the initial stage, the cover concrete spalls off, resulting in loss of load due to a considerable reduction in load bearing area. In final stage, the core concrete, due to confinement by lateral and longitudinal reinforcement, takes extended loads till the crushing of concrete or buckling of the longitudinal steel bars or both occurring simultaneously.

In present experimental program most of the specimens failed at column-slab joint. Initially vertical cracks appeared in the weak concrete layer of slab sandwiched by bottom and top column. It was followed by vertical cracks in anyone of the columns, near the load bearing steel plates. In some of the cases slab developed diagonal cracks starting from the corners of column on confined face and propagated towards the corners of slab. Finally vertical cracks appeared in columns at the slab column-joint and with little more addition of load, specimens failed. Buckling of column reinforcement was observed near the column-slab joint.

In two of the specimens slab did not fail. In these specimens, initially concrete cover spalled off between two ties of anyone of the columns. At the final stage of test, longitudinal reinforcement of column buckled between the same two ties. However, one specimen out of these two, developed hairline vertical cracks in slab, along the column reinforcement, passing through it.

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

4.1.1 EA-1

This specimen was subjected to load on 13th of March 2001. This specimen failed in four stages. At 86 tons load, vertical cracks appeared on the unconfined face of slab. At 96 tons, vertical cracks developed at the upper end of top column and also at the upper end of bottom column simultaneously. At 102 tons of load diagonal cracks appeared in slab at the corners of column on confined side and propagated quickly into the slab. Apart from diagonal cracks perpendicular cracks also appeared on the slab. These cracks were immediately followed by ultimate load. The strains in the slab region were greater as compare to the strains in bottom and top column at the start of testing. After 45 tons of load, till 100 tons, bottom column took more strains as compare to the other two components of the specimen. But just before the ultimate load. This specimen failed at bottom column-slab joint at ultimate load of 103 tons

4.1.2 EA-2

This specimen was tested on 14th of March 2001. At 86 tons, concrete cover between the two lateral ties of top column started spalling off. At 103 tons the concrete cover of bottom column spalled off between the two lateral ties, which was immediately followed by vertical cracks at lower end of bottom column. The propagation of these cracks was very quick and specimen reached immediately its ultimate load of 104 tons. There was very small load interval between second and third stage. It was observed that upto 20 tons of load bottom column took very less strains but thereon strains of bottom column short –up suddenly. The average strains in slab and bottom column were almost same upto 100 tons. After this load concrete between two ties of the bottom column crushed which quite prominent from comparatively higher strain readings of strain gages installed in bottom column. The specimen failed by bottom column.

4.1.3 EA-3

This specimen was tested on 14th of March 2001. At 68 tons, vertical cracks appeared at the upper end of top column but their further propagation was very slow. At 83.5 tons, horizontal cracks appeared at bottom column-slab joint. At 95.2 tons, cracks in slab perpendicular to face of column appeared on confined side. These cracks were followed by further cracks in slab, which originated at the middle of column face and traveled into slab parallel to exposed side. Both the strain gages applied to the column reinforcement of top column were found out of order at the time of testing. Through the testing of this specimen, the strains in the slab region were much greater as compare to the strains in bottom column. After 70 tons of load, the strain gage readings of slab region clearly indicate that concrete in this region is failing rapidly and stress is being transferred to steel reinforcement. The specimen failed at bottom column- slab joint at 97.4 tons.

4.1.4 EB-1

This specimen was tested on 13th of March 2001. At 77 tons, vertical cracks appeared in slab on unconfined face. At 89 tons, vertical cracks appeared at upper end of top column but these cracks did not propagate appreciably and they never crossed the upper most tie of top column. At

105.7 tons, diagonal cracks originated from the corners of top column and crept into the slab, which ultimately reached the edges of slab. At 107.7 tons, vertical cracks at upper end of bottom column and horizontal crack around the upper most tie of bottom column appeared. These cracks propagated very rapidly and specimen did not take any further loads. At the time of testing, it was found that one strain gage in the slab region and two strain gages applied on the reinforcement of top column were out of order. Till 70 tons of load the strains in bottom column and in slab region were almost same. From 70 tons till ultimate load the strains in the slab region were remarkably greater than the strains of bottom column. This strain behavior of specimen indicates that concrete in the slab region started failing after 70 tons of load and stresses in concrete were distributed to steel reinforcement progressively. This specimen failed at slab at 107.7 tons.

4.1.5 EB-2

This specimen was tested on 14^{th} of March 2001. At 72 tons, cracks appeared at the upper end of top column and penetrated about 9” downward before the ultimate failure of specimen. At 98 tons, vertical cracks were observed at unconfined face of slab. At 104.7 tons, horizontal cracks appeared at upper most tie of top column. At 107.2 tons, concrete cover spalled off between the two ties of top column. Column reinforcement between these ties buckled and concrete was crushed. Ultimately this specimen failed mainly due to the failure of top column at 107.8 tons. Although very minute vertical cracks on unconfined face of slabs were observed but slab did not fail. Through the testing, the strains in top column were greater than the strains in bottom column and slab region.

4.1.6 EB-3

This specimen was tested on 15th of March 2001. Failure was observed in three distinct stages. At 63 tons, vertical cracks appeared at the upper end of top column but these cracks could hardly penetrate 4”downward. The number of these cracks increased with the increase of load. At 66 tons, vertical cracks appeared on the unconfined face of slab and propagated through the entire thickness of slab. At the same time more vertical cracks also appeared at the upper end of top column. At 81.2 tons, slab failed on unconfined face and the cracks of slab also traveled into bottom column. Specimen failed at bottom column-slab joint. The top column of this specimen was out of plumb by one centimeter. The strain behavior of this specimen was similar to EB-1.

4.1.7 EC-1

This specimen was tested on 13th of March 2001. At 55.5 tons, vertical cracks appeared on unconfined face of slab. At 62 tons, perpendicular cracks appeared at the corners of column but these cracks never reached edge of the slab. At 67.9 tons, vertical cracks at upper end of bottom column followed by horizontal cracks at upper tie of bottom column also appeared. The column reinforcement between slab and upper tie of bottom column buckled at ultimate load of 70.4 tons. Specimen failed at bottom column-slab joint. Upto 50 tons of load the strains in all three components of the specimens was almost same. After 50 tons of load, the strain gage readings in slab region were much greater than top and bottom column. The strain gage readings of the gages applied on the rectangular tie in the slab region suddenly dropped after the ultimate load.

4.1.8 EC-2

This specimen was tested on 14th of March. At 75.5 tons, vertical cracks appeared at the upper end of top column, which were immediately followed by hairline horizontal cracks along the upper most tie. At 86 tons vertical cracks appeared in slab on unconfined face. At 91 tons, perpendicular cracks appeared in slab on lower and upper side, which propagated immediately and reached edge of the slab. This specimen failed by top column-slab joint at ultimate load of 92 tons. The strain behavior of this specimen was similar to EC-1.

4.1.9 EC-3

This specimen was tested on 15th of March 2001. At 56 tons vertical cracks appeared in slab on unconfined side at a considerably lower load. At 64 tons, the number of these cracks increased in slab on unconfined side and also vertical cracks appeared at the upper end of top column, which propagated very little. At 67.5 tons, diagonal and perpendicular cracks appeared on upper and lower side of slab. These cracks propagated and reached the edge of slab. The vertical cracks of slab penetrated into the top and bottom column as well. This specimen failed at slab at the ultimate load of 68 tons. The strains in the slab region remained greater throughout the testing. The strain behavior of rectangular tie provided in the slab region was similar to EC-1 and EC-2.

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.

The average of the six cylinders has been used throughout the course of this presentation for analysis and other purposes (as given in Fig. 4.3). The cylinders were capped with the melted mixture of sulfur 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.

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. Some of the strain gages got damaged during casting. The reading columns, of these damaged strain gages have been left blank, in the respective tables.

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 behavior 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 behavior 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 (2,4,5) has also been included for the evaluation of the behavior of columns in presence of a weaker floor concrete layer.

5.2 Analysis

5.2.1 Kayani(5) in his research established that sec 10.15 of ACI code is not based on appropriate parameters and behavior of columns with a lower strength floor concrete layer in between, can be compared with the behavior 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:

$$f'_{cp} = 2\lambda_g \frac{f'_{cc} \times f'_{cf}}{f'_{cc} + f'_{cf}} -------------(5-1)$$

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

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

λ_g = 1.16 (for Eq. 5-1)

λ_g = 1.04 (for Eq. 5-2)

These values has been calculated from present and all the previous experimental test results.

5.2.2 The apparent concrete strength has been plotted to verify equations 5-1 and 5-2 are shown in Fig. 5.1 and 5.4. The plots of these values indicate that the current specimens also match the trends observed from earlier studies.

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

a. The provision of rectanguler tie generated a weak plane at the middle of slab portion.

b. The provision of almost 3.5% of longitudinal steel caused slab concrete to fail in most of the cases. Nevertheless, two out of nine specimens also failed at colunms and vertical cracks were observed in the columns of rest of the seven specimens. This means that for the values of steel ratios greater than 0.035, it is quite likely that specimen will not fail due to crushing of slab concrete. This concludes that excessive steel provides the doweling and confining effect to bring the weaker layer of slab at power to column concrete.

c. In general the sequence of crack initiation was as under:

(1) Vertical cracks in slab.

(2) Vertical cracks at the upper end of top column.

(3) Diagonal or horizontal cracks in slab.

5.2 Interpretation

The values of f'_{cp} in relation to the parameters used in Eq. 5-1 and 5-2 based on gross area of the section, alongwith previous test data, were ploted in Fig. 5.1 and 5.4. These values were multiplied with slab thickness, square root of slab thickness, ratio of slab thickness to least column dimension and square root of ratio of slab thickness to least columbn dimension. These values are plotted in Fig 5.2, 5.3, 5.5 and 5.6. Again these values were multiplied with ratio of slab thickness to column dimension and square root of ratio of slab thickness to column dimension. These values are plotted in Fig. 5.7, 5.8, 5.9 and 5.10. Keeping in view the scatter of test results and simplicity Fig. 5.1 is recommended. Then using the parameters of Fig. 5.1, comparison of test results with apparent concrete strength calculated by using Eq.5-1 and Eq. 5-2 is plotted in Fig. 5.11. It is quite clear from Fig. 5.11 that trend line of Eq. 5-2 is much closer to the trendline of test results. But due to simplicity Eq. 5.1 is recommended. In order to close the gap between the trend lines of test results and the trend line of values of apparent concrete strengths calculated by Eq. 5-1 it is calculated that value of λg be modified from 1 to 1.16.

5.4.1 Apparent Concrete Strength Vs Ratio Of Product And Sum of Column and Slab Concrete Strengths.

Detailed calculations of previous and present test results are calculated and 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 5.1, it can be seen that Eq. 5-1 is also valid for the current test data.

5.4.2 Square Root of apparent Concrete Strength VS Ratio of Square Roots of Product and Sum of Concrete Strengths.

The plot for the equation with square roots is given in Fig 5.4.

$$\sqrt{f'_{cp}} = 2\lambda g \frac{\sqrt{f'_{cc}} \times \sqrt{f'_{cf}}}{\sqrt{f'_{cc}} + \sqrt{f'_{cf}}}$$

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

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

(*b/ts*) ratio also seems to be a major factor in behavior of slab – column joint which should be investigated further.

Amount of longitudinal reinforcement affects the behavior 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. The calculations of apparent concrete strength based on gross area are given in a table 5.4.

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 concrete's. 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 alongwith 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 to include, for calculation of load carrying capacity of columns as under:

$$P_\circ = 0.85 f'_{cp} (A_g - A_{st}) + f_y \times A_{st}$$

Where

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

And value of λ_g is 1.16 i.e. for edge column.

b. The amount and detailing of lateral and longitudinal reinforcement can be used to improve the load carrying capacity of slab concrete, in slab-column joints.

c. Modulus of elasticity of concrete may be a function of f'cc only.

d. Ratio of least column dimension and thickness of slab, effects the behavior of joint, which may be verified.

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

6.2 Recommendations

Experimental program should be expanded to include:

a. Enhanced ranges of (*b/ts*) ratio and concrete strengths should be used in test to analyze its effect on load carrying capacity of columns specimens.

b. Confinement of high strength concrete should be studied in detail to

Propose a theoretical model.

c. 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.

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

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

f. Effects of spiral should also be studied.

LIST OF REFERENCES

1. Park, Robert and William L. Gamble, “Reinforced Concrete Slabs” *Jhon Willy & Sons*, 1980, 618p.
2. Bianchini, A. C., R.E. Woods, and C.E. Kesler, “Effect of Floor Concrete Strength on Column Strength” *Journal ACI*, Proc. Vol. 56, No. 11, May 1960, pp. 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, No5, May, 1991.
5. Kayani, M>K>R., “Load Transfer From High Strength Concrete Columns Through Lower Strength Concrete Slabs” *PhD. Dessertation*, University of Illinois, Urbana, Illinois, 1992.
6. Sheikh, Shamim A., and S. M. Uzumeri, “Strength and Ductility of Tied Concrete Columns” *Journal of Structural Division*, ASCE, Vol. 106, No. ST5, May, 1980, pp. 1079-1102.
7. Park, Robert, Priestley, M>J>N>, and Wayne D. Gill, “Ductility of Square-Confined Concrete Columns” *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, Sept. 1961, 54p.
9. Vellenas, J., V.V Bertero, and E.P. Popov, “Concrete Confined by Rectangular Hoops and Subjected to Axial Loads” *Report No. UCB/EERC-77/13*, Earthquake Engineering Research Center, University of Califonia, Berkeley, Aug. 1977, 105p.
10. Scott, B.D., R. Park, and M.J.N. Priestly, “Stress-Strain Behavior of Concrete Confined by Overlapping Hoops at 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 Canterbury, Christchurch, Feb. 1984, 442p.
12. Sheikh, Shamim A., and S.M. Uzumeri, “Analytical Model for Concrete Confinement in Tied Columns” *Journal of Structural Division,* ASCE, Vol. 108, No. ST12; Dec., 1982, pp 2703-2722.
13. Kent., D.C., and R. Park, “Flexural Members with Confined Concrete” *Journal of Structural Division,* ASCE, Vol. 97, No. ST7, Jul., 1971, pp1969-1990.
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.