NSF/RA- 770698
PB 290458

CESRL REPORT NO. 77-4
DECEMBER 1977

DESIGN AND CONSTRUCTION OF A FLOOR-WALL REACTION SYSTEM

By

KYLE A. WOODWARD
and
JAMES O. JIRSA

Report on a Research Project
Sponsored by
Research Applied to National Needs Division
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Grant ENV75–00192

DEPARTMENT OF CIVIL ENGINEERING / Structures Research Laboratory
THE UNIVERSITY OF TEXAS, AUSTIN, TEXAS

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50272-101

REPORT DOCUMENTATION PAGE	1. REPORT NO. NSF/RA-770698	2.	PB 290 458

4. Title and Subtitle
Design and Construction of a Floor-Wall Reaction System, CESRL Report No. 77-4

5. Report Date
December 1977

6.

7. Author(s)
K.A. Woodward, J.O. Jirsa

8. Performing Organization Rept. No.
CESRL 77-4

9. Performing Organization Name and Address
The University of Texas
Structures Research Laboratory
Department of Civil Engineering
Austin, Texas 78712

10. Project/Task/Work Unit No.

11. Contract(C) or Grant(G) No.
(C)
(G) ENV7500192

12. Sponsoring Organization Name and Address
Applied Science and Research Applications (ASRA)
National Science Foundation
1800 G Street, N.W.
Washington, D.C. 20550

13. Type of Report & Period Covered

14.

15. Supplementary Notes

16. Abstract (Limit: 200 words)

A Floor-Wall Reaction System was designed and constructed by the University of Texas at Austin Civil Engineering Structures Laboratory to give researchers the ability to test large-scale models using bilateral loadings in addition to axial loads. This report documents and describes the conception, design, and construction of this system in order to enhance its use and availability. The background of the project is described and detailed information is presented on the analysis, design, construction, and floor-wall loading capabilities of the floor-wall system. The appendices present a photographic description of the system's construction, construction drawings, and a table and graph of material properties.

17. Document Analysis a. Descriptors

Structural engineering	Floors	Stresses
Construction	Walls	Models
Structural design	Loads	Texas
		Civil engineering

b. Identifiers/Open-Ended Terms

Floor-Wall Reaction System

c. COSATI Field/Group

18. Availability Statement	19. Security Class (This Report)	21. No. of Pages 77
	20. Security Class (This Page)	22. Price A04-A01

(See ANSI-Z39.18) *See Instructions on Reverse* OPTIONAL FORM 272 (4-77) (Formerly NTIS-35) Department of Commerce

CAPITAL SYSTEMS GROUP, INC.
6110 EXECUTIVE BOULEVARD
SUITE 250
ROCKVILLE, MARYLAND 20852

A B S T R A C T

The University of Texas at Austin Civil Engineering Structures Research Laboratory designed and, with the close cooperation of The University of Texas System Office of Facilities Planning and Construction, built a unique test facility through a grant sponsored by the RANN Division of the National Science Foundation. The Floor-Wall Reaction System was constructed to give researchers the ability to test large-scale models using bilateral loadings in addition to axial loads.

This report documents and describes the conception, design, and construction of the Floor-Wall Reaction System to enhance its use and availability.

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CHAPTER 1

INTRODUCTION

1.1 Background

The increased interest in the design of earthquake resistant structures has led to extensive research programs being undertaken to explore the behavior of reinforced concrete members subjected to seismic loadings. To date, almost all of the research has been on members with only unidirectional or planar loads, because of the complex problems involved with testing members bidirectionally. The test facility constructed at the University of Texas Civil Engineering Structures Research Laboratory at Balcones Research Center will provide a means to study the behavior of members, structural subassemblages, and small structures under three-directional loadings with a minimum of difficulty. In this report the new facility, the Floor-Wall Reaction System, or Reaction Wall for short, will be discussed from conception to completion.

The Reaction Wall is part of a larger project to study the behavior of reinforced concrete frame members subjected to bilateral and axial loadings. Because of the size and capabilities of the floor-wall system, the facility will become a multipurpose test floor available to other researchers for a wide variety of testing. The nature of the planned research called for a permanent test facility rather than a temporary testing frame because large load requirements and maximum adaptability to varying test conditions were necessary. Construction of the facility was highly feasible because no other location possessed the many advantages that the Research Center offered.

A major advantage in building the Reaction Wall at the Structures Laboratory of The University of Texas at Austin was the ability to construct it inside the existing Laboratory structure. This allowed construction during the winter, thereby taking advantage of what is usually the contractor's slack period, so that an attractive construction bid could be obtained and the project completed within the period required. Because construction was indoors, no weather delays were anticipated. The existence of the building eliminated costly and time-consuming site preparation, since nothing was located in the area of construction except a 6-in. slab on grade. The Laboratory staff included experienced faculty members who had designed other permanent test facilities and had the expertise to make the fullest possible use of the Reaction Wall. In addition to the faculty, the Laboratory staff included a capable group of technicians to maintain the sophisticated electronic and hydraulic equipment that would be used in conjunction with the Reaction Wall.

Briefly, the Civil Engineering Structures Research Laboratory had the personnel, equipment, and space to build the Reaction Wall at a minimum cost and then make the greatest possible use of it for research purposes.

1.2 Acknowledgements

The construction of the Floor-Wall Reaction System was supported by the RANN Division of the National Science Foundation under Grant ENV75-00192.

The initial design was carried out by Professor Russell H. Brown (Visiting Associate Professor, Summer 1976) and Professor James O. Jirsa, with the consultation of other faculty members in the Structures Laboratory, particularly Professor John E. Breen and Professor Joseph A. Yura. The construction drawings and contract specifications were prepared by the University of Texas System Office of Facilities Planning and Construction under the direction

of Mr. B. J. Broadus, Project Manager, who was extremely helpful in assisting the staff during the planning stages and during construction.

The construction contract was awarded in December 1976 to the Thomas Hinderer Company and construction began in January 1977 with completion in June 1977. Cooperation between the contractor, University of Texas System personnel, and Laboratory staff permitted adjustments in the construction details to be made without difficulty. The attention to details and quality of workmanship on the part of the contractor were in large part responsible for the success of the project.

Materials testing services were provided by Trinity Engineering Testing Corporation of Austin. Post-tensioning materials were furnished by Dyckerhoff and Widmann, Inc. of Chicago.

Mr. Kyle Woodward, Graduate Research Assistant, was responsible for the photographic record of the project and prepared this report as part of the requirements for the Master of Science degree.

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CHAPTER 2

PRELIMINARY DESIGN

Before any attempt was made to develop a configuration for the Reaction Wall, the dimensional restraints imposed by the planned building site were studied. The height of the Reaction Wall above the existing floor was limited to approximately 20 ft. by the clearance required for an overhead crane. The lateral dimensions of the floor slab were limited by existing structural details of the laboratory building, such as roof columns on two sides and a test slab and the end of the building on the other two sides. However, even with these restraints the available floor area for construction exceeded 2,000 sq. ft.

The first general outline of the Reaction Wall was a floor slab with contiguous vertical walls, the walls having buttresses, also contiguous with the slab, spaced along their length. At this point a benefit-cost study was done to determine the best compromise between load capacity and cost for various components of the Reaction Wall. The floor slab was of particular interest, since the cost of its construction was heavily influenced by its depth. The excavation required for the slab had to be made through limestone, which increased in soundness with depth. Therefore, it was important that a depth be picked which gave sufficient load capacity at a low unit cost. The economic study revealed that the optimum depth of the floor slab was between 3 ft.-6 in. and 5 ft., and based on this information a floor slab depth of 4 ft.-6 in. was chosen. The buttresses were also studied to examine the most efficient shape and reinforcing details. At first a sloping buttress was considered to take advantage of the anticipated linear moment diagram in the buttresses, but too many complications arose, such as the difficulties in detailing the reinforcement and post-tensioning. In the end, straight rectangular buttresses were used.

Using the results of the economic study, a specific configuration for the Reaction Wall was proposed. As can be seen in Fig. 2.1, it had a biased strength axis with a very large buttress at the intersection of the walls. The main function of this buttress was to provide a way of testing uniaxially large shear walls using high loads. However, there were three drawbacks to this first configuration that prompted a reevaluation of the design. The first drawback was the varying load capacity along the wall because of the buttress layout. The second drawback was the inefficient use of the available floor area. A large part of the slab area could not be used to anchor test frame members, which limited the adaptibility of the Reaction Wall to changing test requirements. The third drawback was that the floor bolt groups in the proposed slab could not be aligned with the floor bolt groups in the existing test slab. A new configuration was proposed which eliminated these drawbacks and embodied much of what was actually built.

The new configuration (Fig. 2.2) made better use of the floor space by shifting the walls to a north-south east-west orientation and extending them the length of the slab. Also, the wall capacity was made uniform by having identical buttresses around the wall. A drop in the maximum uniaxial load capacity resulted from these changes, but it was not large enough to outweigh the advantage of uniform load capacity along the walls and more uniform construction details. The extension of the walls at their intersection eliminated the need for bending the reinforcement around the corner, a difficult and costly detail.

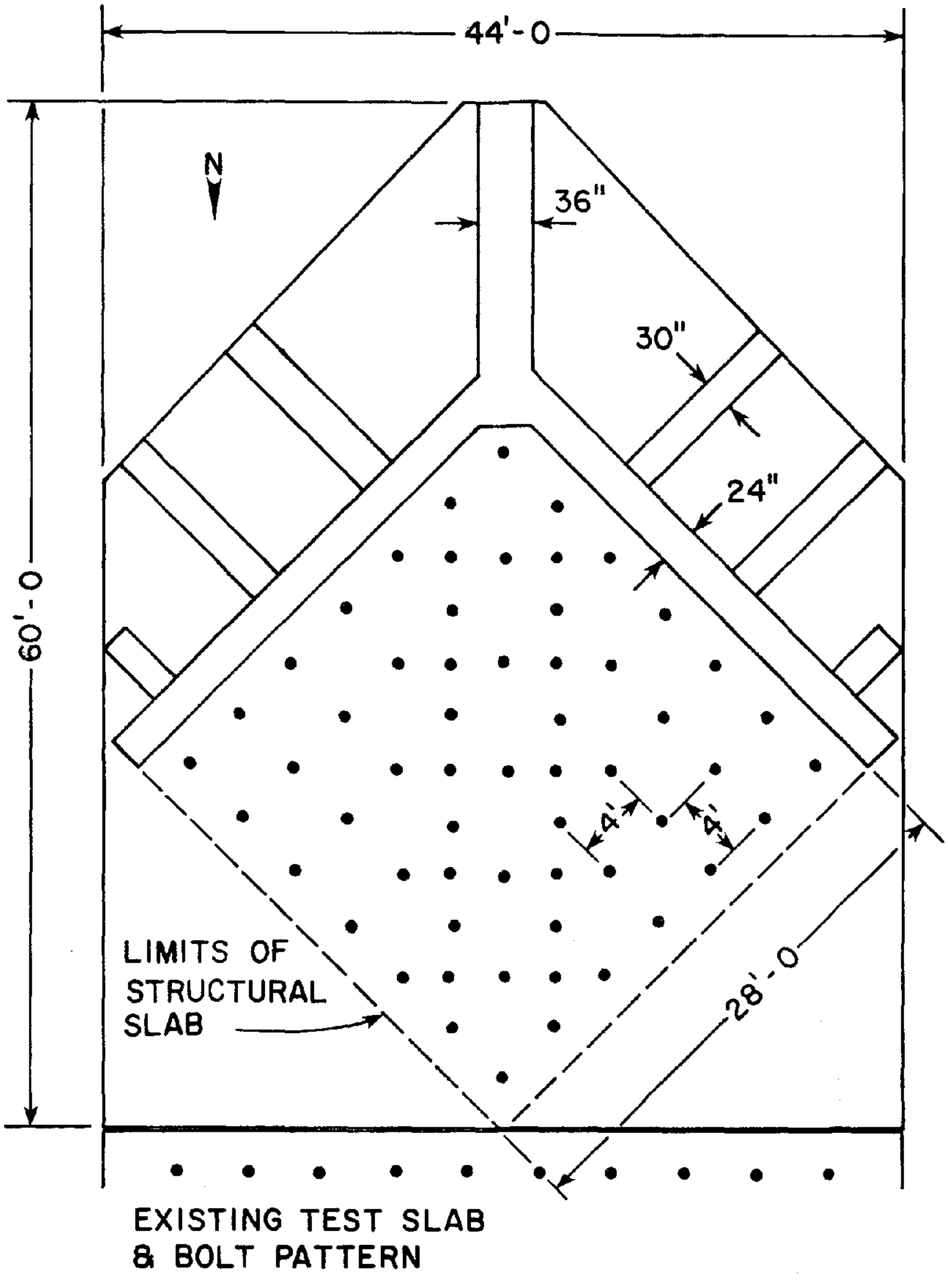

Fig. 2.1 Initially proposed configuration of Reaction Wall

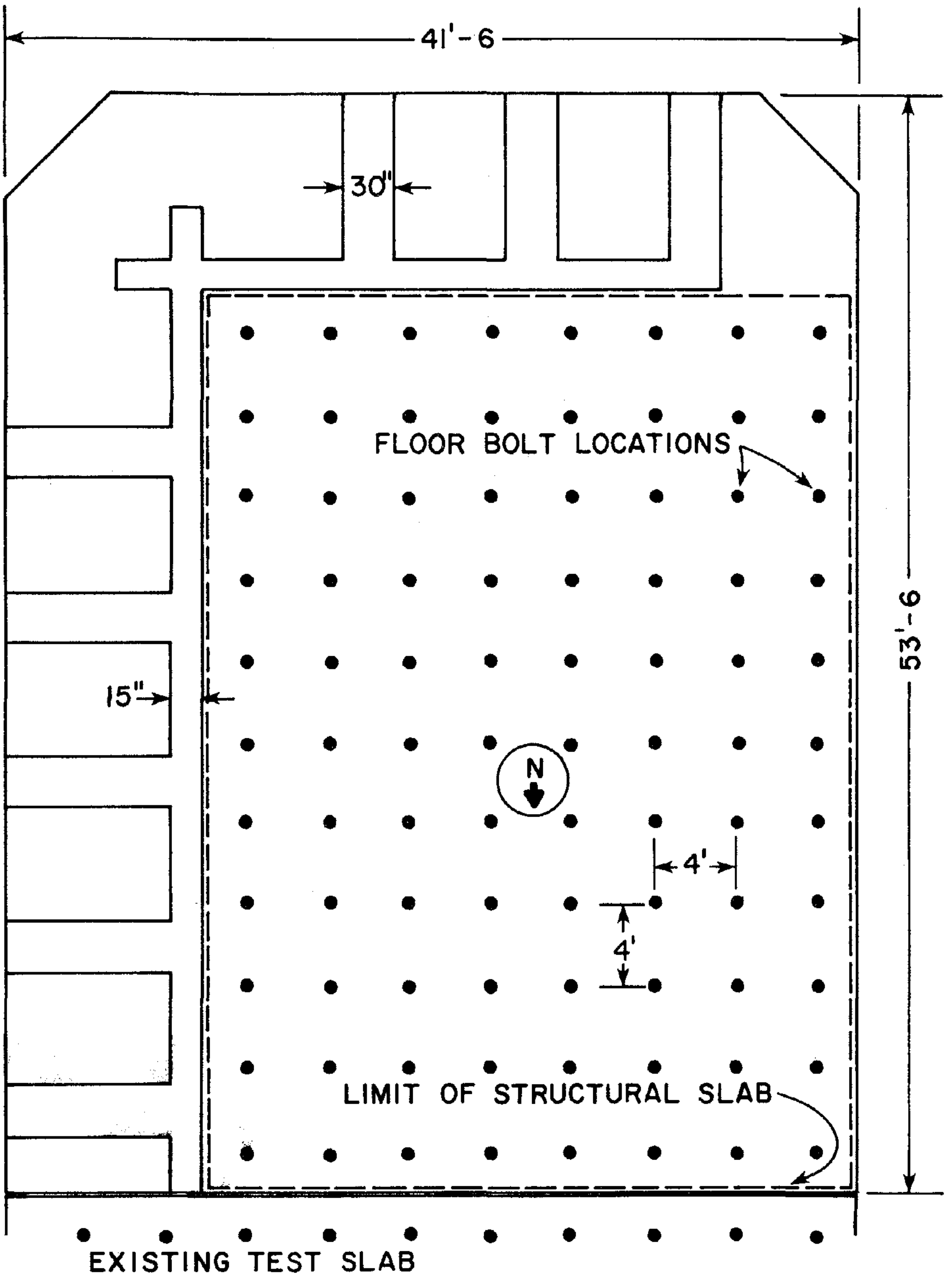

Fig. 2.2 Final proposed configuration of Reaction Wall

CHAPTER 3

ANALYSIS AND DESIGN

3.1 Introduction

The Reaction Wall was divided into three separate components to simplify it to a point where the analysis could be done by hand. This approach neglected the influence of interaction between the components, but it made a relatively quick analysis possible, reducing the time and cost required to design the Reaction Wall. Since each component was looked at independently, each will be discussed similarly.

3.2 Slab

There were three items to be designed in the floor slab--the concrete, the reinforcement, and the method of providing anchorage to the floor slab. The depth of the floor slab had been previously determined to be 4 ft.-6 in., based on the economic study of capacity versus depth.

The concrete used in the floor slab was to have a minimum 28-day compressive cylinder strength of 4000 psi. Primarily, this strength was selected to ensure a good quality concrete which was readily available locally.

Studies showed that the in-place cost of single large diameter reinforcing bars was approximately 18 percent higher than the in-place cost of several smaller diameter rebars which were bundled. Because of this, bundled bars were used in the floor slab with each bundle consisting of three #11 rebars, forming a triangle as seen from the ends. Cost of the reinforcement was a major concern because such a large amount of reinforcement was used in the construction of the floor slab. Over 111,000 lbs of reinforcing steel was used for the longitudinal slab reinforcement alone.

At both the top and bottom of the slab a layer of #11 bundled rebars was placed in each direction, as illustrated in Fig. 3.1.

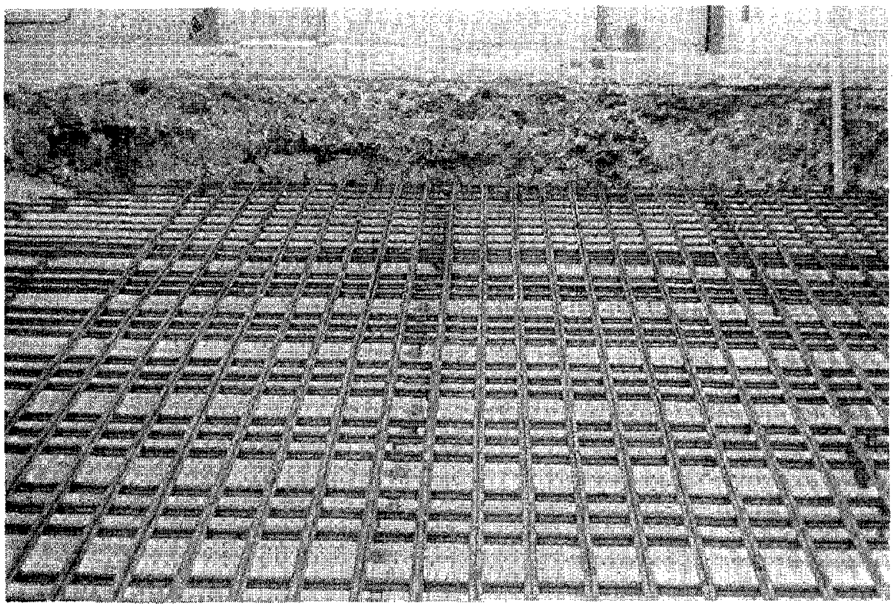

Fig. 3.1 Bottom layer of slab reinforcement

Each layer had a bundle spaced every 12 in. This spacing provided provided sufficient clear space for concrete placement and ensured that the rebars did not interfere with the anchor bolt patterns in the floor slab, which were on 4 ft. centers. The spaces between groups of three bundles in Fig. 3.1 indicate the planned locations of the floor anchorage details.

The principal design criterion for the Reaction Wall was to maximize lateral load capacity, and for this reason as much longitudinal reinforcement as could be reasonably fit into the floor slab was used. The idea was to have the largest ultimate moment capacity possible in the floor slab, since all lateral loads on the Reaction Wall eventually had to be transferred to the slab. Thus the larger the moment capacity, the greater the lateral load capacity. The ultimate moment capacity of the slab was found by taking a typical 8 ft. wide section of the slab, Fig. 3.2, and analyzing it as an ordinary rectangular section. The 8 ft. came from assuming that the lateral load on a buttress would be transferred through an 8 ft. wide section of the slab. In other words, splitting the distance between the buttresses on either side of the buttress in question. With the ultimate moment capacity of the floor slab known, the ultimate lateral load on a buttress was determined by dividing the floor slab moment capacity by the maximum vertical distance possible between the point of lateral load application and the neutral axis of the floor slab (see Fig. 3.3). It should be noted that in the analysis of the slab the soil reaction to the applied loads was omitted. The Reaction Wall was designed as a self-equilibrating system, ignoring soil-structure interaction in the analysis.

At this point it should be noted that in the design of the Reaction Wall no φ factors were used. Also, the only load factor used was a 35 percent amplification on all loads rather than the load factors recommended by ACI 318-71.[1] These changes were justified on the basis that all loads applied to the Reaction Wall would be known or could be controlled closely. In addition, the quality of the materials used in the Reaction Wall and the construction itself would be carefully supervised.

In addition to equilibrating the lateral loads on the Reaction Wall, the floor slab had to provide anchorage for testing frames and equipment. In a test slab built previously, the anchorage detail consisted of nuts welded to the underside of channels embedded in the slab. A pipe extended through the slab to the

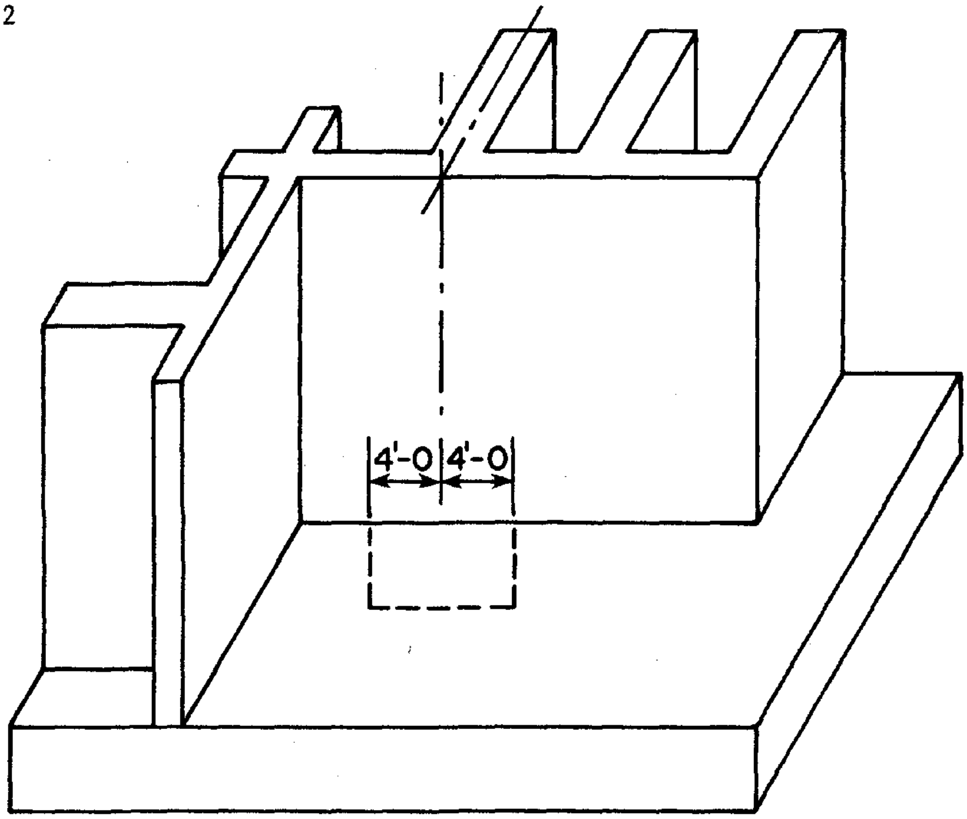

Fig. 3.2 Slab design section

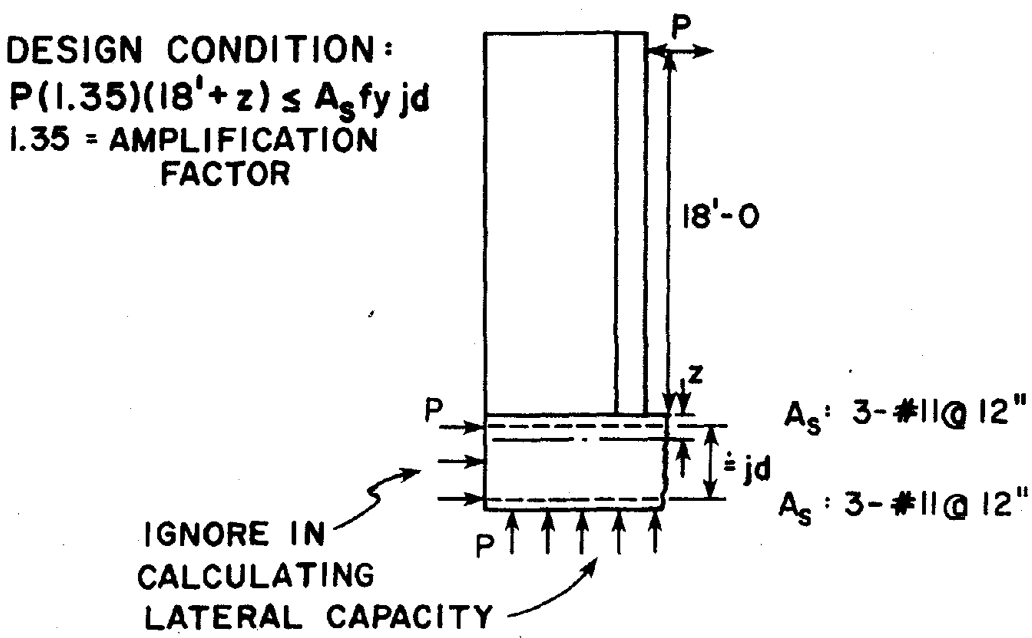

Fig. 3.3 Lateral load capacity determination

channel to provide a passageway for threaded rods to be passed through the slab into the nuts. Because the system had proven very successful, the same method was adopted for the new construction with only minor modifications in the manner of fabrication.

The floor slab bolt pattern was a continuation of the pattern used in the existing test slab, since the two slabs were contiguous. The pattern shown in Fig. 3.4 consists of bolt groups on a 4 ft. grid with each group made up of four bolts on an 8 in. square pattern.

The detail at the channel is shown in Fig. 3.5. Each group was designed for a 200 kip load. A 1-1/4 in. diameter bolt was selected and 2H heavy hex nuts were welded to the underside of the channel. Metal caps and galvanized conduit were welded to the channels for watertightness to prevent cement paste from intruding and filling the void. A bolt with a yield strength of 60 ksi would provide around 50 kips tensile strength. The 2H nuts have greater strength and could be used with higher strength bolts to increase the anchorage capacity. The channels were 40 ft. long to extend the length of the floor slab in an east-west direction. The channel replaced every fourth bar bundle in the bottom layer of slab reinforcement in the east-west direction. To provide anchorage at the end under the wall and permit the channel to be fully developed as tension reinforcement, studs were welded to the channels.

The contractor elected to have a fabricator provide the entire anchorage assembly for placement in the floor slab. The fabricator drilled holes in the channels to form the pattern of a bolt group. On the underside of the channels, nuts were welded at each of the holes and a metal cap was then welded over the nut to seal it. Although the specifications did not require the nuts to be held tightly in place during welding, it would have been desirable to have done so because a check showed that some nuts became skewed during welding and would not permit the threaded anchor rods to

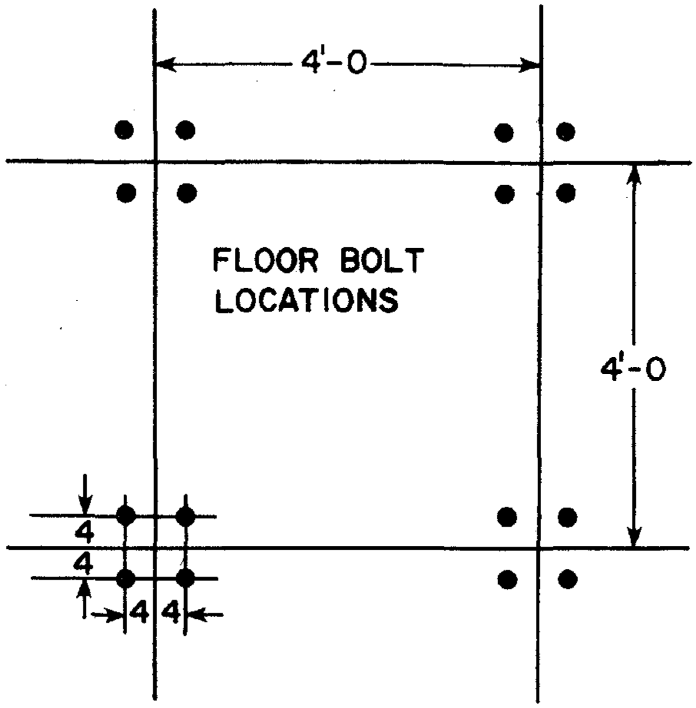

Fig. 3.4 Floor slab bolt pattern

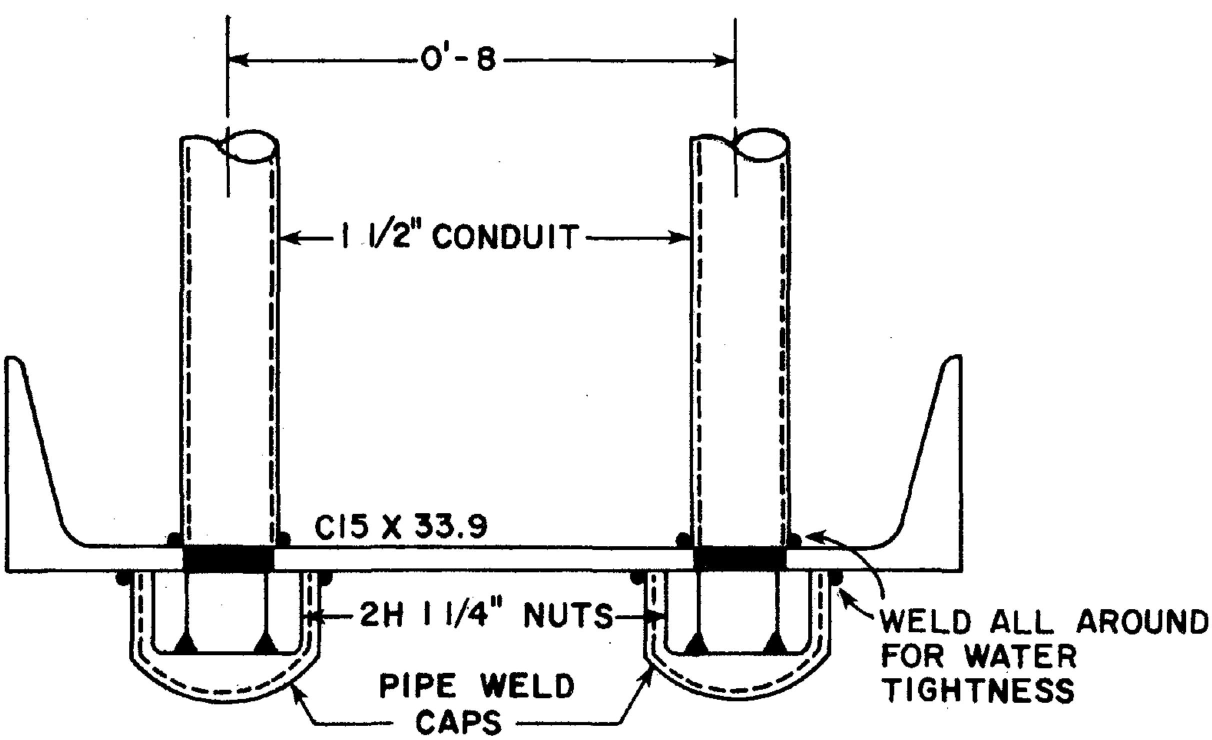

Fig. 3.5 Floor slab anchor bolt assembly

engage the threads. These problems will be discussed in Chapter 4. On the top of each channel, conduit was welded at each hole location to provide the passageway through the floor slab for the threaded rods. At one end of each channel shear studs were attached to provide the necessary shear transfer capacity along the channels. These channels were placed at 4 ft. intervals in the bottom of the floor slab with the reinforcing steel thus forming the bolt groups at 4 ft. intervals. The channels are shown in place in the floor slab in Fig. 3.6.

It should be noted that the shear capacity of the floor slab in a "pull-out" mode (two-way shear) was more than adequate for the loads anticipated on the anchor bolt groups which were 200 kips rated load per bolt group, based on four 1-1/4 in. diameter bolts of ASTM A193, Grade B7 or better.

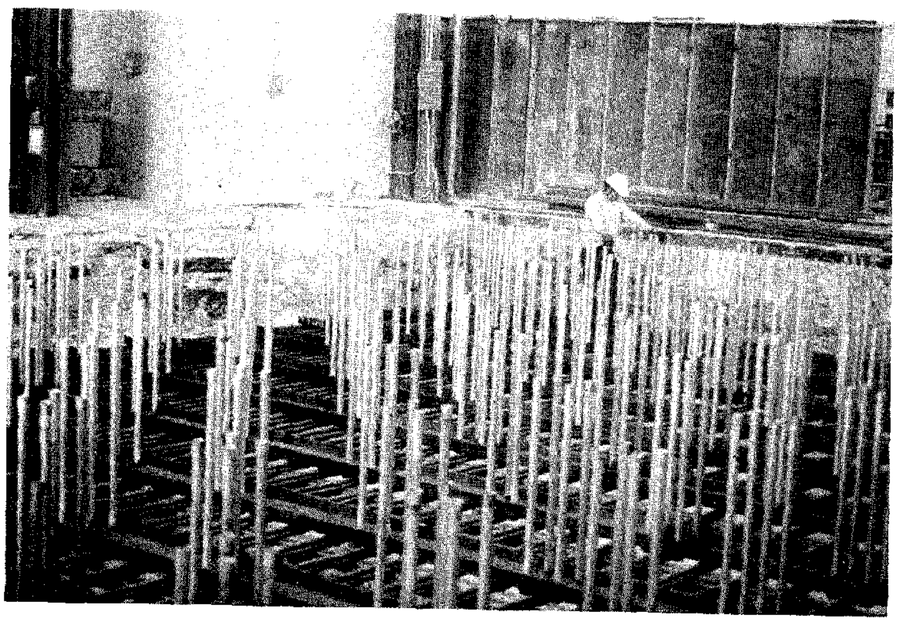

Fig. 3.6 Channel assemblies in place

3.3 Buttresses

Essentially, there were two major design variables for the buttresses, the dimensions and the reinforcement. The concrete strength was to be identical to that of the floor slab for the same reasons.

The dimensions of the buttresses were set for reasons other than load capacity because there were preconditions which limited the options available to the designers. One precondition was that the wall bolt groups, holes in the walls to pass a bolt through, but not threaded in any way, were to be aligned with the floor slab bolt groups. This meant that there would be 4 ft. between bolt groups so that a buttress would either have to fit within the 4 ft. distance or extend far enough to encompass two wall bolt groups (see Fig. 3.7). It was quickly obvious that the less thick buttress combined with a medium thick wall was the most desirable choice. Therefore, the buttresses were dimensioned to fit within the 4 ft. spacing. Subtracting the required clearances for the wall bolt groups resulted in buttresses 30 in. wide. Next, the depth was selected through a very similar process of elimination.

Since the floor slab bolt spacing was 4 ft. between bolt groups, the buttress depths could only be changed in 4 ft. increments in order to add or delete rows of floor slab bolt groups efficiently. Using the available space, the buttresses could be 4 ft., 8 ft., 12 ft., and so on, but the 4 ft. depth was eliminated immediately as being unrealistically short and providing too small a lateral force capacity. It was also decided that anything larger than 8 ft. would reduce the available floor space too much. This left only one choice--8 ft.

After sizing the buttress, the reinforcement was designed to provide sufficient shear and moment capacity to resist the design load previously calculated from the ultimate moment capacity of the

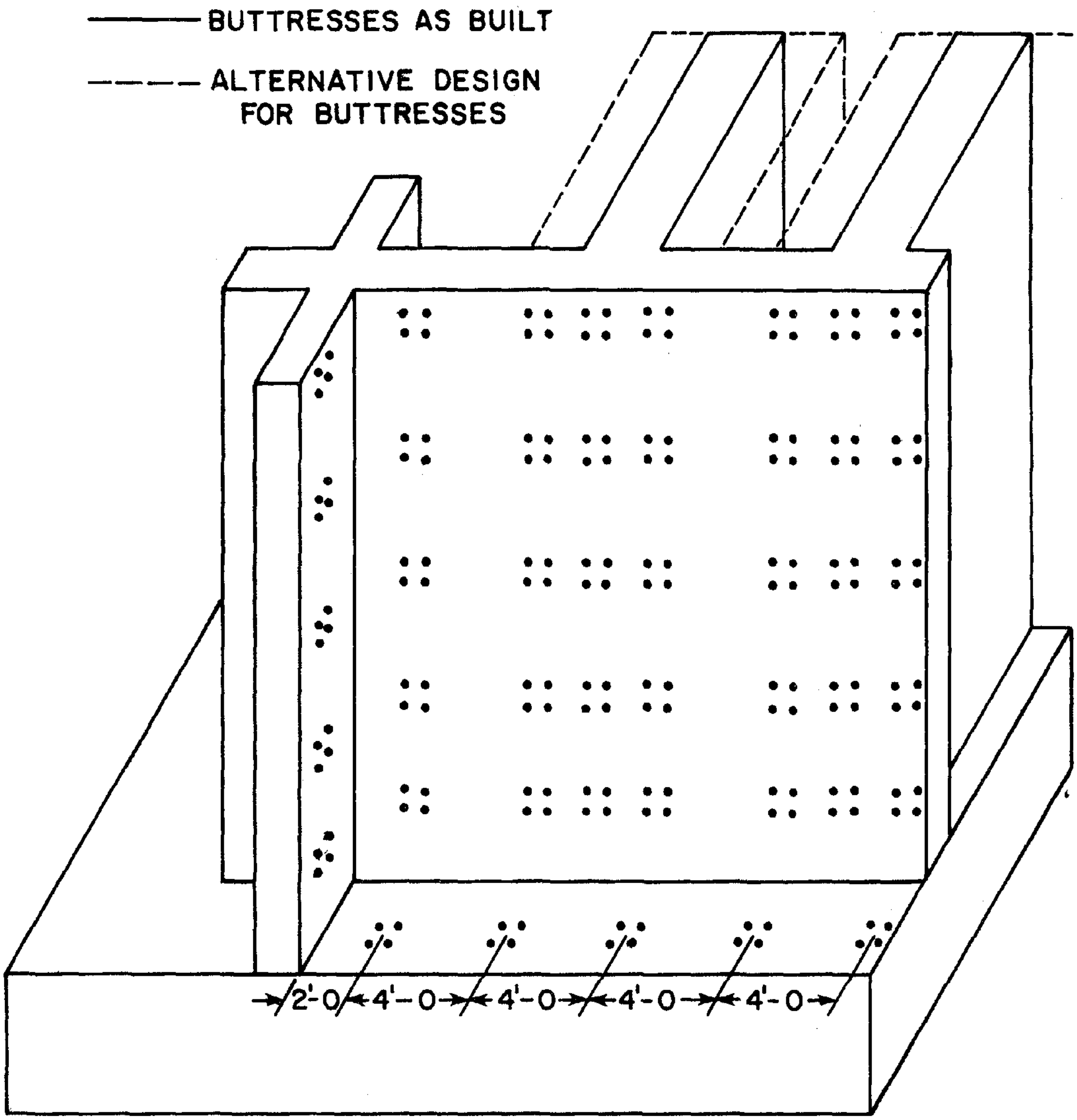

Fig. 3.7 Buttress width alternatives

floor slab. The design lateral load was assumed to act at the top buttress load point and as it was the only load the shear would then be constant along the height of the buttress with a linearly varying moment diagram having its maximum value at the base of the buttress. To analyze the buttress, a tee-shaped section was selected at the base of the buttress, which included 4 ft. of wall on either side of the buttress centerline, splitting the distance between buttresses on either side. The section analyzed is illustrated in Fig. 3.8.

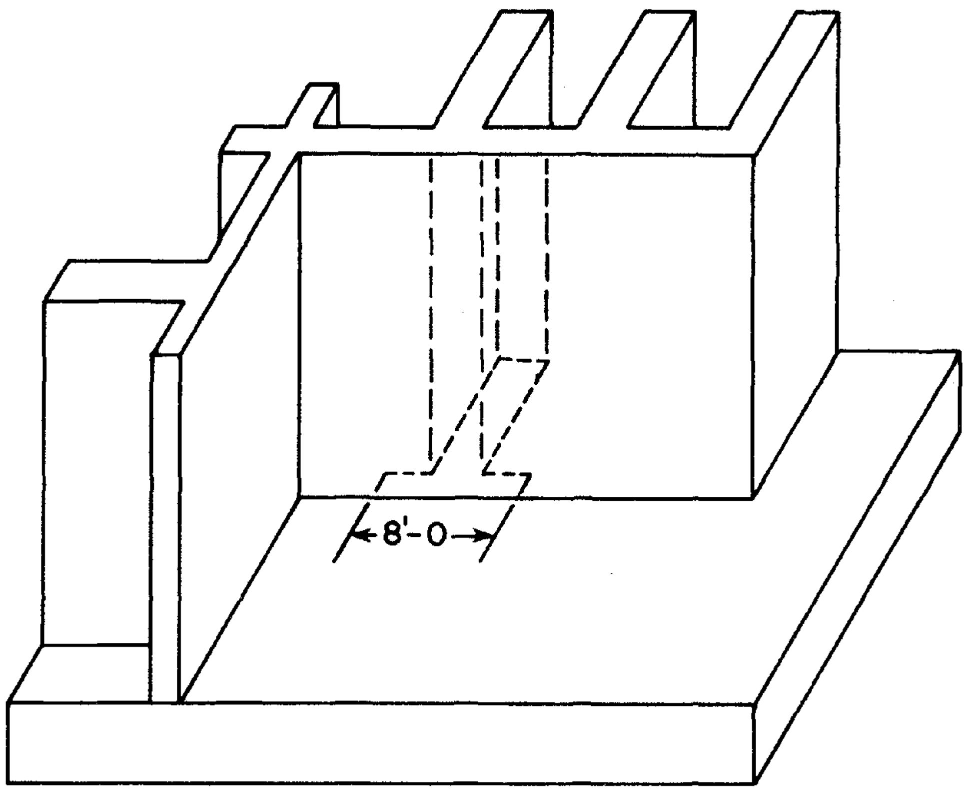

Fig. 3.8 Buttress design section

Before the analysis of the buttresses was begun, consideration was given to post-tensioning the buttresses so that an active restraint was present to close any shear cracks which might occur in the buttresses and to improve shear capacity. Also, the post-tensioning tendons served as the main longitudinal reinforcement and provided an ultimate moment capacity more than adequate for the design lateral load. Post-tensioning was more desirable than using standard reinforcement, which would also provide strength but only after permanent cracking had occurred. The maintenance of an uncracked structure was deemed necessary because of the possibility of using the Reaction Wall for fatigue testing.

Several post-tensioning systems were considered and cable tendons were rejected because of the difficulties in stressing and anchoring the short tendons. Threaded bar systems were decided on and Dywidag threadbar was selected because it was the least expensive system available and was ideal for meeting the design requirements of ease of restressing and possibility of making positive connections to the exposed threadbar ends.

Dywidag threadbar is a high-strength steel reinforcing bar which is rolled with deformations in the form of threads. As a result, splicing two threadbars together is easily accomplished by using a threaded steel coupler into which the two bars are screwed. Ends are anchored with special nuts tightened against end plates as the bar is tensioned.

Sixteen Dywidag threadbars were used in each buttress. Bars with a nominal diameter of 1-1/4 in. and an ultimate tensile strength of 160 ksi were used. The bars were stressed to 120 ksi and then released to 112 ksi, where they were then anchored. The stress after losses was estimated to be 96 ksi.

The permissible shear stress in the buttresses was computed following the recommendations in Sec. 11.5.2 of ACI 318-71. The post-tensioning yielded a 28 percent increase in the permissible shear stress, but a minimum amount of reinforcement was still required. To provide this minimum, #5 bars at 9 in. spacings were used both horizontally and vertically in the buttresses. This reinforcement gave an additional 85 percent increase in the permissible shear stress over that with just the post-tensioning. Table 3.1 describes the steps used in calculating the shear capacities.

3.4 Walls

The last component designed was the wall. The buttresses were to be the principal load-carrying members and the wall was designed to distribute the load to the buttresses without becoming too thick. As with the buttresses, post-tensioning was used, but for the walls it was necessary to provide shear capacity and control flexural cracking, since the thickness of the wall was to be kept to a minimum.

The wall was analyzed in a simplistic manner by taking a strip of the wall between the intersection of the walls and an adjacent buttress and treating it as a beam fixed at each end. Design loads were applied at the wall load points. The strip had its centerline along a bolt group centerline and extended 2 ft. on either side of it to split the distance between bolt groups on either side (see Fig. 3.9). The ultimate shear capacity at the buttress wall intersection was deficient for a critical load case (Fig. 3.10) and rather than increase the post-tensioning force, bent bars were used at the intersection, which extended from the walls into the buttresses forming hangers, as shown in Fig. 3.11. A minimum amount of temperature and shrinkage reinforcement was provided vertically and horizontally (#5 bars at 12 in.) in the walls for serviceability and shear capacity.

3.5 Finite Element Study

Once the design of the Reaction Wall was complete, a more sophisticated analysis was undertaken and presented as a special report,[2] the results of which will be summarized in this section.

The primary focus of the study was to check the adequacy of the walls, especially at the buttress-wall intersections under

TABLE 3.1 SHEAR CAPACITY CALCULATIONS

Case I: No axial compression; no shear reinforcement

$$v_{c1} = 2\sqrt{f'_c} = 2\sqrt{4000} = 126 \text{ psi}$$

Case II: Axial compression; no shear reinforcement

$$v_{c2} = 2(1 + 0.0005\ N_u/A_g)\sqrt{f'_c}$$

$$N_u = 16 \text{ bars x } 1.25 \text{ sq. in. per bar x } 96 \text{ ksi} = 1920 \text{ kips}$$

$$v_{c2} = 2\left(1 + 0.0005\ \frac{(1920)\quad(1000)}{(30 \text{ in.})(114 \text{ in.})}\right)\sqrt{4000} = 162 \text{ psi}$$

$$\%\Delta = \frac{162 - 126}{126} \text{ x } 100\% = 28\%$$

Case III: Axial compression; shear reinforcement

$$A_v = (v_u - v_c)\frac{b_w s}{f_y} \qquad A_v = 2 \text{ x } 0.31 \text{ sq. in.} = 0.62 \text{ sq. in.}$$

$$s = 9 \text{ in.}$$

$$v_u - v_c = \frac{0.62(60000)}{30(9)} \qquad f_y = 60000 \text{ psi}$$

$$v_u - v_c = 138 \text{ psi} \qquad b_w = 30 \text{ in.}$$

$$v_{c3} = 138 \text{ psi} + 162 \text{ psi} = 300 \text{ psi}$$

$$\%\Delta = \frac{300 - 162}{162} \text{ x } 100\% = 85\%$$

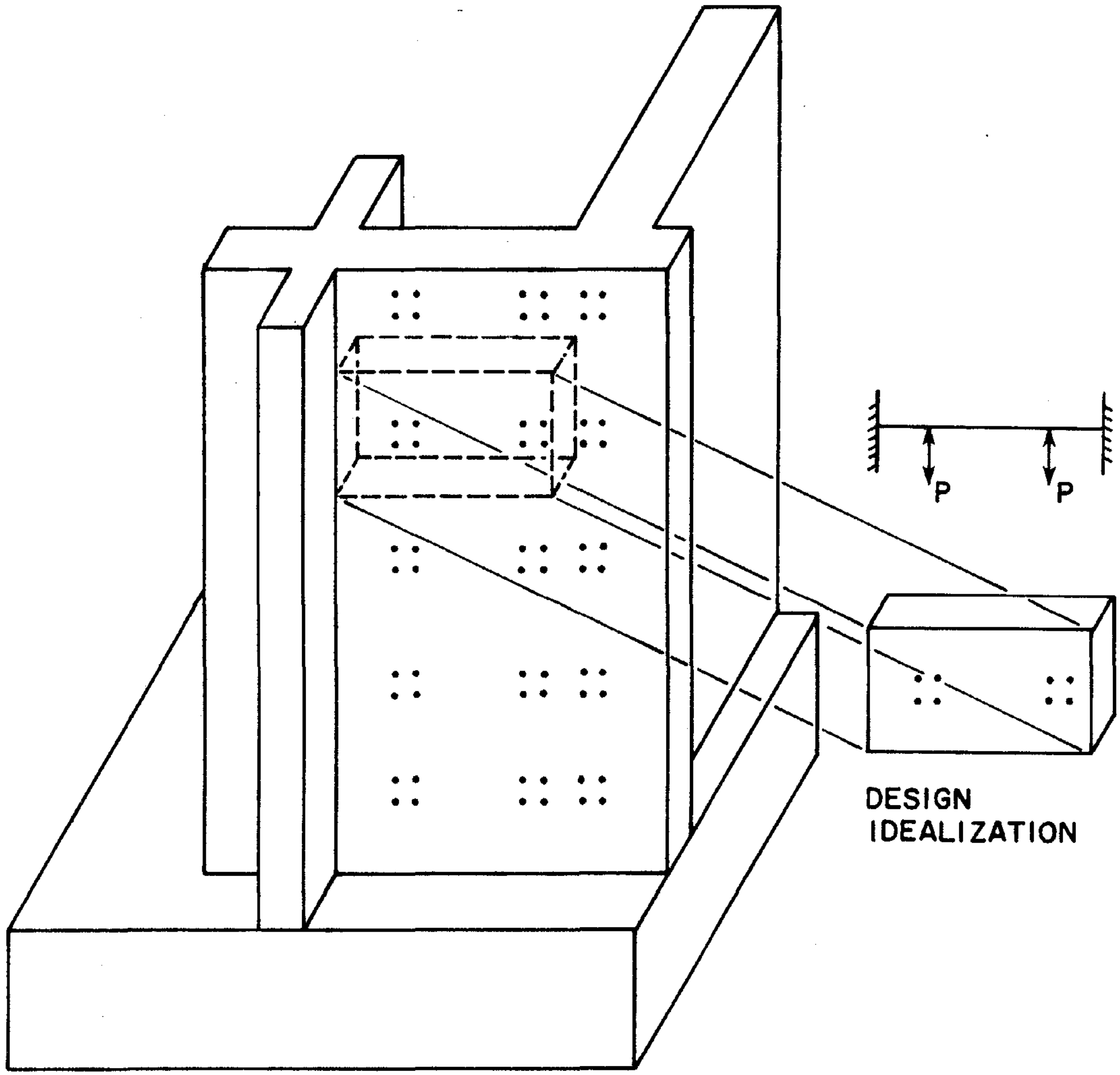

Fig. 3.9 Wall design section

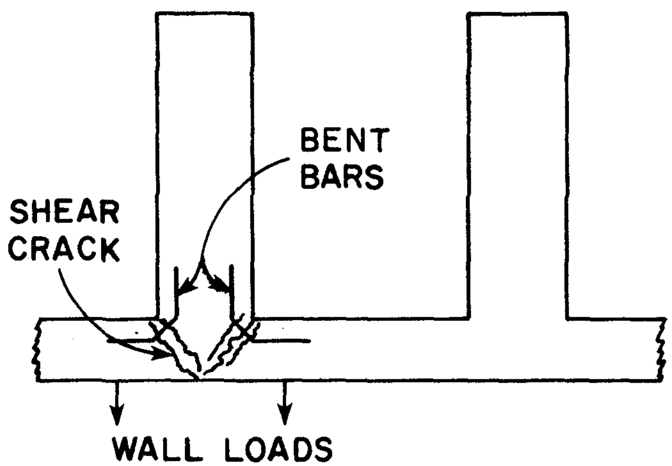

Fig. 3.10 Bent bar detail

Fig. 3.11 Bent bar detail

various critical loading conditions. A two-dimensional finite element program was tailored to the Reaction Wall geometry to obtain the desired information. The program was a major step forward compared to the hand analysis, but it too had limitations which made the use of the results questionable for all except general trend studies. Essentially, the Reaction Wall was too complicated a structure to adequately model as a two-dimensional assembly of elements. The attempt led to the use of a number of assumptions, none of which could be sufficiently tested within the framework of the study to determine their validity.

With the limitations of the program noted and considered, several load cases were analyzed and the results studied.

The first load case was the design wall load of 100 kips applied at the top-most wall load point adjacent to an interior buttress. Preliminary work showed that even though each load point could be made up of a load distributed over a bearing area or up to four individual points of loading because of the pipe groups, the results were not affected by using only a single concentrated load to represent other load cases. There were several more load cases where the wall load was shifted down toward the floor slab along the same line of load points as used in the first case. Loads were also applied at load points adjacent to an exterior buttress, and, finally, adjacent to a buttress closest to the wall intersection.

The results of these cases gave a clear indication that locally very high stresses were present at the buttress-wall intersections. To reduce the stresses and provide an extra margin of strength, the wall thicknesses were increased from the original 15 in. to 18 in.

CHAPTER 4

CONSTRUCTION

4.1 Introduction

Construction was begun in early January 1977 and the Reaction Wall was accepted by The University of Texas in mid-June. The construction period was slightly longer than originally anticipated due to several unexpected difficulties. The Reaction Wall was built at a cost below the engineer's estimate and the final results were generally acceptable. In the following sections, the construction will be discussed chronologically. A pictorial description of the construction is presented in Appendix A. Also, excerpts from the construction plans are shown for certain elements of the wall in Appendix B.

Prior to the construction of the floor slab, a suggestion was made by the contractor that the concrete for the floor slab be pumped rather than placed using dump buckets and an overhead crane as initially proposed. The advantages of pumping were so significant, faster placement and less chance of delays, that the suggestion was accepted. The original concrete specifications had to be altered, reducing the aggregate sizes and increasing the slump to accommodate the use of pumps.

The only other major departure from the original construction concept was the sequences of construction of the wall and buttresses. It was anticipated that the contractor would choose to reuse formwork. Therefore, three vertical and one horizontal construction joints were specified in the plans. However, for purposes of form alignment and elimination of bulkheads at vertical joints, the contractor elected to eliminate vertical joints and to cast the entire wall and buttress section in two stages with just the one horizontal joint. Formwork will be discussed in greater detail later.

4.2 Floor Slab

During the excavation for the floor slab, it was discovered that existing machinery foundations were larger than described in the original building plans. Removal of the rather massive concrete slabs delayed the excavation somewhat and increased excavation costs.

After the excavation was completed, at a depth of about 4 ft.-8 in., a thin layer of low strength concrete was pumpted into the excavation to form a clean, level working surface (mud slab) on which to place the floor slab reinforcement. The pumping operation went very smoothly, eliminating any doubts regarding the feasibility of pumping for the remainder of the construction.

With the mud slab cast and hardened (Fig. 4.1), placement of the floor slab reinforcement began. For proper cover on the bottom layer (3 in.) brick chairs were used. To form the bundles of rebars, the contractor began "tack" welding rather than tying the bars together with wire. Welding was stopped as soon as it became known to the project staff, but not before thirteen bundles were tack welded (Fig. 4.2). The decision was made immediately to replace all welded bars. Although such a practice was not anticipated, it would have been desirable to expressly prohibit any welding of bars during fabrication.

Using tied bundles, placement of the reinforcement in the bottom layers proceeded quickly. At this point, the prefabricated assemblies for the floor bolts were to be placed directly on the bottom layers of reinforcement. Inspection of the assemblies prior to placement revealed that many of the welded nuts were unusable, the threaded rods could not be engaged in the nuts. It was also

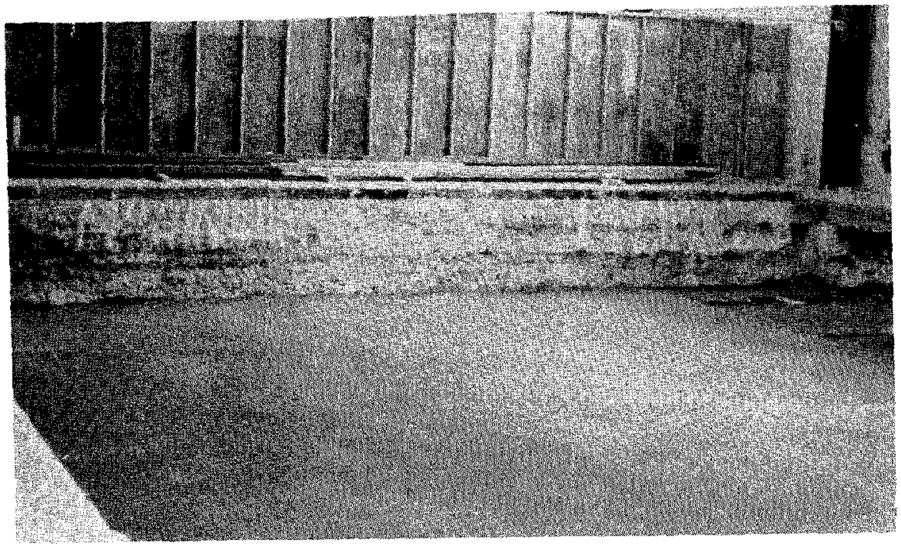

Fig. 4.1 Hardened mud slab

Fig. 4.2 Welded rebar

discovered that the conduits were not always properly aligned at right angles with the channel to form the 8 in. square pattern at their free ends (Fig. 4.3). In order to correct the problems, the assemblies were returned to the fabricator to be thoroughly inspected and repaired. In most cases the problem could be alleviated by running a tap into the threads and cleaning the nuts. In some cases the nuts were skewed during welding and had to be replaced. Conduit misalignment was not corrected because the contractor felt he could provide proper dimensional tolerances prior to casting.

With the anchorage assemblies in place (see Fig. 3.6), the upper slab reinforcement was placed. The upper layers of slab reinforcement were supported on triangular chairs made of reinforcing bars and can be seen in Fig. 4.4. Placement of the top layers of steel proceeded smoothly and quickly.

After the floor slab reinforcing steel was placed, special measures were taken to correct the alignment of the conduits for the floor anchors. Basically, there were two dimensions which had to be maintained. The first was the 4 ft. center-to-center distance between bolt groups, and the second was the 8 in. center-to-center distance between the individual conduits in a group. The tolerances were 1/4 and 1/8 in., respectively.

To ensure the 8 in. pattern, a square wooden template drilled with holes to go over the conduits was used to position the conduits. With the square templates in place, lengths of 2x4 boards were nailed to position the groups at 4 ft. centers and to prevent the group from rotating. As shown in Fig. 4.4, the result was a large framework designed to keep everything in its proper place.

The framework could not be kept in place during the entire casting of the floor slab, since the templates were below the floor slab surface. This meant that in order for the conduits to remain in their correct positions the concrete that was placed prior to

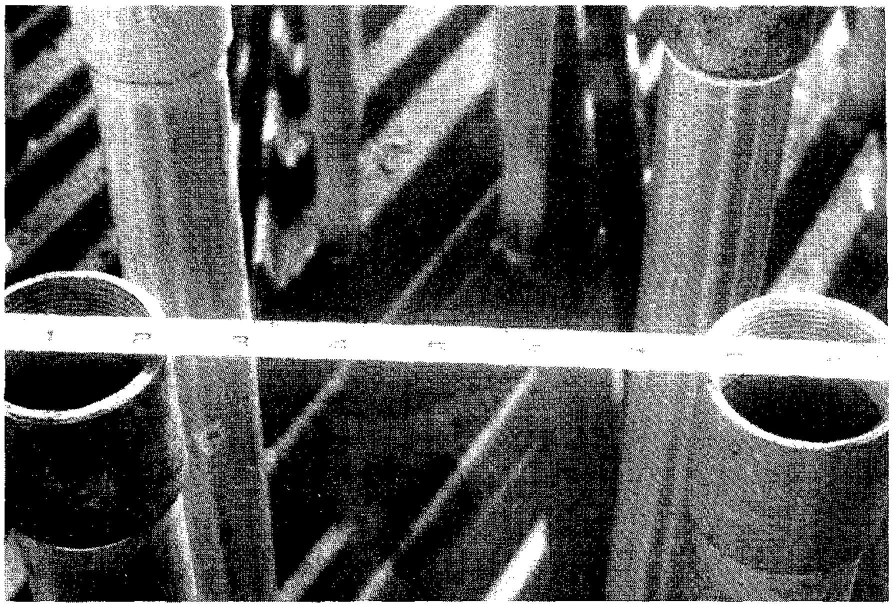

Fig. 4.3 Conduit misalignment

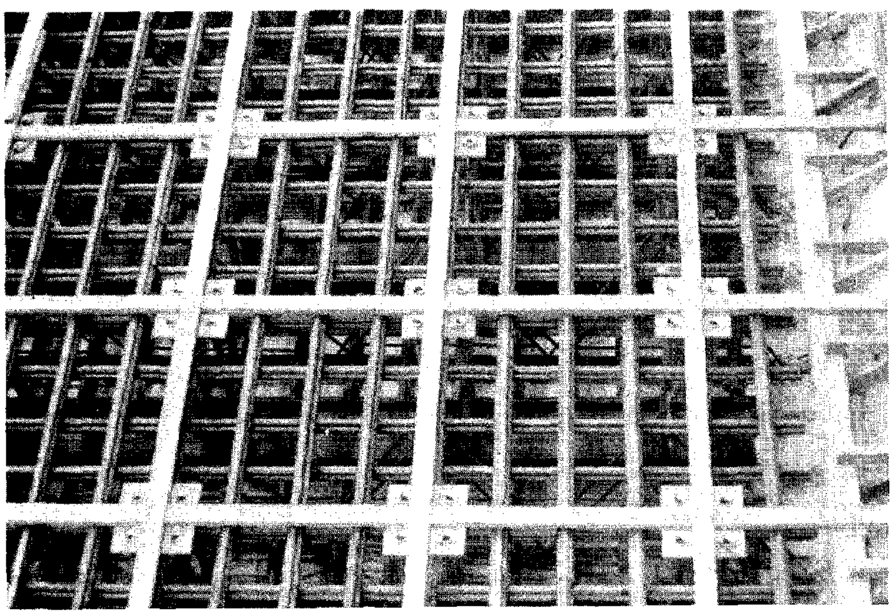

Fig. 4.4 Conduit alignment framework

removal of the framework would have to be stiff enough to restrain any movements by the conduit. Unfortunately, the concrete in the floor slab was not sufficiently stiff and there was some shifting of the conduits, but almost all were within the specified tolerances. The most serious misalignment was 1/2 in. between two bolt groups.

Perhaps the smoothest operation of the entire project was placement of concrete in the floor slab. A crew of ten men and two pump trucks working simultaneously placed over 350 cubic yards of concrete in eight hours. The floor slab was monolithically cast in four equal layers with mechanical vibrators used to assure continuity between the layers. As can be seen in Fig. 4.3, the top of each conduit was fitted with a threaded sleeve. Prior to casting the sleeves were set at the proper elevation with a level and provided the control points for screeding and finishing the floor surface. However, the use of the sleeves as a guide for screeding and finishing resulted in an uneven surface. For a more level surface, a different method would have been needed. After the floor slab was cast, the surface was float-finished and a curing sealant applied to it.

4.3 Buttresses

The buttresses and walls were cast together, but for the sake of clarity each will be discussed individually.

The buttresses were formed after the floor slab had hardened, but provisions had been made during construction of the floor slab to provide continuity between the buttresses and the floor slab. Dowels were cast in the floor slab to provide anchorage for the vertical supplemental steel in the buttresses and lengths of threadbars were also cast into the floor slab (see Fig. 4.5).

The threadbar, surrounded by sheathing, was anchored at the bottom of the floor slab by "A" bell anchors (Fig. 4.6) and

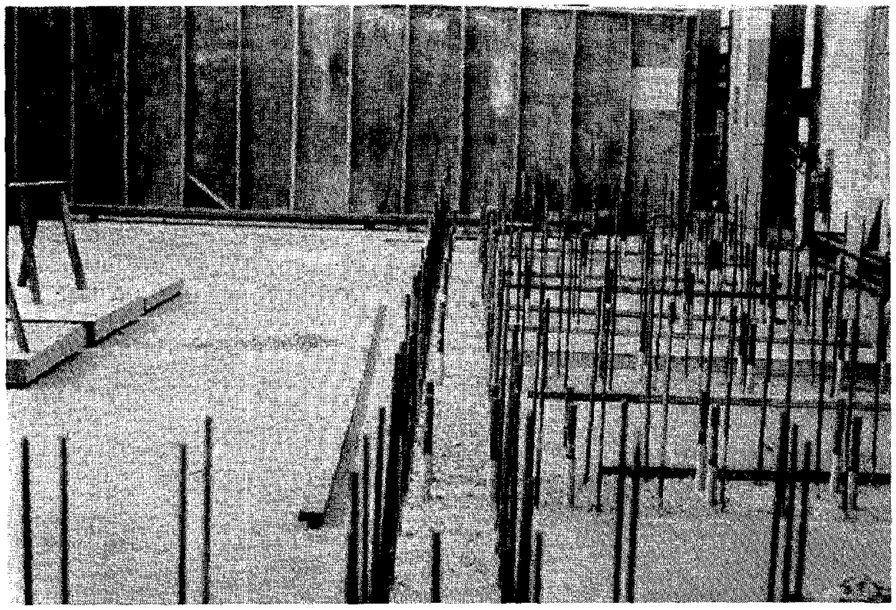

Fig. 4.5 Dowels in slab

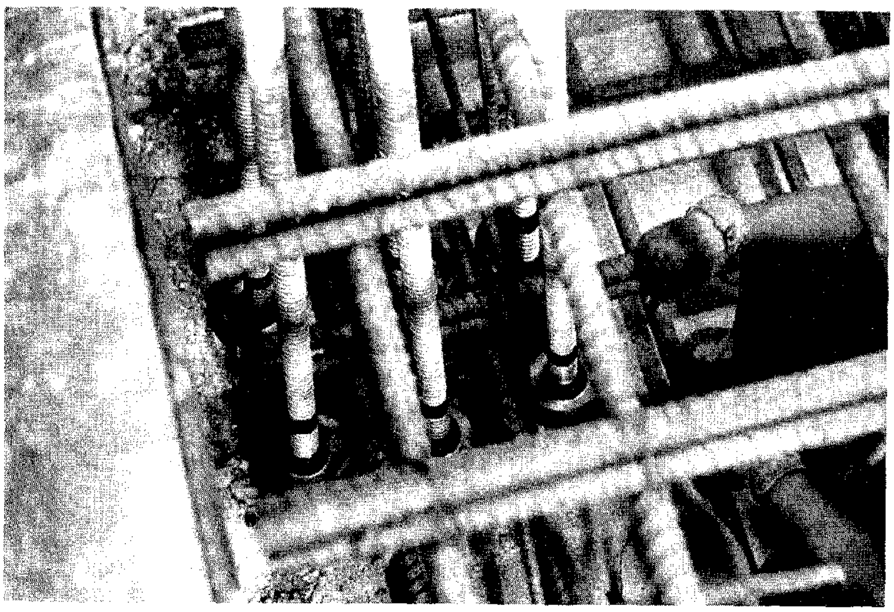

Fig. 4.6 "A" bell anchors in place

extended above the slab so that a second threadbar in the buttress could be spliced to it. Threadbar extensions can be seen in Fig. 4.5 with the sheathing about a foot below the end of the threadbar. The "A" bell anchor was a circular band of steel which had a nut welded at its center (see Fig. 4.7). The circular band provided confinement to the concrete against which the nut was bearing. The threadbar was screwed into the nut to provide load transfer from the threadbar to the concrete.

Each buttress had bolt groups at five levels spaced along its height (see Fig. 3.7). At each level the group was formed by a welded assembly of four pipes which extended the depth of the buttresses. As can be seen in Fig. 4.8, the four pipes had a steel template welded at each end to keep them in the correct 8 in. square pattern. The steel template was also designed to serve as a bearing plate on the wall surface. After removing the forms, bolts could be passed through the buttresses and anchored at either end by a bearing plate and nut.

Originally, the bolt groups in the buttresses were to be positioned with the lowest group 2 ft. above the floor slab surface and the others at 4 ft. intervals. However, because of interference between the threadbar splices, the lowest bolt group had to be shifted upwards 8 in. Fig. 4.9 shows the lowest bolt group and the threadbar splices after the bolt group had been moved up. It is clear that the bolt groups could not have passed between the large diameter sheathing at the splice points if they had been left in their planned position. The rather long splice sheaths provided a void for movement of the splice during post-tensioning.

4.4 Walls

The walls were originally planned to have several vertical construction joints so that duplication of formwork could be avoided, but the contractor, after studying the plans, removed the construction

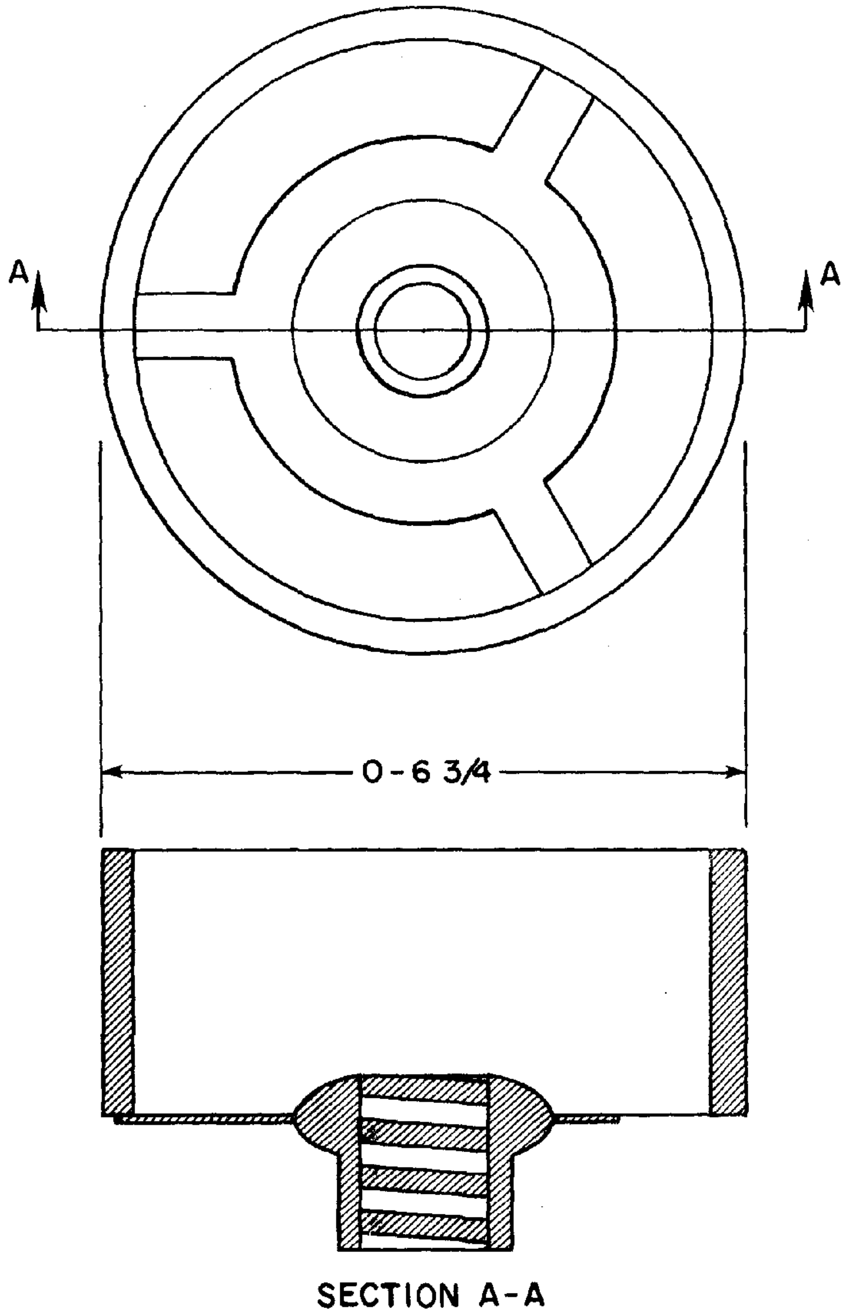

Fig. 4.7 "A" bell anchor

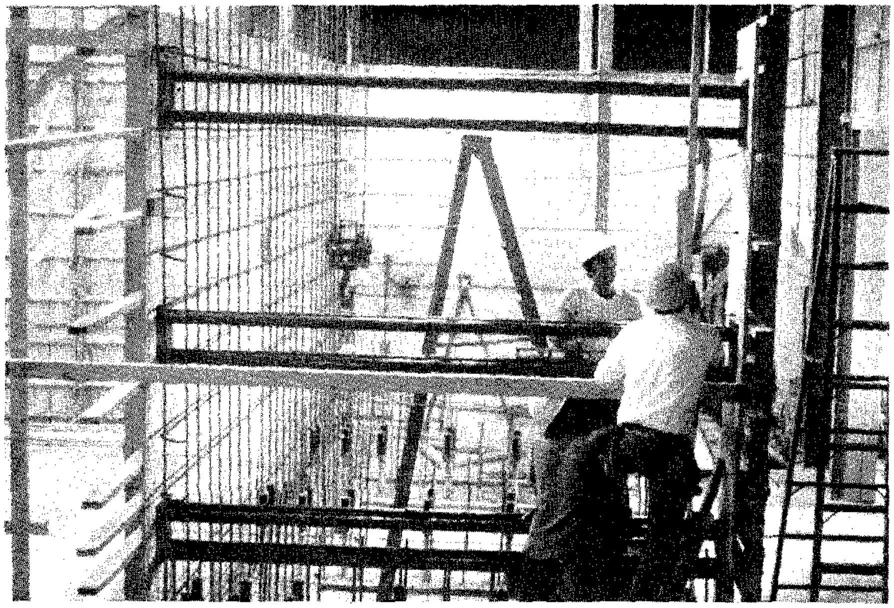

Fig. 4.8 Buttress pipe assemblies

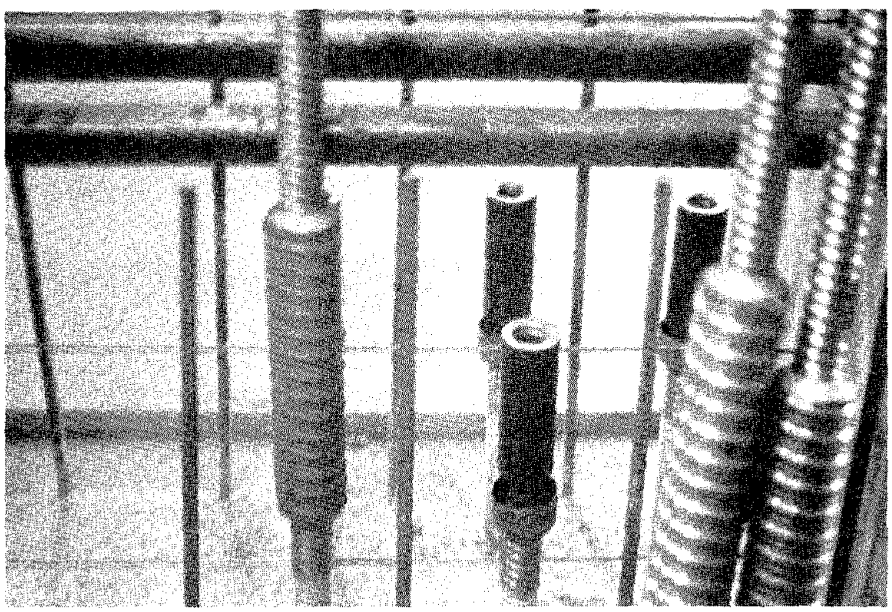

Fig. 4.9 Interference at lowest pipe assembly

joints because he felt it would be cheaper to cast the walls as a unit and would permit an alignment of wall formwork and a smoother wall surface. Since the buttresses and walls were cast together, the walls included the horizontal construction joint.

Like the buttresses, all of the vertical supplemental steel in the walls had dowels in the slab to provide continuity, but since the post-tensioning in the walls was horizontal, there were no embedded threadbars in the floor slab for the walls. The threadbars in the walls were installed in two sections and spliced so that their lengths would not make shipping and handling difficult.

The bolt groups in the walls were formed using pieces of pipe that extended the depth of the wall. Instead of using templates at each end to keep them in position, they were held in place and sealed against concrete intrusion by circular blocks of wood nailed to the wall forms. The only difficulty encountered with this method was the problem of closing up the forms. It was easy to place the pipes on the blocks when only one side of the form was up, but when the other side was erected it was quite difficult to make all of the blocks on it mesh with the pipes already in place. Even though there were no conflicts between the lowest wall bolt group and threadbar splices, the lowest wall bolt group was shifted upwards along with the lowest buttress bolt group for the sake of uniformity.

Casting of the walls went on simultaneously with the buttresses, so the same problems were encountered in both. All of the supplemental steel, fabricated pipe assemblies, and Dywidag bars were placed before the buttresses and walls were cast. The casting, however, was done in two stages, with the first lift being approximately 12 ft. and the second 7 ft. During the casting of the first stage, the single pump truck used was clogged by a substandard batch of concrete and while no bad concrete was placed in the buttresses or walls the resulting delay allowed a cold joint to form, as shown in Fig. 4.10. After removal of the forms from the first lift, some superficial honeycombing was found (Fig. 4.11), but it was quickly repaired using a sand and cement mix. The second lift went smoothly with no problems during or after the pumping.

4.5 Formwork

An integral part of the success of any concrete structure is the design and construction of its formwork. The Reaction Wall was no different and, while no formwork was required for the floor slab, careful consideration was given to the formwork for the buttresses and walls.

The key objectives of the formwork were to minimize surface irregularities on the working faces of the walls and to ensure that the pipe group templates were flush with the wall surfaces, so that bearing plates would fit properly. In addition, the formwork was to be securely held in place so no concrete would enter the pipes of the buttress groups, since these were not fitted with plugs as were the wall pipes.

With no vertical construction joints present, the formwork was easily erected. First, the entire working surfaces of the walls were formed so the wall would be continuous and uniform (Figs. 4.12 and 4.13). Second, the formwork was erected for the first stage around the buttresses and behind the walls once the reinforcing steel was in place.

After the first stage had been cast and sufficiently cured, the forms from behind the walls and around the buttresses were removed and moved upwards to the second stage (Fig. 4.14).

It should be noted again that the forms on the working surface, front face, of the walls were not moved or removed until the entire wall had been cast and cured (Fig. 4.15).

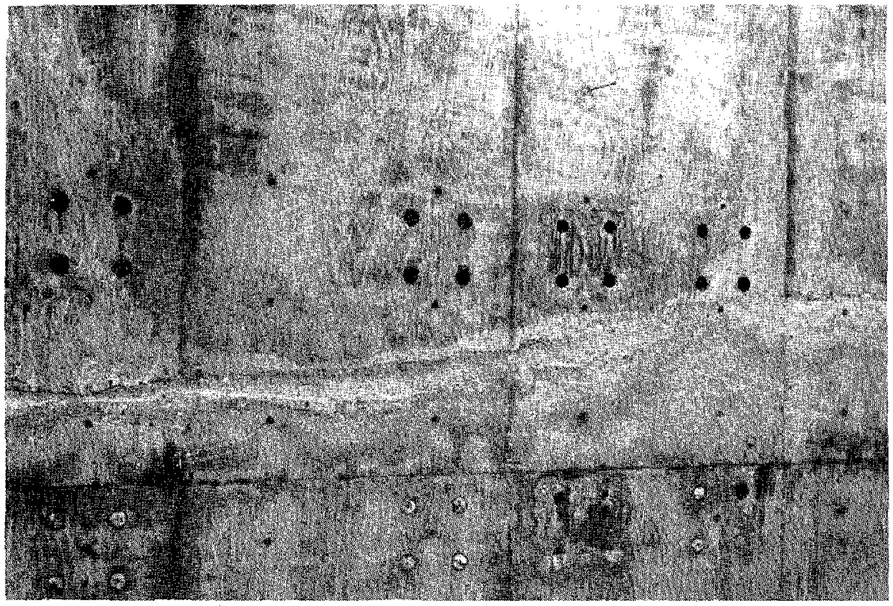

Fig. 4.10 Wall cold joint

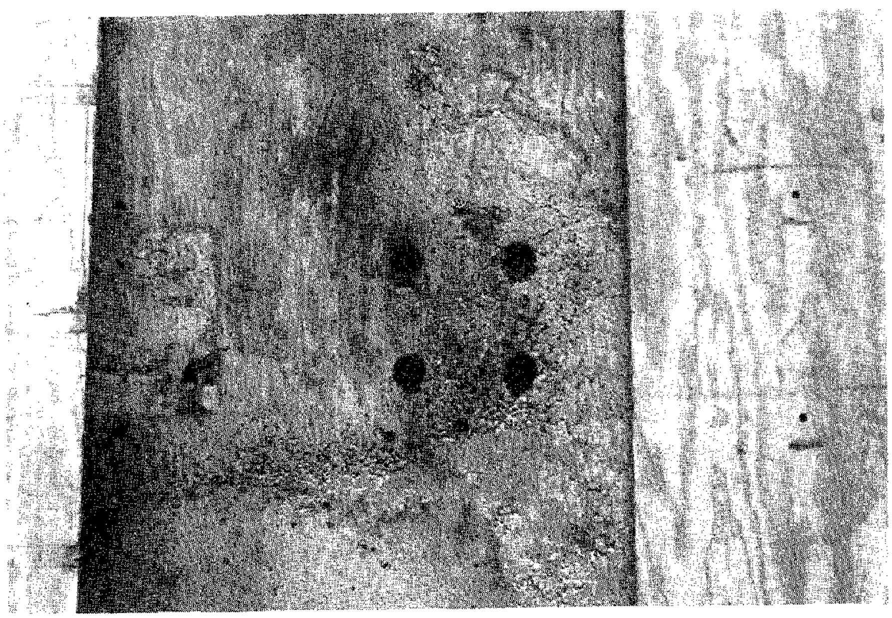

Fig. 4.11 Wall honeycombing

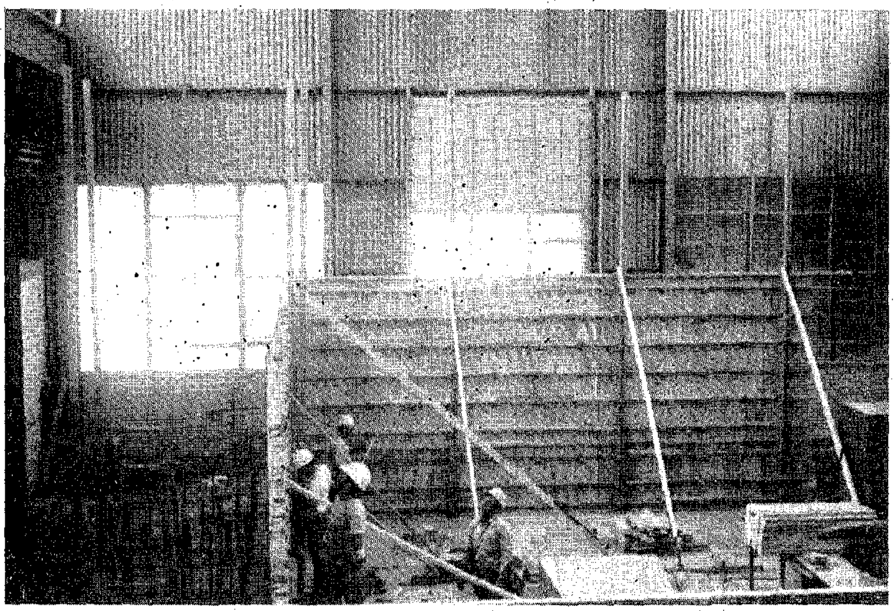

Fig. 4.12 Erecting wall forms

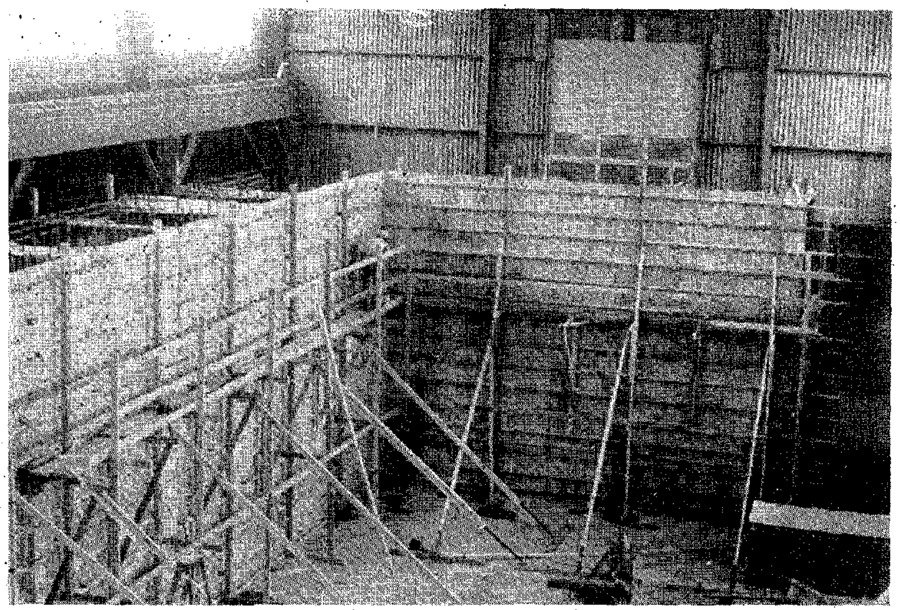

Fig. 4.13 Wall forms complete

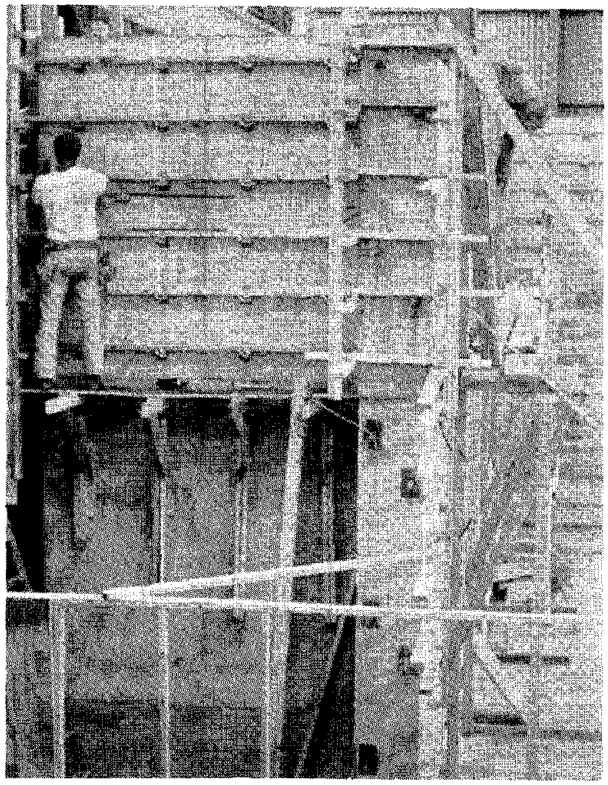

Fig. 4.14 Second stage formwork

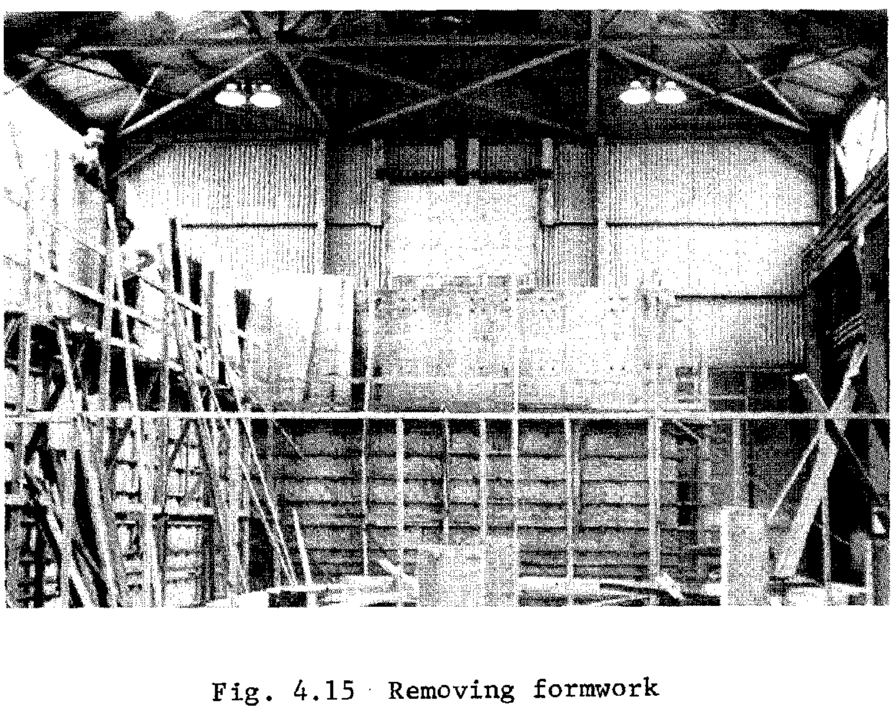

Fig. 4.15 Removing formwork

With the removal of the forms it became apparent that the formwork had only partly satisfied its objective. The working surfaces of the walls were fairly free from irregularities, but the forms had expanded under the weight of the concrete and vibration during placement. As a result, the buttress pipe templates were not flush with the concrete surface and the pipes had varying amounts of concrete in them. There was no concrete in the wall pipes; the plugs had effectively sealed the ends.

Form-ties were used at a spacing of 16 in. vertically and 24 in. horizontally, with one long form-tie running the depth of the buttresses at the center every 16 in. vertically. From the results obtained, it was fairly obvious that the spacings for the buttress form-ties were far too liberal, especially considering their long lengths compared to the wall form-ties.

4.6 Post-tensioning

During installation of the Dywidag threadbars, no problems were encountered and the construction crew found them easy to work with. The buttress threadbars, sixteen per buttress, were kept in alignment during casting by using a template for each buttress and tying the threadbars securely to the buttress pipe assemblies. The wall threadbars needed no templates, but were ties to the wall reinforcement at close intervals to ensure that they were level with no sags anywhere along their lengths.

"A" bell anchors were used only in the floor slab. Elsewhere bearing plates and anchor nuts were used to terminate the threadbars. The bearing plates were rectangular with a hole drilled in the middle to pass the threadbar through. The hole was then ground out to form a spherical seat for the anchor nut which had a spherical head (Fig. 4.16). The plate was set out on the threadbar before casting, using a plastic ring which kept the plate in the desired position. The plastic ring created problems later in the

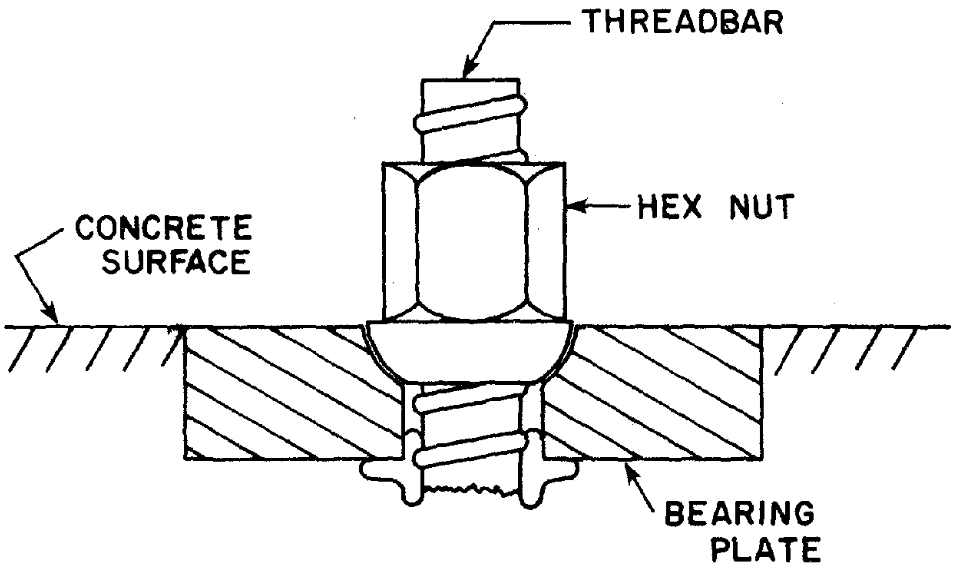

Fig. 4.16 Anchor nut and bearing plate

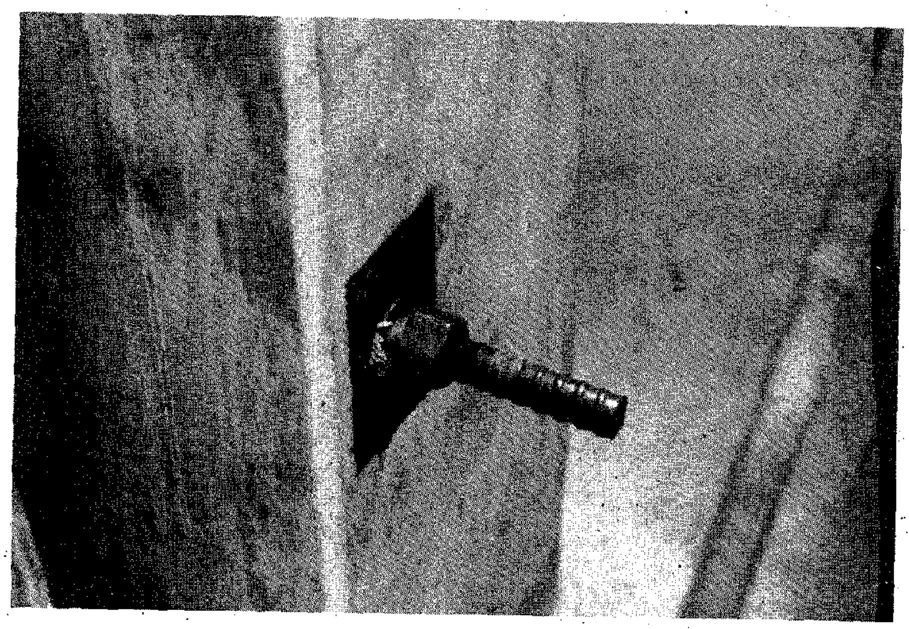

Fig. 4.17 Anchor assembly on wall

tensioning of the threadbars which were both irritating and time-consuming to fix. During tensioning the plastic ring would be drawn up into the plate and prevent the anchor nut from seating properly. Consequently, the plastic ring had to be dug out from the plate hole repeatedly for the same threadbar until enough of the ring had been removed to allow the anchor nut to seat properly. This meant that the threadbar might have to be tensioned and completely released several times before the problem was corrected. The plate was placed so that it fit flush with the concrete, as can be seen in Fig. 4.17, where a plate and nut on a wall threadbar is shown.

The whole operation of installing and tensioning the Dywidag threadbars was made simpler by the fact that no grouting was to be done after the post-tensioning. Since the capability of retensioning the threadbars at some later time was desired, the threadbars could not be grouted.

Tensioning the Dywidag threadbars was a very simple operation. The only equipment needed was a hydraulic pump, hydraulic centerhole ram (Fig. 4.18), mandrel, wrench, and calipers. The mandrel was screwed onto the threadbar to provide a length sufficient to pass through the hydraulic ram (Fig. 4.19). The ram was then placed over the mandrel, threadbar, and threadbar anchor nut so that it rested on the bearing plate. A large nut was then screwed onto the mandrel until it was seated against the ram's piston. The piston was extended using the hydraulic pump, elongating the threadbar. When the correct elongation was reached, the threadbar anchor nut was turned with a ratchet assembly, which was an integral part of the ram, until the nut seated against the bearing plate, preventing the threadbar from relaxing (Fig. 4.20). The piston was then retracted (Fig. 4.21) and the mandrel and ram removed from the threadbar and shifted to the next bar to be tensioned.

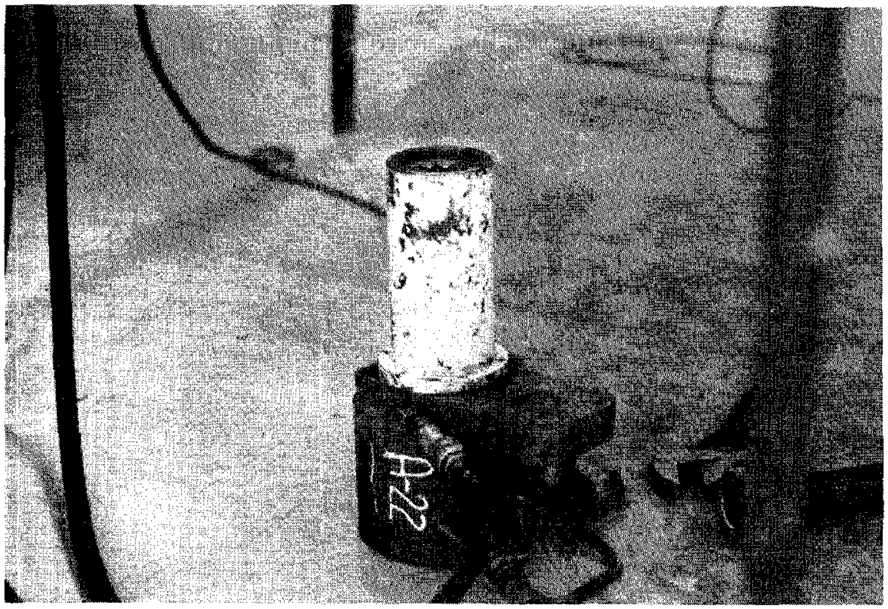

Fig. 4.18 Hydraulic ram

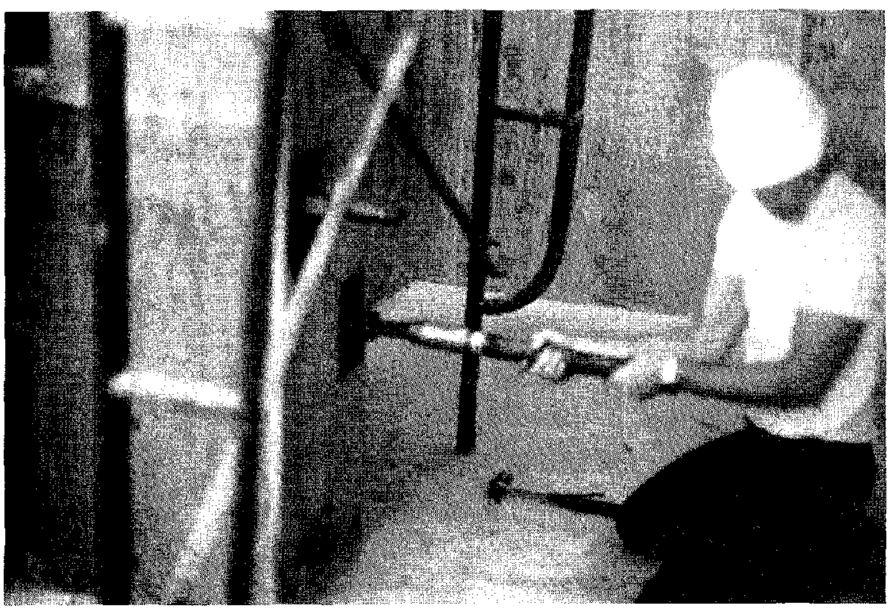

Fig. 4.19 Mandrel

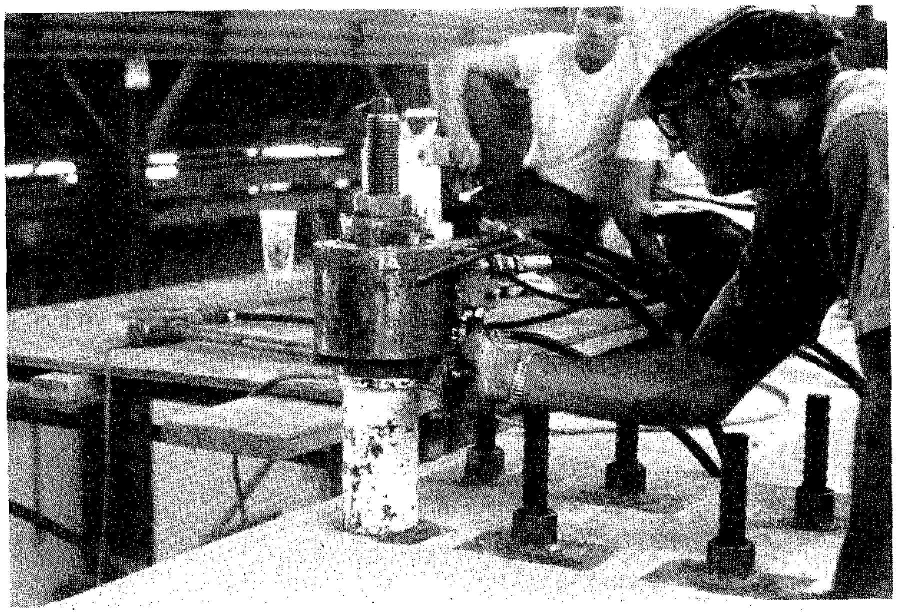

Fig. 4.20 Seating nut on bearing plate

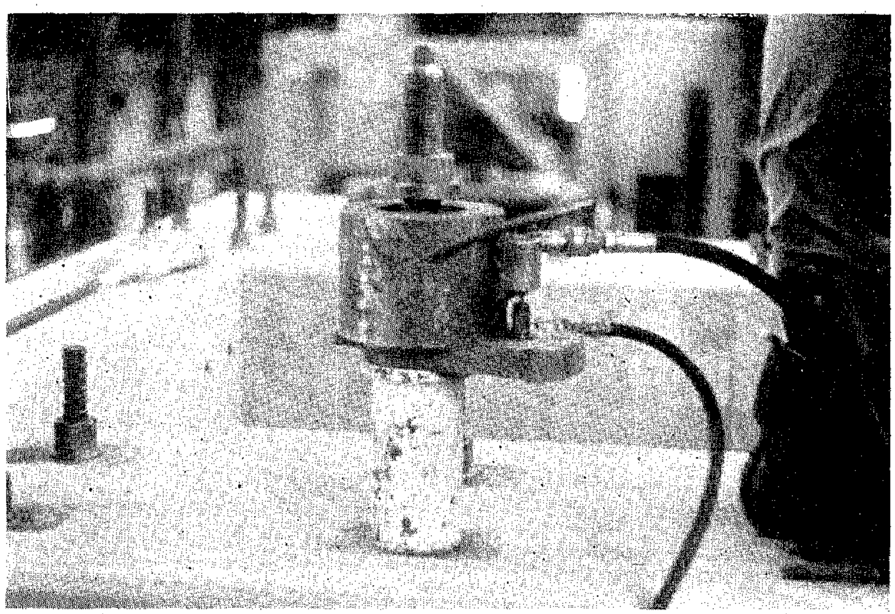

Fig. 4.21 Ram in retracted position

The proper elongation for each threadbar was calculated before tensioning by the suppliers of the Dywidag system. The post-tensioning operation was monitored in three ways to provide safeguards against incorrect stressing. First, the force on the threadbar was determined by measuring the hydraulic pressure in the ram. Second, the number of turns of the anchor nut to seat it on the bearing plate was monitored. Third, elongation of the threadbar was measured using calipers (Fig. 4.22). After all stressing was completed, the ends of the bars were coated to prevent rust and corrosion.

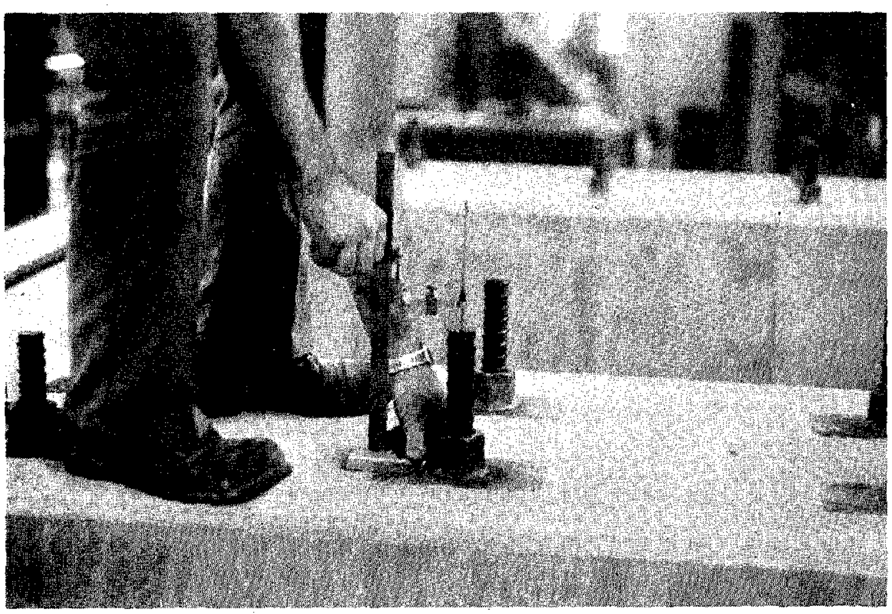

Fig. 4.22 Elongation measurement with calipers

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CHAPTER 5

FLOOR-WALL REACTION SYSTEM LOADING CAPABILITIES

5.1 Introduction

As an aid to the users of the Floor-Wall Reaction System, the maximum loads which can be applied to the various components of the system are catalogued in the remainder of this chapter. In addition, a schematic sketch of these levels is presented in Fig. 5.1.

5.2 Buttress

A. At no time shall the applied load on the buttress exceed 300 kips. Nor shall the combination of applied loads at any buttress load points and adjacent wall load points exceed the above value. This limit is controlled by the shear capacity of the buttress.

B. At no time shall the moment acting on the buttress parallel to the plane of the slab due to the applied loads on the buttress and its adjacent wall load points exceed 5400 k-ft.

Loads acting away from the buttress may be governed by the tensile capacity of the attachments to the wall.

5.3 Wall

A. At no time shall the applied load at any single wall load point exceed:

1. 100 kips when directed away from the working face of the wall
2. 100 kips when directed toward the working face of the wall

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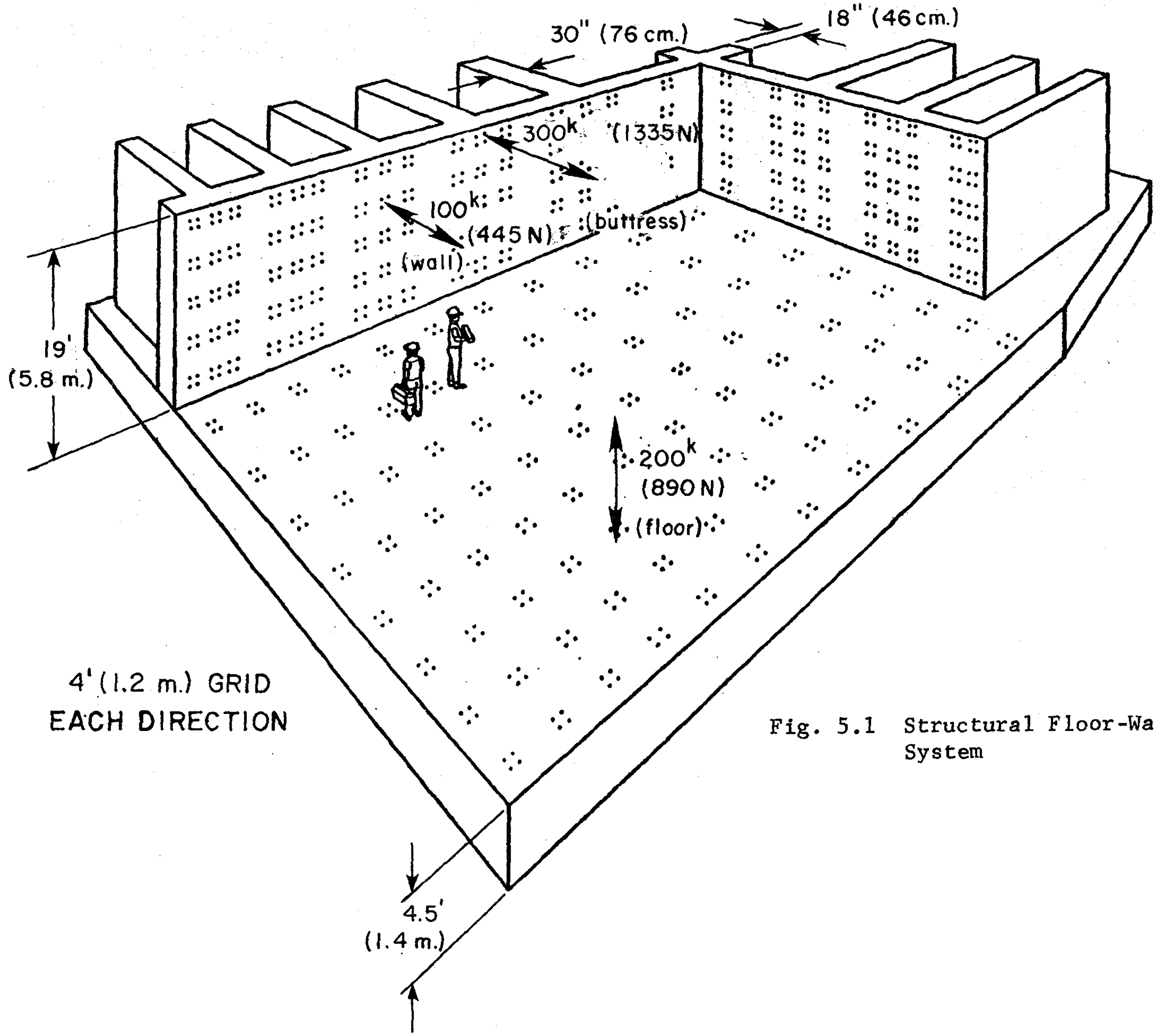

Fig. 5.1 Structural Floor-Wall Reaction System

5.4 Slab

A. At no time shall the applied load at any floor slab anchor bolt location exceed:

1. 50 kips when directed away from the slab
2. 50 kips when directed toward the slab

B. This provision governs only the structural slab and the encased anchor assembly. No statement is made concerning the anchor rod capacities.

Note: With ASTM A193, Grade B7 1-1/4 in. steel rods, the full capacity of the floor slab anchor bolt assemblies can be obtained.

R E F E R E N C E S

1. American Concrete Institute, Building Code Requirements for Reinforced Concrete (ACI 318-71), Detroit, 1971.

2. Young, Kenneth E., "Structural Analysis of Floor-Wall Reaction System," unpublished Special M.S. Report, The University of Texas at Austin, 1976.

A P P E N D I X A

Pictorial Description of the Construction of the Floor-Wall Reaction System

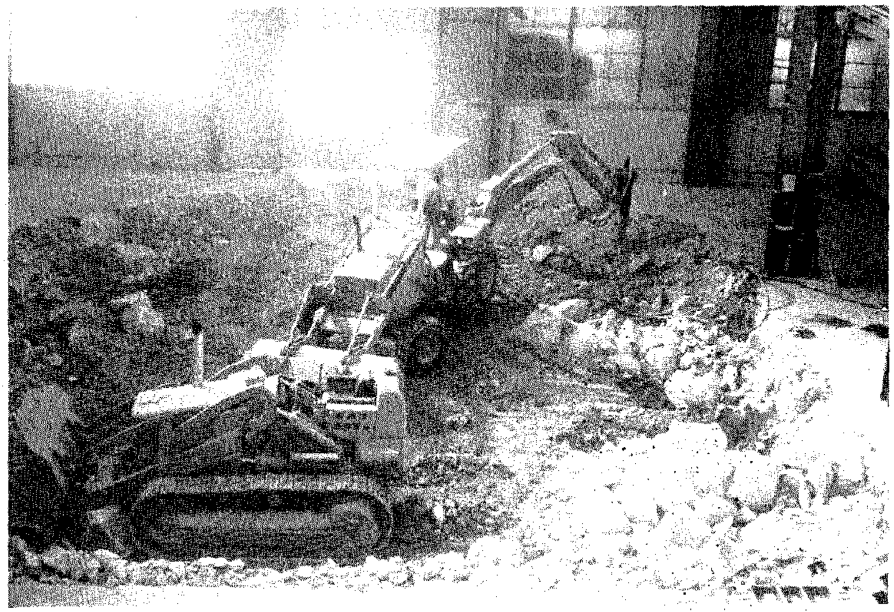

Fig. A1 Excavation for the floor slab

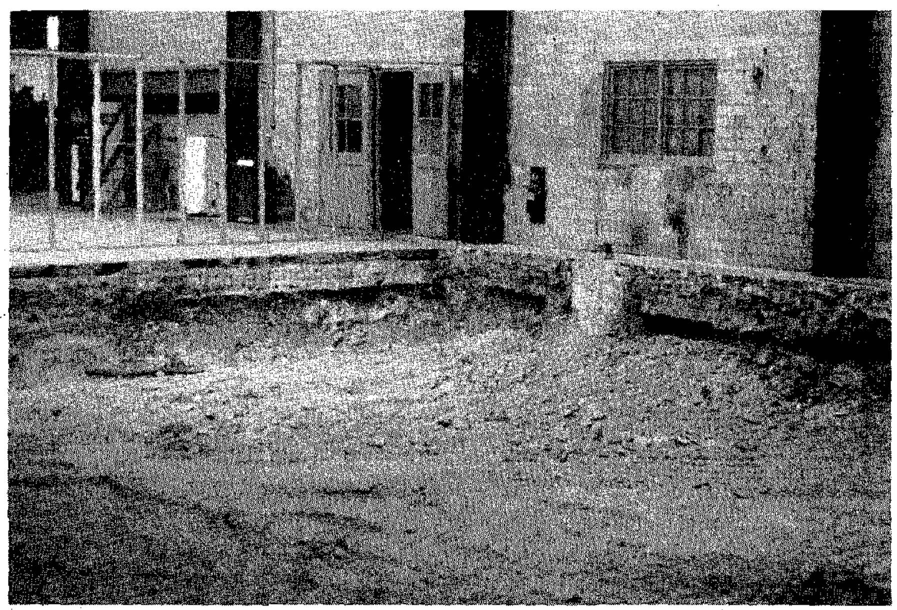

Fig. A2 Completed excavation for the floor slab

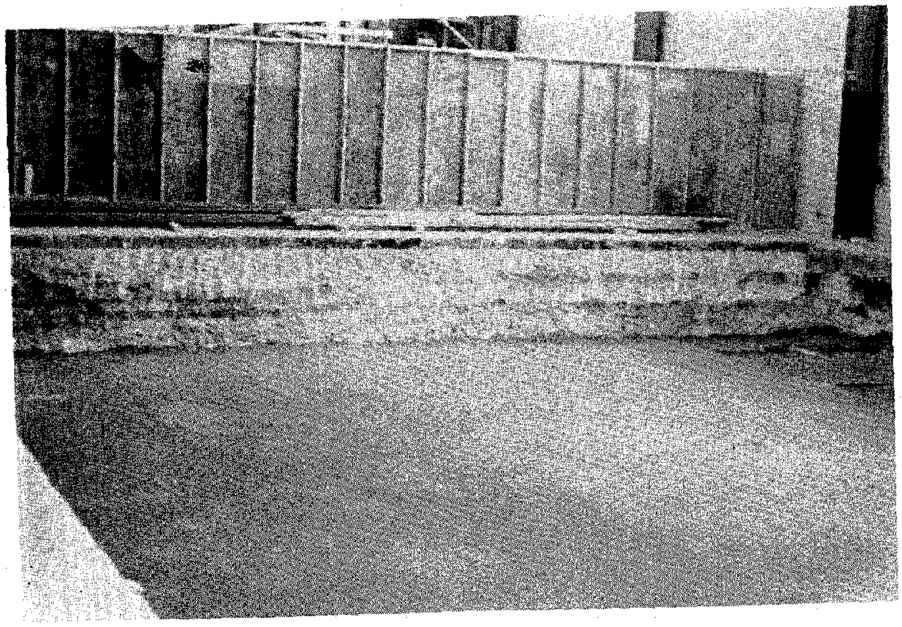

Fig. A3 Mud slab after pumping

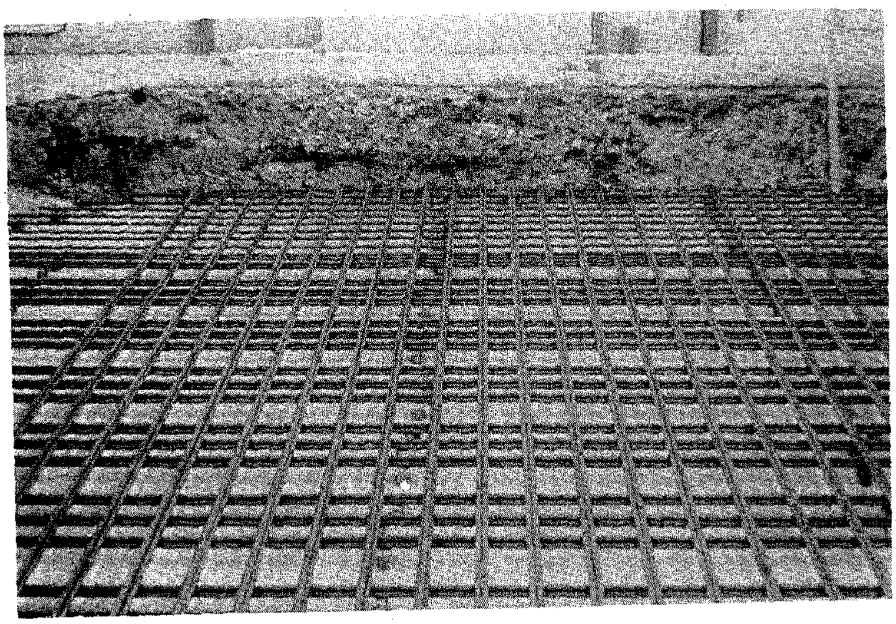

Fig. A4 Bottom two layers of floor slab reinforcing steel in place

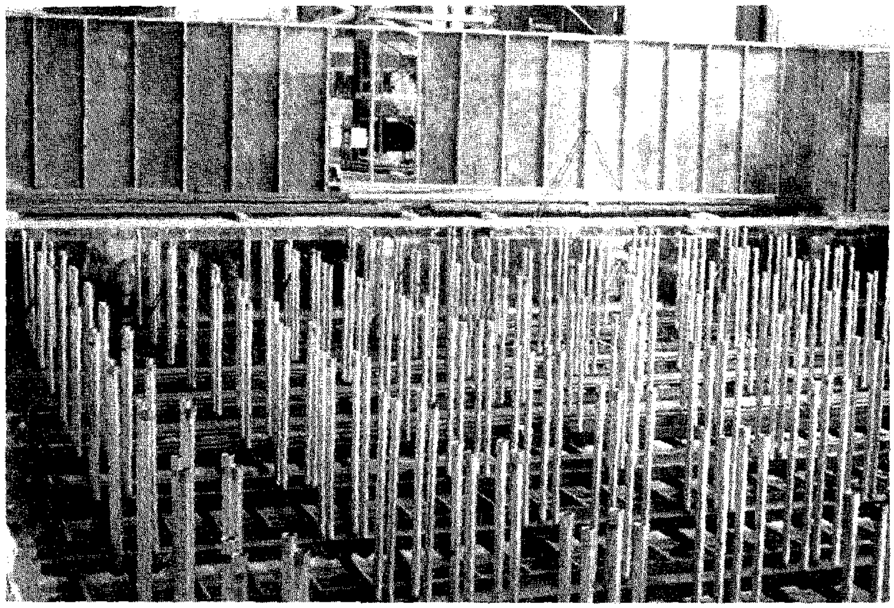

Fig. A5 Floor slab anchor bolt assemblies in place

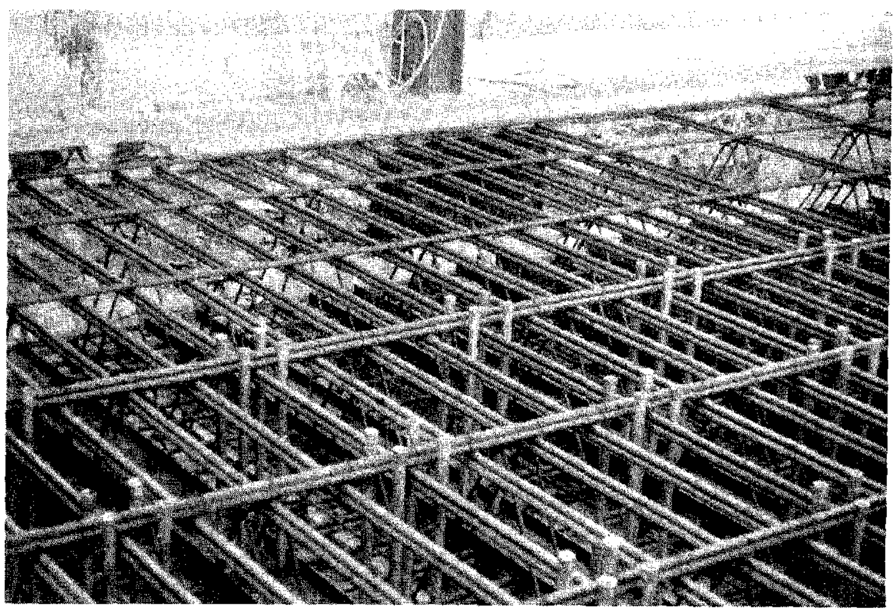

Fig. A6 Partial placement of upper layer floor slab reinforcing steel

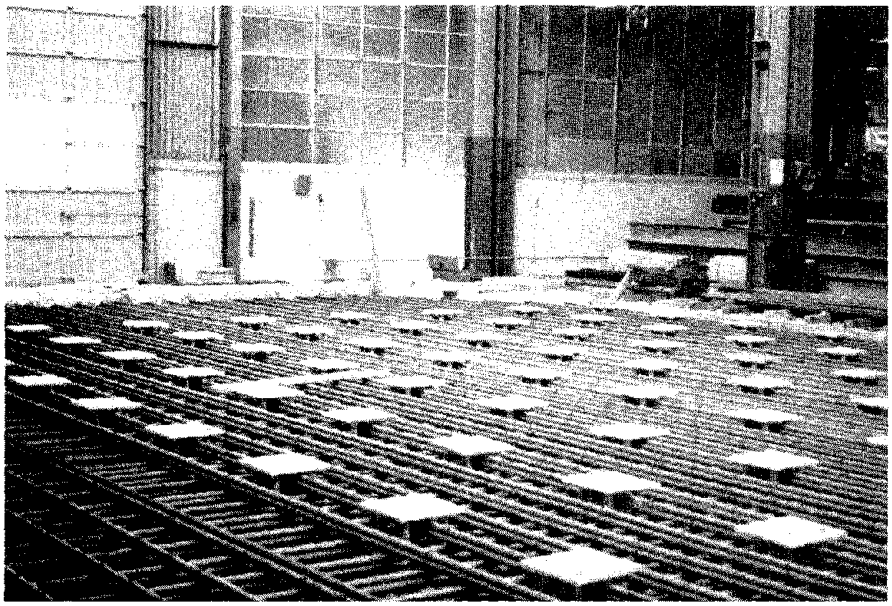

Fig. A7 Templates in place on anchor bolt conduit

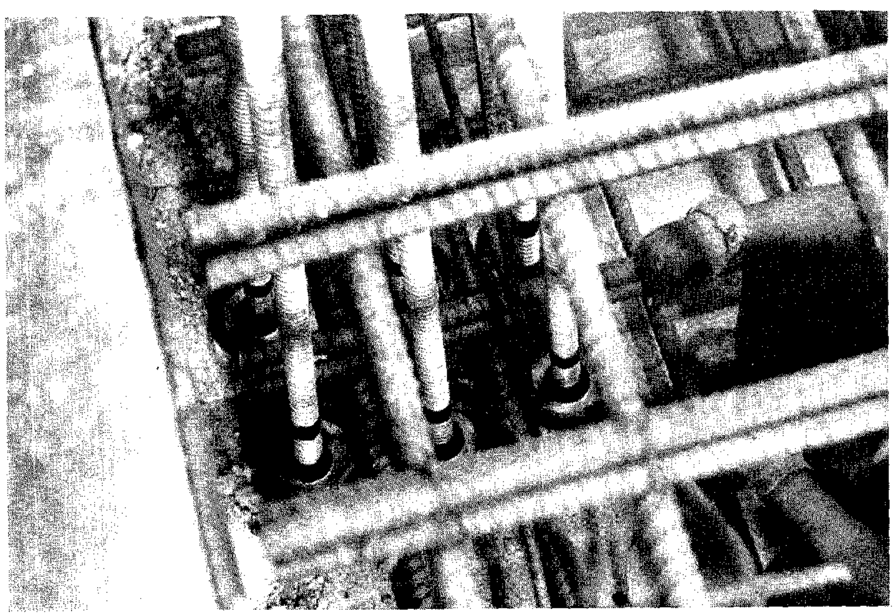

Fig. A8 "A" bell anchors and floor slab threadbar being placed

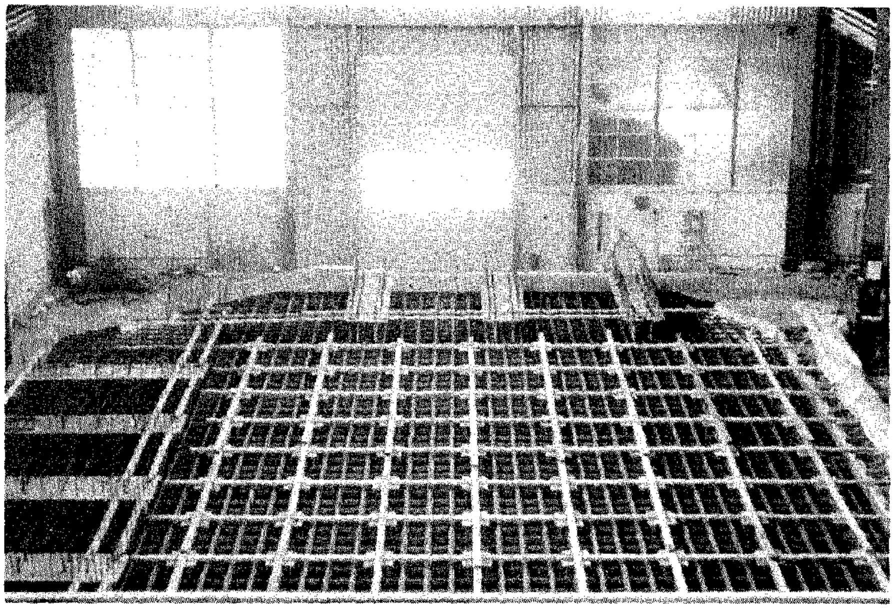

Fig. A9 Overview of floor slab just prior to casting

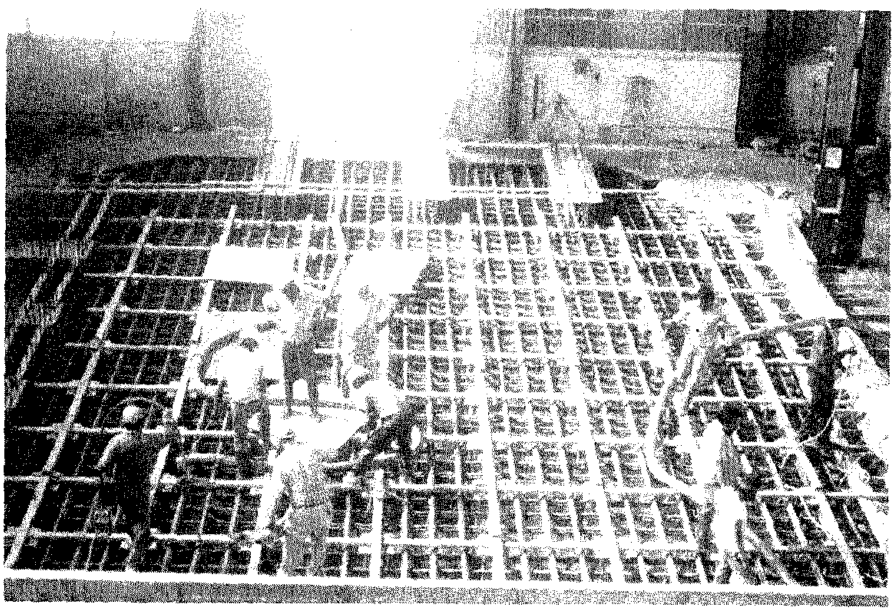

Fig. A10 Overview of floor slab casting operation

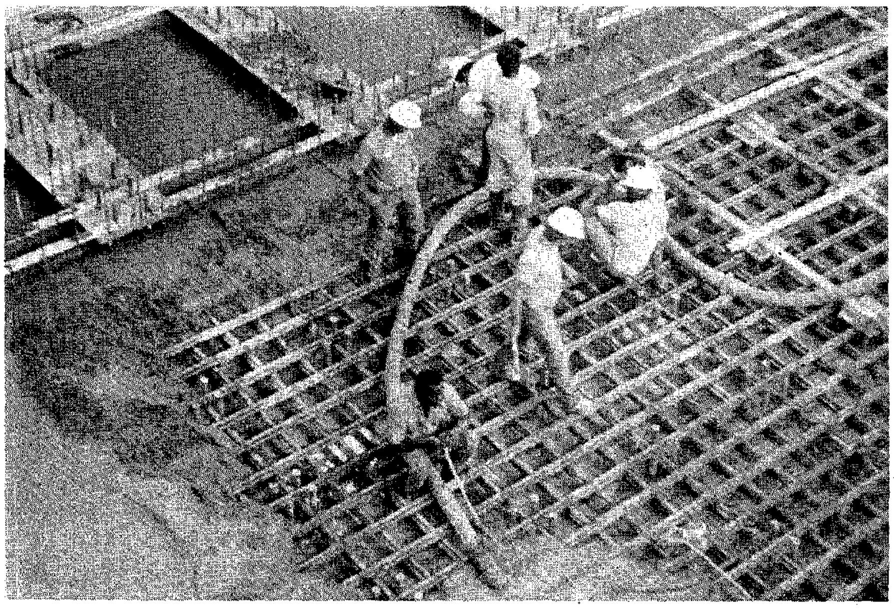

Fig. A11 Overview of floor slab casting operation

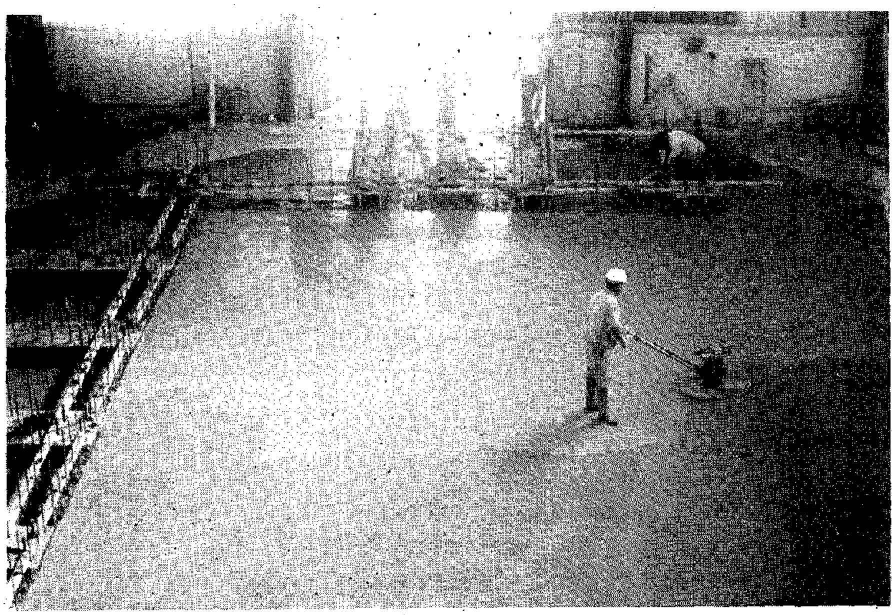

Fig. A12 Float finishing of the finished floor slab

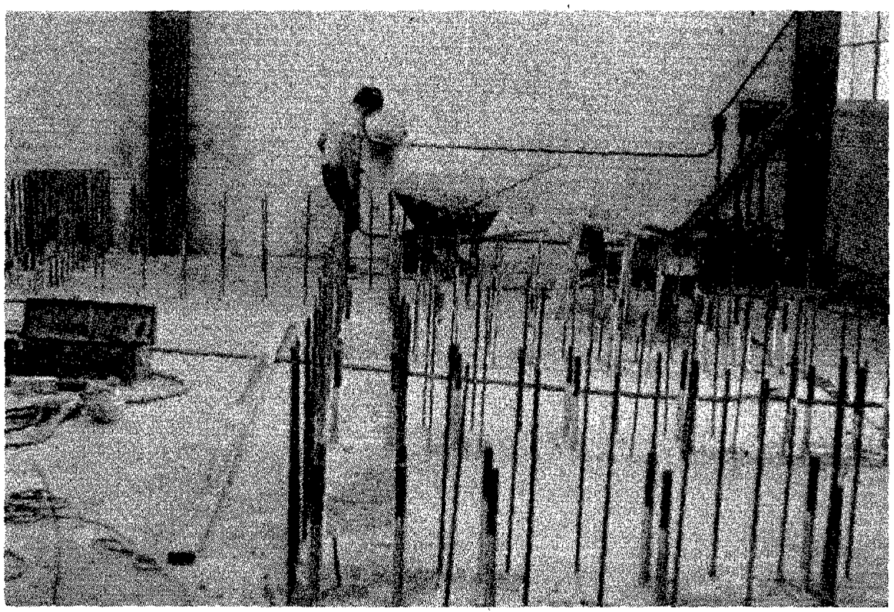

Fig. A13 Dowels in floor slab

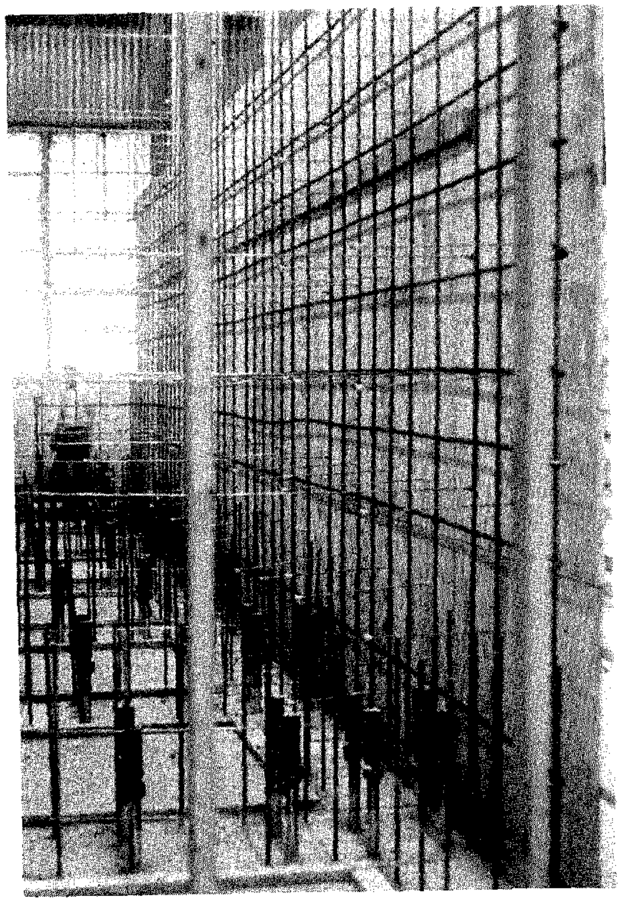

Fig. A14 Wall forms being erected with some wall steel in place

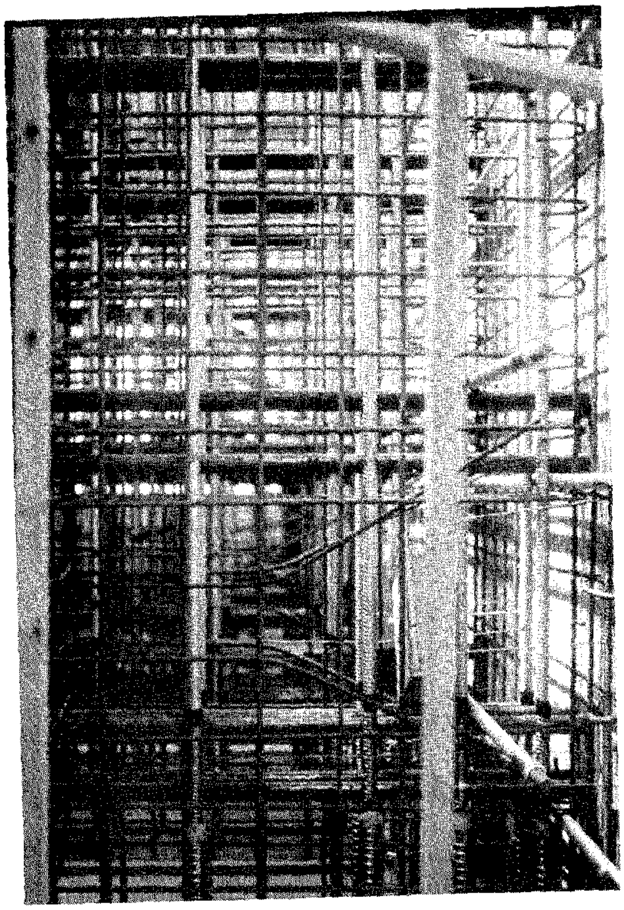

Fig. A15 Buttress steel in place

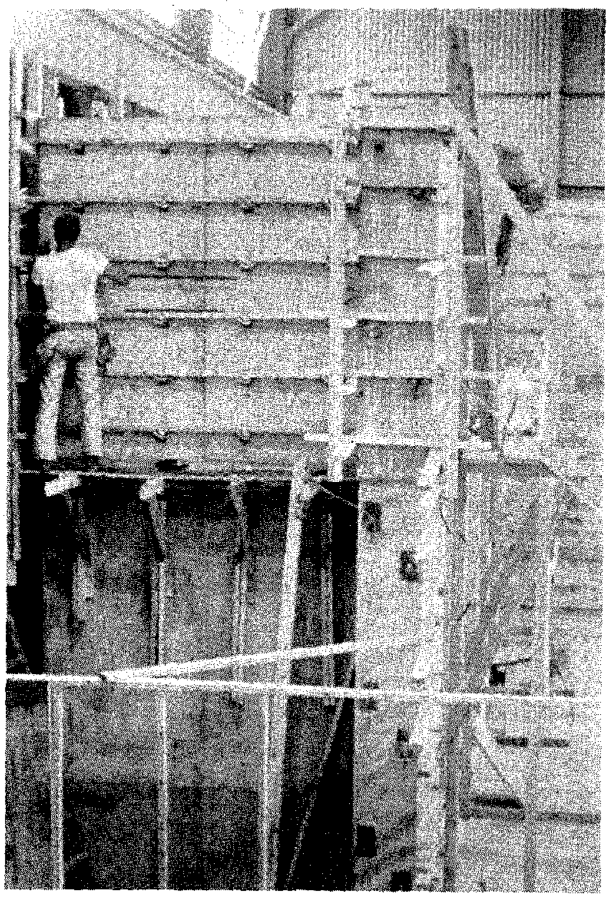

Fig. A16 Erecting second stage formwork for buttresses and walls

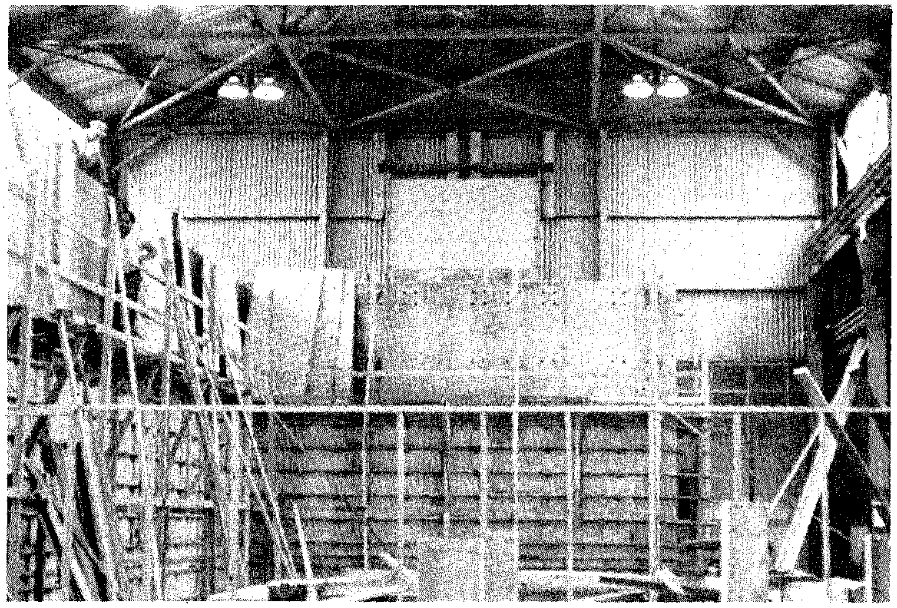

Fig. A17 Removing formwork after final casting

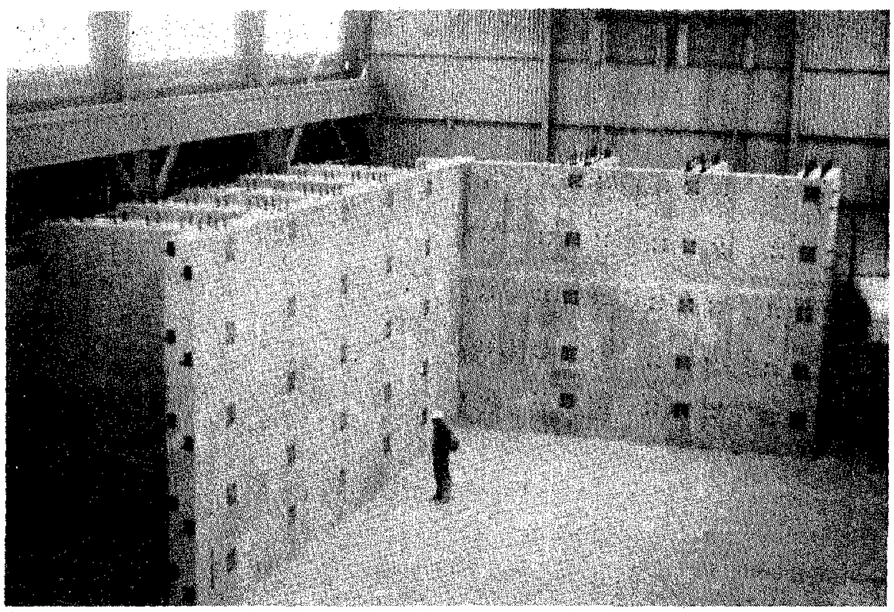

Fig. A18 Completed Reaction Wall

A P P E N D I X   B

Construction Drawings

for

Floor-Wall Reaction System
Civil Engineering Structures Research Laboratory

as prepared by

Office of Facilities Planning and Construction
The University of Texas System
210 West 6th St., Austin, TX 78701

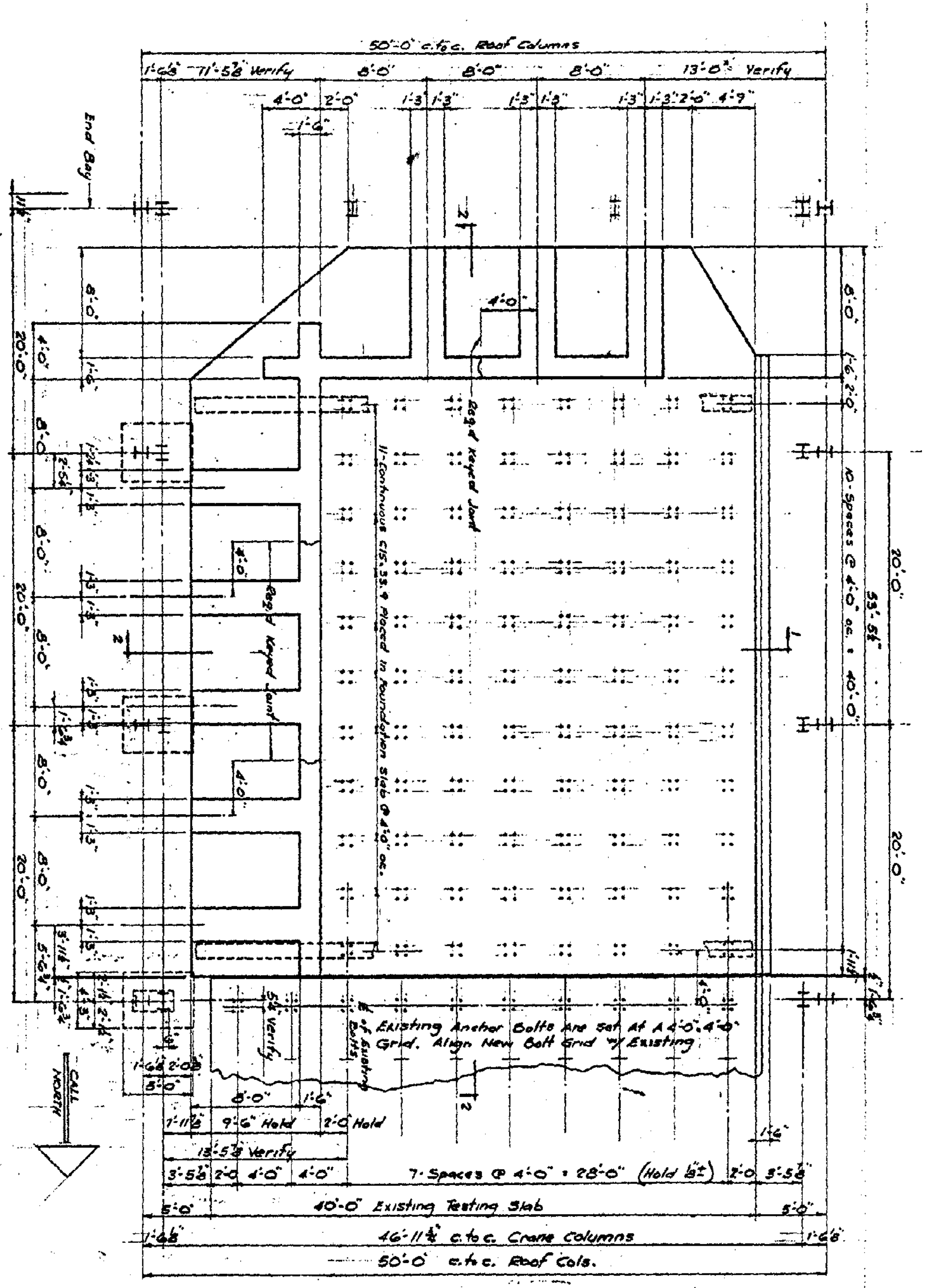

Fig. B1 Plan of Floor-Wall Reaction System

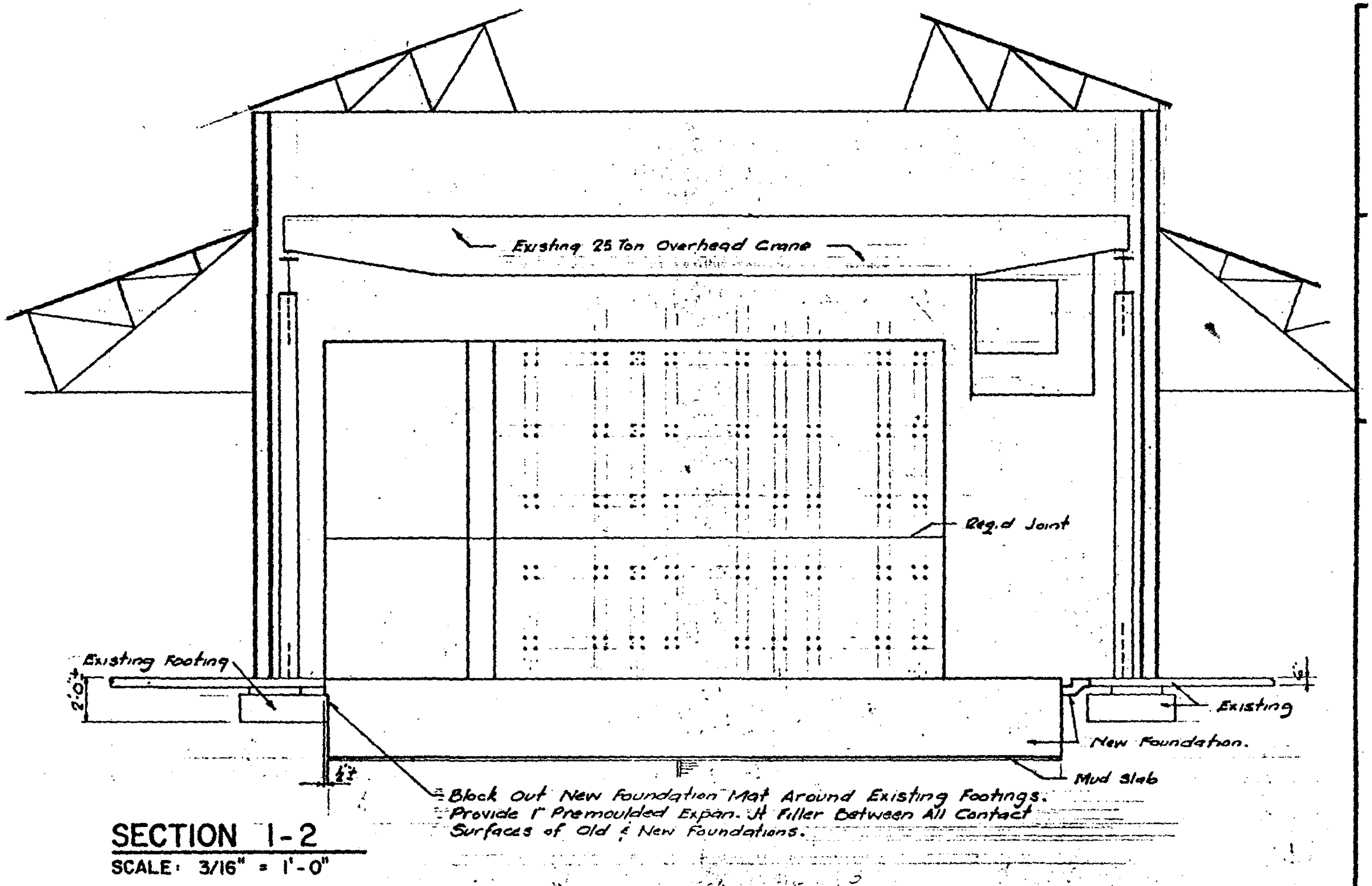

Fig. B2 Floor-Wall Reaction System in building

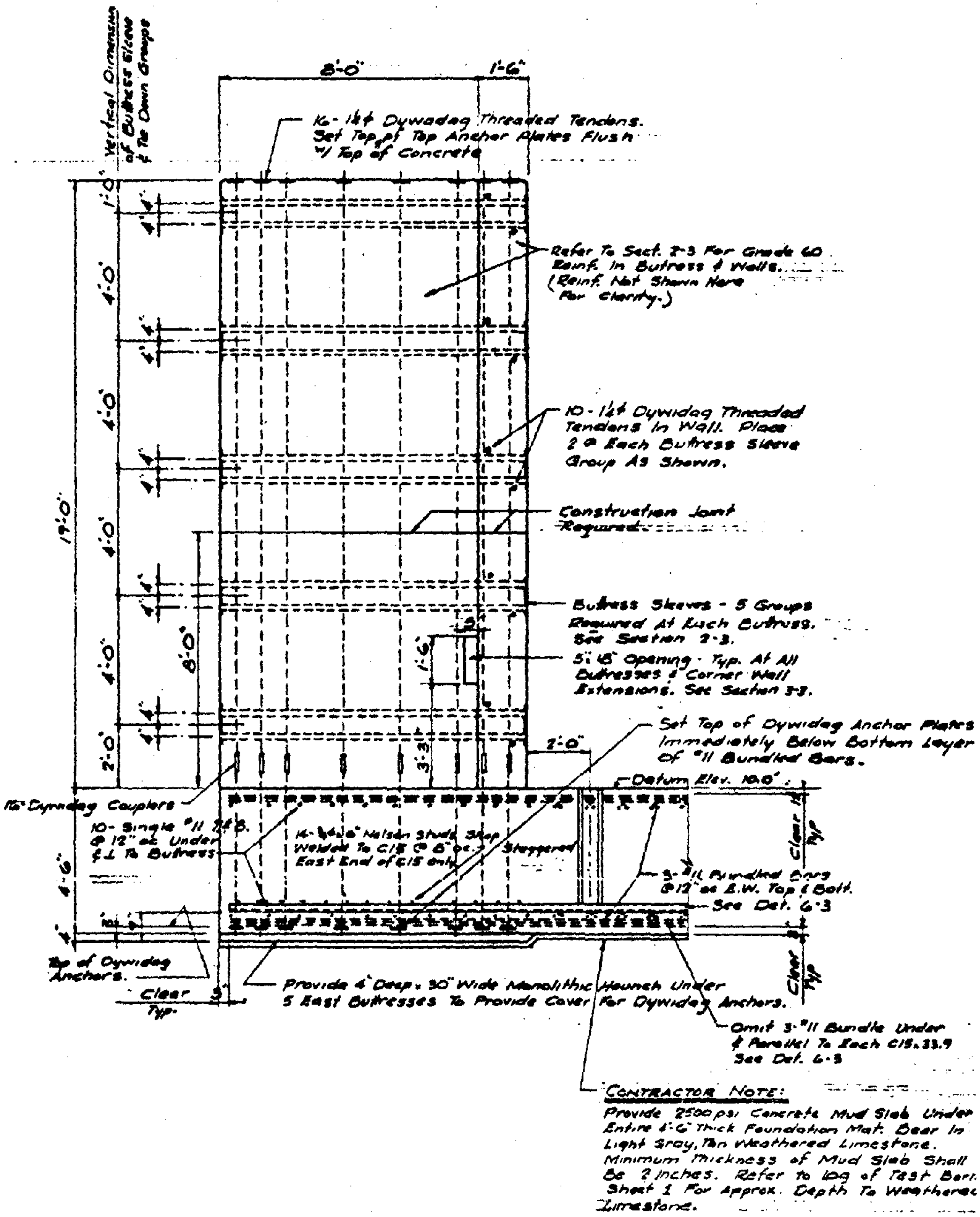

Fig. B3 Elevation of buttress and slab

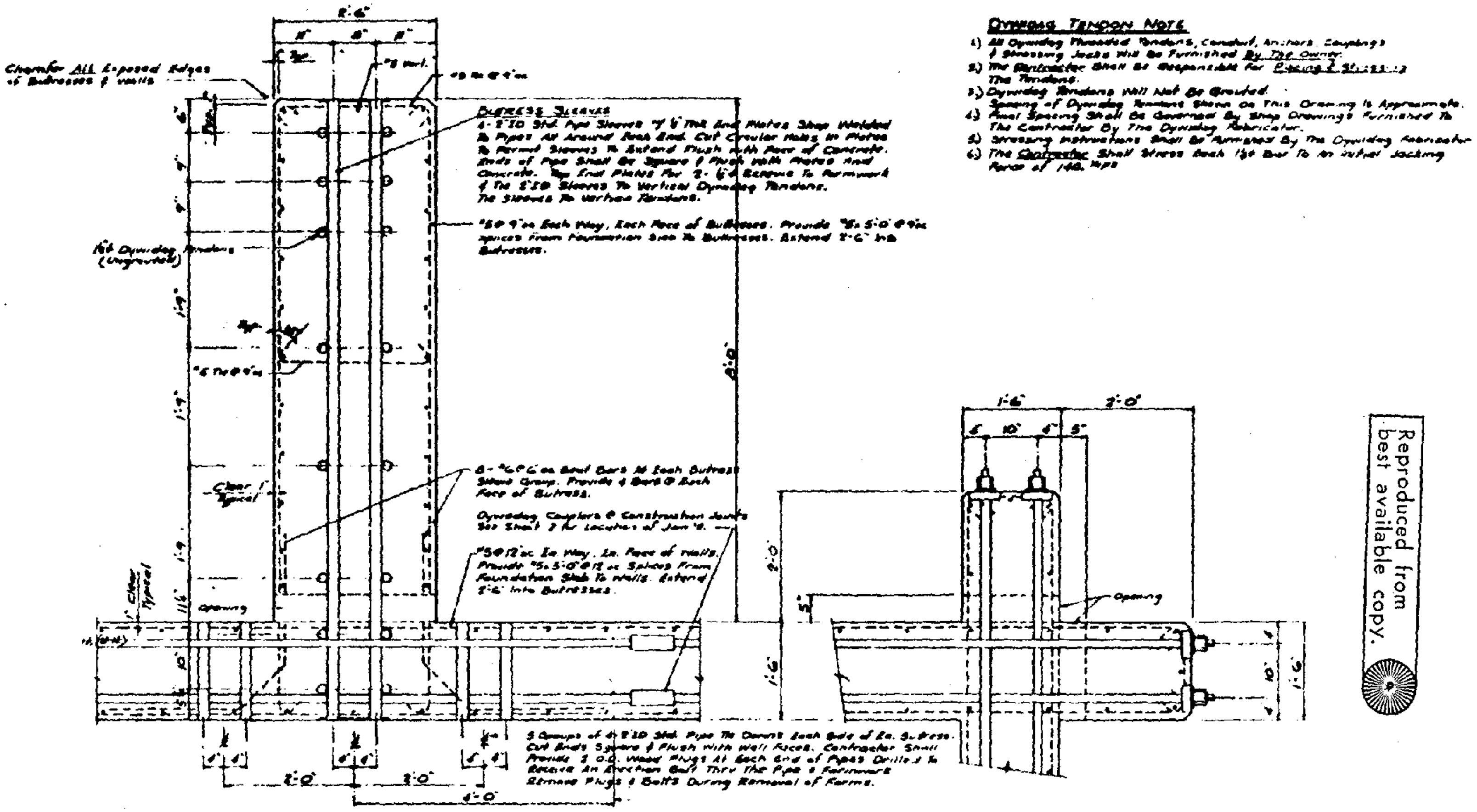

Fig. B4 Plan of buttress and wall

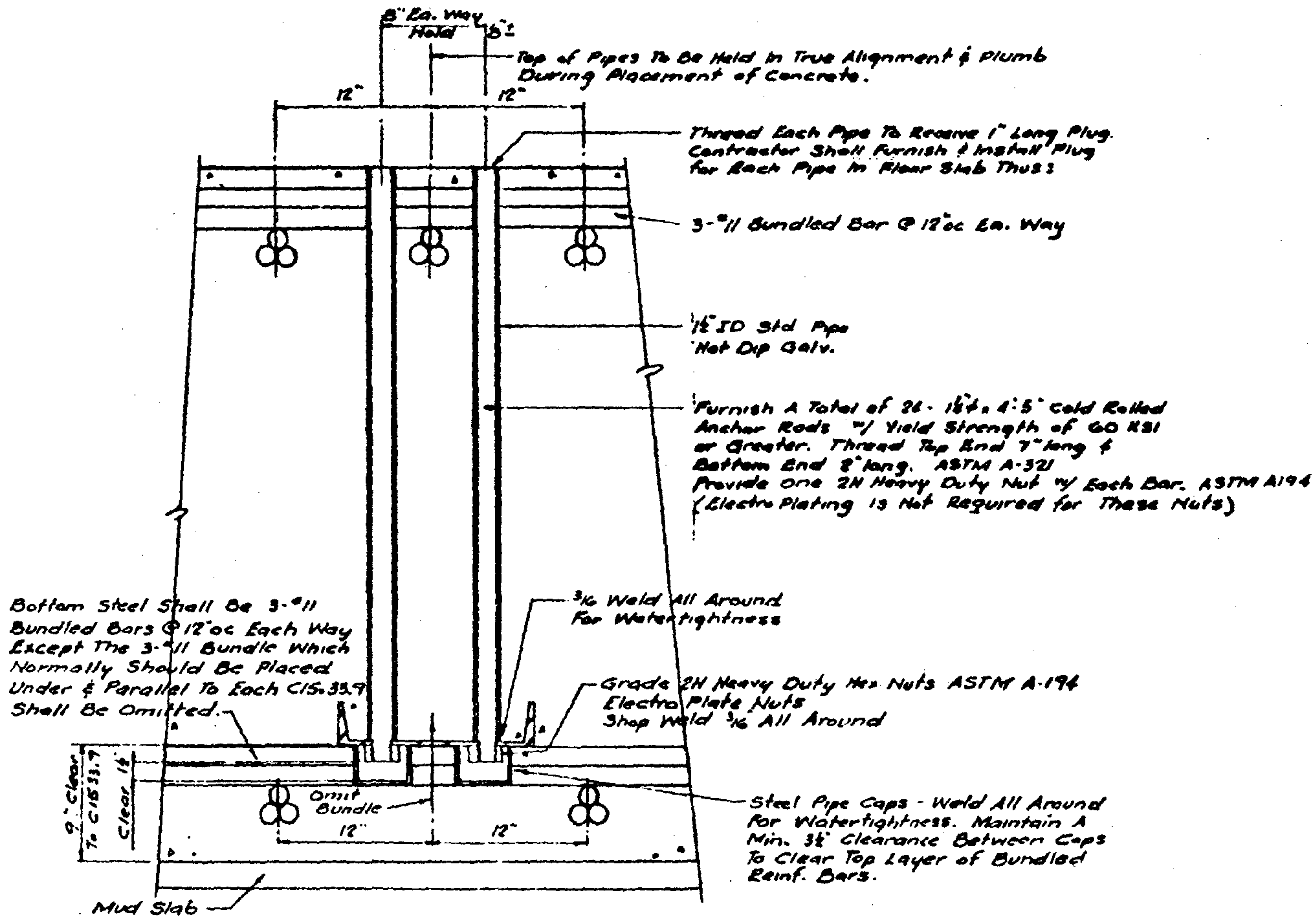

Fig. B5 Elevation of floor slab anchor bolt assembly

A P P E N D I X   C

Material Properties

CONCRETE PROPERTIES*

Typical Mix

Class of concrete: 4000 psi

Water-cement ratio: 5.73 gallons per sack of concrete

Mix design per cubic yard:

Cement	517 lbs
Sand	1370 lbs
Gravel	1795 lbs
Admixture	Airsene 16-1/2 oz
Sacks per cu yd	5.5

Compressive Cylinder Test Results

Pour	Yards Out	Average 7-day Strength	Average 14-day Strength	Average 28-day Strength
Floor Slab	28	3600	4500	5200
	76	3800	4600	5200
	124	3600	4700	5100
	179	3700	4400	5500
	234	4800	5700	6400
	282	4200	5100	5700
	331	4600	5500	6000
1st Wall	21	3200	3800	4600
(Bottom 12 ft)	70	4800	5600	5800
	112	5400	6000	6500
2nd Wall	21	4300	5300	5500
(Top 7 ft)	63	4000	4600	5200

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*Information provided by Trinity Engineering Testing Corporation, Austin, Texas.

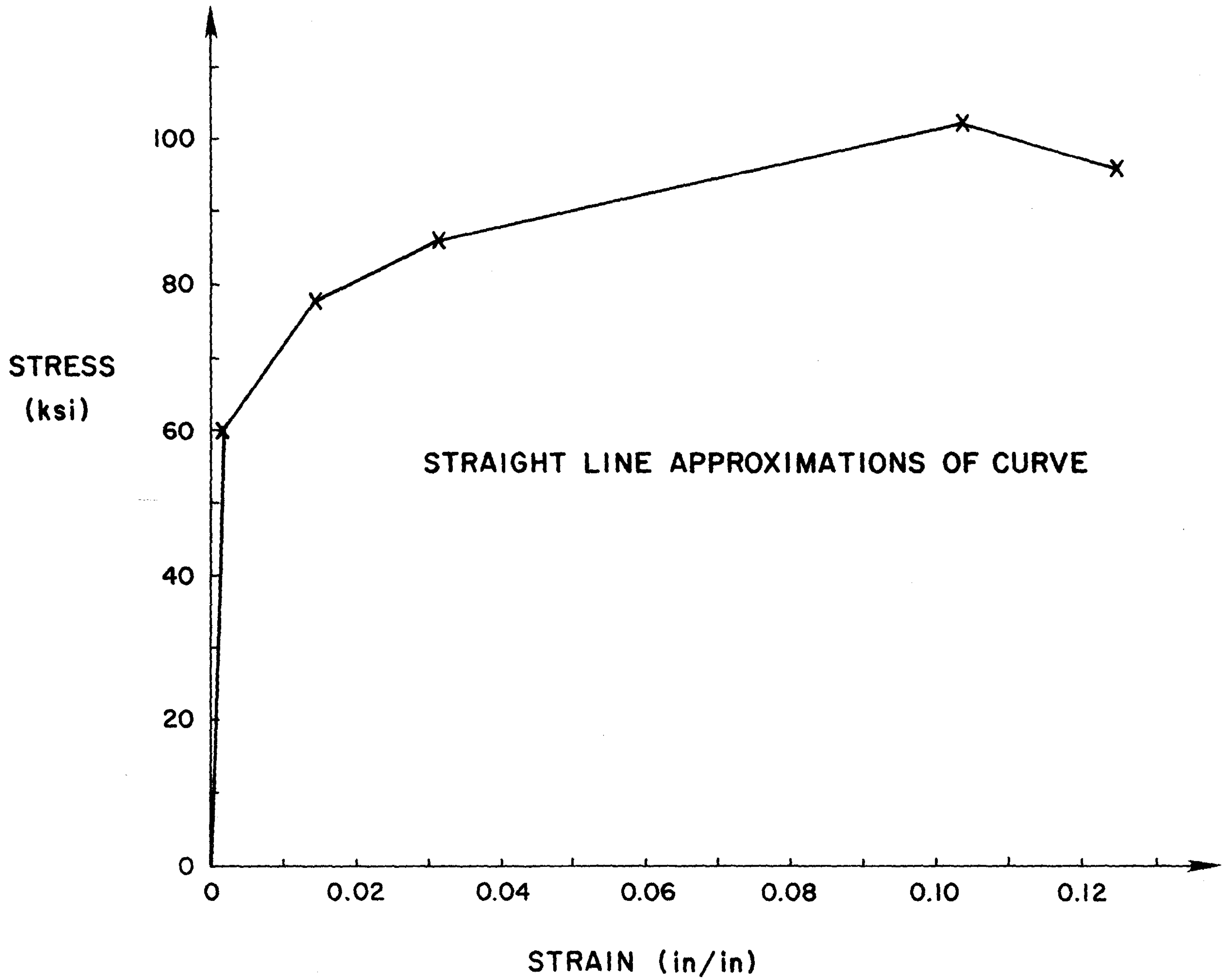

Fig. C1 Stress-strain curve for the #11 rebar used in the floor slab