LOAD TRANSFER FROM HIGH

STRENGTH CONCRETE COLUMNS THROUGH

ORDINARY STRENGTH CONCRETE SLABS

Thesis

Of

Master of Science in Civil Engineering

By

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MILITARY COLLEGE OF ENGINEERING, RISALPUR

NATIONAL UNIVERSITY OF SCIENCES & TECHNOLOGY

2001

This is to certify that the

Thesis entitled

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

Submitted By

MAJOR EHSAN ULLAH KHAN

Has been accepted towards the partial fulfillment

Of

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CHAPTER 1

INTRODUCTION

1.1 General

The progress of human development has often been reckoned in terms of the material used by society, e.g the Stone Age in the prehistoric period and than progressing through bronze, iron, and successively more sophisticated materials and their combination in this modern age. Early people used material largely as they were produced by nature, but today we have come to rely increasingly on engineered materials.

Most engineering designs involve selection and manipulation of materials, and many of these designs are largely or partly mechanical. They require that the resulting structure support applied loads without fracture or excessive deformation. This requires that the designer be able to determine the magnitude and direction of internal forces that may cause rupture or slippage of molecular bonds and to provide enough material of suitable strength to ensure that these events do not occur. As the mechanics of materials is studied, it must be kept mind that real designs must satisfy a number of criteria in addition to mechanical reliability. Cost has always been very important because material cost money, and the designer must use only enough material to satisfy the strength and serviceability requirements. At its core, however, the design problem for load bearing structures involves ensuring the mechanical integrity of the material. Analysis or design problems in the mechanics of materials generally involves two major areas:

a. Determination of internal forces setup within the materials by loads or displacements imposed on it. This is a largely mathematical undertaking, termed stress analysis. These internal forces are often independent of the choice of material used in the structure, and are often possible to carry out this analysis without much specific knowledge of the material itself. This the mechanics in “mechanics of material.”

b. Understanding the material’s response to these internal forces. Material may stretch or distort, this deformation may be reversible or permanent, or the material may fracture in any of several ways. This part of the problem is most certainly materials-specific; it is the materials in “mechanics of materials.”

The use of high-strength concrete is a common feature in tall building construction in the advanced countries of the world. Substantial economies can be achieved in construction by using high strength concrete. 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 more 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 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.

1.2 **Background**

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 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.

The second study on the subject was also carried out 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" slab for all other specimens. All columns were 10" square and were reinforced with 4#6 bars and had ¼ " 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 extended the range 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 is 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 (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. 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 **Objectives and Scope**

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. The main object of the experimental program reported herein was to understand the load transfer mechanism of a high strength concrete column through a layer of lower strength slab concretes when loaded vertically in compression. It was also intended to determine the effects of confinement on the behavior of slab concrete. These objectives lead to design of specimens in the manner given below.

a. Twelve sandwich 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. Except three straight columns of 34.5-inch height which have no slab intervening.

b. The typical layout of the test specimen is shown in Fig. 1.1. 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: -

    (1) Behavior of the sandwich column specimens under axial compressive loads with varying ratios of slab thickness and slab confinement.

(2) Development of the relationship for estimating the load carrying capacity of the column.

1.5 **Experimental Program**

A total of twelve specimens representing sandwiched columns were cast and tested. Details are as under:-

a. Out of which nine specimens consisted of a bottom column, slab portion and a top column as shown in the fig 1.1. All columns were 6” square and 15” long. Details are asunder: -

(1) Three specimens had 3” slab thickness, which gave specimen a total height of 33”. No tie in the slab region.

(2) Three specimens had 4.5” slab thickness, which gave specimen a total height of 34.5”. No tie in the slab region.

(3) Three specimen had 4.5” slab thickness giving specimen a total height of 34.5” with an additional tie in the slab region in order to increase the confinement of slab

b. Three specimens were cast as 34.5” long columns with no slab intervening.

c. In all the specimens longitudinal reinforcement was provided of 4 #5 bars of grade 60 steel.

d. Each column had two rectangular #3 ties. First tie at 3” from the end and second one at 6” from first tie.

e. Three Strain gages were placed on each diagonally opposite longitudinal bars. That makes a total of six strain gages in a specimen.

f. Specimens having tie in the slab region were provided two additional strain gages on the tie.

SPECIMENS, MATERIALS, AND FABRICATION

2.1 Description of Specimens

In this study 12 sandwich column specimens with 6"×6" column section having a total length of 33-inch, and 34.5-inch including a slab concrete layer 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. This was primarily done to keep the vertical alignment of the longitudinal bars. 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 same 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 a steel plate 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.

2.2 Categories of Specimens

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

2.2.1 Category SA

The sandwich concrete layer of ordinary strength concrete was 3" thick. These specimens contained 4, #5 longitudinal steel bars. Two double rectangular ties, starting at 3" spacing from end of the column and 6” center to center, were provided in both the columns. No tie was provided at the center of the slab. Total height of the specimen was 33“.

2.2.2 **Category SB**

The sandwich concrete layer of ordinary strength concrete was 4.5" thick. These specimens contained 4, #5 longitudinal steel bars. Two double rectangular ties, starting at 3" spacing from end of the column and 6” center to center, were provided in both the columns. No tie was provided at the center of the slab. Total height of the specimen was 34.5“.

2.2.3 **Category SC**

An additional tie at the center of the 4.50-inch thick slab was provided and spacing in between ties in both the columns was kept at 6" c/c. Rest is same as the category SB.

2.2.4 **Category SD**

No intervening slab was provided in this category. These specimens contained 4, #5 longitudinal steel bars. Spacing between the ties was kept 6”. Total height of the specimen was 34.5“.

Details of categories SA, SB, SC, and SD are given in Fig. 2.1-2.2.

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 kg
b.	Sand	= 9.65 kg
c.	Aggregate	= 24.14 kg
d.	Water	= 4.83 kg
e.	Superplasticizer	=7724 gm

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

a.	Cement	= 32.9 %
b.	Sand	= 16.44 %
c.	Aggregate	= 41.12 %
d.	Water	= 8.23 %
e.	Superplasticizer	= 1.32 %

Specimens of each category were cast with the above mix. Six cylinders were cast from each batch of concrete. These cylinders were tested for compressive strength at 28 days and on the test date.

2.4. **Floor Concrete Mix**

a.	Cement	= 10 %
b.	Sand	= 30 %
c.	Aggregate	= 60 %
d.	W/C Ratio	= 0.8

The entire casting of slab portions with six cylinders was completed from one batch of concrete mix.

2.5 **Designation of the Test Specimens**

The specimens are designated as SA-1, SA-2, SA-3, SB-1, SB-2, SB-3, SC-1, SC-2, SC-3, SD-1, SD-2, and SD-3. The first alphabet indicates the type of specimen where as the second alphabet is for category to which it belongs-.

2.6 **Materials**

2.6.1 **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 11.36, abrasion value of 13.4% and impact value of 8.33%.

c. **Fine Aggregate**. The fines consisted of sand with a fineness modulus of 3.28.

d. **Admixtures**. Polymer type dispersion was used as admixture.

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

2.6.2 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.

2.6.3 Reinforcing Steel.

The reinforcement mainly consisted of deformed # 5 and # 3 bars. Grade 6o # 5 bars were used for longitudinal reinforcement and for laterally tying the longitudinal bars #3 bars were used.

2.7 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″ thick properly seasoned deodar and partal wooden planks were used for this purpose. The specimens were cast in upright position. The steel reinforcements were welded to 1/2″ steel plate as shown in Fig 2.5. 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 three 15 “ tall columns (upper or lower) and the associated 4” 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 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 of the slab, and pouring of the slab concrete. Six control cylinders were also cast from 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.

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. Specimens in all the four categories had the strain gages on the two diagonally opposite longitudinal bars both in the columns and slab portions. Only in category “SC” specimens two additional strain gages were applied at the central tie in slab portion. The details of the location of the strain gages in different specimens are given in Fig. 2.6 and 2.7.

The deformations were filed 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.

CHAPTER 3

INSTRUMENTATION, TEST SETUP, AND TESTING PROCEDURE

3.1 Instrumentation

Strain gages were applied to the column reinforcing bars in all specimens. 8 strain gages were applied in each specimen, 2 each in the top column, bottom column, and slab portion on diagonally opposite reinforcing bars. Two strain gages were applied on the tie placed in the slab region. The locations of strain gages are shown in Fig. 2.7 to 2.8.

The electrical strain gages were of the foil type EA series gages. This series is a general-purpose family of constant 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 % strain for 0.240"(6mm*)-gage* length. These gages had a resistance of 120.0 ± 0.3 % ohms at 24 ^{o}C and gage factor of 2.060± 0.5 % at 24 ^{o}C.

3.2 Test Setup

The specimens were subjected to the axial compression in steps, 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 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 Sandwich 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 in at the rate of about 10 to 15 ton. On appearance of cracks or any unusual change the load was noted and observations recorded. The cracks were marked along with the load readings on the specimens and updated after each load increment. Strain gage readings were also recorded after each increment of 5 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 20-30 minutes to its completion.

EXPERIMENTAL RESULTS

4.1 Behavior Of Specimens

Tied columns with substantial lateral reinforcement and appropriate detailing fail 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 the load bearing area. In final stage, the core concrete, due to the confinement by the 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 specimen failed due to the crushing of slab concrete immediately followed by buckling of steel in the slab region. In one case the specimen failed due to the failure of top column and buckling of steel in the same region.

The sandwiched concrete layer used in different categories of the present program demonstrated no significant change till ultimate failure of specimen or slightly before that. Eight out of nine specimens failed at the slab-column joint regions where as one specimen failed due to crushing of top column concrete.

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

4.1.1 Specimen SA-1

The specimen was tested on 14 Mar 2001. The load was applied at the rate of about 5 to 10 tons per minute. Formation of vertical cracks started in the slab region at 105 tons (6.53ksi). Stress strain curve (FIG. 4.4) also indicates a substantial increase in strains in the slab region at the same load. Cracks propagated through upper column without showing any distresses in the bottom column. Spalling of concrete cover immediately followed stage 1. There was not much of difference between stage 1 and ultimate failure. Specimen failed from the slab region at 107.8 tons.

4.1.2 Specimen SA-2

The specimen was tested on 14 Mar 2001. The load was applied at the rate of about 5 to 10 tons per minute. Longitudinal cracks appeared in the top column and slab simultaneously at 91 tons (5.66 ksi). stress strain curve (FIG.4.5) also indicates a marked increase in the strains in slab and top column region at about 5 to 6 ksi of stress. Upper portion of bottom column showed some signs of distresses at 104 tons (6.48 ksi) in the form cracks. The specimen ultimately failed from slab region and top column at 109 tons (6.8 ksi)

4.1.3 **Specimen SA-3**

This specimen was subjected to testing on 14 Mar 2001. Two strain gages were out of order. The load was applied at the rate of about 5 to 10 tons per minute. At 57 tons (3.55 ksi) cracks at the upper end of the top column appeared which is clear from the stress strain curve (FIG 4.6) as well. At 64 tons (3.99 ksi) cracks at the lower end of the top column appeared. At 99.7 tons (6.20 ksi) cracks appeared in the slab portion. At 102.8 tons (6.4 ksi) specimen experienced reversal of strains in the slab region and it ultimately failed by top column and slab at the same load.

4.1.3 **Specimen SB-1**

The specimen was tested on 14 Mar 2001. One strain gage slab region was not in proper working condition at the time of testing. The load was applied at the rate of 5 to 10 ton till the failure load was reached. At 80 tons (5 ksi) cracks started appearing in the slab region. Excessive strains in the slab region around same stress level can be seen in the stress strain curve (FIG 4.7). At 99 tons (6.2 ksi) more vertical cracks appeared in the slab region and lower end of the bottom column. Bottom column cracks did not propagate and ultimately specimen failed in the slab region at ultimate load of 102.7 ton (6.4 ksi). Stress and strain relaxation in the top and bottom column (FIG 4.7) clearly indicates that the failure was solely from slab region and top and bottom column were intact.

4.1.5 **Specimen SB-2**

Testing of specimen started on 14 Mar 2001. With all the six strain gages in proper working conditions. Specimen was out of plum towards one side. Initiation of the cracks started from slab at (6.4 ksi) 102.3 tonsFIG 4.8). The failure plain formed at the center of slab portion opposite to the side of inclination. Failure of specimen was due to sudden bursting of concrete in slab portion, with almost negligible amount of distress in top and bottom column. Specimen failed at ultimate load of 106 tons

4.1.6 **Specimen SB-3**

Testing of specimen was on 14 Mar 2001. With all the six strain gages in proper working conditions. Initiation of the cracks was from top column upper portion and slab region at 107 tons. Slab portion cracks propagated very rapidly. The failure plain formed at the center of slab portion at 109.7 tons. Failure of the specimen was due to crushing of concrete in slab portion, with almost negligible amount of distress in top and bottom column. Specimen failed at ultimate load of 109.7 tons.

4.1.7 **Specimen SC-1**

The specimen was tested on 15 Mar 2001. Two out of the eight strain gages did not function at the time of testing. No signs of distress at the initial stages of loading. At 90 tons of load vertical cracks appeared in the slab region, which immediately propagated into the lower end of the top column. At 97 tons of load the specimen failed due to the failure of slab. 7” deep crack in bottom column upper portion was also observed at the time of ultimate failure. Longitudinal steel buckled in the slab and upper end of the lower column.

4.1.8 **Specimen SC-2**

Testing was on 15 March 2001. Two out of eight gages were not working. The load was applied at the rate of 5 to 10 ton. Formation of cracks initiated in slab at 94 tons and started propagating in the top column. Cracks appeared in the lower end of the top column at 98 tons of load. The cracks progressed vertically downwards to the slab portion. Failure was due to sudden bursting of slab concrete at ultimate load of 100.2 tons.

4.1.9 **Specimen SC-3**

The specimen was subjected to testing on 15 Mar 99. All the eight gages were working properly. The load was applied at the rate of 5 to 10 tons. At 94 ton of load cracks started appearing in slab portion. These cracks progressed vertically upwards with the increase in load. The ultimate failure took place at 101.6 tons of load due to crushing of slab concrete above the tie, provided in the center of slab.

4.1.10 **Specimen SD-1**

The specimen was tested on 15 Mar 2001. One gage was not working properly. The specimen was a straight column without any slab. The load was applied at the rate of about 5 to 10 tons. At 43.2 tons of load longitudinal cracks appeared at the top. At 81 tons again crack appeared at about 2” from the top. Column failed at 116.6 from upper half of the column. Longitudinal steel also buckled at the same place.

4.1.11 **Specimen SD-2**

The specimen was tested on 15 Mar 2001. All the six gages were in proper working condition. The load was increased at the rate of about 5 to 10 tons. At 87 tons vertical cracks appeared at the upper portion of the column. At

98 tons same type of cracks appeared in the lower portion of upper half. At 108 tons of load vertical cracks appeared at the lower end of the column and the cracks propagated quickly upwards. At 118 more cracks appeared in the lower end and ultimately the column failed at the load of 122.6 tons from lower end of the column.

4.1.12 **Specimen SD-3**

The specimen was subjected to loading on 15 Mar 2001. The strain readings revealed that one gage was not in proper working condition. The load was increased at the rate of about 5 to 10 tons. Longitudinal cracks appeared at the upper end at 105 tons of load. At 113 tons lower end of the specimen also showed signs of cracking. At 125 tons more cracks appeared at the upper end, which started propagating towards the mid portion. Finally the specimen failed at an ultimate load of 134.4 tons.

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 Table 4.1 and 4.2 respectively.

From the results of testing of these cylinders it was observed that the cylinder strengths varied a lot 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 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. 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 – 4.15. The strain values for each gage have been plotted against the load in Figs. 4.4 – 4.15. 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. Almost all the strain gages remained in proper working condition except in few specimens. The general trend of the behavior of the specimen 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 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 twelve 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**

The main objective of this research investigation is to study and evaluate the structural behavior and current design code procedure for slab-column connections constructed with combination of high-strength concrete columns and ordinary-strength concrete slabs.

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:

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

Or

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

Where λ_Γ is constant whose value depends upon the location of the column in the structure. The value assigned to this constant for sandwich/corner columns is:

λ_G = 0.90 (for Eq. 5-1)

λ_G = 0.95 (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 -5.2. The plots of these values indicate that the current specimens match 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 the current experimental program, was due to crushing of concrete in slab portion. Only one of the total specimens exhibited failure of specimens due to crushing of column concrete which indicates that slab concrete did not fail although being of lower strength.

b. The amounts of longitudinal and lateral steel and their distribution was appropriate enough to let the slab behave as envisaged.

c. The percentage of longitudinal steel and the arrangement of lateral steel viz-a-viz the column dimensions provided a weak plane along the edge of the rectangular ties. This was evident from the formation of cracks in columns at the locations of rectangular ties at very high loads.

d. Cracking load of the specimens and ultimate failure loads were so closely placed that we can very conviniently use gross area of concrete for our calculations

5.3 **Interpretation**

The values of f'_{cp} of the current program, alongwith previous test data, calculated on gross area, were ploted in Fig. 5.3 and 5.4, using equations 5.1 and 5.2 .

5.4.1 **Apparent Concrete Strength Vs Ratio Of Product And Sum of Concrete Strengths.**

Detailed calculations of previous and present test results are summarized in table 5.3-5.5. This data for the above parameters has been plotted in Fig 5.3 and compared with Eq. 5.1:

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

From the plot, it can be seen that Eq. 5-1 is 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 above parameters is given in Fig 5.4 with Eq. 5-2:

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

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

5.5 we can very safely say that rectangular ties provided in the weaker portion of the slab-column joint does not improve the load carrying capacity by increasing the confinement in the classical sence. Rectangular ties in the slab region provides a weaker plane for stress concentration and instead of enhancing the load carrying capacity it tends to reduce the capacity.

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

5.7 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.

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

6.1 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. For calculation of load carrying capacity of columns ACI Sec. 10.15 should be amended, as already recommended by kayani in his study (5) as under:

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

Where

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

Where

$$\lambda G = 0.90 \;\; \text{for sandwich and corner columns}$$

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

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

d. Modulus of Elasticity of concrete may be a function ***f'c*** rather than of widely accepted square root of *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 behavior of reinforced concrete members.

6.3 **Recommendations**

Experimental program should be expanded to include:

a. Enhanced ranges of (*h/b*) 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 the presence of moments in addition to the axial loads should also be studied.

f. Effect of spirals should also be studied.

SPECIMEN PART	CYLINDER NO	DATE OF CASTING	28DAYS COMPRESSIVE STRENGTH		DAY OF TETING COMPRESSIVE STRENGTH		AVERAGE COMPRESSIVE STRENGTH OF 28 DAY AND DAY OF TESTING	
			psi	ksi	psi	ksi	psi	ksi
TOP COLUMN	1	7/2/200	7515.51	7.52	7786.07	7.78	7771.04	7.77
	2	7/2/200	7695.88	7.69	7876.26	7.87		
	3	7/2/200	7786.07	7.78	7966.45	7.96		
	AVERAGE		7665.82	7.66	7876.26	7.87		
SLAB	1	4/2/2001	2924.22	2.92	2924.22	2.92	2915.24	2.91
	2	4/2/2001	2655.05	2.65	3103.66	3.10		
	3	4/2/2001	2834.49	2.83	3049.83	3.04		
	AVERAGE		2804.58	2.80	3025.90	3.02		
BOTTOM COLUMN	1	1/2/2001	8507.58	8.51	9138.89	9.13	8861.88	8.86
	2	1/2/2001	8237.01	8.23	9319.27	9.31		
	3	1/2/2001	8597.78	8.59	9370.80	9.37		
	AVERAGE		8447.45	8.44	9276.32	9.27		
AVERAGE COMPRESSIVE STRENGTH OF TOP AND BOTTOM COLUMN							8316.46	8.316

Table 4.1 Average Compressive Strength of concrete cylinders

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