SLAB-COLUMN CONNECTIONS
WITH MISPLACED REINFORCEMENT

by

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TO MY PARENTS & ELKE

ABSTRACT

This thesis discusses the structural performance of slab-column connections containing misplaced top reinforcement before and after repair. A series of shear and moment transfer tests were performed. The effects of degree of misplacement on the cracking behaviour, load-deflection response and ultimate capacity are studied. The effectiveness of a structural topping repair method was also examined. In addition, the results of this experimental programme are compared with the companion direct punching shear tests and flexural tests conducted by Lee (see reference 13).

From the test results, it is concluded that a decrease in effective depth results in a reduction in shear and flexural capacities as well as flexural stiffness, and also an increase in crack widths. A small reduction in effective depth from the proper position would lead to a large and significant decrease in shear capacity. The structural topping repair method effectively improved the overall structural performance.

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RESUME

Cette thèse se propose d'étudier la performance structurale des joints dalle-poteau dont l'armature du haut est mal placée, avant et après la réparation. Une série d'essais sur le transfert du cisaillement et du moment fléchissant ont été effectués. Les effets du mauvais placement, à différent degrés, sur le phénomène de fissuration, sur la relation charge-déformation et sur la capacité ultime sont etudiés. L'efficacité de la méthode de réparation au moyen d'un recouvrement structural a aussi été examiné. De plus, les résultats expérimentaux de ce programme sont comparés à ceux des essais compagnons de cisaillement direct par effoncement et des essais de flexion effectués par Lee (void la référence 13).

A partir des résultats expérimentaux on conclut qu'une réduction de la hauteur efficace entraîne une réduction de la résistance à l'effort tranchant, de la résistance à la flexion et de la rigidité en flexion, ainsi qu'une augmentation de la largeur des fissurations. Une petite réduction de la hauteur efficace peut entraîner une importante diminution de la résistance au cisaillement. La méthode de réparation au moyen d'un recouvrement structural a efficacement amelioré le comportement structural dans son ensemble.

ACKNOWLEDGEMENTS

The author would like to take this opportunity to express his gratitude to his research supervisor, Prof. P.J. Harris and his internal advisor, Prof. D. Mitchell for their guidance and encouragement during the research work. Their careful review of the entire thesis and making precious suggestions and criticisms are deeply appreciated.

Thanks are extended to the laboratory technicians, Brian and Ron, for their enormous help in constructing the test specimens.

Also, special thanks to my friends Anna Lau and Joseph Fung for typing the manuscript.

LIST OF FIGURES

LIST OF TABLES

TABLE OF CONTENTS

CHAPTER 1

INTRODUCTION

It is well known that the effective depth of reinforcement is one of the major parameters which significantly affects the overall structural performance of a slab system. A decrease in the effective depth due to misplaced reinforcement could not only lead to possible serviceability and durability problems, but also, to a reduction of the flexural, one way shear and punching shear strengths. There have been many examples of unacceptable serviceability, local failures and even total structural collapses due to the effects caused by misplaced reinforcement. A very recent collapse of a five-storey, flat-plate structure in Florida in which misplaced reinforcement contributed to the failure is described in references 14 and 15. Since punching shear failures are brittle and have led to several collapses, it is particularly vital to gain more knowledge of the effect of misplaced reinforcement on the punching shear strength of slab-column connections.

Although the ACI[2] and CSA[9] codes give tolerances of reinforcement placement of the order of ±8 mm for usual slab thickness, it has been found[16] from field investigations that the average variation of the insitu effective depth measured to the top bars is of the order of ±10 mm. The engineer does not usually directly account for such a variation of effective depth in the design of slabs.

The engineer may be confronted with a situation where reinforcement has been misplaced, that is, the variation of effective depth is beyond the code tolerances. If the top reinforcement in the column region is misplaced such that a significant reduction of effective depth results, then the engineer must determine the effect of the misplacement on both the flexural and punching shear capacities of the slab.

The present codes (References 2 and 9) provide a rational and accurate method for predicting flexural strength which can easily be used to also predict the effects of misplacement on the flexural capacity, the punching shear capacity however can only be calculated by empirical equations that may not be accurate when extended to predict the punching shear strength of slab-column connections containing misplaced steel.

In order to provide designers with some guidance for determining the effect of misplaced reinforcement on the serviceability and strength as well as offering guidance on the repair of such slabs, experimental programmes were conducted at McGill University. Previous research work carried out by Lee[13] had studied the behaviour and repair of slabs containing improperly placed reinforcement. A series of flexural and punching shear tests were performed. In addition, the effectiveness of two repair methods, one with a structural topping and one with extra bars epoxied into grooves cut in the top surface of the slab were examined.

The research work presented in this thesis is a continuation of Lee's work with a view to studying the behaviour of slab-column connections transferring both shear and moment and having various degree of misplacement of reinforcement. The effect of varying the effective depth of top reinforcement on the safety and serviceability of slabs is examined. One of the specimens containing misplaced top reinforcement was repaired with a structural topping in order to investigate the effects of this repair on the structural performance. The test results of this investigation are compared with Lee's results in order to study the effects of degree of misplacement and the effects of repair on the behaviour of slab-column connections subjected to shear and shear combined with moment transfer.

CHAPTER 2

CRITICAL STAGES OF SLABS DURING AND AFTER CONSTRUCTION

This chapter discusses the different loading conditions imposed on a slab structure during the construction stages and during the service life of the structure.

2.1 Design Loading Conditions of the Slab During Service

During service, the loads superimposed on a slab are assumed to be its service live and dead load. With the multiplication of each service load by appropriate load factors, the design load is obtained. As specified in section 9.2 of ACI 318-77 Code[2], this load is equal to 1.4 times the dead load plus 1.7 times the live load.

In designing slab-column connections, the ACI 318-77 slab provisions require a consideration of both uniform and pattern gravity loads. For most practical situations the effects of factored service pattern loading is less critical for interior columns than for exterior columns. For interior columns, uniform loading, that is direct punching shear usually governs the design. Slabs are usually designed for the factored service loads without considering the loads imposed during the construction stages.

2.2 Estimated Loading Conditions during Construction Stages

During construction, loads sustained by each floor slab would depend on the following factors[6] :-

(1) The number of levels of shores and reshores,

(2) The construction cycle,

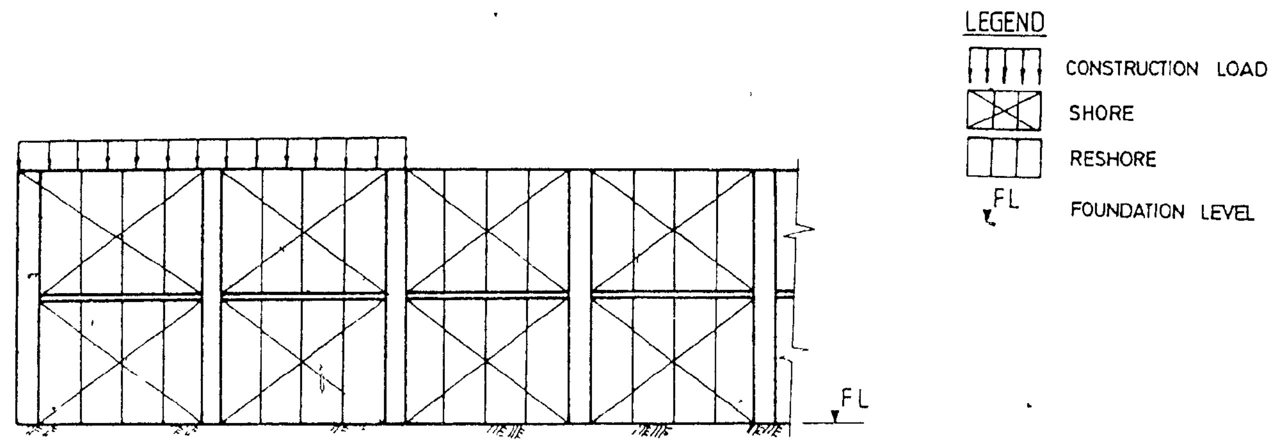
(3) The relative stiffness

and (4) The rate of strength gain of the concrete with age, temperature and humidity.

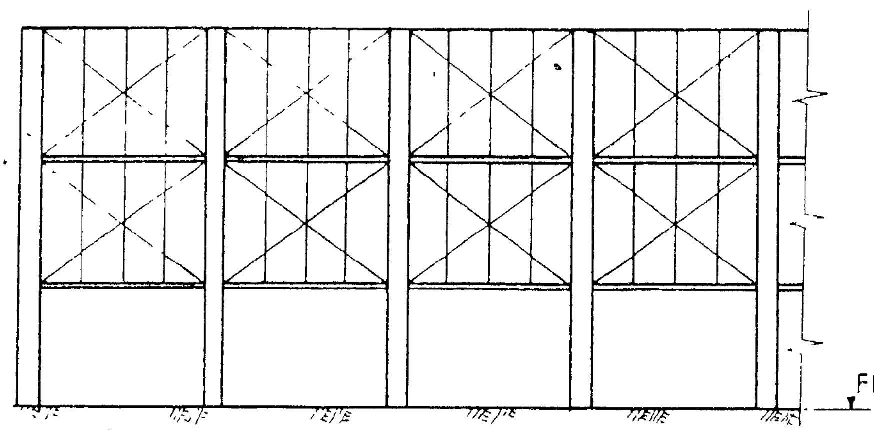
In order to simplify the problem of estimating the construction loads acting on each floor, different construction stages of shoring are described in the next Sections. Reference 6 provides a summary of studies of flat slab structures during construction.

2.2.1 Stage 1 - Loads Transmitted Directly to the Foundation Through Shores

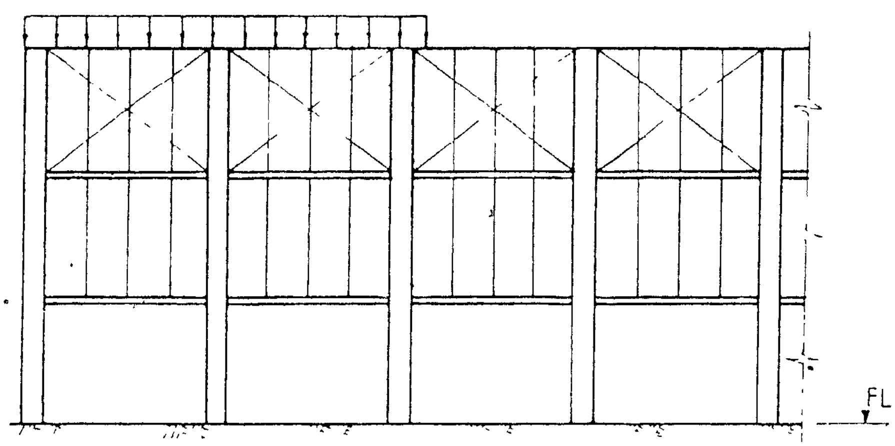
Fig. 2.1a shows the first construction stage which involves provision of shores that support all the finished floors. The total construction loads including the weight of each slab are directly transmitted to the foundation through the shores. At this stage, slabs are subjected to virtually no loading.



(a) Stage 1 - Loads Transmitted Directly to the Foundation Through Shores



(b) Stage 2 - Removal of First Floor Shores



(c) Stage 3 - Typical Upper Floor Shores and Reshores

Fig. 2.1 - Different Construction Stages of Shoring

2.2.2 Stage 2 - Removal of First Floor Shores

After the slabs have gained sufficient strength, the simultaneous removal of the shores in the first floor requires that each individual floor slab carry approximately its own dead weight (see Fig. 2.1b). If shores of one span are removed well before those in adjacent spans, this can cause significant unbalanced moment. The loading induced from such an operation could be damaging. This could be a more serious problem if the slabs contain misplaced reinforcement.

2.2.3 Stage 3 - Typical Upper Floor Shores and Reshores

The third construction stage, as shown in Fig. 2.1c has the shores and reshores in the first storey removed, while concrete is being placed on the top floor. At this stage, the construction loads are distributed through shores and reshores and shared between a number of supporting floors in direct proportional to their stiffnesses.

Current construction practice consists of a sequence of concrete placement which usually progresses from one side of the structure to the other side. As illustrated in Fig. 2.1c, this will cause significant moments to be transferred at the slab-column connections along the column line at the loading edge of the concrete placement. As the placement proceeds, the load pattern changes causing direct punching shear at the connection which previously had to transfer moment.

Unlike the pattern gravity loading at the service stage as described earlier, a study using simplified analyses[12] of loads on slabs has revealed that the effects of the construction stage loading can be more critical than the factored service loading. This is particularly true for flat plate structures which commonly have relatively low live load/dead load ratios. The pattern loading caused by the concrete placing sequence is usually ignored. In fact the shoring and reshoring is usually designed assuming a uniform gravity loading during construction.

2.2.4 Estimated Maximum Construction Load

In order to get a reasonable estimation of the maximum loading on the supporting slabs during construction, the design requirements for a number of levels of shoring and reshoring are considered. The formwork standards[4] reported by ACI committee 347 recommend that the maximum load imposed on a slab during construction be equal to or less than the slab capacity as determined by the design load and the concrete strength at the time of stripping and shoring. Actual field measurements during construction together with analyses[6] indicate that even if proper shoring is provided, the construction loads imposed on a floor slab may appreciably exceed the design loads (i.e. the factored load for which the slab is designed). This increased load during the construction stages together with the reduced capacity of the slabs due to partial curing of the concrete during different stages of construction often produce the most critical design stage in the life of a slab structure.

2.3 Shear Strength of Slabs Designed Using ACI 318-77

The above studies predict that many slabs would fail during construction. However, it is observed in practice that, in fact, they do not fail nor do they show signs of severe damage. This can be explained by the fact that the calculated ultimate shear strength of a slab-column connection using the empirical design equations given in ACI 318-77 is not necessarily a good indication of the actual capacity of the connection. The design equation, given by both ACI 318-71[1] and ACI 318-77[2] for calculating the limiting shear stress at a distance d/2 away from the column face was chosen to provide a lower bound[10,11] for the scattered test results and a lower bound for large ratios of column dimension to effective depth, c/d. Fig. 2.2 shows the calculated nominal shear stress at failure for a large number of experimental results. This figure is taken from reference 7, in which research concerning the shear strength of slabs is summarized. It is also worth noting that most of the results falling below the ACI 318-71 provision are for specimens with high concrete strengths, ring reinforcement only, or a very low percentage of reinforcement[7]. For typical slab systems the inherent conservatism[6] contained in the ACI 318-77 expression is estimated to be at least 1.4.

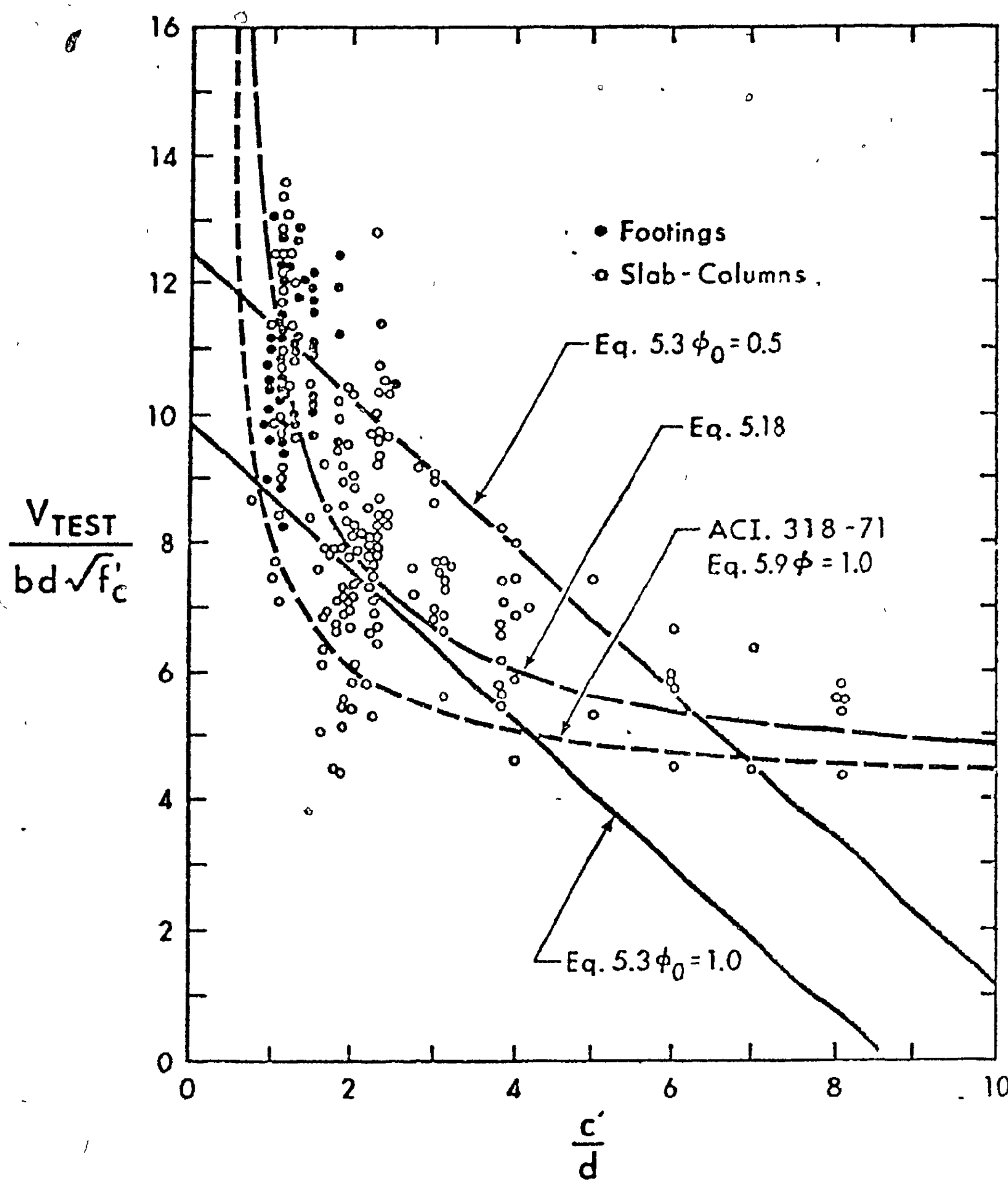
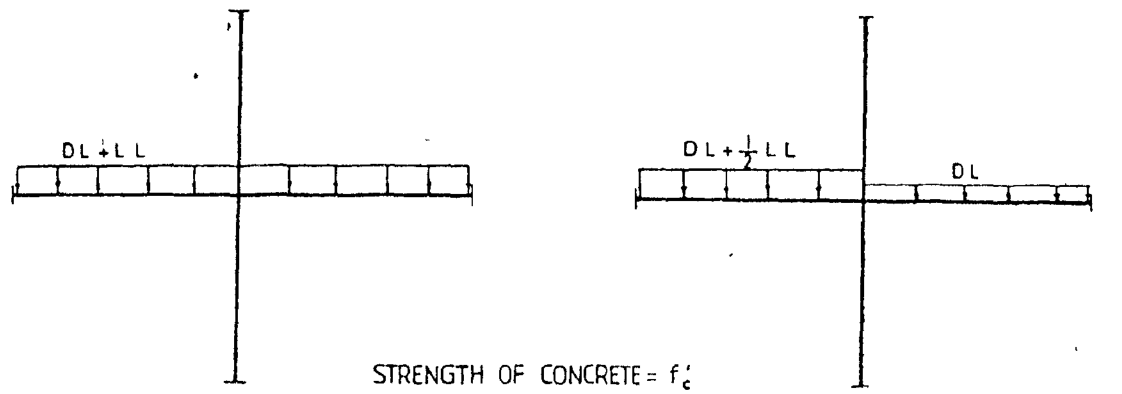


Fig. 2.2 - Relationship Between $V_{TEST}/bd\sqrt{f'_c}$ and c'/d for Specimens Loaded Through Square or Circular Areas (from reference 7)

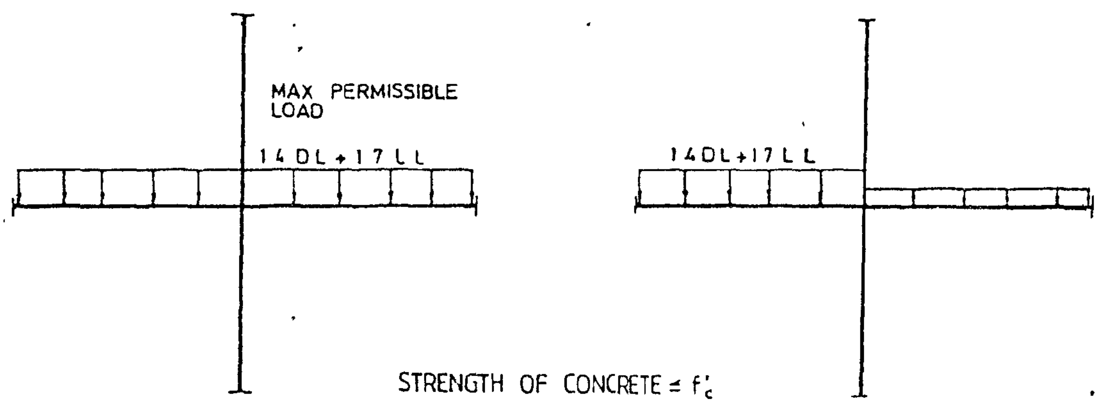
2.4 Effect of Construction Loading on Slabs with Misplaced Reinforcement

With this inherent extra "safety factor", typical slab systems which are properly designed and well constructed would rarely have structural strength problems. However, for slabs containing improperly placed top reinforcement, the resulting reduction of shear strength could partially or totally eliminate this inherent "safety factor". This misplacement could therefore lead to structural demage or collapse.

Fig. 2.3 in comparing some possible loading conditions acting on typical slabs emphasizes the severity of the construction loads. The severity of the construction loading is aggravated by the rapid construction techniques which lead to early removal of shores and reshores, and often lack proper inspection before placing concrete and during the construction stages. If in addition to these problems the reinforcement in the slab is misplaced then the consequences could be disastrous. In particular if the effective depth of the top reinforcement is reduced then a brittle punching shear failure may occur.



(a) Service Stage



(b) Construction Stage

Fig. 2.3 - Comparison of Possible Loading Conditions at Service Stage and Construction Stage

The early signs of misplaced top reinforcement which usually appear during construction are significant sagging of the slab and severe cracking around the column regions. In particular, slabs with misplaced top reinforcement exhibit severe cracks which radiate out from the column[13] on the top surface of the slab.

The loading conditions in Fig. 2.3 emphasize the need to appreciate the effects of misplacement of reinforcement on both the direct punching shear strength and on the strength of slab-column connections transferring shear and moment. It is important to realize that each slab-column connection is subjected to both types of loadings during different stages of construction and that any structural damage is accumulated as the construction progresses.

CHAPTER 3

EXPERIMENTAL PROGRAMME

This chapter describes the test specimens, the material properties, the test set-up, the instrumentation, and the loading method.

3.1 Test Specimens

The experiments consisted of a series of slab-column specimens subjected to shear and moment transfer. The first three specimens S1, S2 and S3 had identical geometry and identical amount of reinforcement within the moment transfer region with the only intended variable being the position of the top reinforcing mat in the thickness of the slab. Since these specimens were cast at different times the concrete strength varied. The slab thickness for all specimens was 150 mm.

Specimen S1 had the top reinforcement placed such that the clear concrete cover was 20 mm. This specimen was the control specimen with the steel placed in its proper location and having a concrete cover corresponding to the specified cover in the code[9]. Specimens S2 and S3 had the top steel mats placed such that they had concrete clear cover of 65 mm and 90 mm respectively. These two specimens represented slab-column connections having different degrees of misplacement.

Specimen RS2, in its unrepaired condition, was identical in every respect (except for the concrete strength) to specimen S2. The purpose of this specimen was to study the repair of a slab-column connection with misplaced reinforcement.

Fig. 3.1 shows the geometry of the test specimens and the test set-up. A photograph of a specimen before testing is given in Fig. 3.2. A typical specimen consisted of a 2300 mm square slab supported by a 225 mm square column. The column was hinged and laterally restrained at the top and the bottom. The columns had

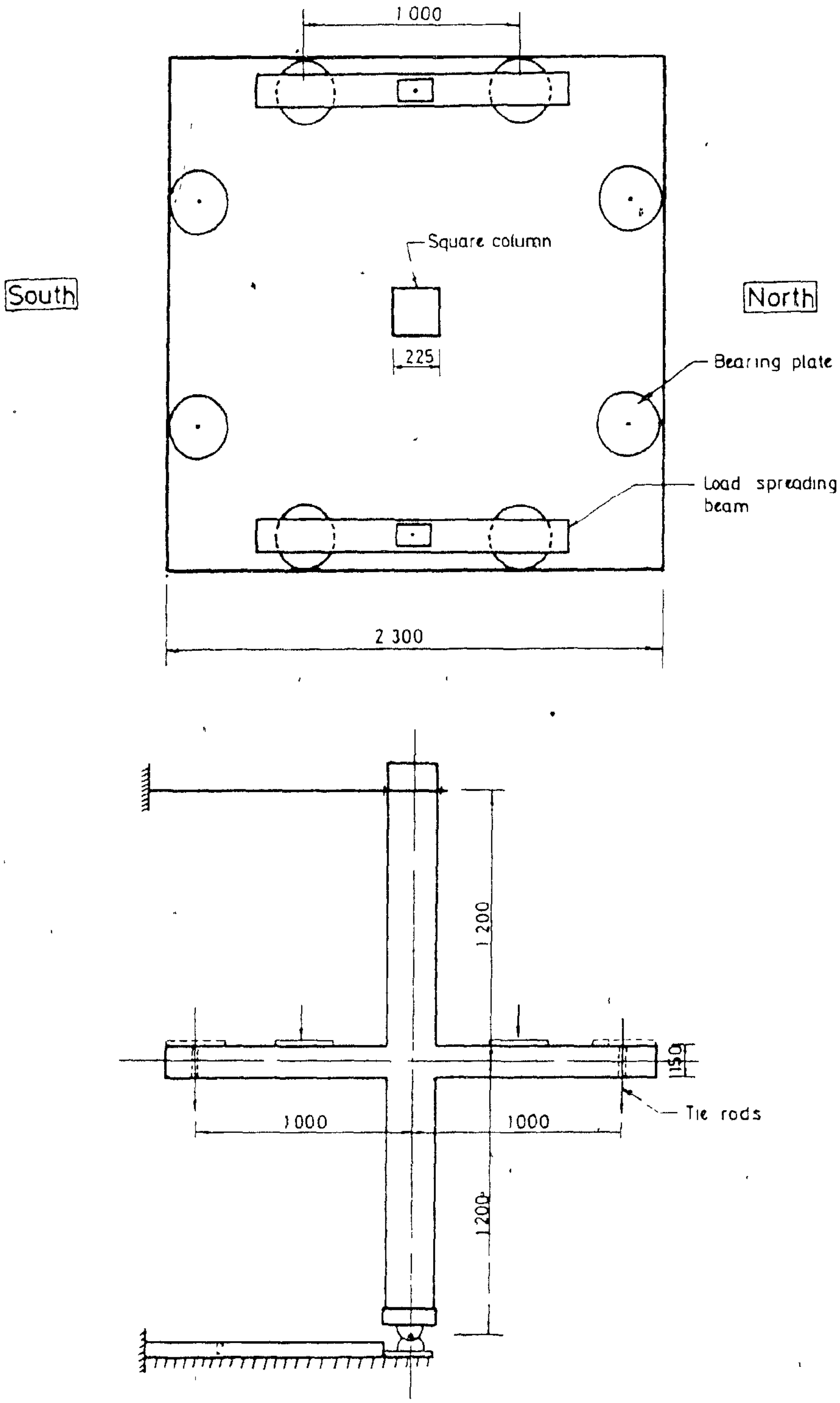


Fig. 3.1 - Shear and Moment Transfer Test Set-up

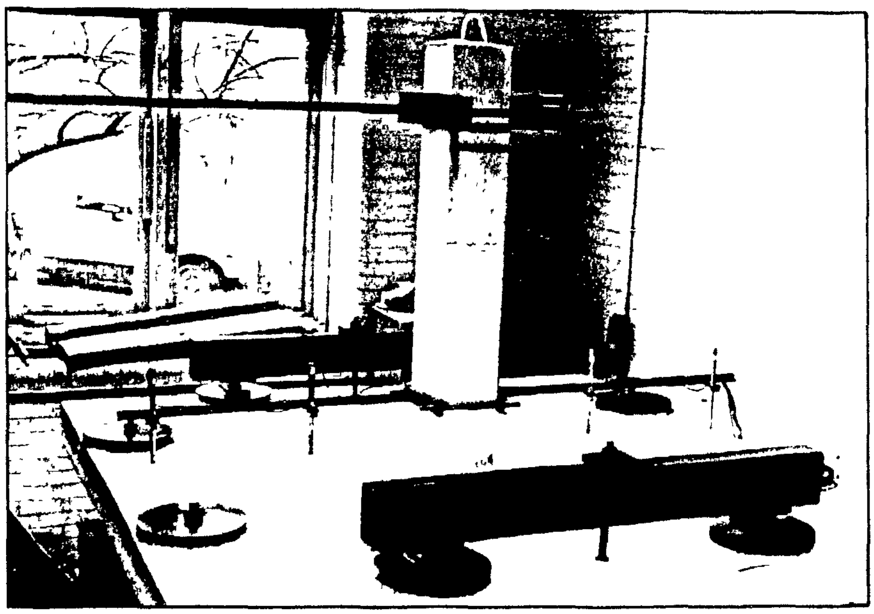


Fig. 3.2 - Specimen Before Testing

a 30 mm concrete cover and were reinforced with 8 #15M deformed bars, three on each side to resist the axial load and the bending caused by the transfer of moments through the slab-column connection. The column ties consisted of 8 mm diameter reinforcing bars. They were placed 75 mm apart outside the region of the slab. The slab was 150 mm thick, with 18 #15M reinforcing bars being placed in each direction. To allow proper transfer of moments from the slab to column, a concentration of top reinforcement in the immediate column region was provided. The details of the top reinforcement layout are given in Fig. 3.3 and 3.4. Specimens S2 and S3 had extra top reinforcement added at the edges of the slab to increase the flexural resistance of each specimen and to prevent general flexural yielding. In order to provide proper end anchorage, the top bars were welded at each end to a 50 mm x 50 mm steel angle which was embedded in the concrete around the perimeter of the slab. Fig. 3.5 shows the details of the welded connection. Properties of the test specimens are given in Table 3.1.

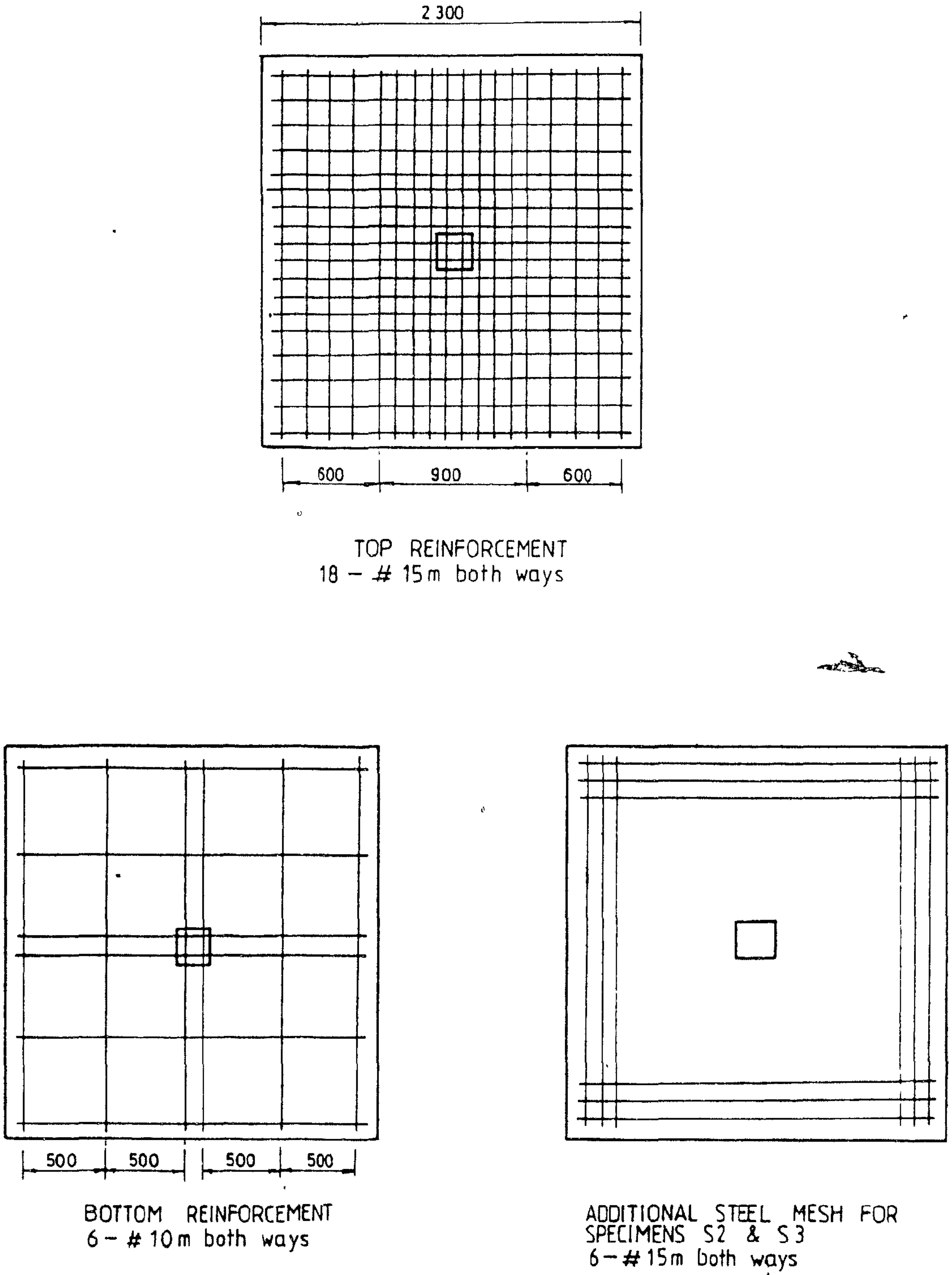


Fig. 3.3 - Reinforcement Details of Shear and Moment Transfer Test Specimens

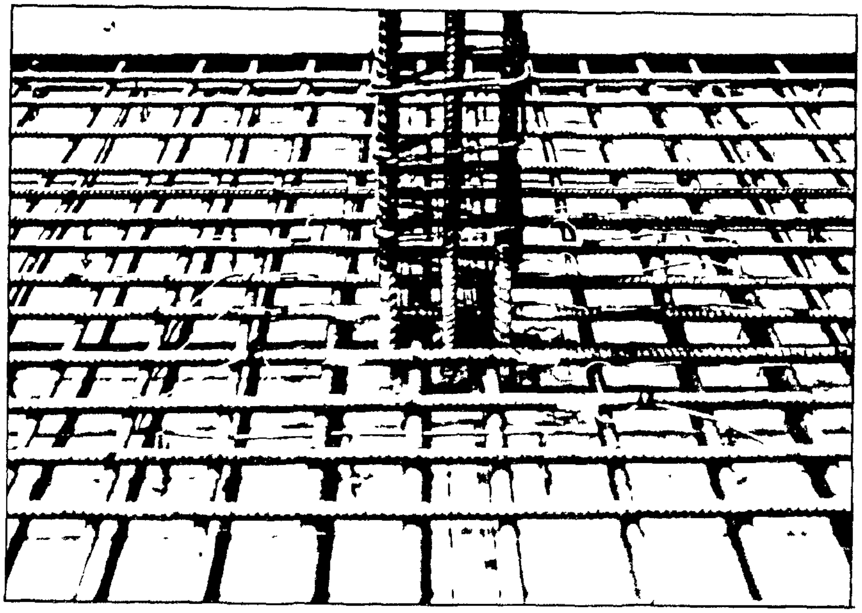


Fig. 3.4 - Layout of Top Reinforcing Bars

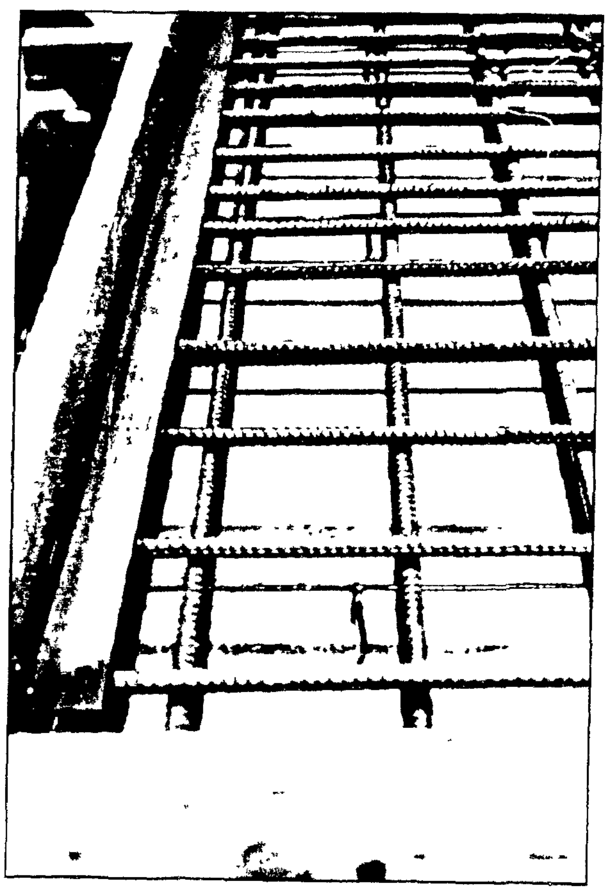


Fig. 3.5 - Top Bars Welded to Edge Steel Angle

Shear and Moment Transfer tests*							
Specimen	Cover mm	dav mm	fc' Mpa	Vn kN	Vexp kN	Vn/Vexp	Comments
S1	20	114	39.6	160.5	235.9	1.47	
S2	65	69	36.3	74.3	157.8	2.12	
S3	90	44	33.6	39.6	130.0	3.28	
RS2	65	69					unrepair condition
	20	163	35.4	268.8	276.4	1.03	'topping' repair
Punching Shear Tests (Reference 13)*							
PS1	20	114	25.0	257.0	404.4	1.57	
PS2	65	69	35.1	160.0	252.4	1.58	
PS3	90	44	39.9	99.0	248.4	2.51	
PS2AR	65	69					unrepair condition
	20	163	27.8	443.0	496.9	1.13	'topping' repair
PS2BR	65	69					unrepair condition
	6	131	25.8	304.0	394.4	1.30	'groove-epoxy' repair

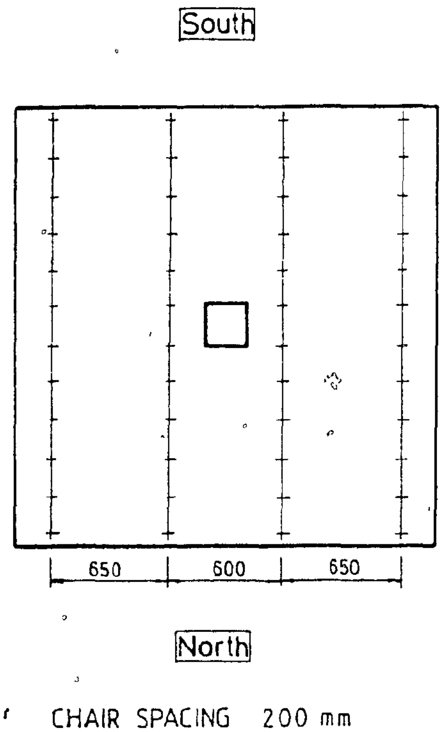
Note: For all shear and moment transfer test specimens
fy = 460 Mpa for all #15M
& fy = 410 Mpa for all #10M
For all punching shear test specimens
fy = 337 Mpa for all #15M
& fy = 503 Mpa for all #10M

* Dead weight of slab of each specimen was taken into account. It is estimated to be equal to 18.6 kN for shear and moment transfer test specimens and 11.4 kN for punching shear test specimens.

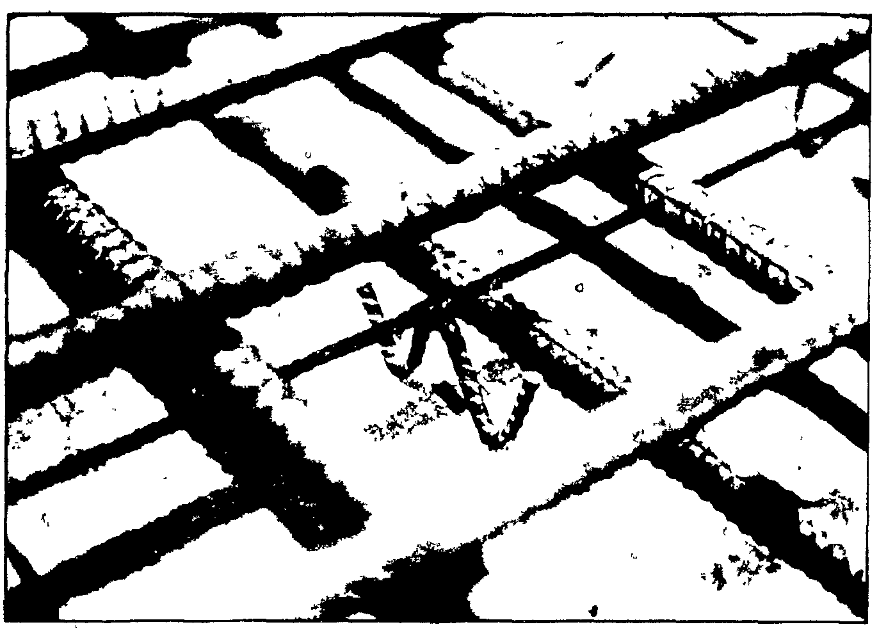
Table 3.1 - Details of Test Specimens and Comparison of Predicted and Measured Capacities

To provide proper support for the top reinforcing mat, continuous high chairs made from 4 mm diameter bars were used (see Fig. 3.6). This form of support conformed with the Concrete Reinforcing Steel Institution Standards[8]. Before the placing of the concrete and after testing, the effective depth of the principal reinforcement for each specimen was carefully measured. Due to the special care taken during the construction, the actual location of the steel bars were within 2 mm of the desired location.

The bottom reinforcing steel mat consisted of 6 #10M bars in each direction as shown in Fig. 3.3. Two of these bottom bars in each direction passed through the column. They acted as temperature and shrinkage reinforcement and also provided tension capacity on the bottom surface of the slab which is sometimes needed when large moment transfers are present.



(a) Layout of Continuous High Chairs



(b) Details of Continuous High Chairs

Fig. 3.6 - Continuous High Chairs

The slab thickness and column size as well as the percentage of reinforcement within the moment transfer region were chosen to be identical to Lee's[13] test specimens. This was done in order to make a comparison of both sets of experimental results possible.

3.2 Loading Arrangement

Loads were applied by means of hydraulic jacks placed below the strong floor. To control, measure and apply the loads, three hydraulic pumps (P1, P2 and P3), four load cells (C1, C2, C3 and C4) and six jacks (J1 to J6) were used. They were arranged as shown in Fig. 3.7. One pump, P1, controlled the load at the east and west sides, while the other two controlled the south and north sides. With two load spreading beams (see Fig. 3.1 and 3.2) placed across the east and west sides of the slab, a total of eight point loads were applied to the top surface of the slab. For each load step, the applied loads were identical at the south, west and east sides, while a larger load was applied at the north side. These loads were transferred by high strength steel tension rods to 300 mm diameter circular bearing plates on the top slab surface, thus producing shear and unbalanced moment around the column.

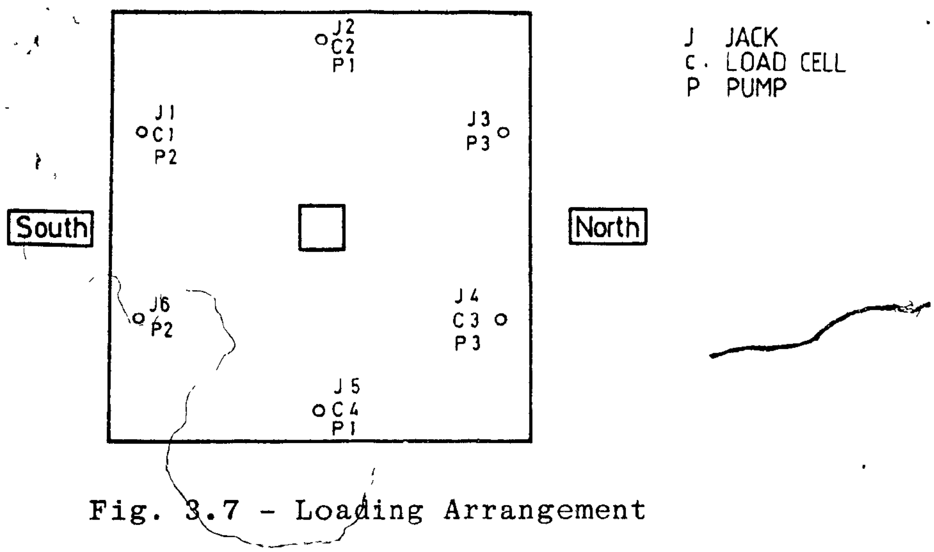


Fig. 3.7 - Loading Arrangement

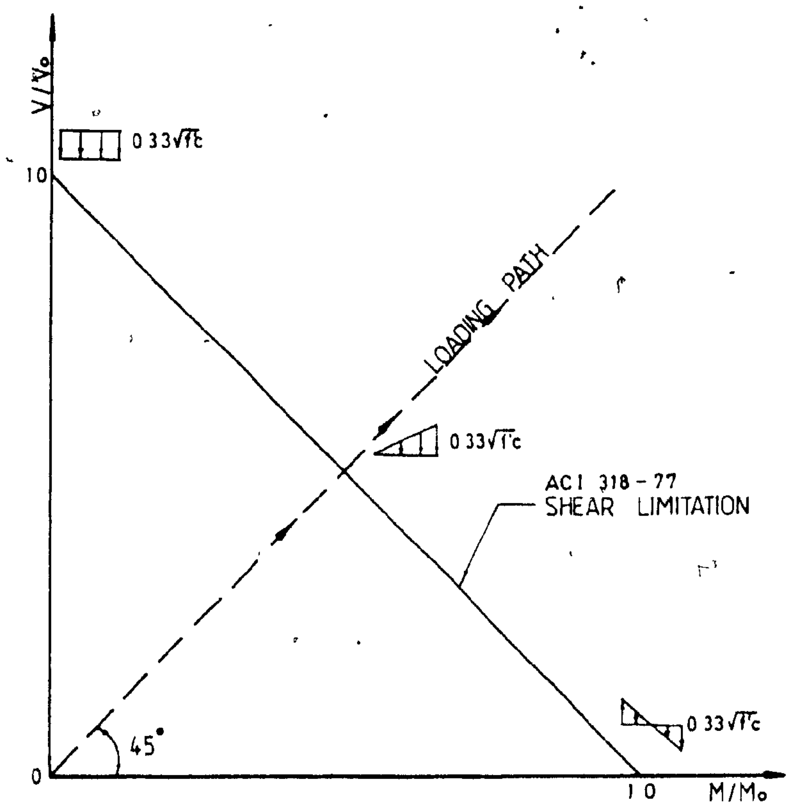


Fig. 3.8 - Loading Path

3.3 Loading Path and Loading Sequence

Fig. 3.8 shows a shear vs. unbalanced moment interaction diagram. This interaction diagram has been non-dimensionalized by dividing the applied shear, V, by the pure punching shear strength, V_o and also by dividing the total applied unbalanced moment, M, by the pure unbalanced moment corresponding to shear failure around the column, M_o.

A loading path was chosen such that the connections would be subjected to considerable moment transfer. The path chosen is halfway between the pure punching shear case and the pure moment transfer case as shown in Fig. 3.8.

The specimens were designed using a concrete compressive strength of 30 MPa and a steel yield strength of 400 MPa. The moment to shear ratio, calculated for the chosen geometry and material properties and assuming a concrete cover of 20 mm, was 0.29 m. This eccentricity of loading which corresponds to the loading path shown in Fig. 3.8 was used for all four specimens. In determining the loads to be applied, the dead weight of the slab was taken into account.

The loading was divided into several load steps. To achieve the loading condition for each step, loads were applied in the following sequence. First, using load cell readings, the pump pressure controlling the east and west sides was raised by a small amount to a desired value. This load was maintained until the deflection stabilized, and then, the same procedure was repeated for the north side followed by the south side. Finally, the load cell readings corresponding to the applied load on each side were checked. This procedure was repeated up to failure of the specimen.

3.4 Instrumentation

All electronic data was recorded and stored by a data acquisition and mini-computer system. The reduced data is given in Appendix A.

3.4.1 Deflections

Deflections of the slab relative to the column were measured by four Linear Voltage Differential Transformers attached to a steel frame which was clamped to the sides of the column immediately above the slab. The locations of these transformers are shown in Fig. 3.9 and in the photograph in Fig. 3.2. In addition, dial gauges below the slab measured the movement of the bottom of the slab relative to the column near the column face in order to determine when punching had occurred (see Fig. 3.9).

3.4.2 Concrete Strains

Six 30 mm electrical resistance strain gauges were placed on the bottom surface of the slab in order to measure the strain distribution in the slab around the column peripheries. The locations of these gauges are shown in Fig. 3.10.

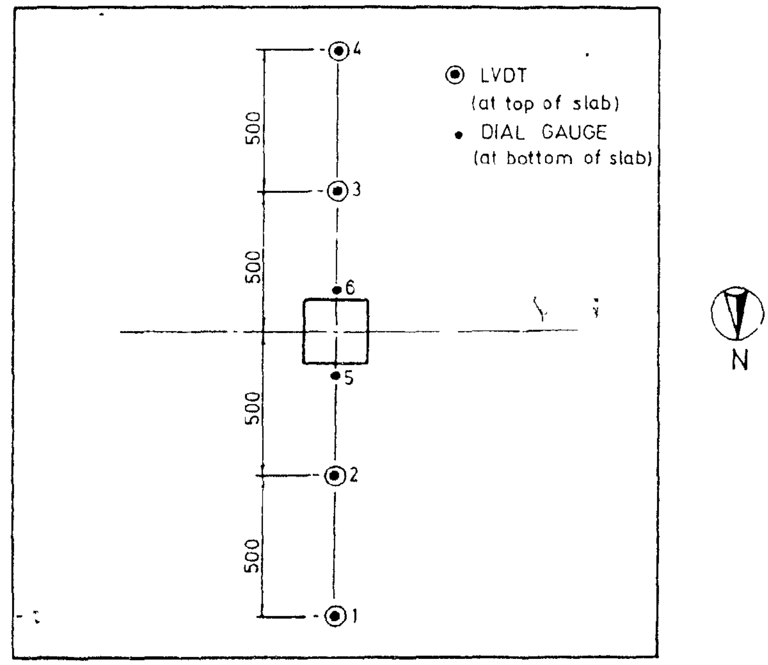


Fig. 3.9 - Locations of Deflection Gauges

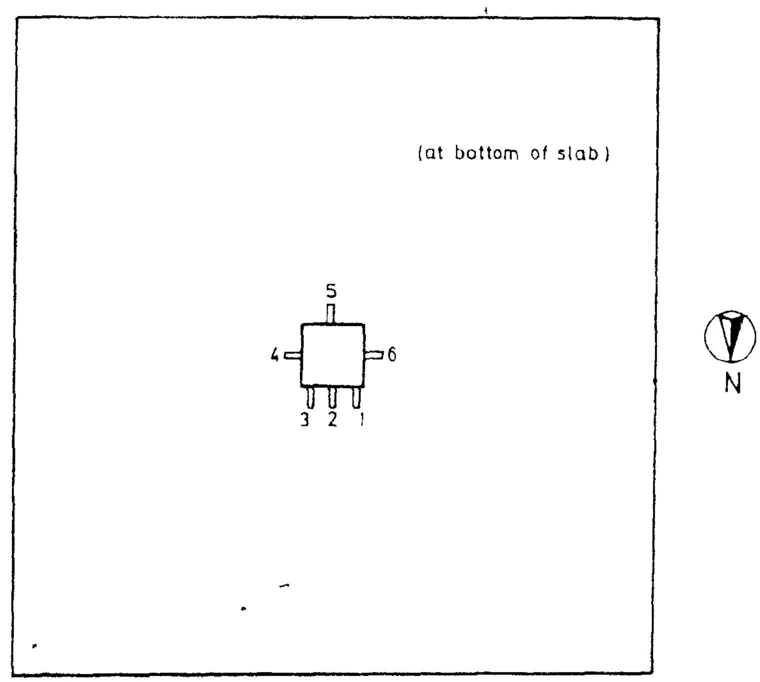


Fig. 3.10 - Locations of Concrete Strain Gauges

3.4.3 Steel Strains

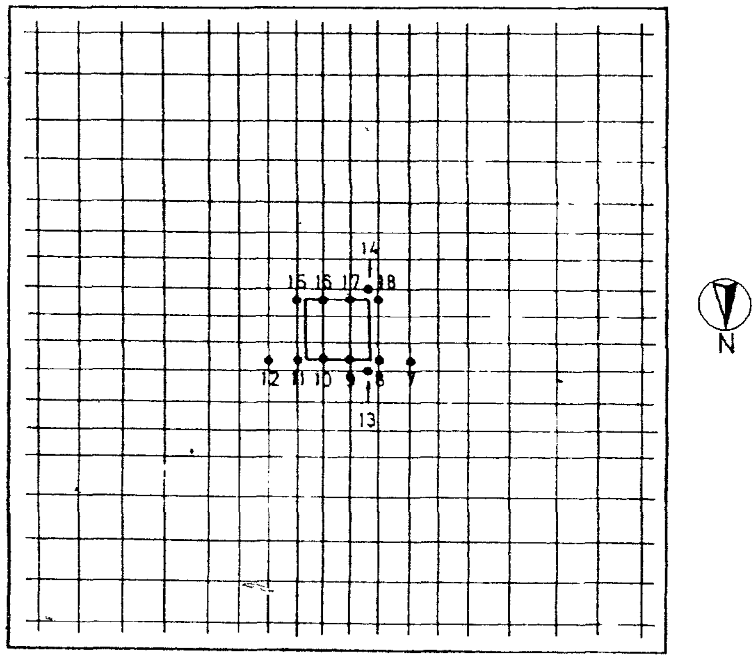
All specimens contained twelve electrical resistance strain gauges, with gauge length of 10 mm on the #15M reinforcing bars as shown in Fig. 3.11. While the repaired specimen, RS2 contained eight additional strain gauges on the additional #10M reinforcing bars. The locations of these gauges are also shown in Fig. 3.11.

3.4.4 Load Measurements

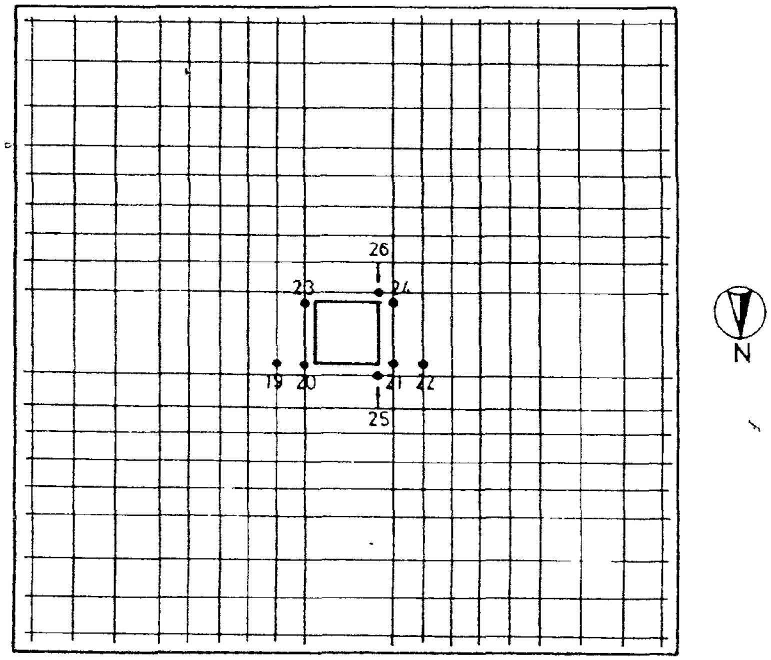
In addition to the pressure gauges on the three hydraulic systems, four load cells positioned as shown in Fig. 3.7 were used to check the loading.

3.5 Loading and Repair of Specimen RS2

Specimen RS2, which was identical in its unrepaired condition to specimen S2, was repaired with an additional structural concrete topping 45 mm thick which contained 18 #10M reinforcing bars in each direction (see Fig. 3.11). In order to facilitate the repair of the top surface of the slab the loading distribution beams shown in Fig. 3.1 and Fig. 3.2 were placed below the slab.



ON 15m BARS OF ALL SPECIMENS



ON 10m BARS OF SPECIMEN RS2

Fig. 3.11 - Locations of Steel Strain Gauges

A high-modulus, high-strength, moisture-insensitive epoxy adhesive was used as a bonding agent between the existing concrete and the topping. Prior to its application on the specimen, a compression test[18], for determining bond strength of epoxy-resin system, as specified by ASTM C882[19] (American Society for Testing and Materials) was carried out in a similar environment to that of the specimen. The cylinder with the inclined epoxy interface before testing is shown in Fig. 3.12. This test was performed to ensure that sufficient bond strength of the epoxy could be reached seven days after the placement of the concrete topping. The cylinder failed in shear parallel to the epoxy interface at a calculated shear stress of 7.6 MPa. The repair procedure is described as follows. First, specimen RS2 was loaded in its unrepaired condition to the design service dead load having the same moment to shear ratio as the other specimen. This service load was calculated from

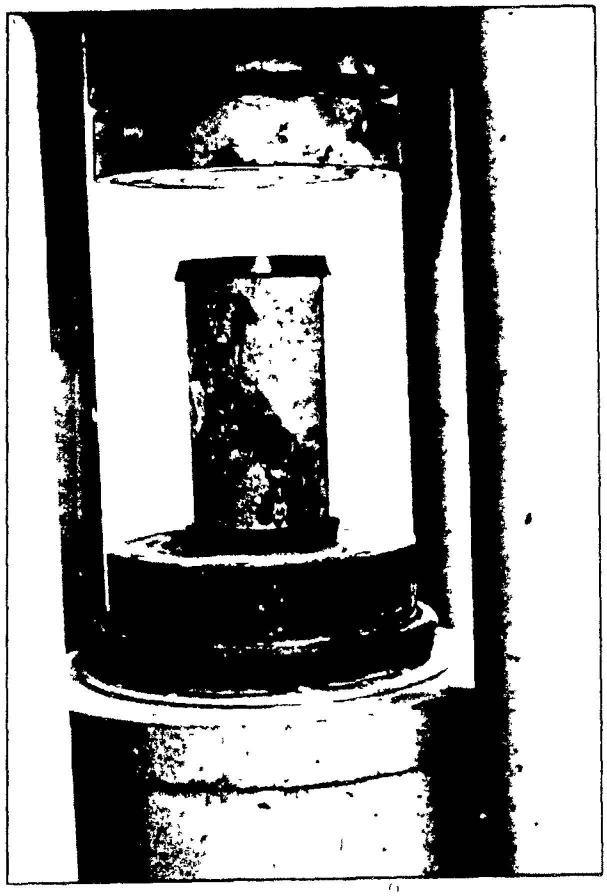


Fig. 3.12 - Compression Test of an Epoxy Resin Bonded Core

$$P_d = \gamma P_u$$

where P_u = ultimate design capacity of specimen S1, with the loading condition following the previously assumed load path

γ = ratio of service dead load to the design ultimate load

$$= \frac{w_d}{1.4 w_d + 1.7 w_l} \quad \text{(Taken as 0.43)}$$

w_d = service dead load

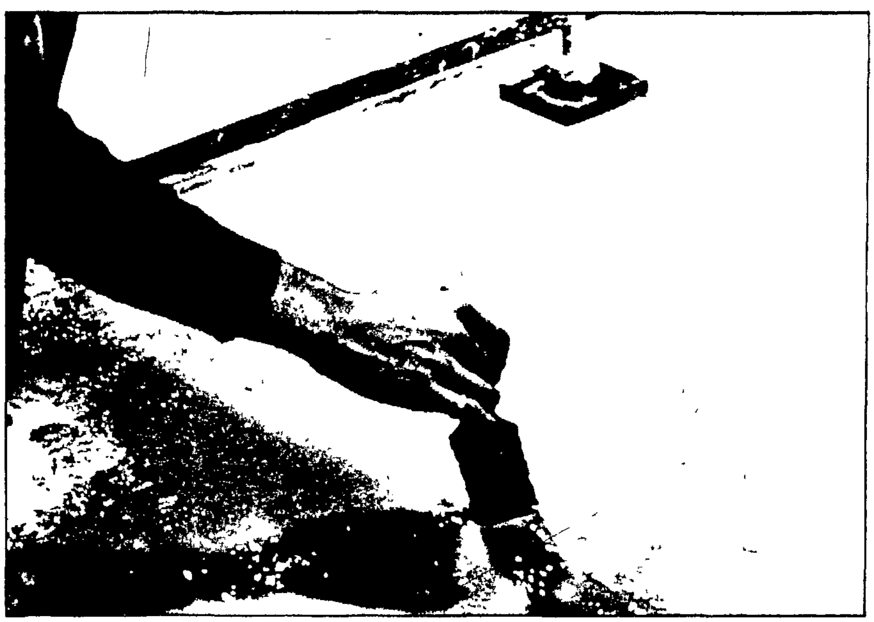
w_l = service live load

After the desired load was reached, the ram of each jack was mechanically locked in position to hold the deflections constant. The slab specimen was then properly supported to simulate the operation of reshoring.

The repair was made two days after the initial loading. During this period of time, observations of the load cell readings indicated that the locking scheme was effective.

Prior to the application of the epoxy resin, the slab surface was roughened using a small bush hammer to improve the bonding surface. The surface was then wire-brushed, air-blown and wet-cleaned. In mixing and placing the epoxy, the manufacturer's recommendations were strictly followed. Particularly precautions were taken in consolidating the topping concrete to disturb the layer of resin as little as possible. Fig. 3.13 shows the bush hammer and the epoxy coating of the top surface of the slab.

(a) Application of Bush Hammer



(b) Application of Epoxy Coating

Fig. 3.13 - Topping Repair

CHAPTER 4

EXPERIMENTAL RESULTS

This chapter first discusses the code provisions for shear combined with unbalanced moment and then presents the experimental results for the specimens S1, S2 and S3 together with the results for the repaired specimen RS2.

4.1 Code Strength Equations

Before examining the test results, a brief review of the present ACI code provisions for determining the shear strength of slab-column connections transferring moments is presented.

The commentary to the ACI code 318-77[3], has adopted the following expressions for determining the maximum factored shear stress at the critical section of an interior column (see Fig. 4.1). For convenience all of the expressions have been expressed in SI units.

$$v_{u(AB)} = \frac{V_u}{\emptyset\ A_c} + \frac{\gamma_v M_u C_{AB}}{\emptyset\ J_c} \tag{4.1}$$

$$v_{u(CD)} = \frac{V_u}{\emptyset\ A_c} - \frac{\gamma_v M_u C_{CD}}{\emptyset\ J_c} \tag{4.2}$$

where = maximum factored shear stress.

$v_u = (0.17 + \frac{0.33}{\beta_c})\sqrt{f'_c}$.

but not greater than $0.33\sqrt{f'_c}$

β_c = ratio of long side to short side of the column

$\emptyset$ = strength reduction factor = 0.85 for shear

f'_c = specified compressive strength of concrete, MPa

V_u = factored shear force

M_u = factored unbalanced moment

$$\gamma_v = 1 - \frac{1}{1 + \frac{2}{3}\sqrt{\frac{C_1 + d}{C_2 + d}}} \tag{4.3}$$

A_c = area of concrete of assumed critical section

$= 2d(C_1 + C_2 + 2d)$ for a rectangular column

J_c = property of assumed critical section analogous to the polar moment of inertia

$$= \frac{d(C_1 + d)^3}{6} + \frac{(C_1 + d)d^3}{6} + \frac{d(C_2 + d)(C_1 + d)^2}{2}$$

for a rectangular column

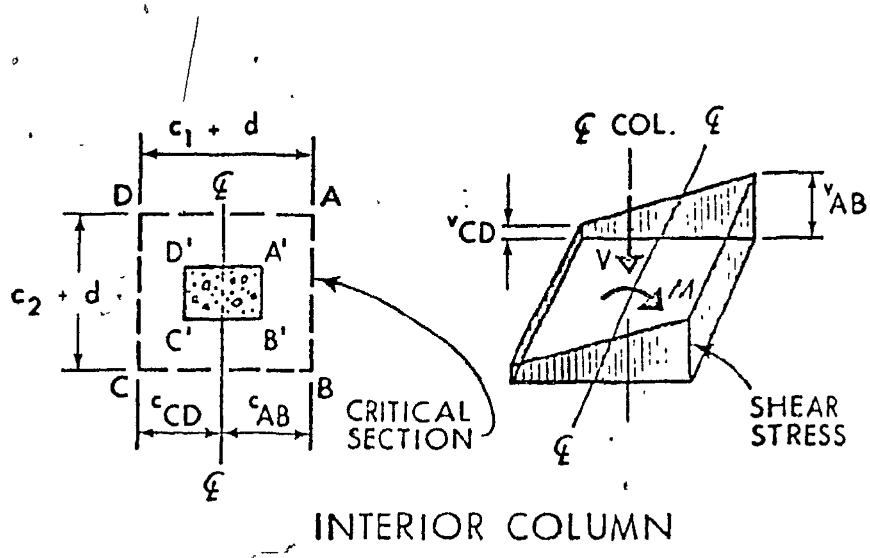


Fig. 4.1 - Moment Transfer for Linearly Varying Shear Stress (from reference 7)

4.2 Load Deflection Responses

The load-deflection responses of specimens S1, S2 and S3 and the repaired specimen RS2 are shown in Fig. 4.2. In this figure the total shear force, V, is plotted against the deflection, Δ , measured at the location halfway between the loading points on the north side. All the specimens failed in shear. It can be seen that, for specimens S1, S2 and S3, the decrease of effective depth resulting from the simulation of misplacement of reinforcement from 114 mm to 69 mm and 44 mm results in a significant decrease in the shear capacity and flexural stiffness. Shear capacity measured from the experiments, V_{exp}, and the nominal shear strength, V_n, calculated according to ACI 318-77 provisions, are shown in Table 3.1 for this investigation and for the punching shear tests performed by Lee[13]. It can be seen that the ACI predictions using the actual average effective depth are conservative for all the specimens.

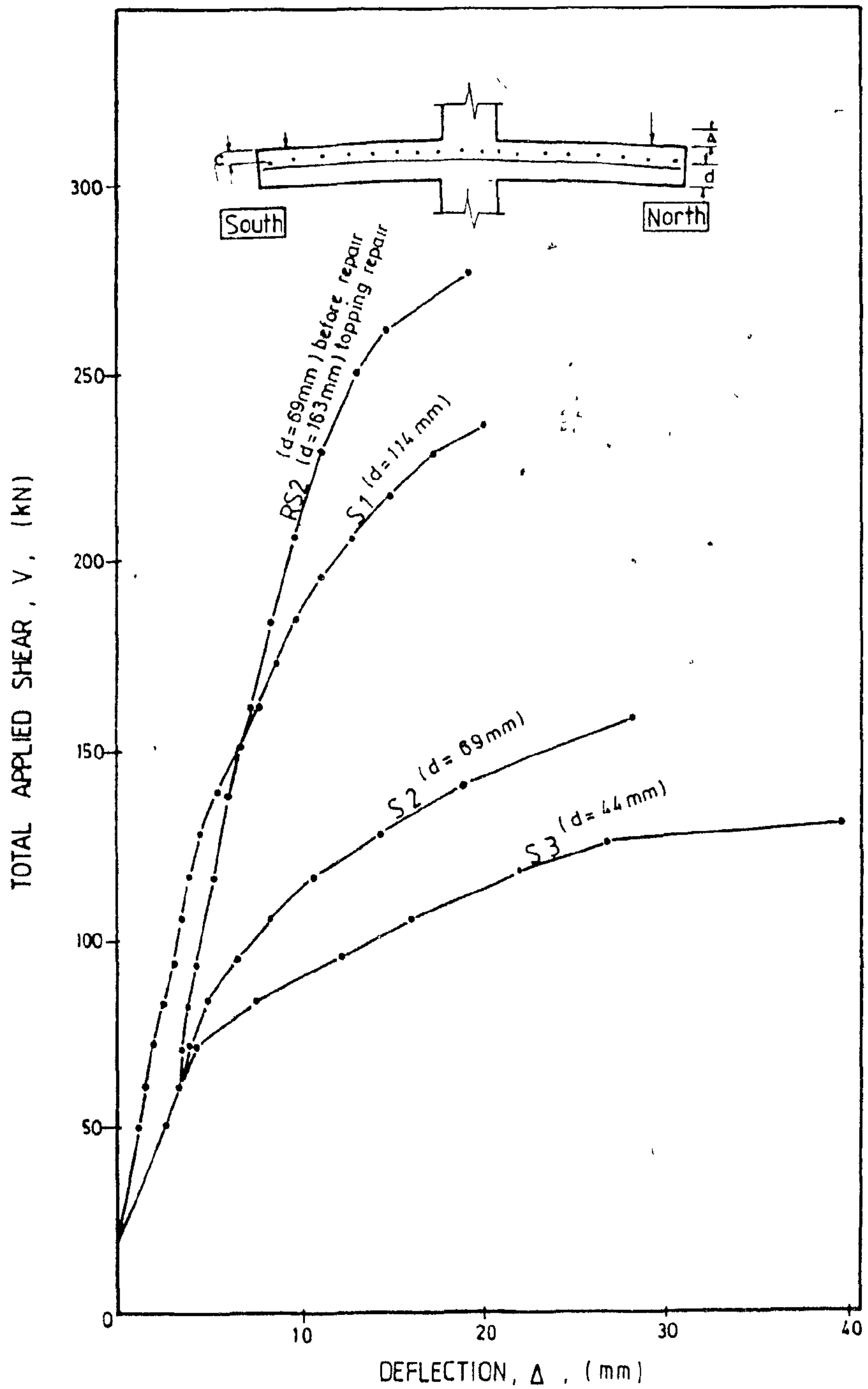


Fig. 4.2 - Effect of Effective Depth on Load-Deflection Response of Shear and Moment Transfer Test Specimens

Fig. 4.3 compares the non-dimensionalized shear vs. moment interactions with the experimental values for the pure punching tests and for the shear and moment transfer tests. These comparisons are made for the three different effective depths. It can be seen that the ACI approach using the actual position of the reinforcement becomes more conservative as the effective depth reduces.

Specimen S1, with the larger effective depth failed in a brittle manner. At failure the top bars ripped out of the top concrete cover and the load dropped off significantly. While the specimens with the smaller effective depths (S2 and S3) displaced a more ductile failure.

The difference in the load-deflection response of specimen S2 before and after repair can be seen in Fig. 4.2. A significant improvement of shear capacity and flexural stiffness was achieved by the topping repair. Table 3.1 indicates that the ACI method conservatively predicts the shear strength of the repaired specimen calculated using the larger effective depth measured to the centre of the additional mat of reinforcement used in the repair.

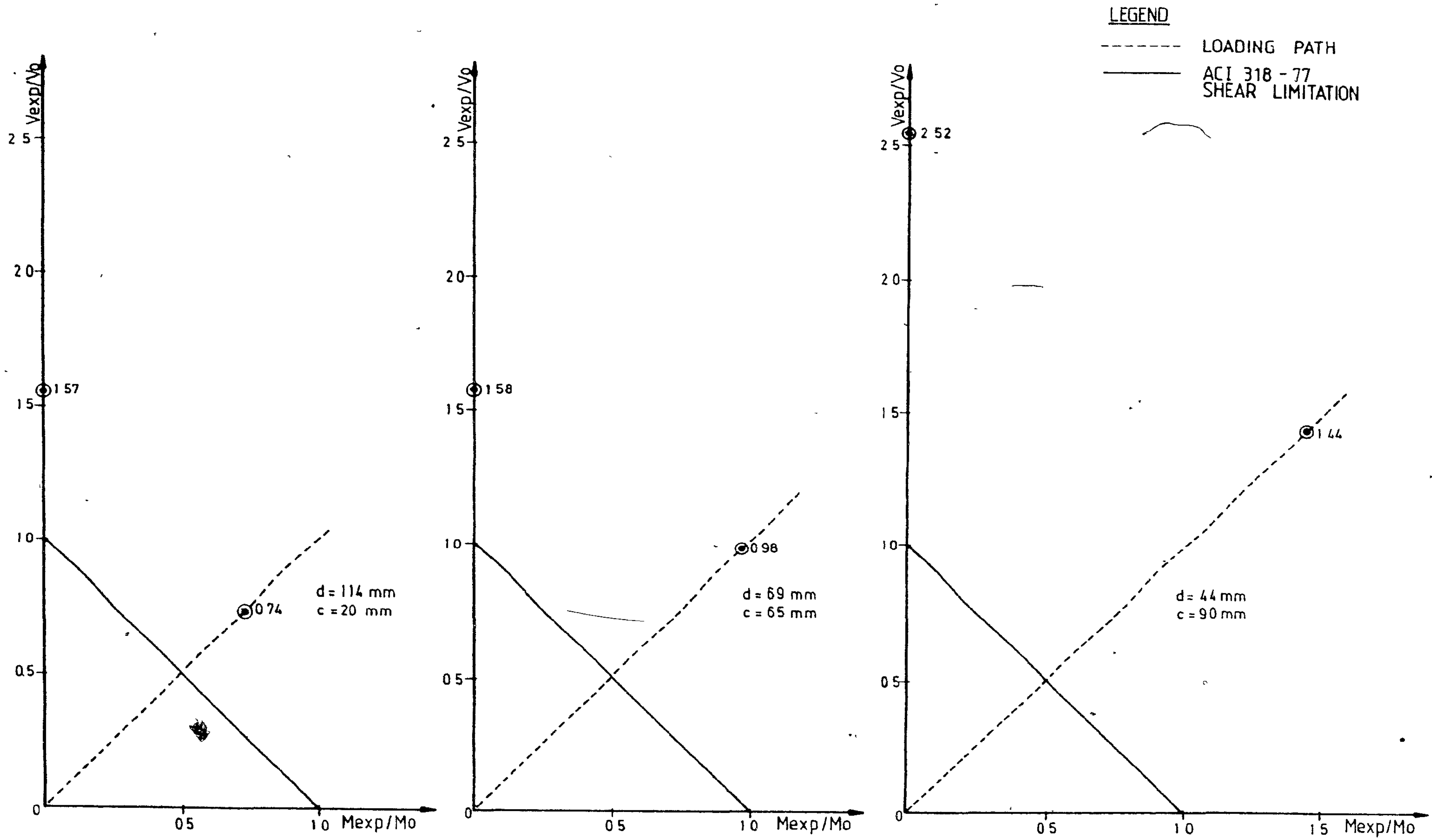


Fig. 4.3 - Comparison of Measured Strengths and Predictions of ACI 318-77

4.3 Cracking Behaviour

Plots of the shear versus maximum crack widths given in Fig. 4.4 show that increases in concrete cover (and the corresponding decrease in effective depth) result in larger maximum crack widths at all levels of loading. As the load increased, a drastic increase of crack width for specimens with the small effective depths was observed. Table 4.1 was prepared in order to assess the effect of "misplacement" of reinforcement on the crack widths at service loads. For each specimen tested by direct shear and by shear combined with moment transfer, the approximate service load shear was calculated by determining 0.6 times the ACI predicted nominal shear capacity and using the effective depth corresponding to proper placement (i.e. clear cover = 20 mm and average effective depth = 114 mm). The service shear was calculated using the measured concrete strength for each specimen. As expected "misplaced" reinforcement leads to an increase in crack widths at service load levels. The direct punching tests gave larger increase in crack widths than the combined loading tests for the same degree of "misplacement". The ACI Commentary[3] limit on crack width is 0.4 mm for interior exposure.

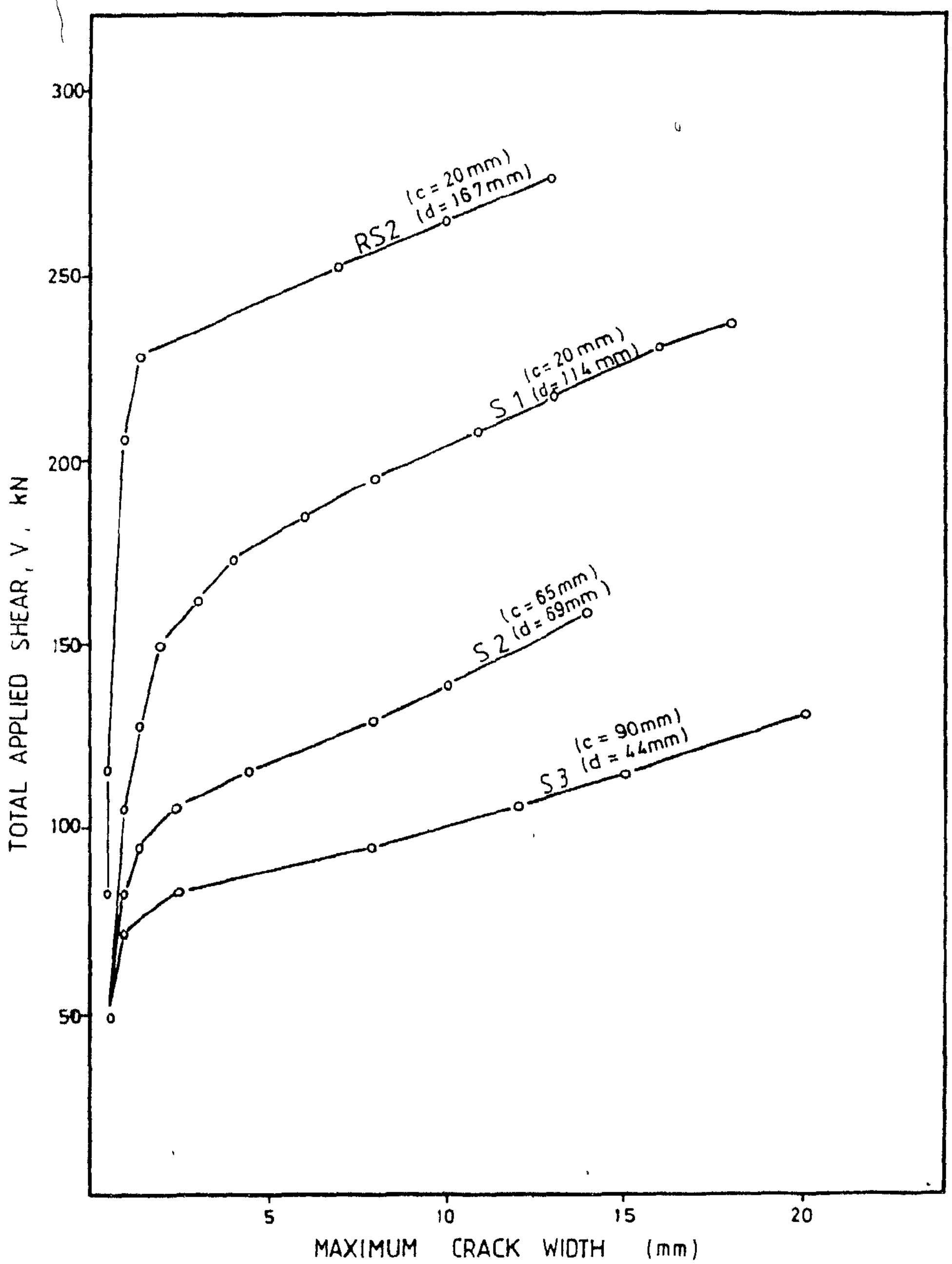


Fig. 4.4 - Effect of Concrete Cover on Maximum Crack Width of Shear and Moment Transfer Test Specimens

Specimen	Cover mm	dav mm	f'c Mpa	Appro.x Service Shear kN	Max. Crack Width mm	ACI Limit on Crack Width mm
Shear and Moment Transfer Tests						
S1	20	114	39.6	96	0.06	0.4
S2	65	69	36.3	92	0.32	
S3	90	44	33.6	89	0.58	
RS2xx	20	163	35.4	91	hairline	
Punching Shear Tests (Reference 13)						
PS1	20	114	25.0	154	0.23	0.4
PS2	65	69	35.1	182	1.20	
PS3	90	44	39.9	195	failed	
PS2ARxx	20	163	27.8	162	0.08	
PS2BRxx	6	131	25.8	156	0.08	

x Approximate service shear taken as 0.60 times the calculated nominal shear using ACI approach and measured concrete strength for each specimen and also using the effective depth assuming proper placement (ie. cover = 20mm and effective depth = 114mm)

xx Repaired specimens after repair

Table 4.1 - Comparison of Measured Maximum Crack Widths and ACI Limit at Service Load

Figure 4.5 shows the crack patterns for the specimens S1, S2 and S3 at the approximate service load. Specimen S1, with 20 mm cover, showed an orthogonal pattern of cracks which followed the lines of the top reinforcement, while specimens S2 and S3, with larger covers, developed radial and tangential cracks around the columns. This clear difference in the crack pattern could enable early recognition of misplaced top reinforcement in slabs. Fig. 4.6 gives close-up photographs of the three specimens comparing the crack patterns near the column after failure. This figure emphasizes the differences in the crack patterns.

The repaired specimen, RS2, exhibited a marked decrease of maximum crack width for all load levels after the topping repair. Table 4.1 compares the maximum crack widths before and after repair for the direct shear test performed by Lee and the combined loading tests of this investigation. It is important to realize that the specimens to be repaired were

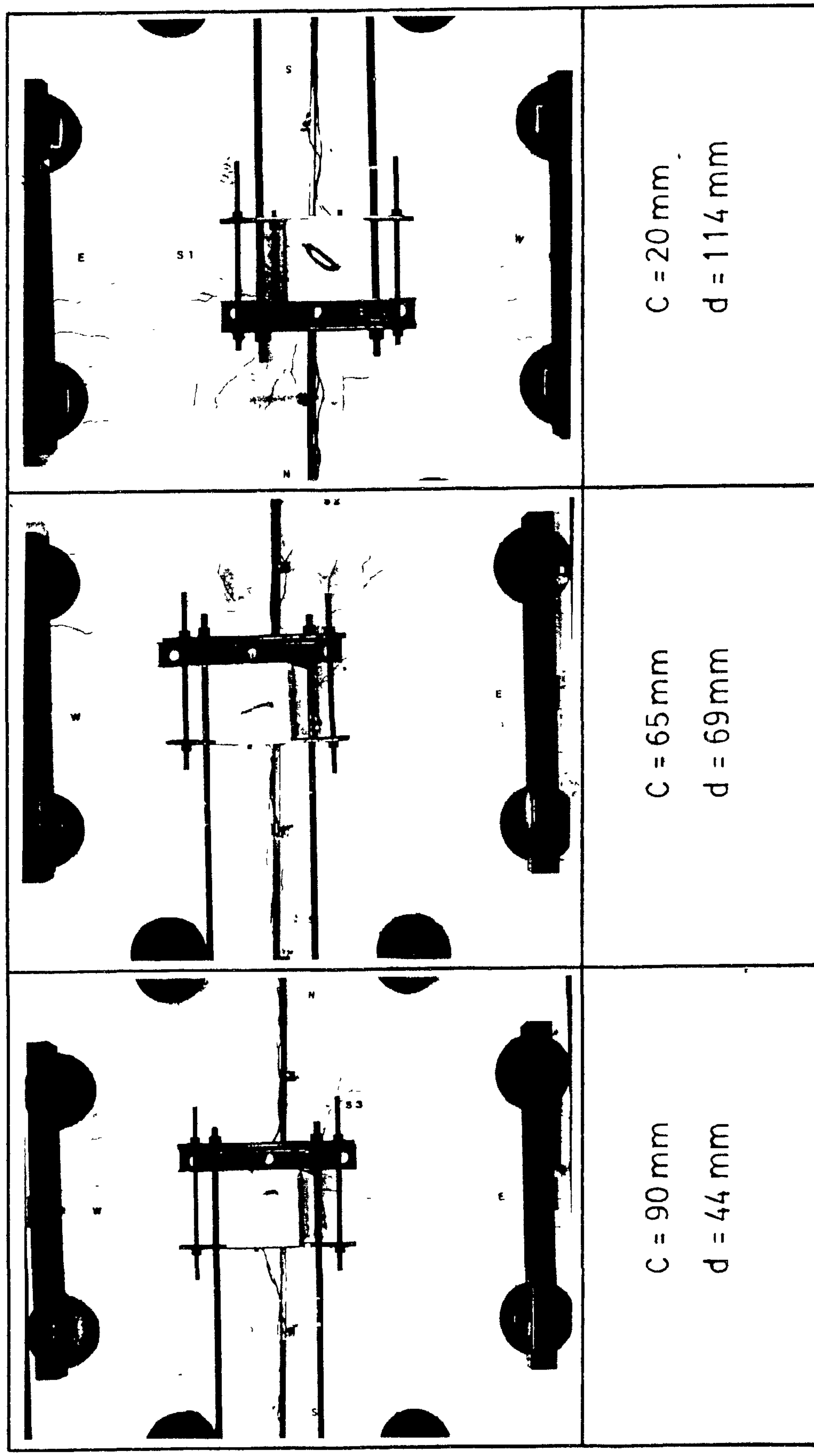


Fig. 4.5 - Crack Patterns of Shear and Moment Transfer Test Specimens at Service Load

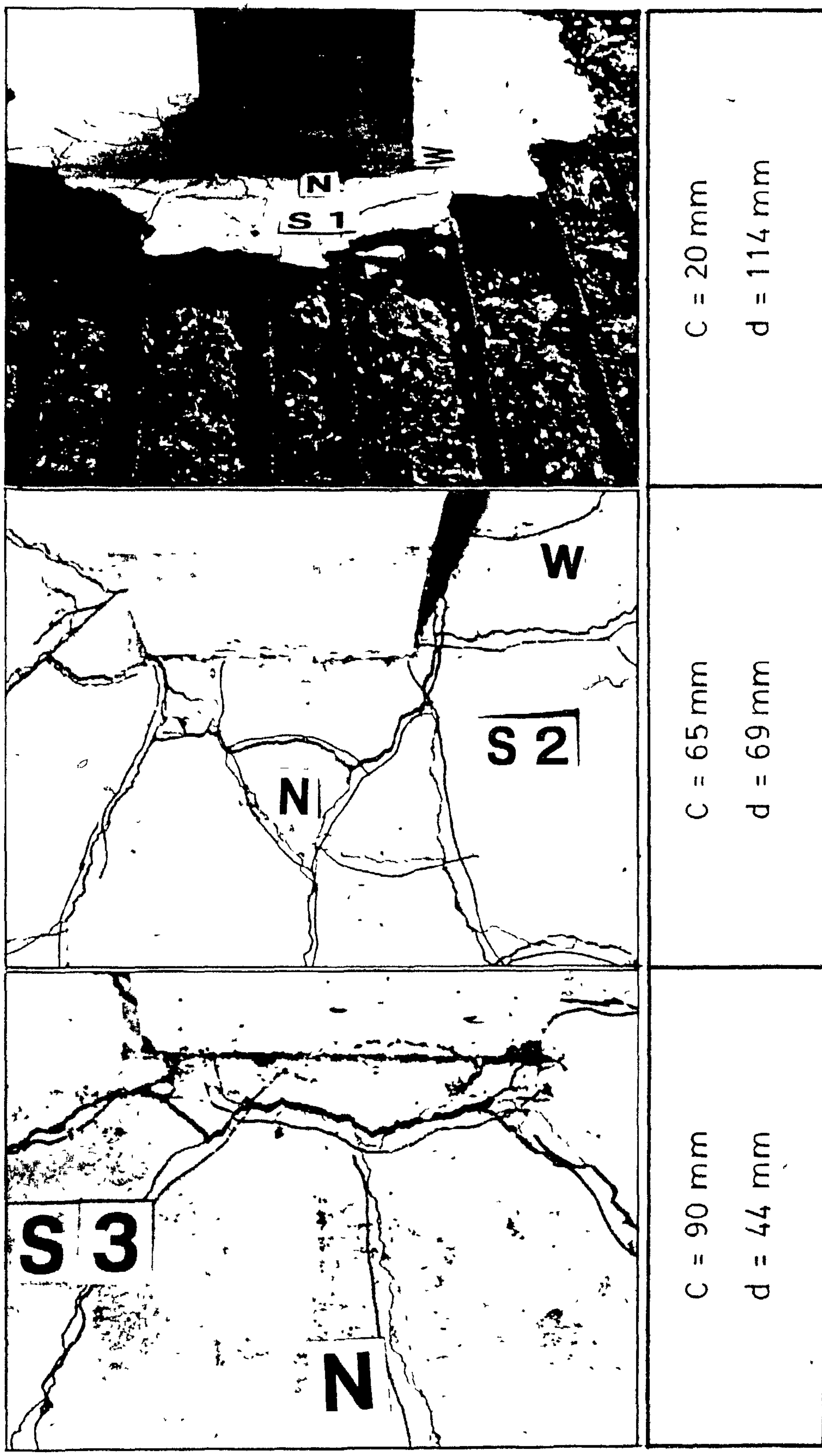


Fig. 4.6 - Crack Patterns of Shear and Moment Transfer Specimens after failure

first loaded to load corresponding to approximate the service dead load. The repair was then made which consisted of a topping repair for specimens RS2 and PS2AR, while extra bars were placed in epoxied grooves cut in the top surface of the slab for Lee's specimen PS2BR. It can be seen from Table 4.1 that these repair methods significantly improved the cracking behaviour. Only very small maximum crack widths were observed at the load approximately corresponding to service dead load plus service live load. Fig. 4.7 compares the crack patterns for specimen RS2 before repair under service dead load and after repair at ultimate. The initial cracking which consisted of cracks radiating from the column before repair changed to an orthogonal crack pattern similar to the pattern for specimen S1 after repair. Both specimens S1 and RS2 after repair had identical concrete clear covers of 20 mm for the topmost reinforcement.

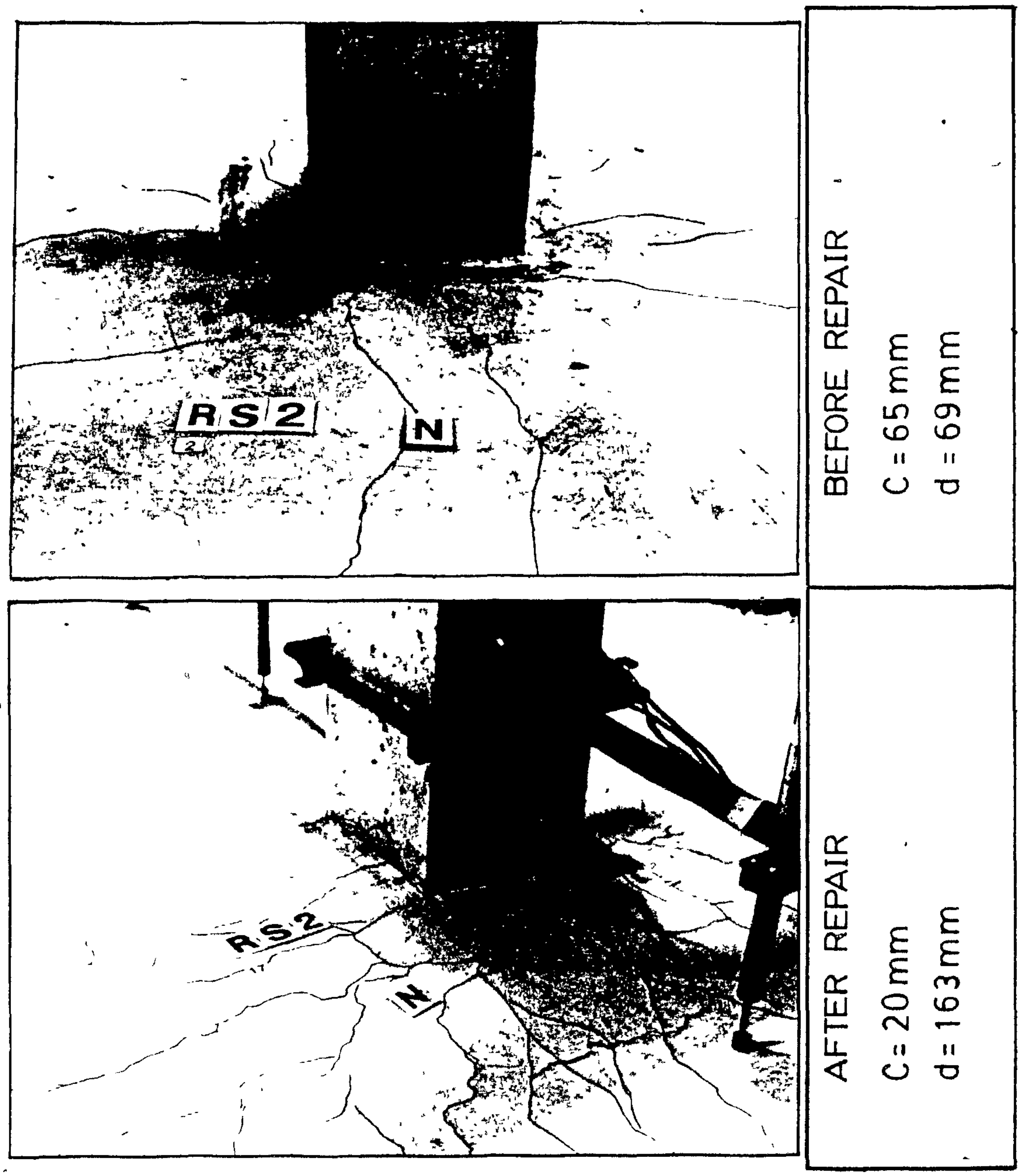


Fig. 4.7 - Crack Patterns of Specimen RS2 before and after repair

Observations before and after repairing of specimen RS2 showed no signs of shrinkage cracks or delamination at the topping interface. The cracks evident at the edges of the slab, as shown in Fig. 4.8, which had developed vertically across the topping and slab interface without interruption indicate that good bond between the topping and the slab was attained.

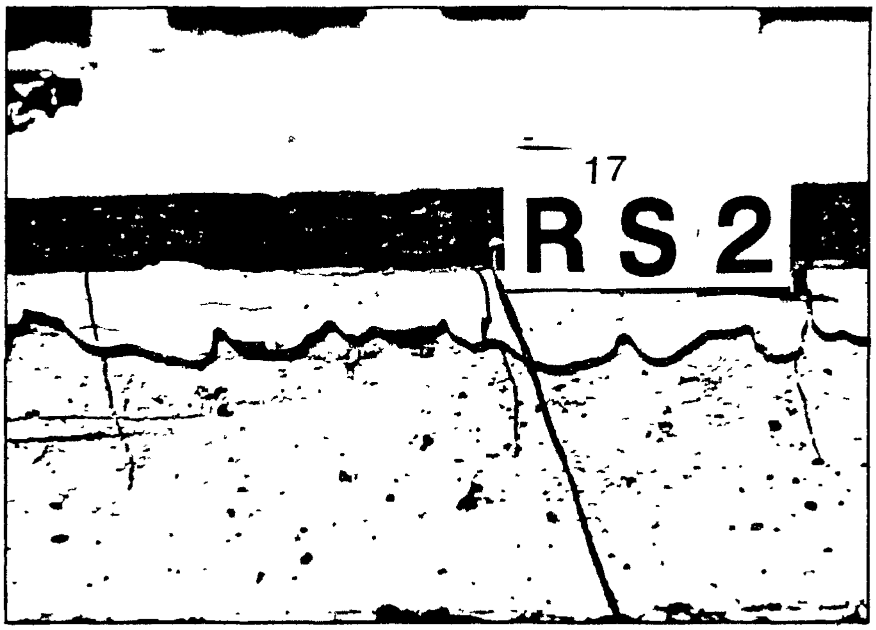


Fig. 4.8 - Vertical Cracks Developed Across Topping and Slab Interface

CHAPTER 5

EVALUATION AND REPAIR RECOMMENDATIONS FOR SLAB-COLUMN CONNECTIONS CONTAINING MISPLACED REINFORCEMENT

This chapter discusses some of the practical implications of the research programme on slab-column connections containing misplaced reinforcement and subjected to direct shear and combined shear and unbalanced moment.

5.1 Inspection for Possible Misplacement of Reinforcement

In inspecting a structure during and after construction, the engineer should be made aware of excessive deflections or unusual cracking. An orthogonal cracking pattern with cracks above reinforcing bars and having small crack widths is usually an indication that the steel has been properly placed.

A cracking pattern which has wide cracks radiating from the column is an early sign of possible misplacement of the top reinforcement in the column regions. Since the misplacement of the top reinforcement could lead to a premature brittle punching shear failure, the need of early recognition of this defect cannot be overemphasized.

A recent structural collapse of the condominium in Cocoa Beach, Florida[14] is an excellent example of the need for early detection of this serious problem. The collapse of this five-storey flat-plate structure during construction was attributed to underdesign for punching shear as well as the misplacement of the top reinforcement 25 mm lower than needed to satisfied the minimum cover requirements. It is worth noting that during the investigation of this serious collapse an interview with a worker revealed that he had seen cracks which he had described as "spider cracks".[15]

It is essential for engineers to appreciate that a "spider-like" cracking pattern which consists of radiating cracks particularly in the column regions may be an early warning of possible misplacement of reinforcement.

Means of determining the position of reinforcement in slabs are described in references 13 and 16.

5.2 Evaluation of Service Performance and Ultimate Strengths of Slab-Column Connections Containing Misplaced Reinforcement

Fig. 5.1 compares the experimental shear-moment interaction diagrams for different positions of reinforcement with the predicted interaction diagram using the ACI approach for a specimen with the reinforcement in its proper place (i.e. clear cover of 20 mm). Fig. 5.1 demonstrates that "direct punching" shear is more sensitive to "misplacement" of reinforcement than the case of shear combined with transfer of moment.

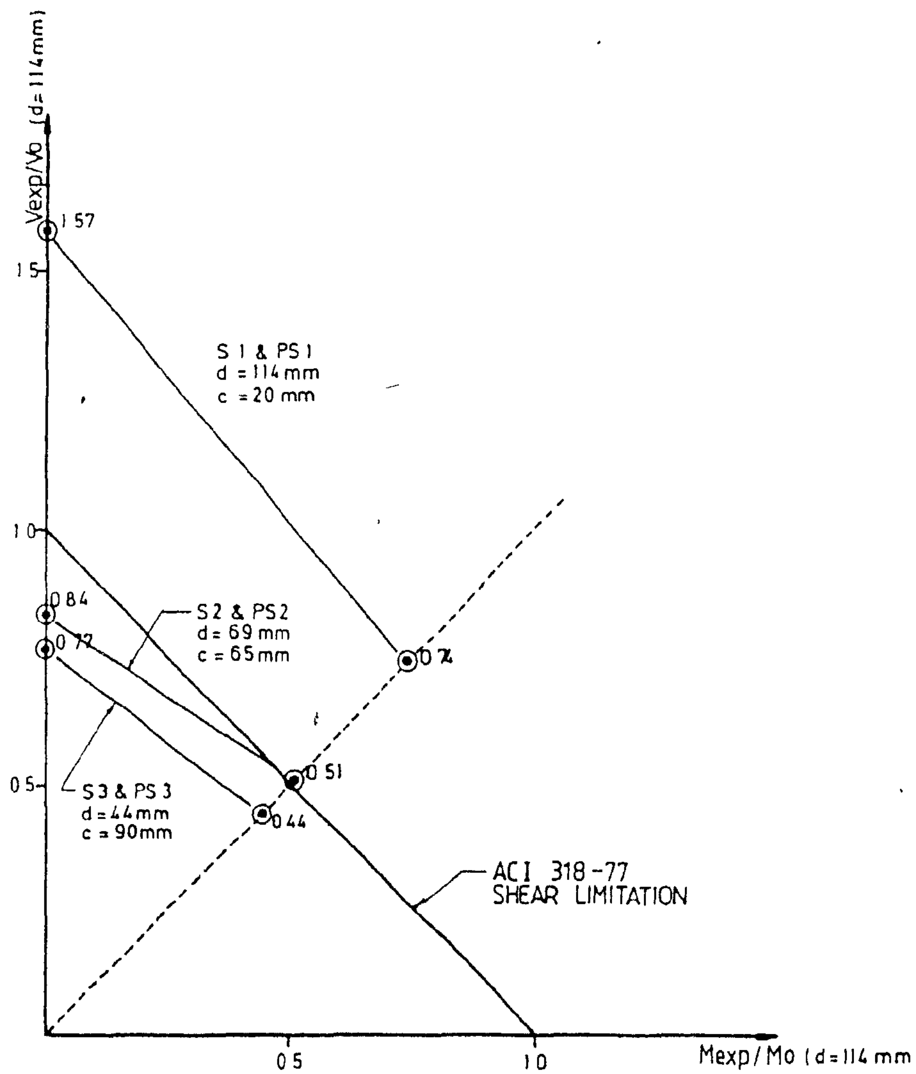


Fig. 5.1 - Effect of Misplaced Reinforcement on Shear Strength of Test Specimens

However for both cases the "misplacement" of reinforcement can eliminate the inherent extra factor of safety. It is important to keep in mind that the loading conditions during construction may be more severe than that for the completed structure.

Fig. 5.2 compares the ratio of the experimental shear capacity divided by the nominal shear capacity, V_{exp}/V_n, with the degree of "misplacement" of the reinforcement. The degree of misplacement is expressed as the ratio of the actual average effective depth, d, to the average effective depth, d_{20}, assuming a 20 mm cover and #15M bars. This experimentally determined relationship has been plotted for pure punching shear tests and for the tests of shear combined with unbalanced moment. These experiments have a column dimension to average effective depth ratio, c/d_{20}, of 1.97. An addition experimental value was obtained from the direct shear test reported in Reference 17. This specimen had an overall slab thickness of 175 mm, an actual clear cover of 30 mm, #15M bars, a 300 mm square column and had a concrete strength of 45.5 MPa. This particular specimen had a c/d_{20} ratio of 2.16. Also shown in Fig. 5.2 are the predictions using the ACI approach for both the direct punching tests and for the combined loading tests. It can be seen that the ACI approach gives conservative predictions of the strengths of all the specimens studied.

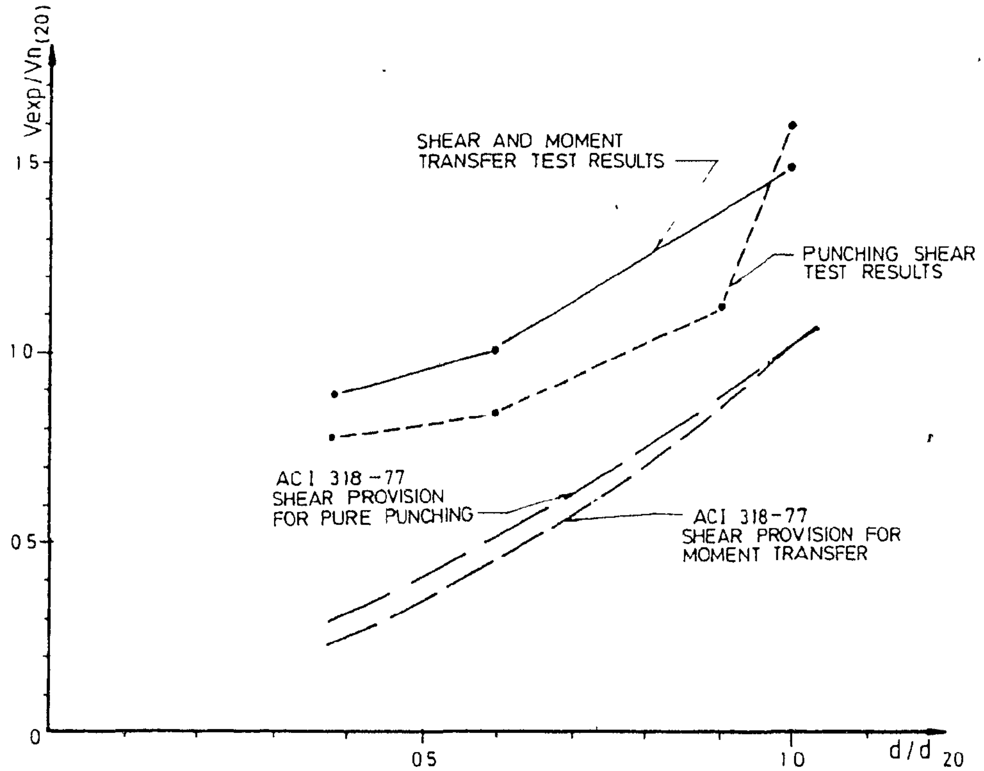


Fig. 5.2 - Relationship Between Shear Strength Ratio and Effective Depth Ratio for Shear Test Specimens

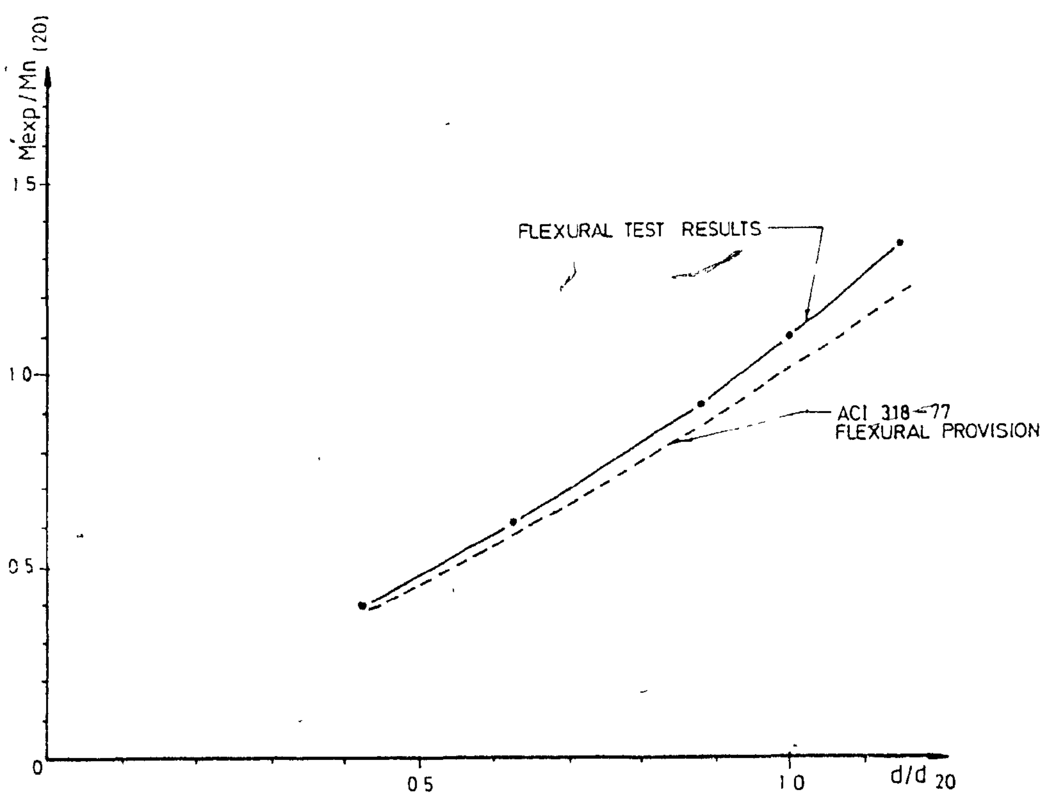


Fig. 5.3 - Relationship Between Shear Strength Ratio and Effective Depth Ratio for One-Way Flexural Test Specimens (*Data obtained from reference 13)

Fig. 5.2 shows that there is a large decrease in capacity resulting from small decreases in effective depth. For the punching shear tests a 10 percent reduction in effective depth, d_{20}, results in a 48 percent reduction of shear capacity. When the steel is greatly misplaced with say an average effective depth which is less than 50 percent of d_{20} then further decreases in effective depth seem to give very small reductions in capacity. It is also noted that the pure punching test results are below the combined loading test results.

Fig. 5.3 gives the relationship between the ratio of experimental moment capacity to nominal flexural strength, M'_{exp}/M_n, versus the effective depth ratio, d/d_{20}. The experimental points were obtained from one-way flexural tests from Reference 13. Also shown is the prediction of flexural capacities using the ACI flexural strength approach. The predictions are very accurate for the full range of effective depths. The fact that the ultimate flexural theory is more rational than the empirical approach for punching shear is apparent when Fig. 5.2 is compared to Fig. 5.3. It can be seen that a reduction of 10 percent in effective depth results in a 18 percent reduction in flexural capacity.

A comparison of Fig. 5.2 with Fig. 5.3 indicates that for the more usual range of possible misplacement the punching shear capacity is more sensitive than the flexural capacity to the actual placement of the reinforcement.

If the variations of placement of reinforcement in actual slabs are ± 10 mm[16], then for relatively thin slabs (e.g. 150 mm) this could lead to a 10 percent reduction in d/d_{20} and a corresponding drop of 48 percent of punching shear capacity. Thus the misplacement of reinforcement can be quite serious for thin slabs.

Fig. 5.4 shows the experimental variation of strength, expressed non-dimensionally, versus the effective depth d/d_{20}. The experimental curve was chosen as the punching shear limit since it was more severe than the combined loading test results over the practical range of steel placement. Also shown are the code tolerance for placement for the specimen

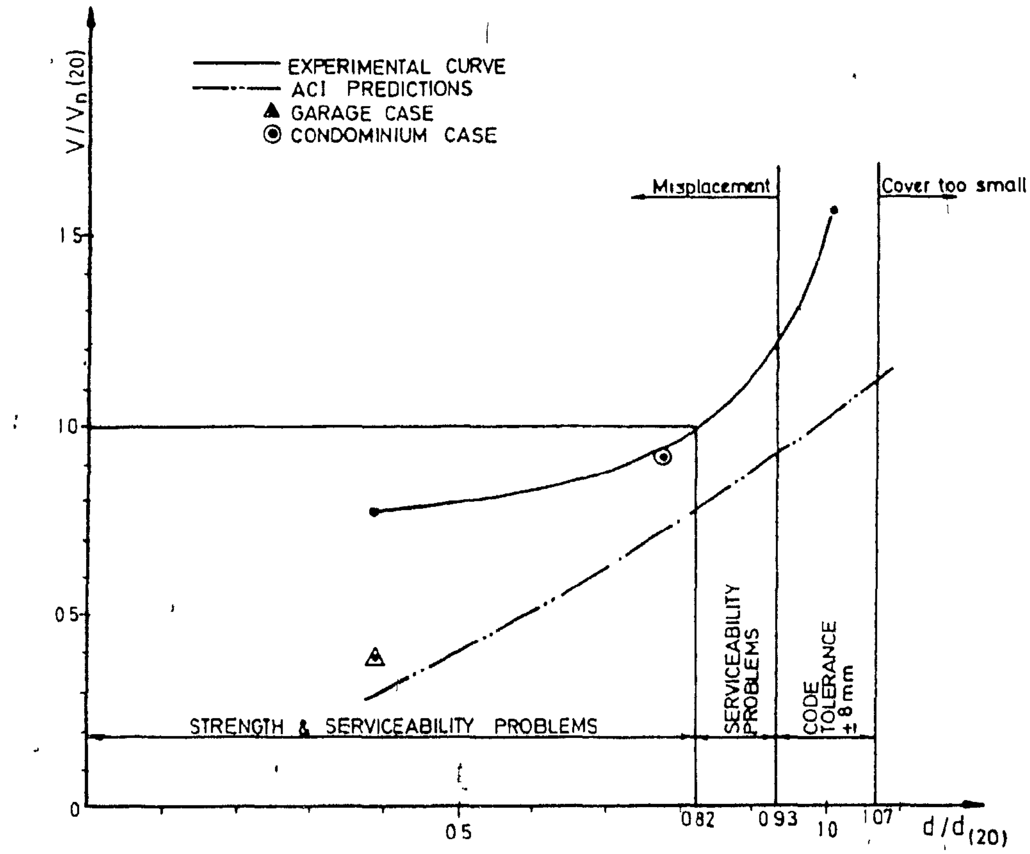


Fig. 5.4 - Evaluation of Structural Performance Using the Experimental Curve for 150 mm thick Slabs

tested. If the effective depth ratio exceeds 1.07 then the concrete cover is too small and may result in problems of corrosion protection and durability. If the effective depth ratio, d/d_{20}, is lower than 0.93 then the steel is classified as being misplaced. As can be seen from Fig. 5.4 there is considerable reserve of strength over the predicted strength for this tolerance limit. In the range of effective depth ratios lower than 0.93 serviceability problem such as excessive cracking or excessive deflections may result. For effective depth ratios less than 0.82 there are definite strength and serviceability problems.

Fig. 5.4 also shows the degree of misplacement and the severity of loading for two structures that have been well documented in the literature. The first example is for an industrial garage, flat-slab structure[13], situated near Montreal. This structure had a severe case of misplacement of top reinforcement in which the maximum concrete cover was measured to be 222 mm instead of 20 mm.

This results in a effective depth ratio calculated to be 0.39. The applied shear force to the nominal shear capacity ratio, V/V_n, was estimated to be 0.34. Field investigations indicated a significant degree of cracking with cracks as wide as 1.6 mm. The largest cracks occurred on the top surface around the column in a "spider-like" pattern. This extreme cracking is to be expected for this severe misplacement. However, the garage structure did not collapse as the code equations would predict.

The second relevant example is the five-storey, flat-plate condominium structure in Cocoa Beach, Florida[14] collapsed during construction. This structure was thoroughly investigated by National Bureau of Standards (NBS). The NBS investigative team reported[15] that "The effective depth is found to fall in the range of 4.97 to 5.35 in. (126 to 136 mm)" at the column strip, and "The value of d = 5.3 in. (135 mm) is assumed in the subsequent calculation of stresses in the structure. Generally, these stresses are somewhat on the low side because the assumed d for the slab is high at most of the columns".

The effective depth of 126 mm was chosen for calculating the effective depth ratio in Fig. 5.4 and the severity of loading corresponding to the analysis of the construction stages. The effective depth ratio and V/V_n ratio were calculated to be 0.78 and 0.91 respectively. The experimental curve predicts that the structure is very close to failure under the construction loading. As mentioned previously, large cracks with a "spider-like" crack pattern were observed prior to collapse.

Fig. 5.4 can be used as a guide in estimating the shear capacities of slab-column connections which are similar to the specimens tested. In evaluating the strength of slab-column connections that are quite different from the test specimens used to prepare Fig. 5.4 it would be necessary to carry out punching shear tests to determine the effect of misplaced reinforcement.

5.3 Repair of Slabs Containing Misplaced Reinforcement

If the evaluation of the structure indicates that repair is necessary due to misplaced reinforcement then the choice of the repair method will depend on the following conditions.

a) <u>Reinforcement misplaced in many regions over a large area of the slab</u>

If the top or bottom reinforcement is misplaced such that the strength or serviceability is inadequate over a large region then a practical repair method could consist of adding a composite structural topping. The adhering surface should be properly prepared and epoxied to ensure proper adherence (see Reference 5).

The structural topping in the negative moment regions should contain an adequate amount of reinforcement to properly control cracking and to provide an increased effective depth, to increase the punching shear capacity and the flexural capacity. Fig. 5.5 summarizes the effects of repair using a structural topping for direct punching shear and for shear combined with moment transfer. The strength of specimens PS2 and S2 containing misplaced reinforcement is underestimated by the current code procedures. Specimens PS2AR and RS2 were identical to specimens PS2 and S2 before repair. The significant strength increases after repair with a structural topping is evident in Fig. 5.5. It can be seen that the ACI approach assuming that the average effective depth measured

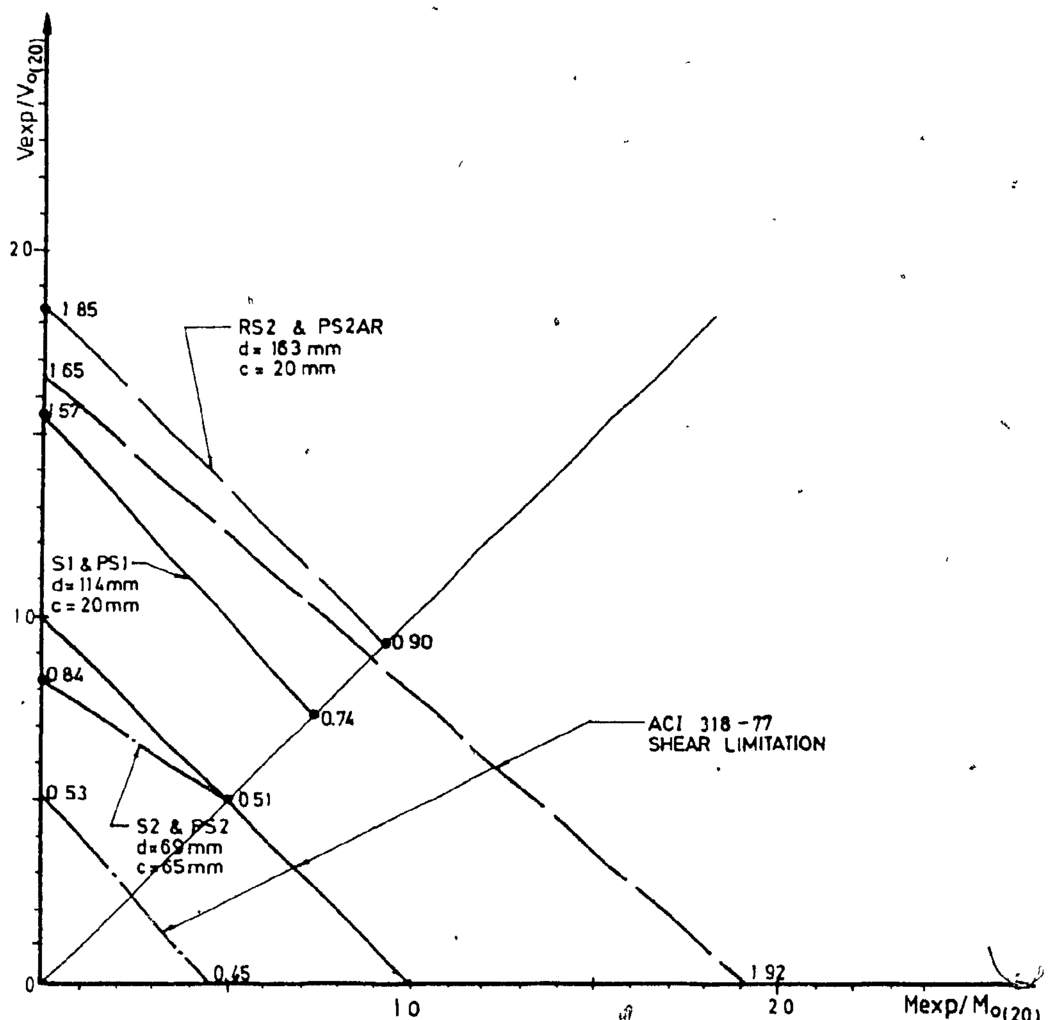


Fig. 5.5 - Comparison of Measured Strengths and Predictions of ACI 318-77 for the Repair Specimens

to the topmost layer of the reinforcement in the topping gives good predictions of the capacities. This method can be used to increase the flexural capacity and the punching shear capacity as well as improving the serviceability performance. Also shown in Fig. 5.5 are the predictions and the experimental results for the two specimens with the steel in the proper location. These two specimens provide a means of comparing the effectiveness of repair with specimens with steel properly placed.

If it is necessary to increase the flexural strength in positive moment regions, when applying the topping repair procedure the increased effective depth would result in an increase in flexural resistance. It is necessary to add temperature and shrinkage reinforcement in the topping due to the differential shrinkage between the topping and the original slab.

b) <u>Top reinforcement misplaced only at a few locations</u>

If the top reinforcement has been misplaced or ends up being misplaced after concrete placement due to lack of support at only a few column locations then it may be economical to consider a different repair procedure. A repair procedure which may be used in these circumstances was investigated by Lee[13]. The repair consisted of placing extra top bars in grooves cut into the top surface of the slab and then filling the grooves with an epoxy mortar. It was demonstrated that this repair procedure successfully increased both the flexural capacity and the punching shear capacity. In addition the serviceability was significantly improved. As with the topping repair technique, the flexural capacity can be calculated using the effective depth measured to the topmost layer of reinforcement and the punching shear capacity can be calculated assuming the average effective depth measured to the topmost layer of steel.

This method of repair may also be necessary if for some reason the slab thickness cannot be increased.

CHAPTER 6

CONCLUSIONS

The purpose of this experimental programme was to investigate the effect of misplaced reinforcement on the punching shear capacity of slab-column connections transferring moment. In addition the effectiveness of a structural topping repair method was also examined. The results from this experimental programme together with the companion direct punching shear tests performed by Lee enabled an investigation of a wide range of loading conditions.

A study of the behaviour of the experiments together with analysis of the results led to the following conclusions.

1) The reduction of the effective depth due to misplaced reinforcement (i.e. placed beyond the code tolerance) could very likely lead to inadequate serviceability performance and strength deficiencies. It is shown that the reduction of flexural and shear strengths due to misplaced reinforcement combined with the severe loading conditions during construction could lead to total structural collapse.

2) The test results show that a decrease in effective depth led to a reduction in shear and flexural capacities as well as flexural stiffness. This test series together with Lee's thesis results indicate that a small reduction in effective depth from the proper position led to large and significant decreases in shear capacity. Misplacement of the reinforcement led to a change of crack pattern from orthogonal to a radial or "spider-like" crack pattern and a corresponding increase of maximum crack width at all load levels. This marked difference in cracking pattern could enable early recognition of the existence of misplaced reinforcement in slabs.

3) Although the ACI Code provision provide a conservative prediction of shear strength of normal slab-column connections, it does not offer an accurate method of determining the punching shear strength of slabs containing misplaced reinforcement.

4) The experimental results when compared with the direct punching shear tests performed by Lee[13] indicate that misplaced reinforcement gives a larger reduction in capacity for the direct shear tests than for the tests with shear combined with moment and for the flexural tests.

5) For the range of loading conditions studied, the specimens repaired using the topping repair method show significant improvement of their overall structural performance leading to satisfactory serviceability and adequate strength. The ACI 318-77 code method conservatively predicts the shear capacity of the repaired specimens. The increase in the effective depth provided by the extra reinforcement with the additional concrete topping proved to be effective.

6) The structural topping repair method provides a procedure for general repair of a slab requiring structural improvement over a large area. The method of embedding top bars into grooves cut in the slab with epoxy mortar provides a possible means of providing more local repair of a slab which requires structural improvement in specific area only.

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NOTATION

A_c	=	area of critical section
b	=	perimeter of loaded area or column
c	=	clear concrete cover
c'	=	side length of square column or diameter of circular column
c_1	=	width of torsion face of column (Fig. 4.1)
c_2	=	width of transverse face of column (Fig. 4.1)
C_{AB} , C_{CD}	=	distance from centroid of critical section to transverse face of critical section (Fig. 4.1)
d, d_{av}	=	average effective depth
d_{20}	=	average effective depth of specimens with 20 mm concrete cover
f'_c	=	concrete cylinder compressive strength
f_y	=	yield strength of reinforcement
J_c	=	polar moment of inertia of critical section

M = applied unbalanced moment

M_o = pure unbalanced moment corresponding to shear failure around the column.

$M_{o(20)}$ = pure unbalanced moment corresponding to shear failure around the column of specimens with 20 mm cover.

M_{exp} = measured value for moment transferred to column at failure.

M'_{exp} = measured flexural strength at failure

$M_{n(20)}$ = nominal flexural strength of one-way flexural test specimens with 20 mm cover.

M_u = factored unbalanced moment

P_d = design service dead load

P_u = ultimate design load

V = applied shear force

V_n = nominal shear strength

$V_{n(20)}$ = nominal shear strength of specimens with 20 mm cover

V_o = pure punching shear strength

$V_{o(20)}$ = pure punching shear strength of specimens with 20 mm cover.

V_u = factored shear force

V_{test}, V_{exp} = measured value for shear transferred to column at failure.

v_u = maximum factored shear stress

w_d = service dead load

w_l = service live load

$\emptyset$ = capacity reduction factor

γ = ratio of service dead load to the design ultimate load.

γ_v = (see eqn. 4.3)

β_c = ratio of long side to short side of the column

Δ = deflection of shear and moment transfer specimen as defined in Fig. 4.2

APPENDIX A

EXPERIMENTAL DATA

SHEAR AND MOMENT TRANSFER TEST   SPECIMEN: S1   COVER= 20mm
EFFECTIVE DEPTH= 114mm

Load kN	Concrete Strains $\times 10^{-6}$						Steel Strains $\times 10^{-6}$												Deflections mm						Max. Crack Width mm	Comments
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	1	2	3	4	5	6		
18.6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	
48.6	211	253	187	47	20	1	21	49	51	47	53	22	12	97	33	29	21	10	.78	.20	.10	.30	.08	.0	-	
60.0	268	317	231	65	50	3	64	89	100	104	97	57	111	132	43	41	40	32	1.29	.33	.17	.54	.16	.5	-	
71.1	362	420	301	83	68	5	73	112	218	237	120	71	212	164	59	65	58	39	1.83	.47	.20	.84	.20	.13	-	
82.4	486	543	389	110	109	5	112	153	301	346	162	109	272	210	81	92	81	59	2.36	.61	.34	.18	.25	.23	.05	first flex.crack
93.7	631	676	481	136	112	10	165	216	384	391	235	145	373	293	113	122	109	91	2.89	.74	.48	1.40	.29	.25	.05	
104.9	779	809	580	157	147	17	237	273	439	472	297	243	468	316	159	157	144	134	3.34	.85	.61	1.62	.32	.28	.10	
116.2	961	972	682	183	164	20	301	346	567	584	358	266	567	456	193	196	190	192	3.89	.99	.71	1.93	.36	.29	.10	
127.5	1210	1165	801	208	199	25	362	420	746	796	424	291	679	550	228	240	251	257	4.67	1.21	.84	2.16	.40	.31	.15	
138.7	1452	1335	916	240	237	28	431	497	1087	1091	501	374	821	639	308	289	319	329	5.69	1.44	.96	2.41	.45	.33	.15	
150.0	1712	1571	1023	271	250	34	446	536	1333	1411	572	391	992	811	386	347	404	395	6.57	1.67	1.06	2.50	.49	.35	.20	
161.3	2270	2002	1246	330	317	35	533	625	1684	1756	635	511	1162	932	442	490	601	1601	7.56	1.91	1.07	2.74	.58	.38	.30	
172.5	2646	2274	2274	384	365	45	649	762	1939	2130	780	604	1331	1011	485	551	696	-	8.58	2.30	1.13	2.97	.64	.45	.40	
183.8	3916	3112	2071	430	390	43	731	859	2621	2800	860	673	1694	1135	573	678	920	-	9.52	2.41	1.24	2.99	.77	.55	.60	
195.1	4474	3459	2407	467	471	43	825	1035	3200	3470	1012	714	1880	1241	622	889	1003	-	10.80	2.73	1.33	3.51	.90	.63	.80	
206.3	-	3657	2801	500	506	35	937	1136	3817	3901	1092	855	1968	1325	-	953	1051	-	12.75	3.29	1.35	3.71	1.13	.72	1.10	
217.6	-	3835	3271	537	532	27	955	1193	-	-	1158	894	1782	1415	-	1000	1083	-	14.81	3.81	1.42	3.98	1.35	.82	1.30	
228.9	-	4011	3412	571	580	25	1039	1301	-	-	1233	1013	1706	1536	-	1053	1116	-	17.45	4.43	1.48	4.00	1.56	.97	1.60	
235.9	-	-	-	-	-	-	1241	1541	-	-	1340	1204	-	1671	-	1078	1286	-	19.53	4.97	1.53	4.03	-	-	1.80	failure

* Dead weight of slab was taken into account.

SHEAR AND MOMENT TRANSFER TEST   SPECIMEN: S2   COVER= 65mm
EFFECTIVE DEPTH= 69MM

Load kN	Concrete Strains x10^{-6}						Steel Strains x10^{-6}												Deflections mm						Max. Crack Width mm	Comments
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	1	2	3	4	5	6		
18.6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	
48.5	346	356	310	81	97	2	70	-	305	312	136	91	92	81	15	24	13	16	2.59	.65	.01	.25	.1	.03	.05	first flex. crack
60.0	724	786	556	147	136	4	153	-	546	567	249	125	201	196	37	51	48	49	3.30	.84	.04	.45	.1	.02	.05	
72.3	937	962	834	312	286	3	217	-	573	598	297	177	286	254	87	69	81	83	4.01	.96	.05	.57	.1	.06	.05	
83.0	1199	1271	1021	510	312	4	315	-	620	694	364	226	365	341	105	94	111	116	4.85	1.23	.06	.66	.1	.08	.10	
93.9	1648	1831	1742	569	701	6	439	-	897	962	652	383	497	404	220	140	203	172	6.71	1.71	.11	.89	.2	.10	.15	
105.1	2074	2294	2014	974	1033	4	597	-	1123	1011	731	482	666	547	394	256	331	314	8.18	2.11	.12	.94	.3	.16	.25	
116.4	2966	3046	2861	1432	1410	1	708	-	1569	1620	1476	709	895	754	428	374	473	386	10.69	2.73	.15	.20	.4	.17	.45	
127.7	3543	3557	3329	1700	1726	0	997	-	2175	2540	1812	1011	1240	956	548	426	574	437	14.30	3.65	.18	1.43	.52	.23	.80	
138.6	3911	3862	3836	2134	2306	2	708	-	2747	3012	2411	1350	1520	-	688	498	648	592	18.82	4.77	.19	1.69	1.00	.30	1.00	
157.8	-	-	-	2540	2571	3	512	-	3456	3847	2871	1771	1911	-	847	744	729	699	27.94	7.13	.22	1.83	1.32	.40	1.40	failure

* Dead weight of slab was taken into account.

SHEAR AND MOMENT TRANSFER TEST SPECIMEN: S3 COVER= 90mm

EFFECTIVE DEPTH= 44mm

Load kN	Concrete Strains $\times 10^{-6}$						Steel Strains $\times 10^{-6}$												Deflections mm						Max. Crack Width mm	Comments
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	1	2	3	4	5	6		
18.6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	
49.0	350	371	320	84	93	2	84	147	341	256	146	77	99	53	14	21	16	15	2.56	.40	.01	.23	.10	.03	.05	first flex. crack
60.1	777	786	598	136	142	4	195	249	535	476	269	188	198	109	41	53	43	45	3.41	.70	.04	.41	.13	.07	.05	
72.0	1243	1254	1011	310	306	8	364	405	608	811	394	349	352	240	64	91	70	87	4.36	1.05	.07	.68	.17	.09	.10	
82.5	1756	1736	1702	595	628	14	584	657	932	1442	648	555	510	347	112	130	121	117	7.62	2.01	.12	.88	.25	.11	.25	
94.9	2307	2310	2130	998	1023	18	947	897	1079	1716	768	933	624	485	164	236	170	221	12.70	3.21	.14	.99	.35	.17	.80	
105.1	2930	3010	2947	1410	1421	19	1106	1576	1710	1948	1496	1082	749	568	300	351	301	331	15.82	3.99	.17	1.12	.55	.19	1.20	
117.0	3501	3495	3455	1675	1820	21	1394	1799	250	2650	1831	1358	945	767	351	410	349	400	21.08	5.31	.18	1.36	.68	.21	1.50	
127.0	3900	3893	3886	2147	2231	15	1642	2011	-	3315	2401	-	1096	891	410	498	405	491	26.67	6.73	.19	1.58	1.05	.32	1.80	
130.0	-	-	-	-	-	-	2013	-	-	4434	2815	-	1321	1122	630	750	636	719	39.66	9.87	.23	1.79	2.15	.42	2.00	failure

* Dead weight of slab was taken into account.

SHEAR AND MOMENT TRANSFER TEST SPECIMEN: RS2 COVER= 65mm
EFFECTIVE DEPTH= 69mm

Load kN	Concrete Strains $\times 10^{-6}$						Steel Strains $\times 10^{-6}$																				Comments
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	
18.6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0									
48.6	331	329	286	57	83	2	72	95	340	320	147	87	86	70	27	21	15	17									first flex. crack
59.9	664	752	516	163	144	3	147	196	559	583	257	137	200	189	39	48	45	47									hold load
																			0	0	0	0	0	0	-	0	while repair
71.7	664	761	552	184	146	3	152	197	561	-	-	142	202	197	43	-	50	-	30	28	28	33	6	7	-	25	loading continues
82.4	752	778	562	211	156	4	162	199	586	-	-	152	206	202	47	-	59	-	60	51	78	68	17	26	-	56	
93.7	819	808	573	256	193	4	187	214	621	-	-	162	210	206	55	-	71	-	120	114	178	132	34	54	-	76	
116.2	873	868	631	280	206	5	217	239	661	-	-	187	215	215	72	-	83	-	201	153	234	217	51	88	-	96	first flex. crack
138.7	1042	898	639	314	217	6	225	264	762	-	-	222	226	220	83	-	96	-	361	236	376	331	71	122	-	163	
161.3	1239	1029	767	369	290	8	237	347	873	-	-	297	256	224	96	-	109	-	459	373	558	409	99	187	-	232	
183.8	1699	1320	926	424	386	11	292	449	1021	-	-	387	317	236	102	-	132	-	652	566	793	600	137	238	-	271	
206.3	2131	1810	1142	492	502	16	317	526	1191	-	-	487	420	247	114	-	158	-	842	809	1096	789	201	391	-	334	
228.9	2376	2095	1424	580	683	17	336	647	1398	-	-	598	476	259	128	-	179	-	1100	1118	1463	1090	331	526	-	394	
251.4	3016	2484	2087	640	879	23	358	839	1600	-	-	668	512	271	143	-	195	-	1408	1373	1784	1347	531	758	-	469	
262.7	3645	3177	2916	665	1064	17	399	1096	1811	-	-	747	564	296	160	-	231	-	1606	1570	1997	1586	871	1082	-	727	
276.4	-	-	-	-	-	-	502	1304	2056	-	-	836	623	321	187	-	254	-	2014	2001	2496	1972	1200	1571	-	1005	failure

Note: Strain gauges 10, 11, 16 and 18 malfunctioned during the repair operation.

* Dead weight of slab was taken into account.

SPECIMEN: RS2 'TOPPING' REPAIR

Load kN	Deflections mm						Max. Crack Width mm	Comments
	1	2	3	4	5	6		
18.6	0	0	0	0	0	0	0	
48.6	2.48	.63	.01	.24	.13	.01	.05	first flex. crack
59.9	3.25	.81	.03	.43	.17	.03	.05	hold the load while repair
71.7	3.50	.88	.03	.46	.19	.04	.00	loading continues
82.4	3.76	.95	.04	.56	.21	.05	.00	
93.7	4.27	1.10	.04	.71	.22	.08	.00	
116.2	5.28	1.39	.05	.86	.26	.11	.05	first flex. crack
138.7	6.04	1.59	.05	1.06	.31	.14	.05	
161.3	7.31	1.94	.06	1.31	.36	.17	.05	
183.8	8.27	1.41	.07	1.61	.42	.20	.05	
206.3	9.62	2.92	.07	1.96	.50	.23	.10	
228.9	11.01	3.12	.08	2.39	.59	.27	.30	
251.4	12.90	3.57	.09	2.79	.68	.31	.70	
262.7	14.65	4.11	1.00	3.11	.75	.35	1.00	
276.4	19.01	5.16	1.10	3.61	.89	.45	1.30	failure