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TECHNICAL REPORT 3439

TEST OF 1/10 SCALE BAY STRUCTURE

REPORT NO. 6

ESTABLISHMENT OF SAFETY DESIGN CRITERIA FOR USE IN ENGINEERING OF EXPLOSIVE FACILITIES AND OPERATIONS

U.S. ARMY MUNITIONS COMMAND

ARTHUR SCHWARTZ
EDWARD COHEN
NORVAL DOBBS

JULY 1966

PICATINNY ARSENAL
DOVER, NEW JERSEY

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ESTABLISHMENT OF SAFETY DESIGN CRITERIA FOR USE IN ENGINEERING OF EXPLOSIVE FACILITIES AND OPERATIONS

BY

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EDWARD COHEN, NORVAL DOBBS -- AMMANN & WHITNEY

Contract DA-28-017-AMC-423(A)

JULY 1966

AMMUNITION ENGINEERING DIRECTORATE
PICATINNY ARSENAL
DOVER, NEW JERSEY

TABLE OF CONTENTS

FOREWORD

This report was prepared jointly by personnel of Picatinny Arsenal and by personnel of Ammann & Whitney, Consulting Engineers, under Contract DA-28-017-AMC-423(A).

The complete performance of the work covered was the result of a cooperative effort.

Supervision and coordination of the program was by Arthur Schwartz and Richard Rindner under the general supervision of Leon W. Saffian and Stanley Wachtell.

Design of the structure and detailing of drawings was done by Ammann & Whitney under the supervision of Norval Dobbs and Edward Cohen with assistance from Edward Laing, Maurice Rubin and Samuel Weisman, with valuable contributions to preparation of the report by Albert J. Bayruns.

The model test structure was fabricated at the Carleton Civil Engineering Laboratories of Columbia University under the direction of Professor Charles W. Thurston.

Testing of the structure was performed at Picatinny Arsenal at the Bear Swamp range by Robert M. Michael and Glenn Ward.

Analysis of the test results was performed by personnel of Picatinny Arsenal and Ammann & Whitney.

ABSTRACT

Three tests of a 1/10 scale specially reinforced concrete test bay were performed to evaluate the explosive capacity of a specific cubicle arrangement and as a first step in establishing the validity of scaled testing of concrete structures.

The tests consisted of firing 2.00, 3.24 and 4.25-lb. charges of Composition B (equivalent to 2,000, 3,240 and 4,250 lbs. on a full scale) in the center of the test bay and evaluating the extent of damage to the bay to estimate its ultimate explosive capacity.

The progressive damage to the structure was recorded and the bay tested until it was considered no longer usable.

A detailed description of test procedures is given with illustrations of damage after each test and sketches of the test bay showing the development of the crack patterns in the floor and walls with each subsequent test.

A detailed calculation of the center wall capacity of the test bay also is included.

SUMMARY

A series of 1/10 scale bay tests were performed to evaluate the explosive capacity of a specific cubicle arrangement and to obtain information pertaining to the establishment of model factors which will relate the results of the 1/10 scale model bay tests to results of other scale model tests (1/3, 1/5 and 1/8) of the same structure as well as to the full-scale unit.

The 1/10 scale structure was a 4-foot-wide, 2-foot-long and 1-foot-high explosive storage bay consisting of three walls and a floor slab with the end wall and roof open (Figure 1). The bay was constructed to simulate adjoining cells that would exist in an actual manufacturing facility. Both the bay dimensions and the diameter of the reinforcement were scaled linearly from a full-scale prototype design. The explosive charge was scaled by the cube-root relationship.

A total of three tests were performed in the structure and documented by high speed and still photography. Post-shot deflection measurements were taken and maps of the crack patterns were drawn for each shot.

In the first test, the donor charge was a 2-lb. spherical charge of Composition B. The charge was located in the center of the cubicle. The damage sustained by the cubicle was very light. The floor slab in the donor cell received the greatest amount of damage. The damage to the side walls -- both the donor and acceptor panels -- was negligible. The back wall suffered slightly more damage than the side walls. A few hairline cracks with one major crack in the center of the back wall (donor panel) were observed. In general, the structure withstood the blast load well.

The second test in the same structure involved 3.24 lbs. of Composition B (consisting of three charges weighing 1.08 lb. each). This test produced only minor additional damage to the walls, although the damage to the back wall was more extensive than in the first test and the previous center crack widened appreciably. In addition, the tension reinforcement at the base of the donor panel failed over a length of about three inches measured along the base of the wall.

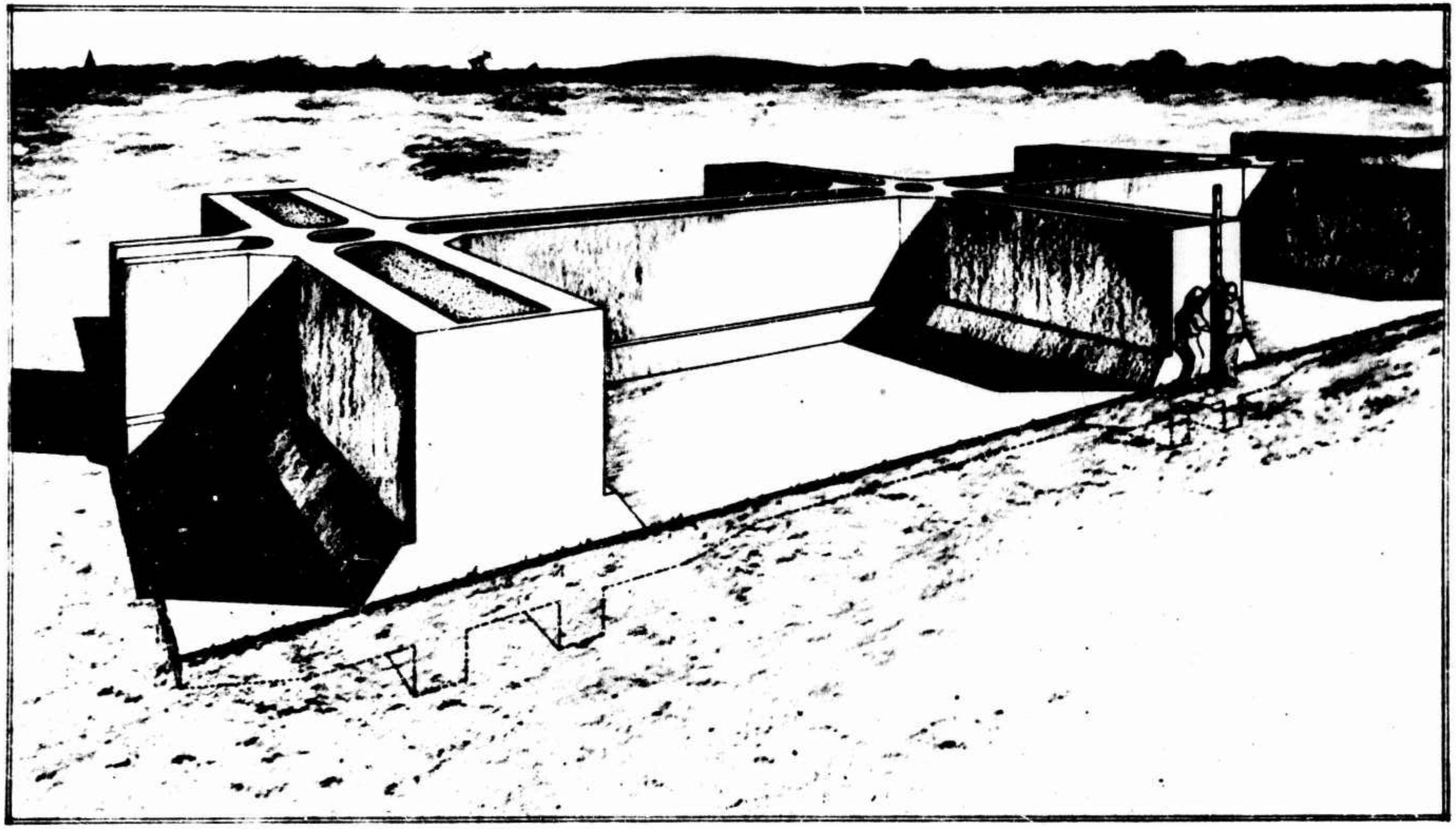

FIGURE 1
PROTOTYPE OF BAY STRUCTURE

The third test involved five centrally located Composition B charges totaling 4.25 lbs. The test resulted in extensive damage to the back wall (incipient failure). The final failure of the structure was caused by the increased load acting on the side wall which tended to split the structure in the middle, in addition to the damage sustained by the wall and the floor slab in two previous tests. The side walls remained basically intact except for slight failure at the intersections of the side and back walls in the donor panel.

A pre-test analysis of the back wall of the structure indicates that the maximum explosive capacity of an interior cell of a multi-cubicle complex would be in the order of 7,000 lbs. (Appendix B).

CONCLUSIONS

Based onthe results of this 1/10 scale bay test (and subsequently confirmed tests of 1/3, 1/5 and 1/8 bay structures), it was demonstrated that a reinforced concrete cubicle type structure when designed in the prescribed manner will withstand the blast output of relatively large quantities of explosives.

For the cell dimensions and charge location considered, the bay structure design is adequate for explosive capacities in excess of 5,000 lbs. in a full-scale arrangement.

The use of heavily reinforced concrete in combination with sand fill (composite construction) appears to be an effective means of confining the blast effects of large explosive quantities in explosive operating facilities.

The use of diagonal shear reinforcement was demonstrated as an effective means of tying the reinforcing within the concrete thereby fully developing the ultimate capacities of the steel and the concrete.

Reinforced concrete haunches (Figure 2) tend to reduce multiple blast pressure reflection build-up in the corners of cubicle type structures thus reducing the formation of additional excessive local stresses in areas of high stress concentrations.

The results of this test (and those of larger scale models of the same structure) indicate that the use of scale model tests appears to be an economical means for predicting the results of full-scale tests.

The most likely cause of the major cracks formed at the center of Wall 2 (back wall) was a combination of three distinct actions of the structure:

1. Vertical downward motion of the back wall due to settlement of the underlying earth fill.

2. Horizontal bending action produced at the base of the wall (pedestal) by the blast load acting normal to the surface of the wall.

3. Tension stresses created in the back by the outward motion of the two side walls.

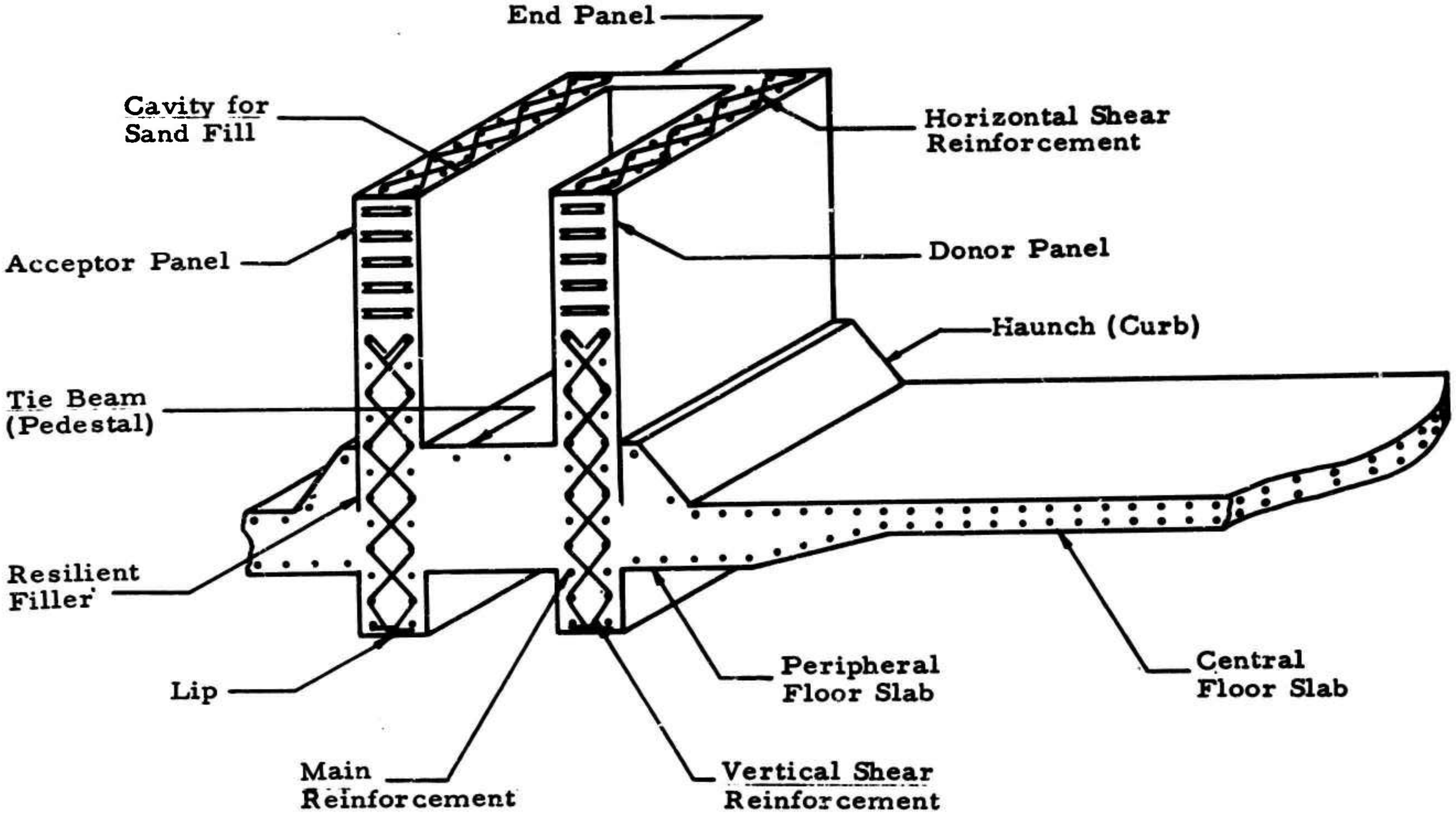

FIGURE 2
TYPICAL WALL AND FLOOR SECTION

BACKGROUND

Picatinny Arsenal, acting under assignment by the Armed Services Explosive Safety Board (ASESB), is engaged in an extensive systematic testing program -- the utlimate objective of which is the establishment of design standards to be employed in the engineering of new explosive storage and explosive manufacturing facilities as well as in the modification of similar existing facilities. Ammann & Whitney, under contract to Picatinny Arsenal, is providing technical assistance on structural problems of the program.

At present, this program is being conducted on the basis of scale models with major emphasis directed toward the study of reinforced concrete for use in cubicle type structures. Model tests utilizing other types of structural materials also are incorporated in the program.

As a part of this program, several approaches were utilized to acquire information from relatively small models (as small as 1/10 scale) to determine the structural capabilities of cubicle type storage and manufacturing facilities. These approaches included the testing of components of cubicle type structures (slab tests) as well as the testing of overall cubicle arrangements (complete cubicles).

Objectives of this test series are to:

1. Establish "model factors" that will relate the test results of model structures to those of their prototypes (full scale) and to each other.

2. Evaluate the explosive handling (storage or manufacturing) capabilities of specific cubicle arrangements.

Primarily this report deals with a particular cubicle arrangement and specifically a 1/10 scale model on which testing was performed during July and August 1965, at Bear Swamp Test Area, Picatinny Arsenal.

Tests also have been performed on 1/3, 1/5 and 1/8 scale models of the storage bay structure. Results of these tests will be covered in a separate report. A full-scale bay will be tested after analysis of the scaled test results is completed.

DESCRIPTION OF PROTOTYPE BAY STRUCTURE

General

In conformance with the criteria prescribed by the ASESB the arrangement of the prototype structure is designed to represent adjoining cells that would exist in an actual explosive manufacturing facility (Figure 1). All structural components are constructed exclusively of reinforced concrete and consist of a floor slab, back wall and two side walls. The front and the top of the structure remain open to the atmosphere. The overall interior dimensions of the prototype cubicle are:

Length.....40 feet, 0 inches
Depth......20 feet, 0 inches
Height.....10 feet, 0 inches

The walls of the structure are built in a composite (sandwich) construction featuring two reinforced concrete panels separated by a sand fill, each wall having an overall thickness of eight feet (Figure 2). For all three walls of the structure, the cross-section is identical -- a 2-foot-thick "donor panel," a 4-foot-wide "cavity" loosely filled with rounded sand and a 2-foot-thick "acceptor panel." In the side walls, closure of the sand cavity is provided by an "end panel." At the intersection of the side wall and the back wall a sand filled cylindrical cavity extends to the "pedestal."

At the exterior of the bases of all three walls are reinforced concrete "haunches," 2-feet-high x 2-feet-wide. Although these haunches are monolithic with the base slab, they are isolated from the walls by means of a 1-1/4-inch-thick rubber base "filler."

Separating each concrete panel of each wall at the base are reinforced concrete pedestals. . However, unlike the haunches, the pedestals are connected by reinforcing steel both to the concrete panels and to the floor slab, thereby forming a "tie beam." The sand fill is situated immediately above the pedestal.

Adjacent to the walls is the peripheral floor slab which has a thickness of two feet and surrounds the 1-foot-thick central floor slab. The transition between the two thicknesses is accomplished by an intermediate taper.

Reinforcement

High strength billet steel bars conforming to ASTM Specification A-432 are used throughout for reinforcement of the concrete. The main horizontal and vertical reinforcement in both the back and the side walls consists of 1-1/8-inch-diameter bars and is identical at both surfaces of each panel (Figure 3). As is the case with the horizontal reinforcement, the adjacent vertical bars in the walls are tied together by means of diagonal shear reinforcement having a diameter of 5/8-inch. The shear reinforcement in the vertical direction is continuous almost over the lower half of the wall height whereas the shear reinforcement in the horizontal direction is continuous over the entire wall length. In the peripheral floor slab the reinforcement is identical to that utilized in the walls -- 1-1/8-inch-diameter bars. On the other hand, in the central floor slab only minimum reinforcement of 1/2-inch-diameter bars is provided.

Basis of Design

The basic concept of this structure differed from those used previously in that a substantially larger amount of reinforcing steel was provided and this augmented reinforcement was completely developed by a shear tie system. Another variation in this design consisted of full development of the reinforcement by the use of a floor system that was the structural equivalent of the walls. These features (together with other advanced characteristics) permitted the realization of a structurally sound and unique design.

Sandwich type wall construction of the proportions incorporated in the prototype affords efficient inexpensive absorption of blast energy by reason of the large volume of low cost sand utilized in the cavity between the donor and the acceptor panels (Figure 2). Although the sand adds mass to the structure, its primary purpose is to assist the concrete panels in absorbing and redistributing a portion of the blast impulse. This action results from the high energy absorbing capacity of sand -- particularly of sand composed of rounded particles -- due to the force required for compacting the sand and displacing the individual particles toward the adjacent interstices and redistributing the blast energy through the various points of contact among the sand particles.

The pedestal or tie beam is provided at the base of the wall to tie the donor panel to the acceptor panel at the base and at the sides -- ensuring a completely monolithic wall.

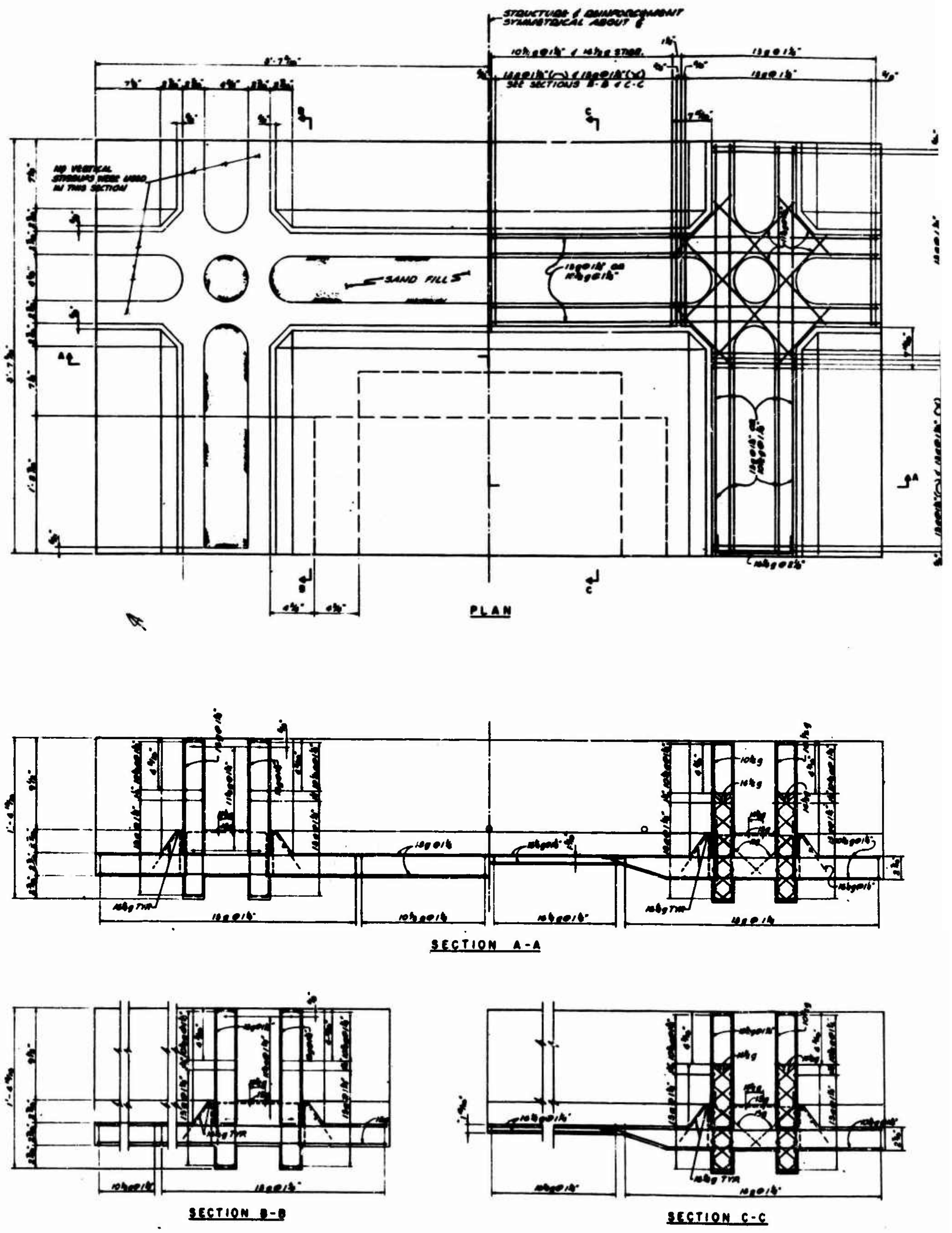

FIGURE 3
BAY STRUCTURE - CONSTRUCTIO

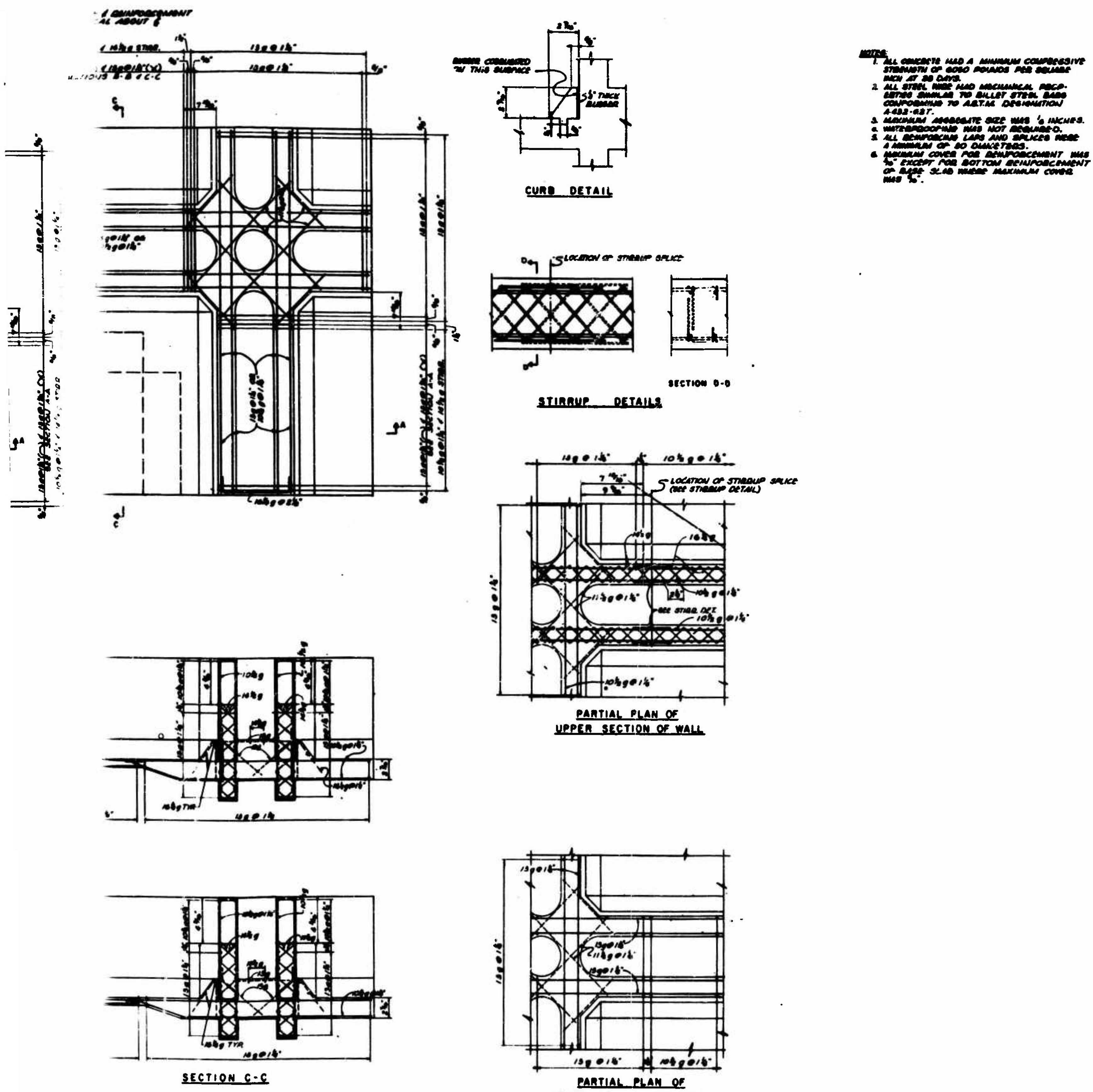

FIGURE 3
ION ιE - CONSTRUCTION DETAILS

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A non-integral haunch or curb was included at the base of each wall panel to prevent a build-up of the reflected pressure due to the blast. In this design a resilient filler was inserted between the vertical face of the haunch and the wall panel to cushion the load acting on the haunch and thus reduce transmittal of the load to the pedestal. However, this filler was eliminated after the first 1/10 scale bay was built because of construction difficulties.

At the base of each wall panel a lip provides anchorage for the vertical reinforcing bars in such a manner that both panels act monolithically with the floor slab. In addition, construction is facilitated by the lips in that they serve as a means of support for the vertical reinforcement.

Closure of the sand cavity in the side walls is the principal purpose of the end panels. However, by reason of their shear action these panels also provide a certain amount of supplementary strength to the side walls.

A cylindrical cavity is provided at the intersection of the side wall and the back wall to prevent concentrations of stress at that area.

Extending a predetermined distance from each wall is the peripheral floor slab having the thickness required to develop the full strength of the wall. The thickness of the central floor slab is only 50% that of the peripheral slab, not only to reduce construction costs, but also to permit the load to be borne partially by the underlying soil and not solely by the walls.

In addition to the main horizontal and vertical reinforcement (Figure 4), all three walls contain diagonal shear reinforcement (Figure 5) so designed as to impart shear strength to the individual panels. Furthermore, the shear reinforcement ties the main reinforcement together thus preventing tension failure of the concrete (Figure 6). Vertical shear reinforcement is located in the region of high shearing forces along the floor slab supports and is discontinuous in the area of nominal shear stresses. Horizontal shear reinforcement is situated in the upper portion of the panel where the shearing stresses are greatest.

The method of calculating the maximum capacity of the back wall of this structure is given in Appendix B.

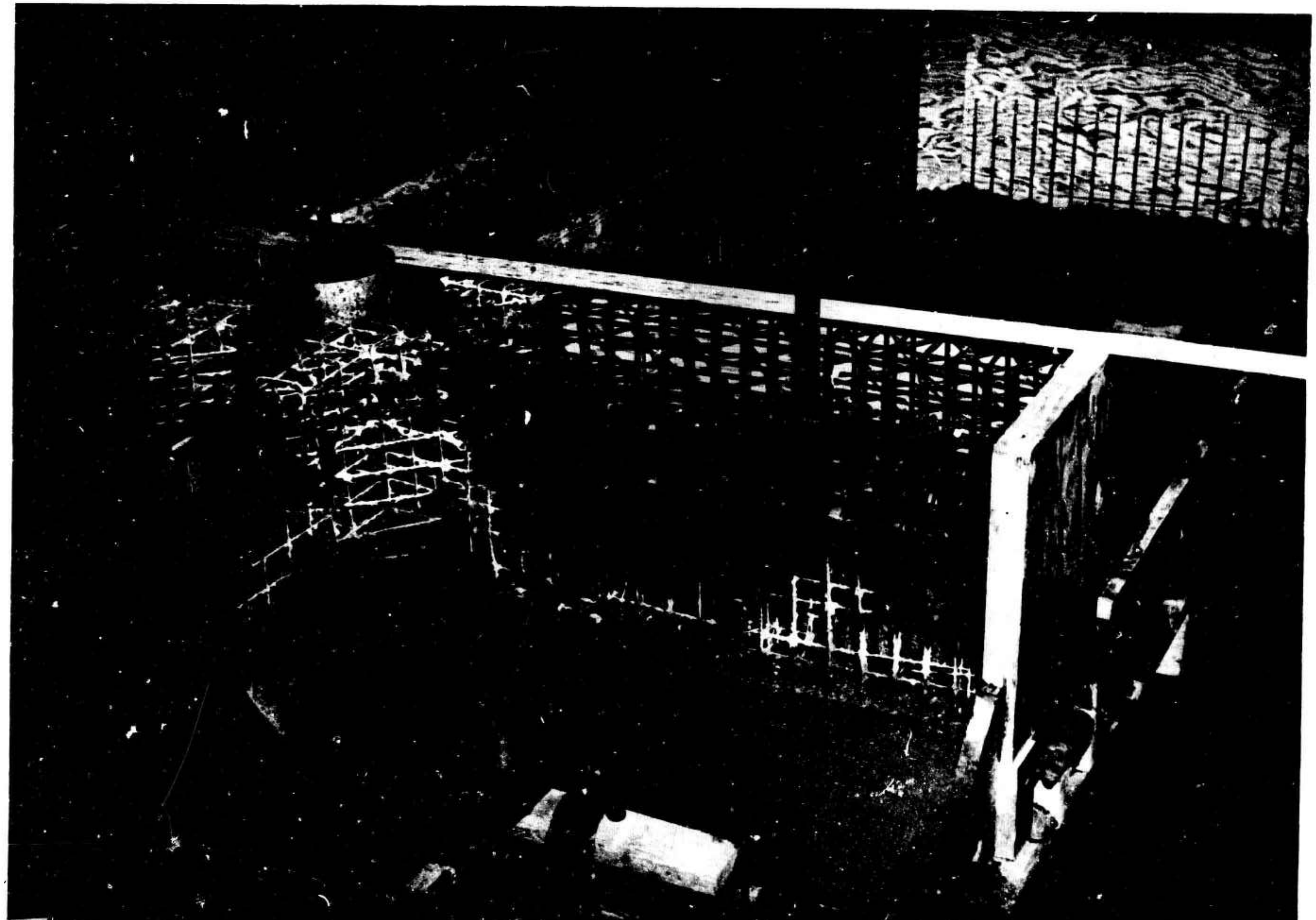


FIGURE 4
MAIN WALL REINFORCEMENT

FIGURE 5
DIAGONAL SHEAR REINFORCEMENT

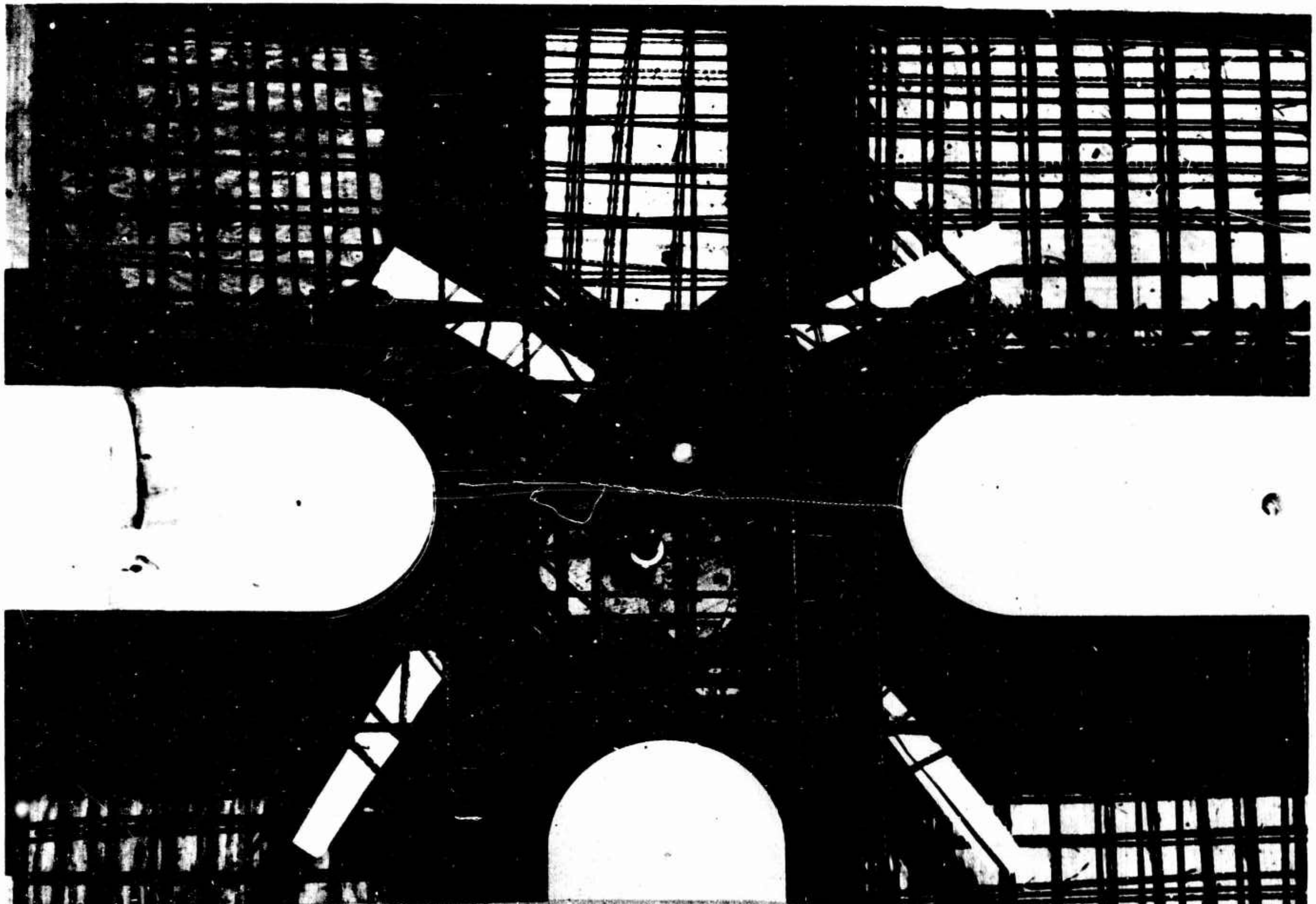


FIGURE 6
HORIZONTAL STEEL SHEAR REINFORCEMENT AT
INTERSECTION OF BACK AND SIDE WALLS

DESCRIPTION OF TEST STRUCTURE

For this test, the structure was a 1/10 scale model of the prototype cubicle (Figure 7 and 8).

For determining the size of the test specimen, the Geometrical Scaling Method was used (Reference 1). Both the dimensions, reinforcement and concrete aggregate of the test structure were scaled in accordance with the size of the model. The dimensions of the structure as well as the sizes and spacing of the reinforcing bars were varied in direct proportion to the scaling factor of the model; however, the area of reinforcement was scaled as a function of the square of the scaling factor. If, as was inevitable in certain instances, the exact scaled bar sizes and spacing could not be provided in the model because the particular wire sizes required were not available, the sizes and the spacing were so adjusted as to furnish the proper scaling of the reinforcement area and thereby maintain the correct scaled strength of the structure. The sand used in the cavity was the same grain size and type specified for the prototype.

The overall interior dimensions of the model were:

Length.....4 feet, 0 inches
Depth......2 feet, 0 inches
Height.....1 foot, 0 inches

The walls consisted of a 2-7/16-inch donor panel and a 2-7/16-inch acceptor panel separated by a 4-3/4-inch cavity loosely filled with sand having a density of about 80 lbs. per cubic foot.

The peripheral floor slab was 2-7/16-inch-thick and the central floor slab was 1-3/16-inch-thick.

Reinforcing bars were of cold drawn steel wire (AISI C-1040) that was annealed to simulate the mechanical properties of the high strength bars utilized in the prototype. The main horizontal and vertical reinforcement consisted of No. 1-1/2-gage wire (0.12-inch-diameter).

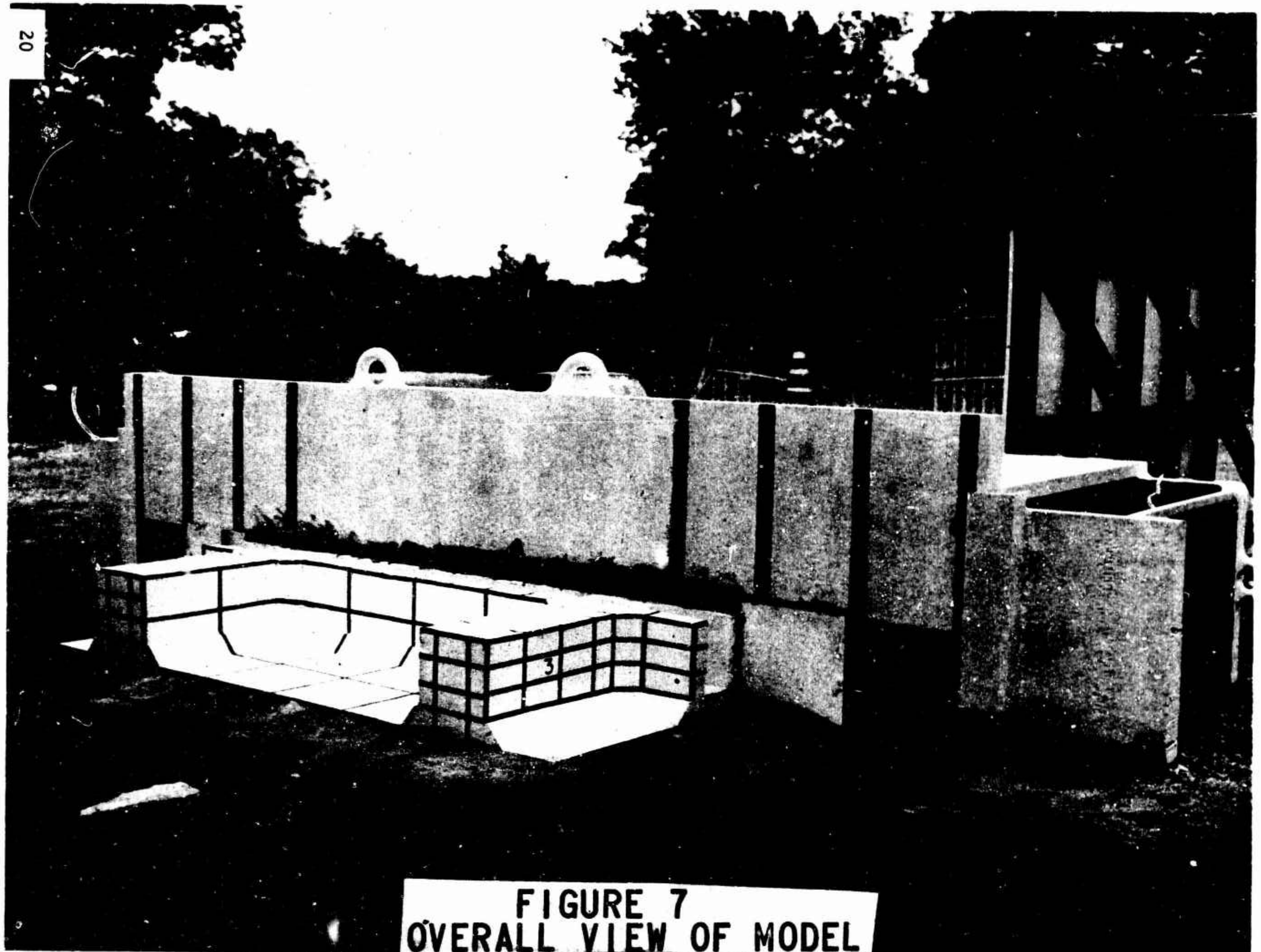


FIGURE 7
OVERALL VIEW OF MODEL

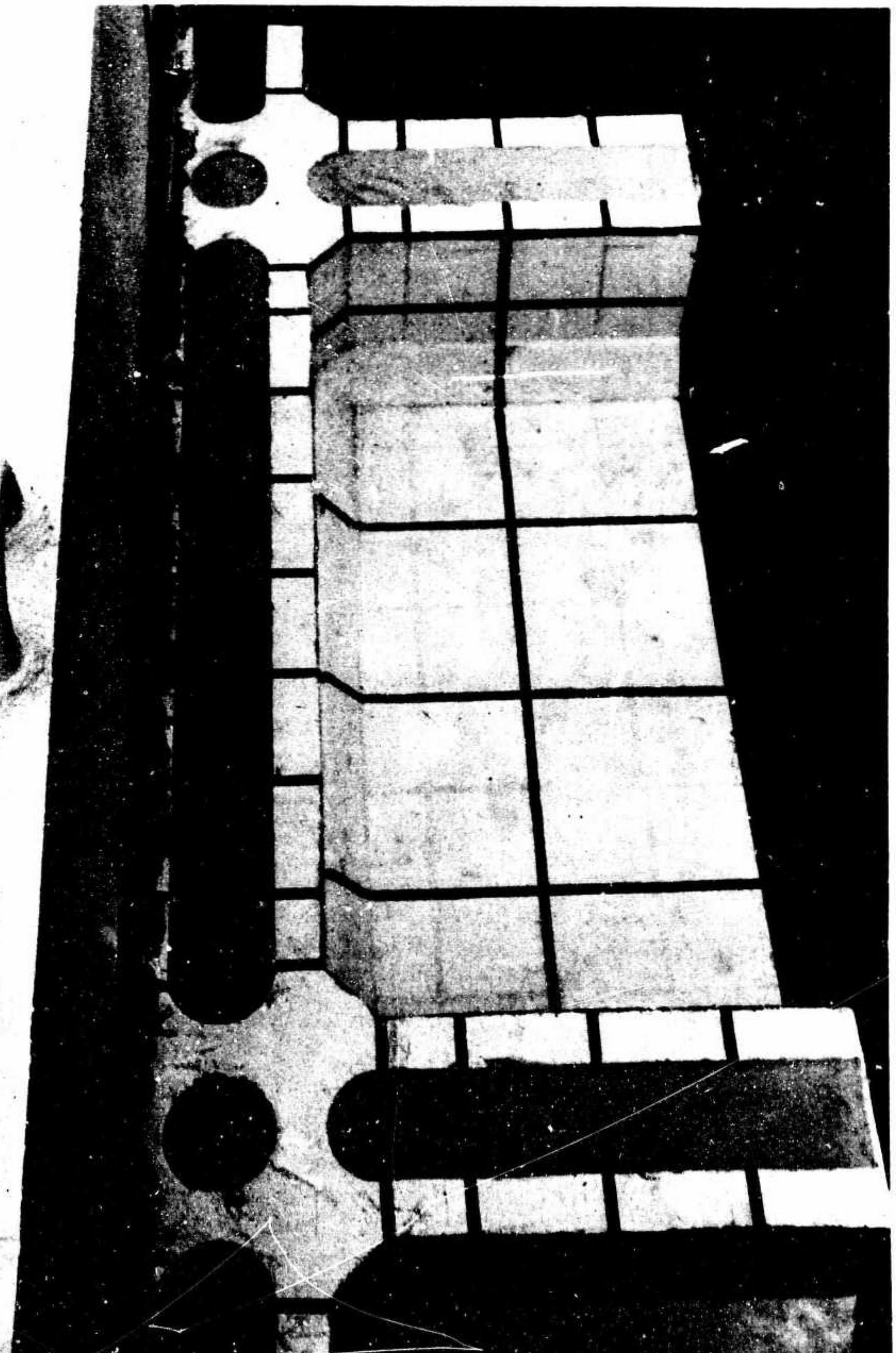


FIGURE 8
PLAN VIEW OF MODEL

DESCRIPTION OF TEST SET-UP

General

Each test set-up comprised three components:

Test structure
Donor charges
Photographic coverage (both still and motion pictures)

The same structure was tested three times with successively larger donor charges and the photographic coverage was modified as experiences dictated. Figure 9 illustrates the preliminary layout of the test facility.

Donor Charges

Charge weights were varied in the successive rounds of the test. In the first round, the weight of the charge was a scale model of the 2,000-lb. prototype or 2 lbs. In subsequent rounds the charge weight was established on the basis of the damage sustained by the test structure in the preceding round. Properties of the donor charges are in Table 1.

All charges were bare spherical Composition B explosive. Initiation of all charges was accomplished at the center of the sphere by means of engineer's special blasting caps placed in a radial hole drilled to the center of the charge. In all cases the detonator side of the charge was placed away from the back wall of the structure.

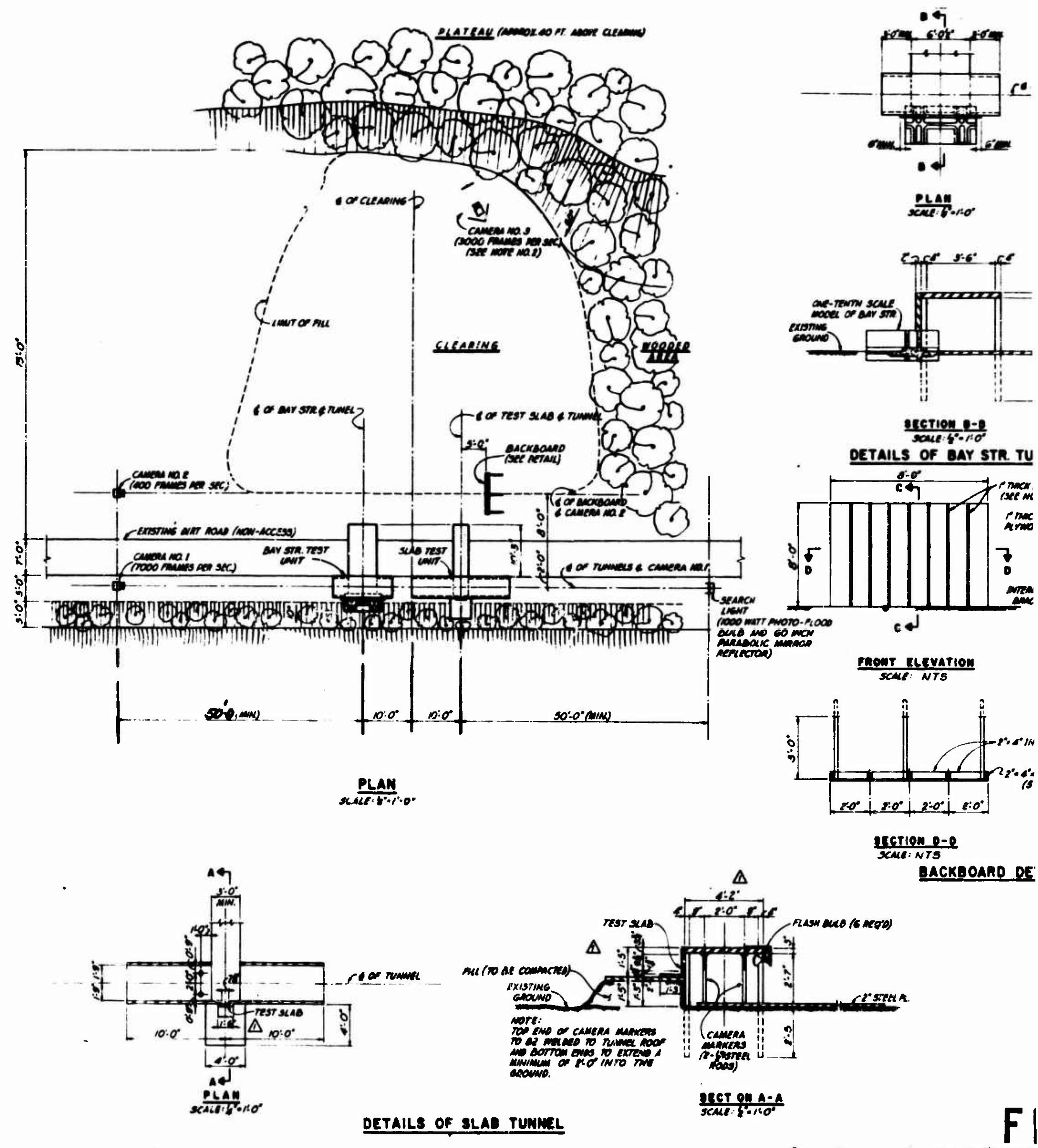
In Round 1 the charge consisted of a single sphere (Figure 10) whereas several spheres arranged in clusters were used in Round 2 and 3 (Figure 11). In all cases the centroid of the explosive was located at the center of cubicle -- midway between the side walls, one foot from the back wall and six inches above the floor slab. In those rounds where the cluster arrangement was employed the individual charges were placed in a plane parallel to the back wall. In the third round, the large charge was placed closer to the floor slab. Here again this type of arrangement was selected for the purpose of directing the path of the reflected shock wave formed at the interfaces of the various charges towards the top of the structure.

Figure 12 illustrates the charge arrangements used in all three rounds.

TABLE 1

PHYSICAL PROPERTIES OF THE EXPLOSIVES

Round Number	Type of Charge	Unit Weight (lbs.)	Quantity	Total Weight (lbs.)	Distance to Back Wall (ft.)	Scaled Distance $(ft./lb.^{1/3})$	Equivalent Full-Scale Weight (lbs.)
1	Single	2.00	1	2.00	1.0	0.795	2,000
2	Cluster	1.08	3	3.24	1.0	0.675	3,240
3	Cluster	0.81 1.00	4 1	3.24 1.00 4.24	1.0	0.620	4,240



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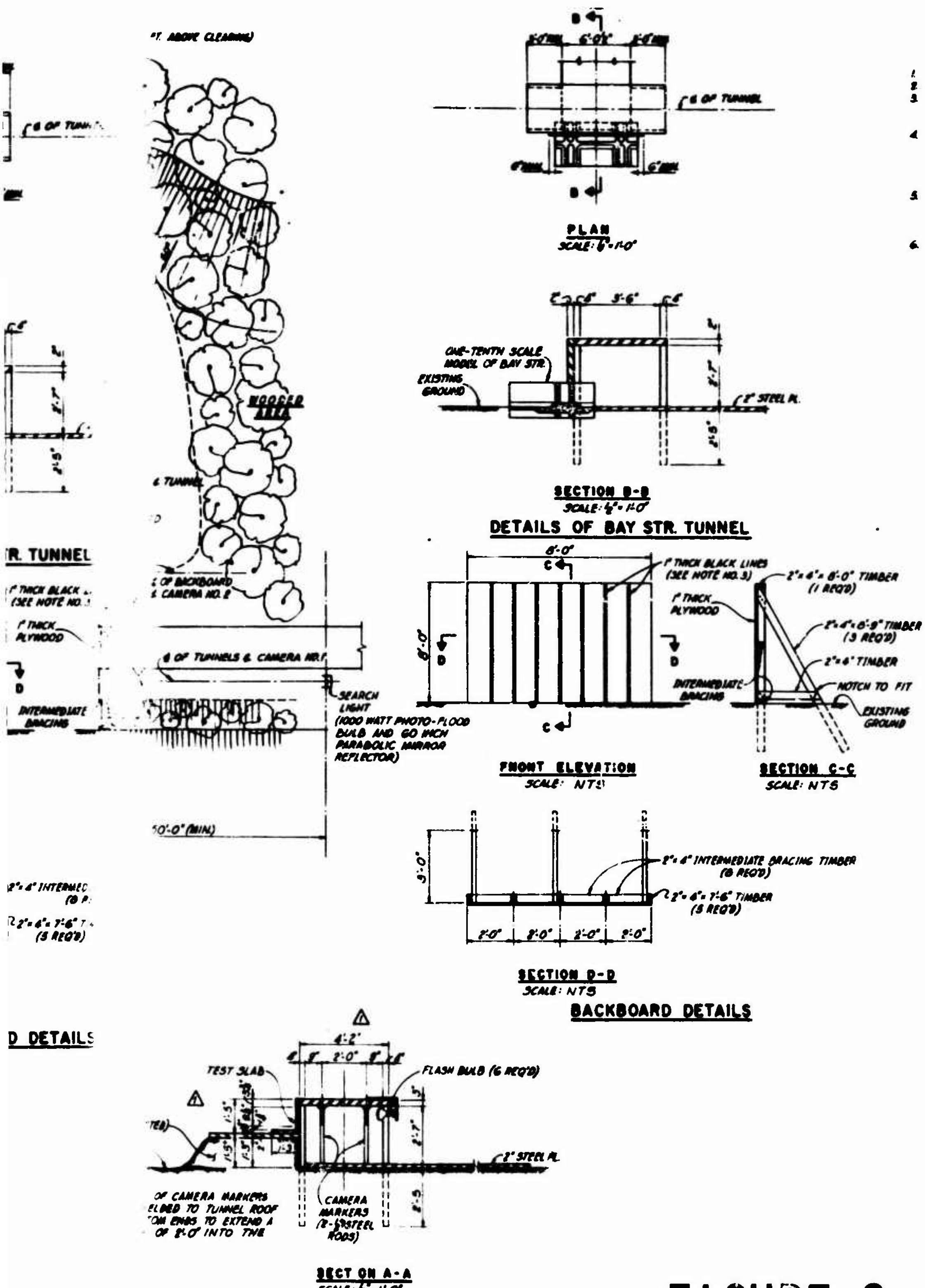
SITE LAYOUT



NOTES:

1. ALL OVERALL DIMENSIONS OF AREA ARE APPROXIMATE.
2. [illegible]DE BARRICADE AND MIRROR IF NECESSARY.
3. PAINT FACE OF BACKBOARD WHITE. ALSO PROVIDE ONE INCH THICK BLACK LINES ONE FOOT ON CENTER THE FULL HEIGHT OF THE BOARD.
4. THE REAR SURFACE OF THE SEALING BLOCKS, THE SURFACE OF THE SLAB ADJACENT TO THE TUNNEL WALL AND ABJOINING SURFACES OF THE BLOCKS AND SLAB ARE TO BE COATED WITH GREASE BEFORE EACH TEST.
5. 2" STEEL PLATES EXTENDING FROM REAR OF TUNNELS TO BE MARKED ACROSS THEIR ENTIRE WIDTH AT ONE FOOT INTERVALS ALONG THEIR ENTIRE LENGTH.
6. BOLT DIAGONAL LEG SUPPORTS AND BRACING TO BACKBOARD.

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FIGURE 9
SITE LAYOUT AND TEST SET-UP

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FIGURE 10
TEST SET-UP FOR ROUND 1

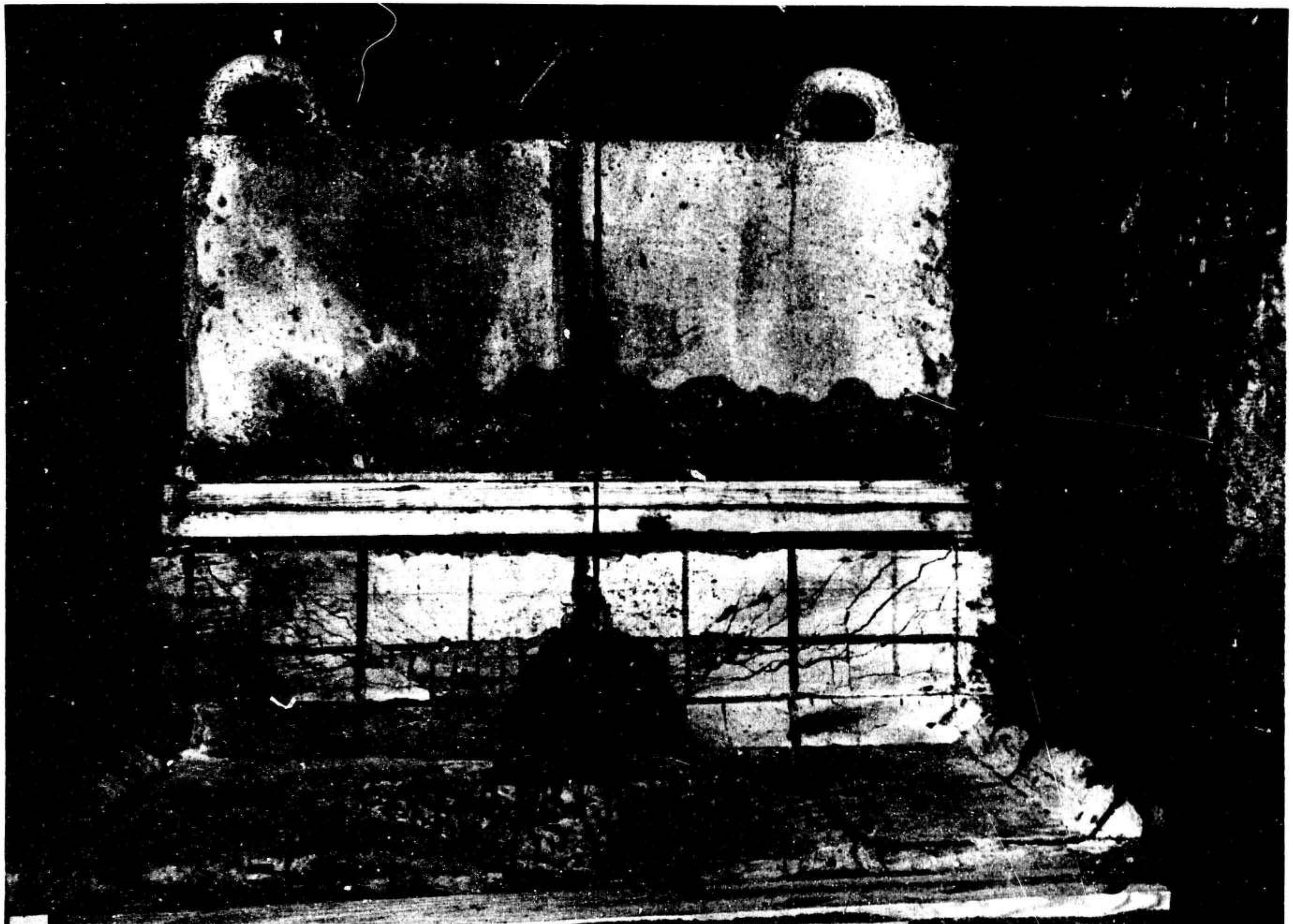

FIGURE 11
TEST SET-UP FOR ROUND 3

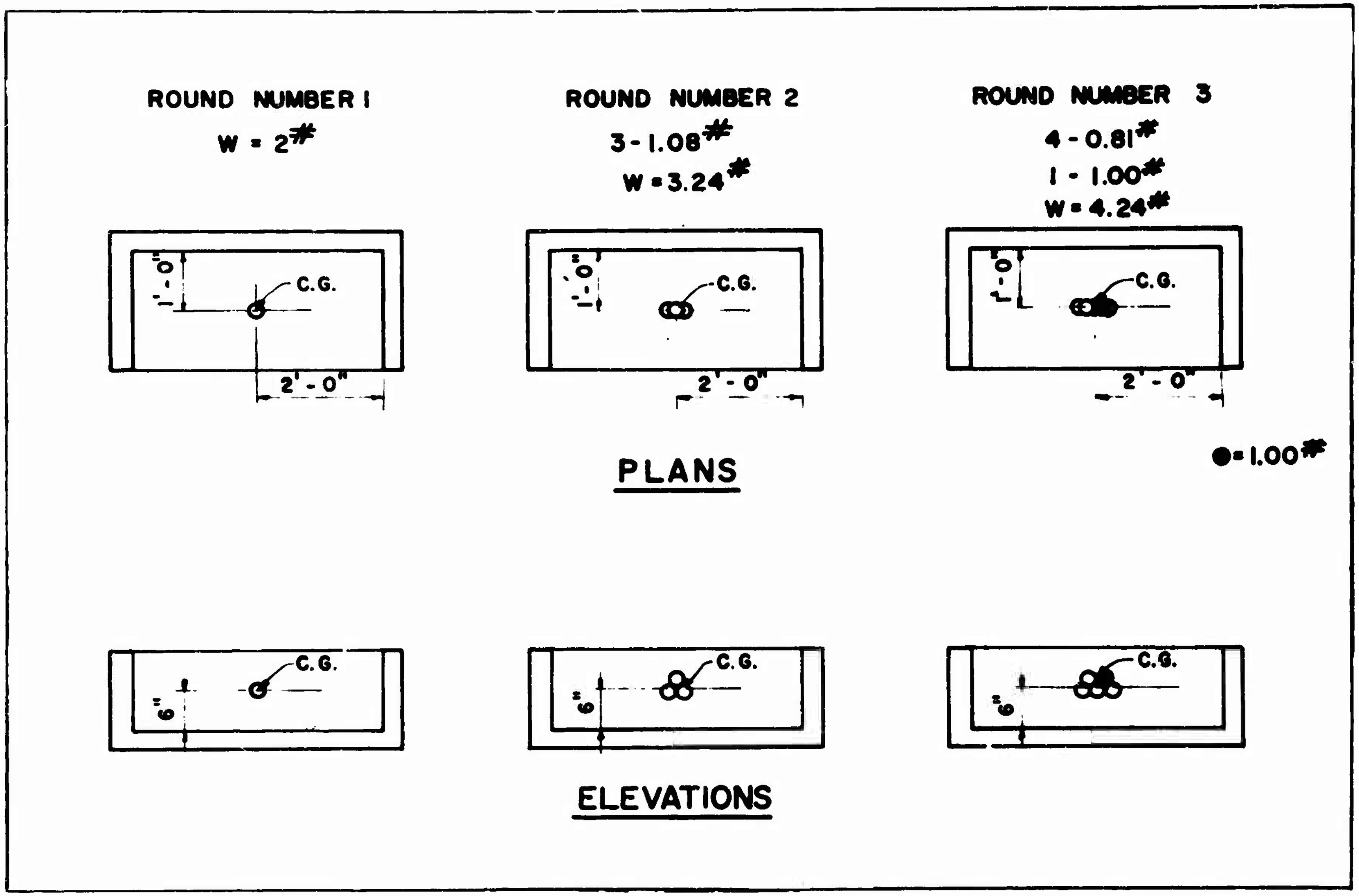

FIGURE 12
CHARGE LAYOUT

Photographic Coverage

Two types of photographic coverage of the tests were used: still photography and motion pictures. The still photographs recorded both pre- and post-shot test arrangements and results. The motion pictures were employed principally to determine the damage characteristics of the test structure including fragment velocities and fragment distribution. Camera layout and speeds are indicated in Figure 13.

Two basic motion picture arrangements were employed (Reference 2):

The backboard method
The rear wall viewing method

The primary purpose of the first method was to determine fragment velocities and the second method recorded the manner in which the test structure was damaged.

To assist in the viewing of the rear surfaces of the walls, flash shields were utilized to restrain the gases, smoke and dust formed by the explosion.

In the case of the back wall, a steel tunnel arrangement was employed to provide a seal around the edges of the wall. The sealing was accomplished by a series of steel plates (part of the tunnel) resting on the top and against the sides of the walls (Figure 14). To reduce the accumulation of dust the contact surfaces of the plates and the walls were coated with grease.

Shielding of the side walls was effected by means of Celotex or plywood sheeting that extended above and beyond the walls thereby producing a longer path over which the smoke and dust were required to travel above or around the walls (Figure 10 and 15).

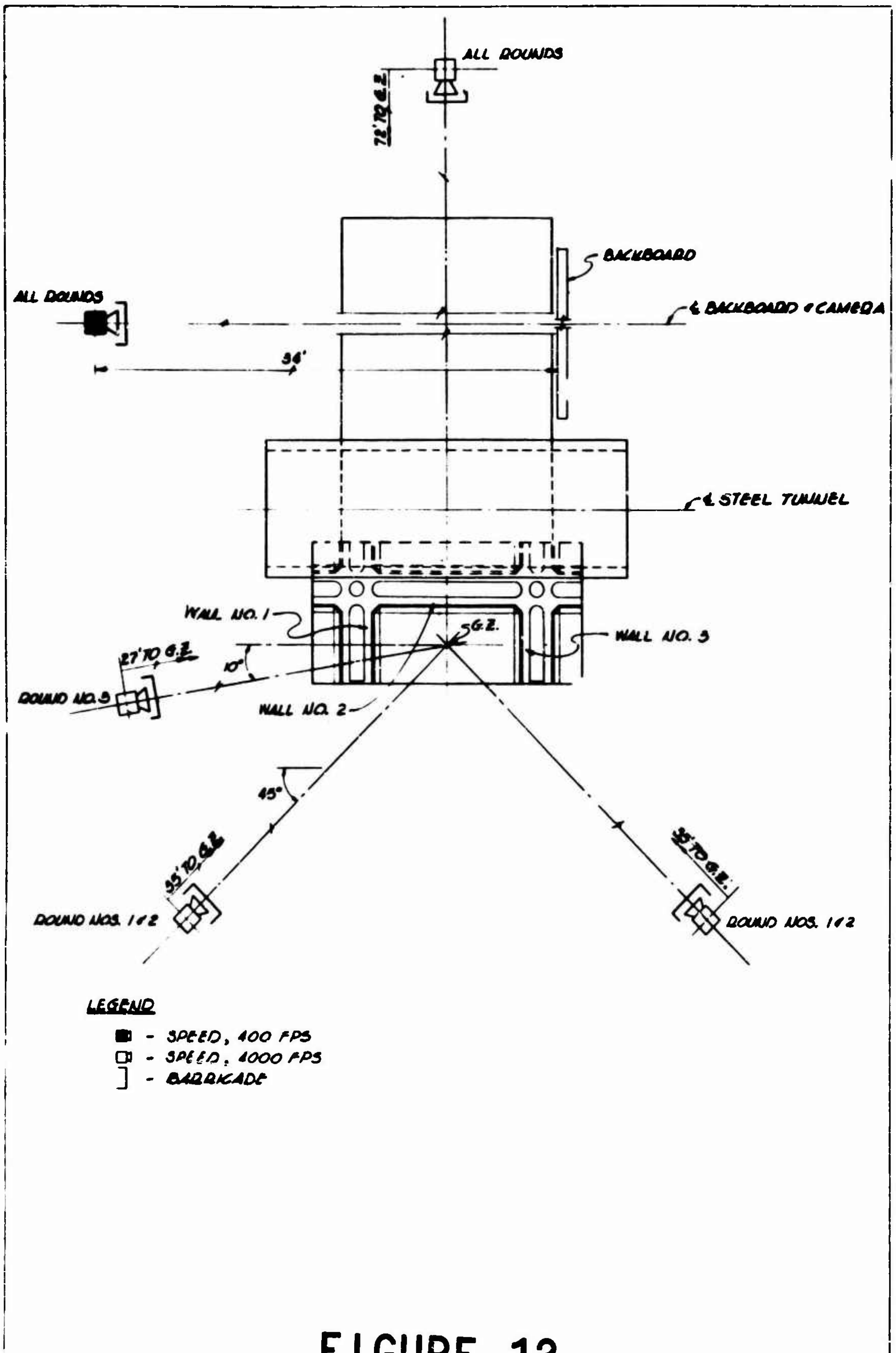

FIGURE 13
CAMERA LAYOUT

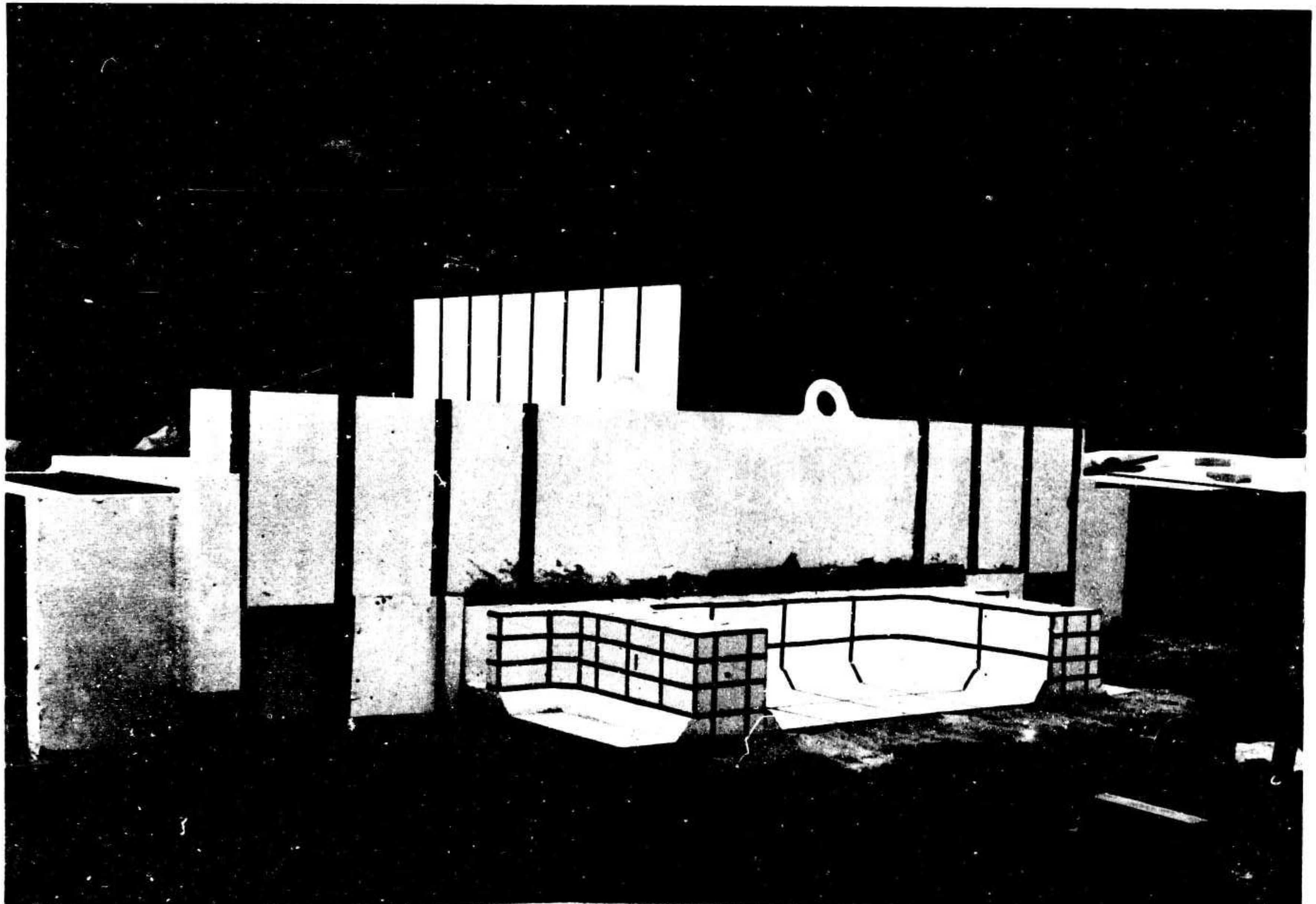

FIGURE 14
STEEL PLATE ARRANGEMENT FOR SEALING
OF BACK WALL

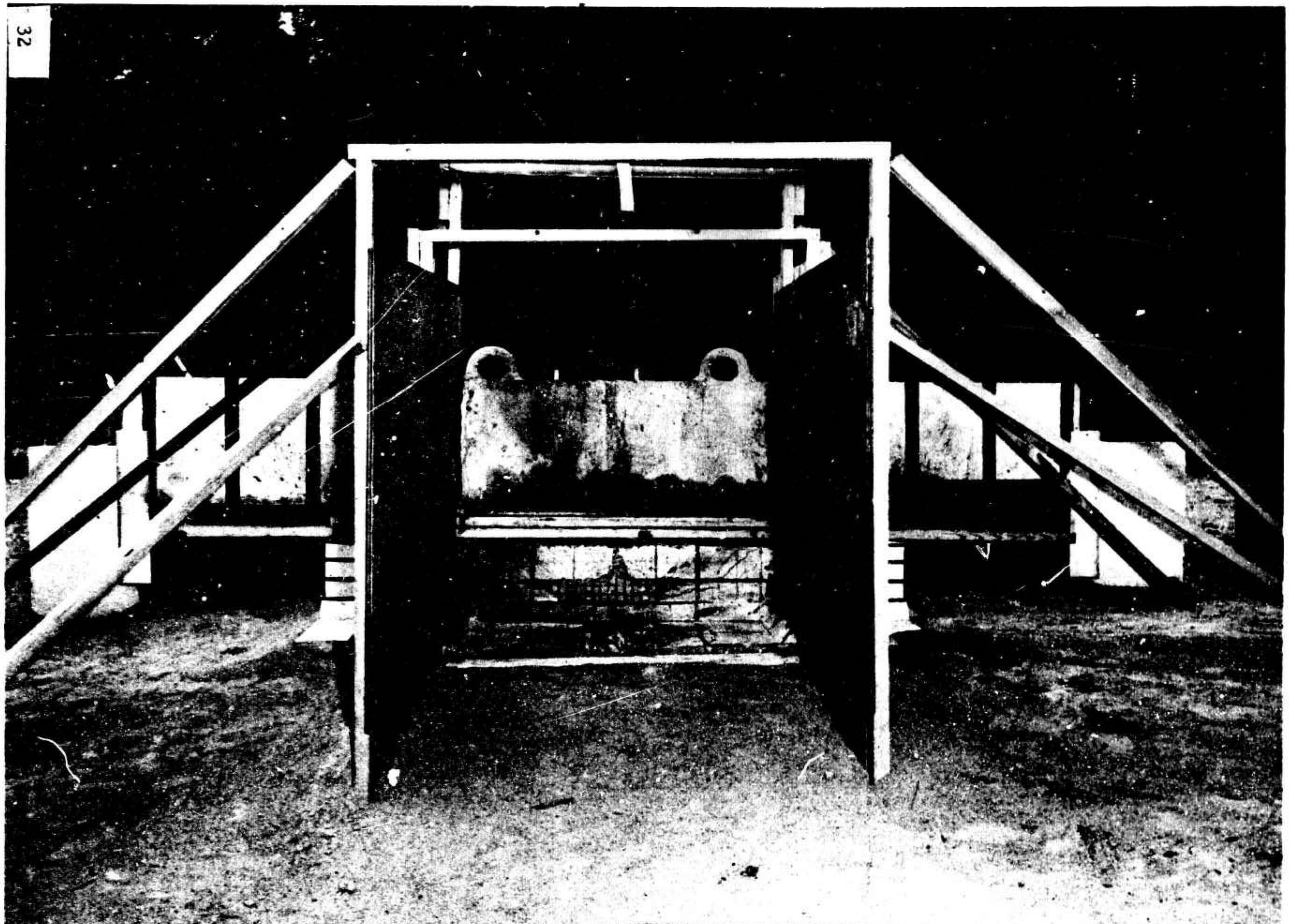

FIGURE 15
FLASH SHIELD USED IN ROUND 2 AND 3

TEST RESULTS

Description of Damage

Round 1 (W = 2 lbs.) -- Slight damage was sustained by the structure as a whole although certain sections suffered more than others (Figure 16). As expected, the floor slab in the donor cell sustained the greatest amount of damage and the donor panel of the back wall exhibited somewhat less damage. The damage to the donor panels of the side walls was substantially less than that sustained by the back wall. The tie beam at the base of the back wall was subjected to slight damage and the damage to the tie beam was slightly less than that of the floor slab in the cell at the acceptor side of the back wall. All other portions of the structure remained intact.

Figure 17 and 18 are a plan view and elevations, respectively, of the damage sustained by the test structure.

Floor Slab -- The main damage to the floor slab of the donor cell occurred near ground zero. Spalling of the slab surface took place directly below the charge while slightly greater damage to the floor slab occurred closer to the back wall (Figure 19). At this latter section, the concrete cover was displaced and the reinforcement was exposed at the inner edge of the thinner portion of the slab.

Generally, the floor slab displayed a circular pattern of cracking that followed the dishing action of the slab. The circular cracks were mainly hairline in nature except at the periphery of the damage where the cracks varied from 1/32- to 1/8-inch in width. In addition, some larger cracking was evident at the intersection of Wall 2 (back wall) haunch and the floor slab indicating displacement of the floor slab away from the walls (Figure 17). The total length of the haunch cracks was about one foot and the cracking straddled the center line of the cell.

Figure 20 is an overall view of the damage sustained by the floor slab.

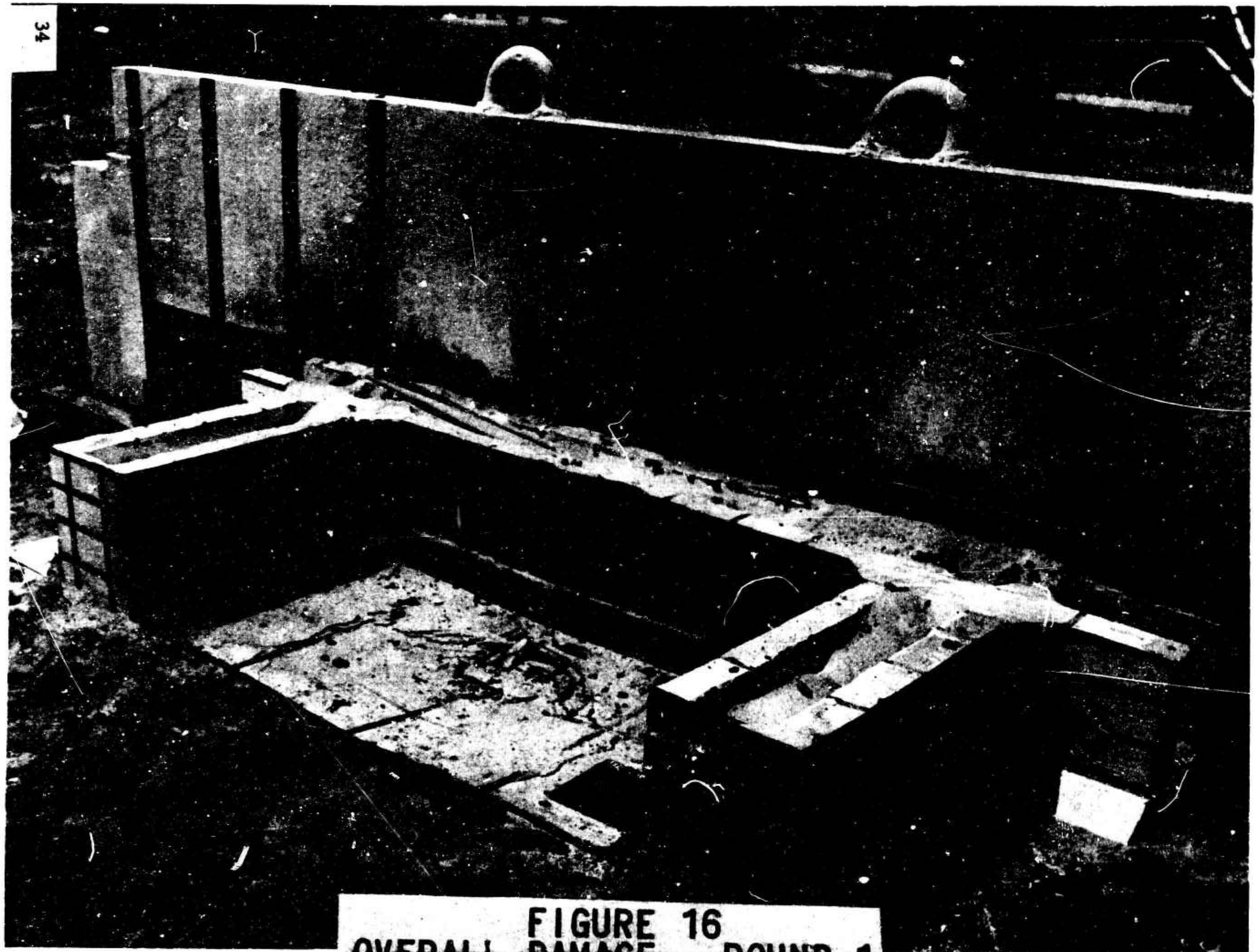

FIGURE 16
OVERALL DAMAGE - ROUND 1

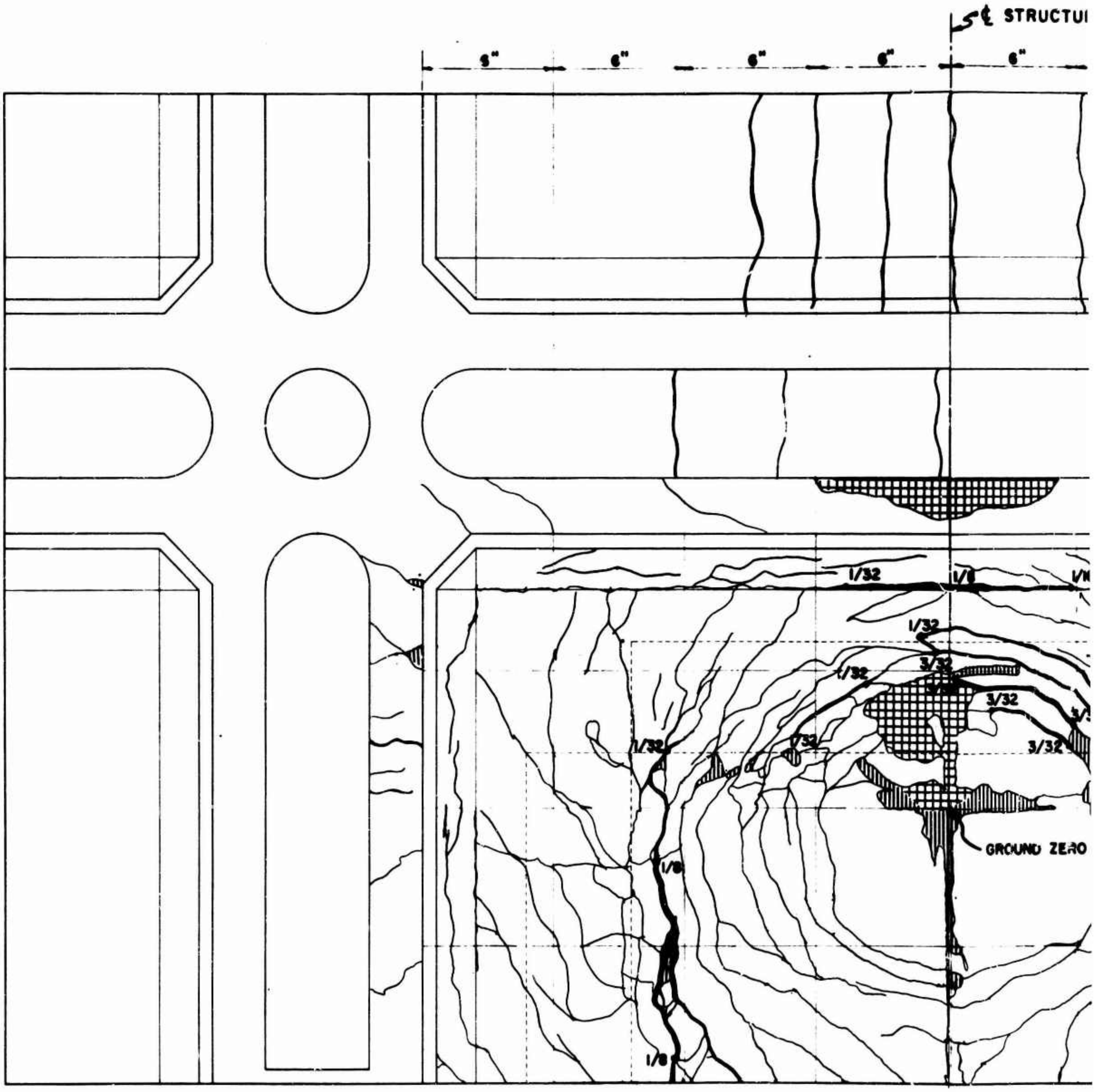

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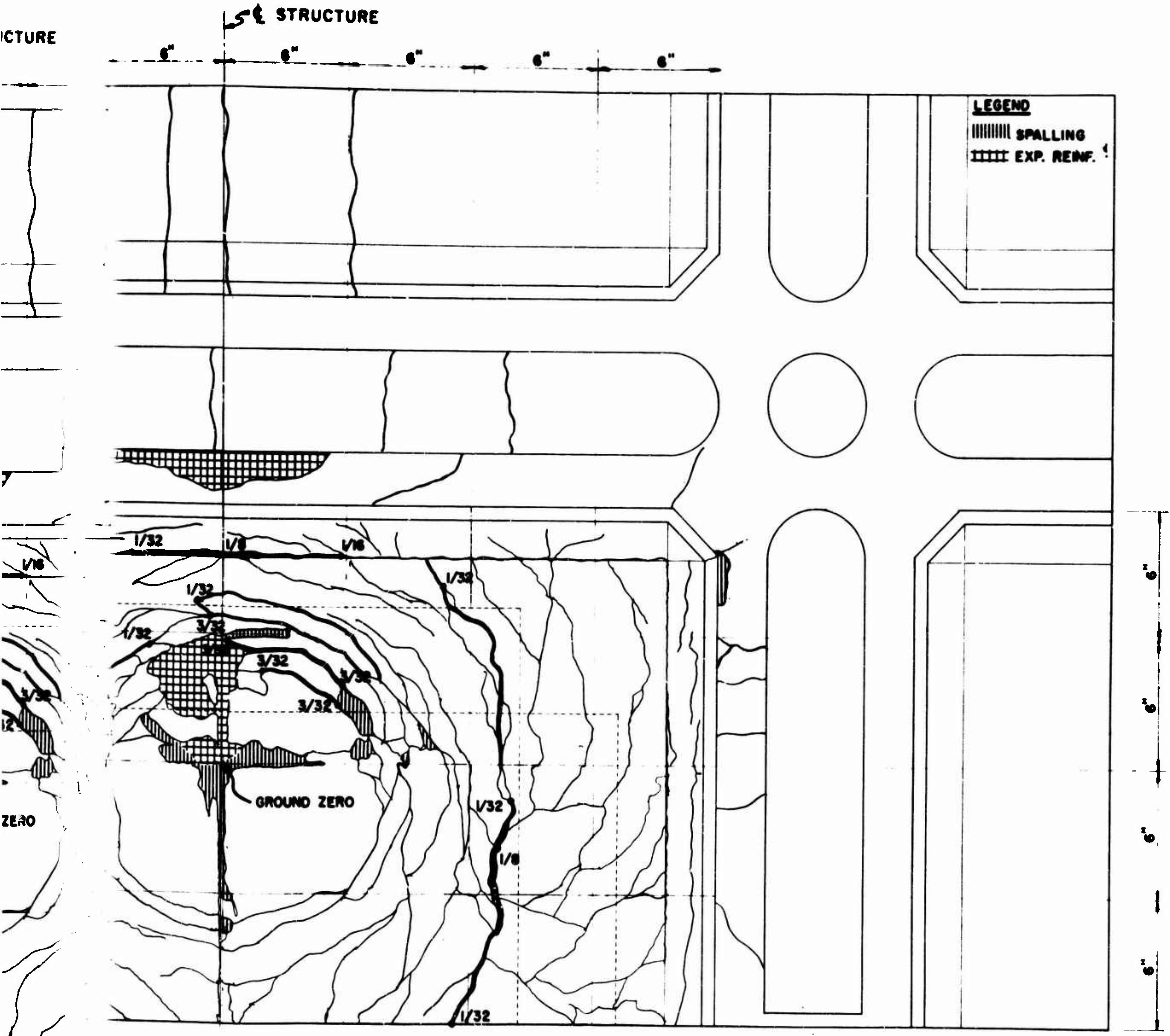
FIGURE 17
DAMAGE TO STRUCTURE - PLAN VI



NOTE:

NOTE: CRACK DIMENSIONS NOT SHOWN ARE LESS THAN 1/32"

FIGURE 17

VIE TURE - PLAN VIEW - ROUND 1

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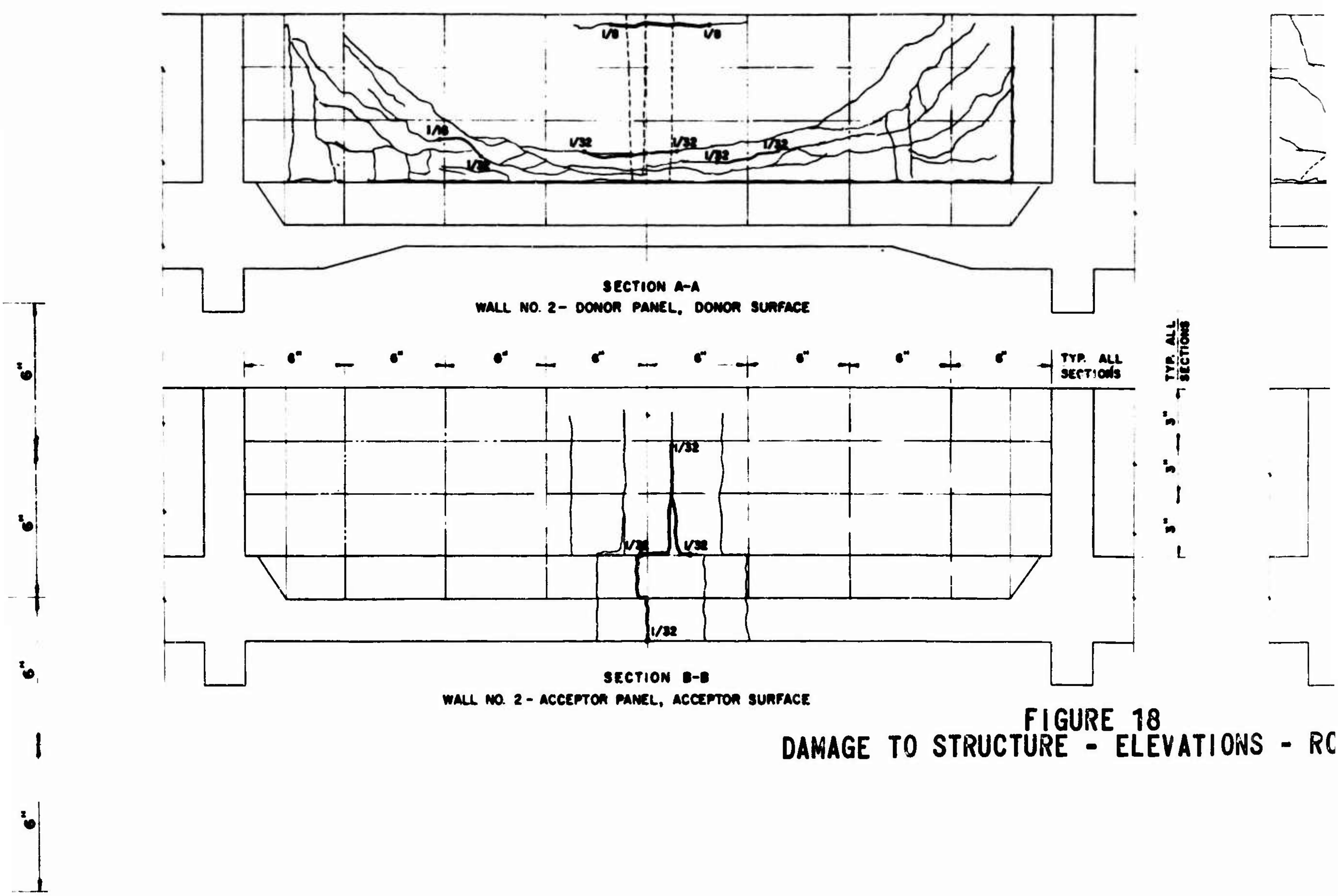

FIGURE 18
DAMAGE TO STRUCTURE - ELEVATIONS - RC

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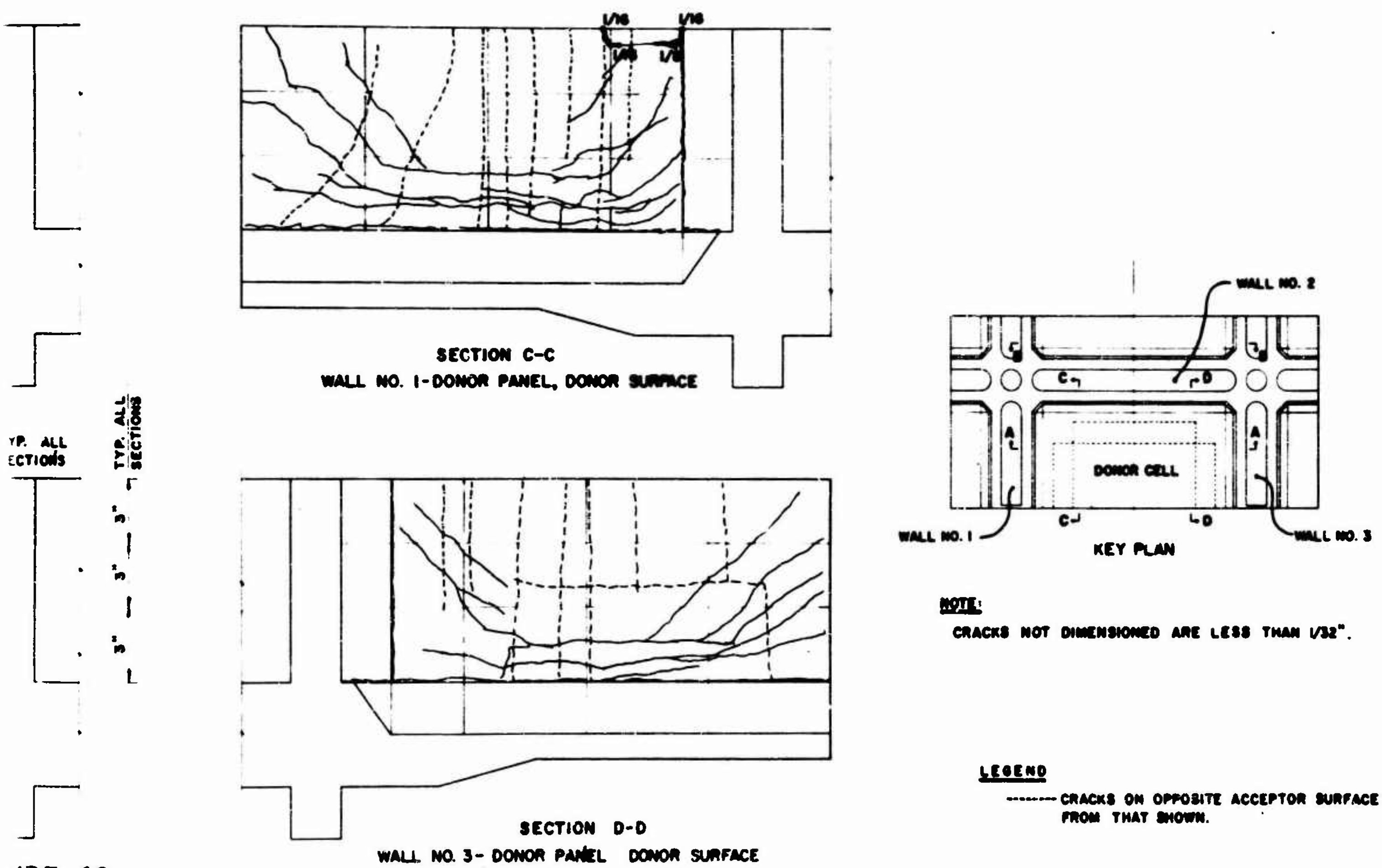

URE 18
- ELEVATIONS - ROUND 1

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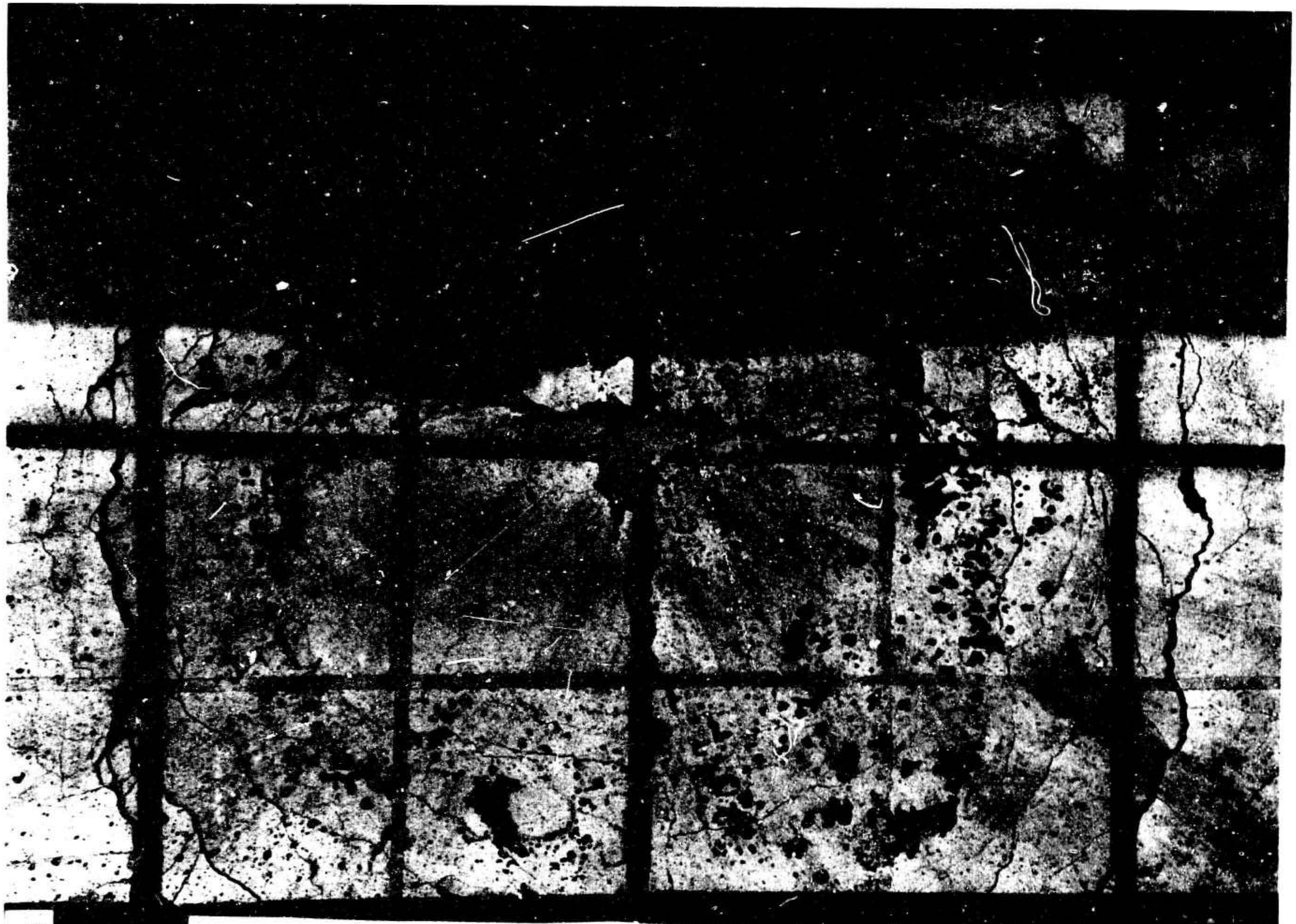

FIGURE 19
AREA OF MAIN DAMAGE TO FLOOR SLAB - ROUND 1

FIGURE 20
OVERALL DAMAGE TO FLOOR SLAB - ROUND 1

Wall 1 -- Slight damage was sustained by the donor panel of Wall 1 whereas there was no damage to the acceptor panel (Figure 21). On both surfaces of the donor panel, the major portion of the cracks were hairline in size. On the donor surface the general pattern of the cracks was circular and started in the upper corner at the inersection of Wall 1 with Wall 2; from there the cracks proceeded downward toward the center of the wall near the top of the haunch and then extended upward toward the top corner of the wall at the open end of the structure. The circular pattern indicated that a certain amount of support was afforded by the end panel which acted as a shear wall and as a compression strut in transferring a portion of the donor-panel load to the acceptor panel (Figure 22). Only minor cracking of the end panel was evident at its intersection with the donor panel.

The major damage to the donor panel of Wall 1 occurred at the top of the wall adjacent to its intersection with Wall 2 (Figure 18 and 22). In this instance, 1/16- and 1/8-inch-wide cracks were formed by the relatively large stresses developed at the inter-sectional area. In addition, continuous hairline cracks were created along the support ends of the donor surface of the donor panel -- at its juncture with Wall 2 and at the haunch (Figure 18). Straining of the donor surface reinforcement (tension reinforcement) was the cause of the hairline cracking.

On the acceptor surface of the donor panel there appeared a series of vertical hairline cracks extending from the top of the panel to its base in the vicinity of the interior third point of the panel. This cracking was indicative of positive straining of the horizontal reinforcement on this face of the panel.

Wall 2 -- As anticipated this wall received the most damage (Figure 23). Both the donor and the acceptor panels were damaged, the major portion being sustained by the donor panel.

The crack pattern of the donor surface of the donor panel was similar to that of the donor panel of Wall 1 (Figure 18 and 23). In this case the cracking commenced at each of the upper corners where the back wall intersects the two side walls and proceeded diagonally downward towards the center of the wall near the haunch. This pattern is typical of panels supported on three sides (two side walls and the floor slab) and free on the fourth (top of wall). The widths of the individual cracks varied from hairline near the upper

FIGURE 21
DONOR SURFACE OF WALL 1 - ROUND 1

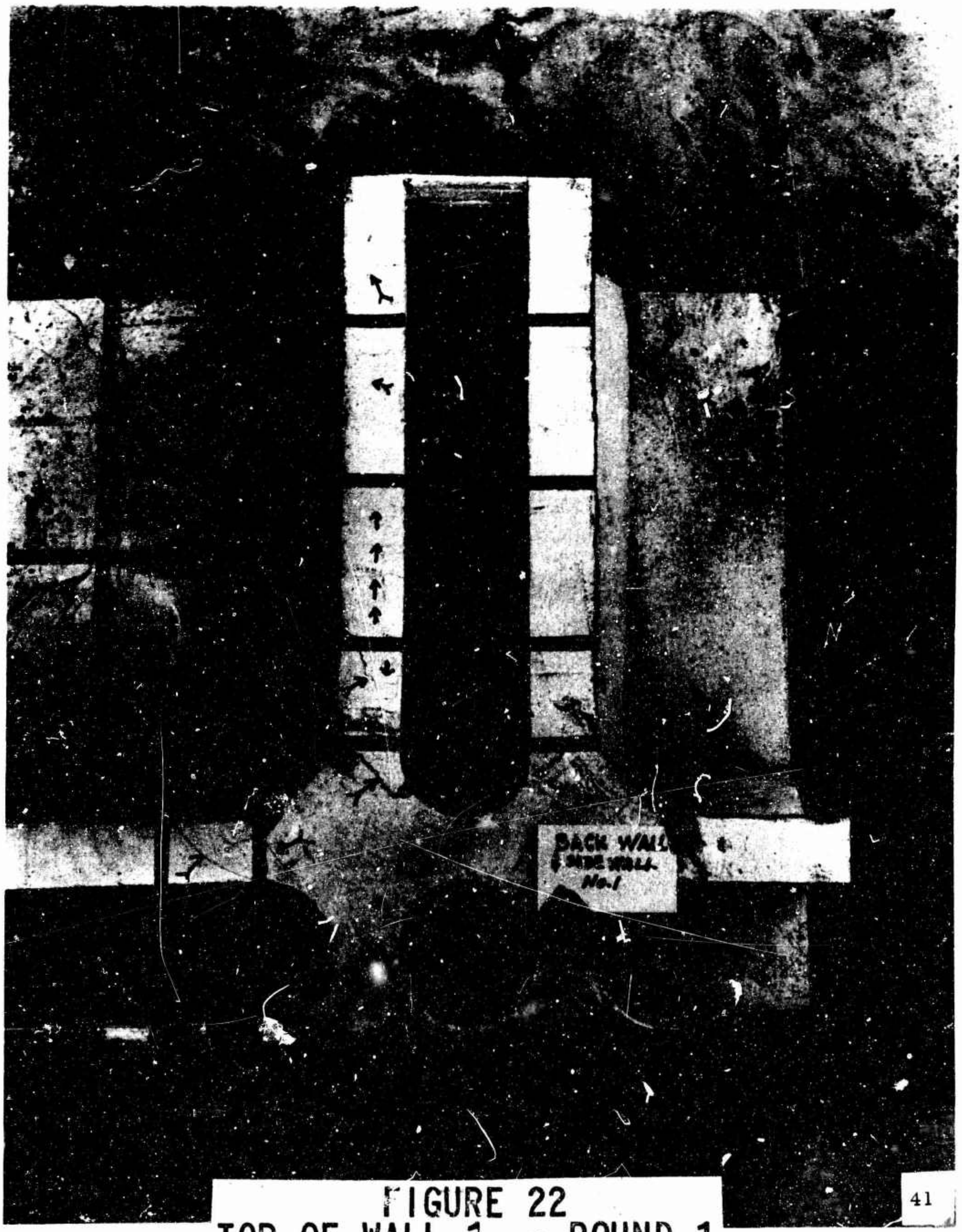

FIGURE 22
TOP OF WALL 1 - ROUND 1

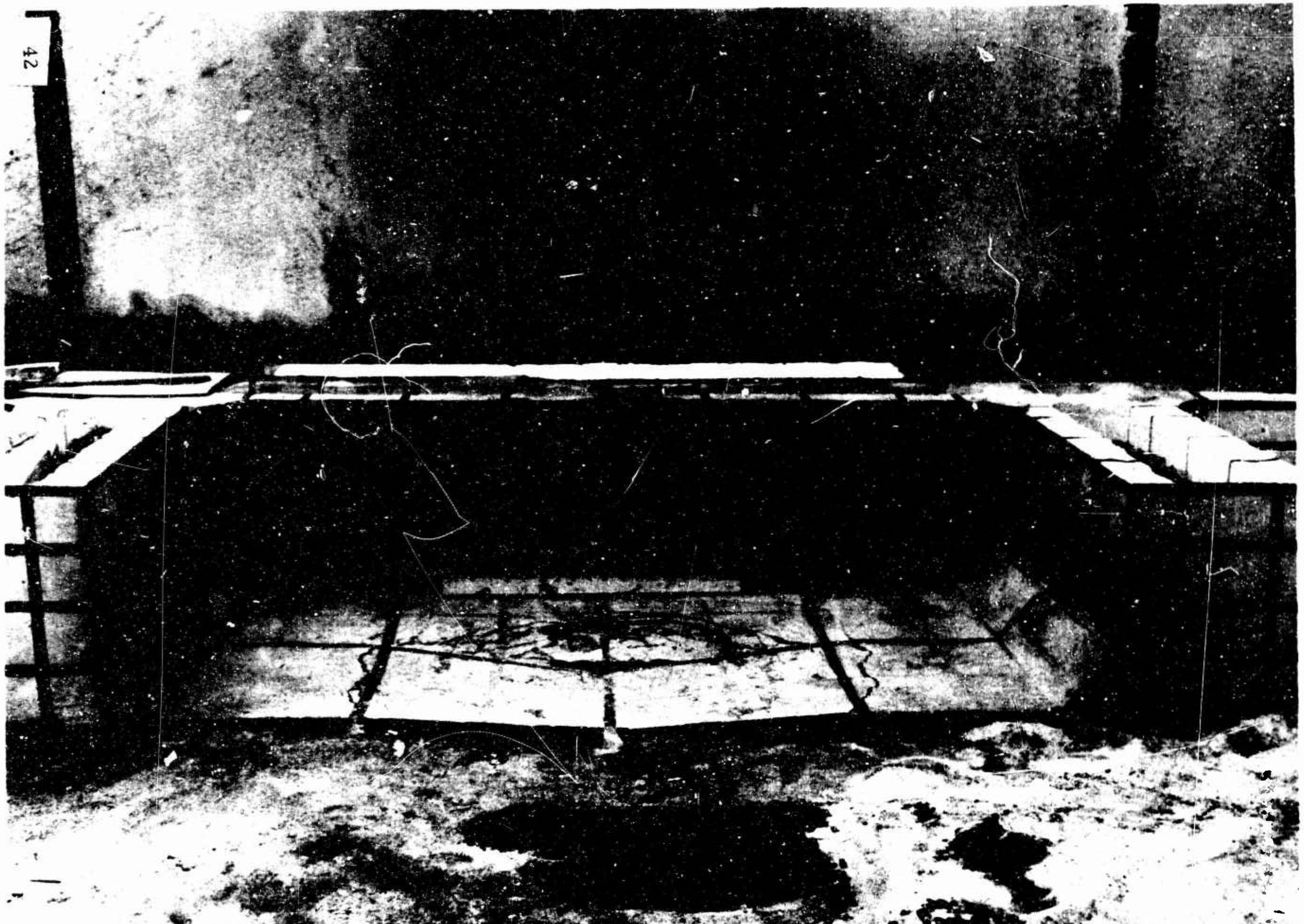

FIGURE 23
DAMAGE OF DONOR PANEL OF WALL 2 - ROUND 1

portion of the panel to 1/32-inch near the lower section of the wall, the larger cracks being definitely due to the cantilever action of the wall at that point. In addition, a 1/8-inch-wide horizontal crack, about one foot in length, was formed at the top central portion of the panel at the upper limit of the panel reinforcement and was the result of the spalling that occurred at the top corner of the acceptor side of the donor panel. As in the case of Wall 1, hairline cracks were evident along the perimeter of the panel supports also resulting from the straining of the tension reinforcement.

On the acceptor surface of the donor panel, vertical hairline cracks extended the full height of the panel at the mid-span of the wall and were the result of the straining of the horizontal reinforcement (positive reinforcement) at this face of the panel.

The main feature of the damage sustained by the acceptor panel of Wall 2 was a 1/32-inch-wide crack near the center of the wall. The crack extended vertically from about six inches below the top of the wall to both of the wall haunches and continued through the floor slab of the acceptor cell of the structure and also through the tie beam separating the donor and acceptor panels at the base of the wall (Figure 17, 18 and 24). In a like manner several auxiliary cracks appeared parallel and adjacent to the main crack and were of the hairline type.

It is theorized that the cracks in the back wall acceptor panel and those in the floor slab of the acceptor cell may have resulted from the horizontal bending action (of the floor slab and the tie beam at the base of the wall) produced by the load applied to the face of the wall proper. However, the settlement of the center of the wall (caused by the floor slab load) relative to the ends of the wall may have produced tension cracks in the lower section of the wall. At this time it is believed that the settlement of the wall was the more significant of the two factors.

It will be seen later that this damage sustained in the acceptor cell of the cubicle was an important factor in the collapse of Wall 2 in subsequent rounds.

Wall 3 -- Of the three walls under consideration this side wall incurred the least damage (Figure 25). As in the case of Wall 1, only the donor panel of this wall was damaged with the acceptor panel remaining intact. Little damage was sustained by the end panel.

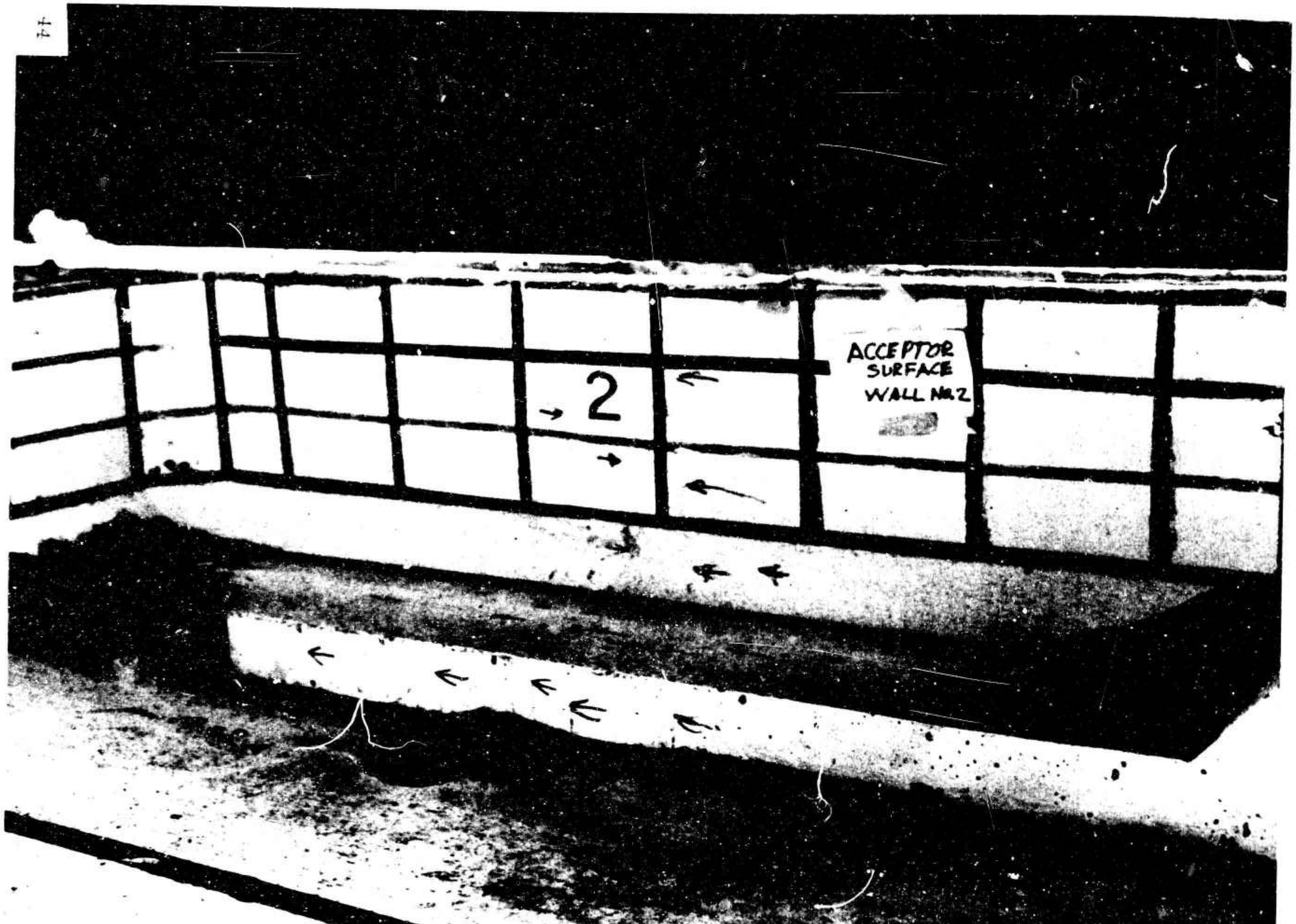

FIGURE 24
DAMAGE TO THE ACCEPTOR PANEL OF WALL 2 - ROUND 1

FIGURE 25
DAMAGE TO DONOR PANEL OF WALL 3 - ROUND 3

The crack pattern of the donor surface of the donor panel was similar to those of Wall 1 and 2 -- circular in nature and extending from the upper corners of the wall to the lower central section (near the haunch) of the panel. All cracks were of the hairline variety. As in the case of the other side wall the effective restraint of the end panel of the wall was evident from the conformation of the crack pattern (Figure 26). Also fissured were the sections of panel adjacent to the panel supports.

Like that of Wall 1, the acceptor surface of the donor panel of Wall 3 displayed vertical hairline cracks extending the full height of the wall; but unlike Wall 1, cracks were spread over a large area of the wall (Figure 18).

Round 2 (W= 3.24 lbs.)-- In this round, the structure suffered greater damage (Figure 27). This included failure of the floor slab, deformation of the back wall donor panel, slight deflection of the side wall donor panels, and no fissuring whatsoever of the side wall acceptor panels.

Figure 28 and 29 are plan views and elevations of the damage sustained by the structure.

Rupture of reinforcement, fragmentation of concrete and complete collapse (Figure 30) were the chief types of damage sustained by the thin, central portion of the floor slab. Extensive cracking was evident in the thick sections of the slab adjacent to the side walls and also in the haunches of the walls (Figure 28).

On the side walls, damage was more marked on the donor panel of Wall 1 (Figure 31) than on the corresponding panel of Wall 3 (Figure 32), the bulk of the damage being light cracking and small spalled areas in the proximity of the back wall. Deflections of the donor panels were substantially similar in magnitude -- in the order of 1/4 inch. Although the end panels exhibited several scattered fissures, the acceptor panels displayed no damage whatsoever (Figure 31 and 32).

Damage to Wall 2 was very extensive in the donor panel (Figure 33) and relatively light in the acceptor panel (Figure 34). Reinforcement was bared over a considerable area of the donor surface of the donor panel and over more than 50% of the top surface of the same panel (Figure 33). Extensive cracking was visible throughout the

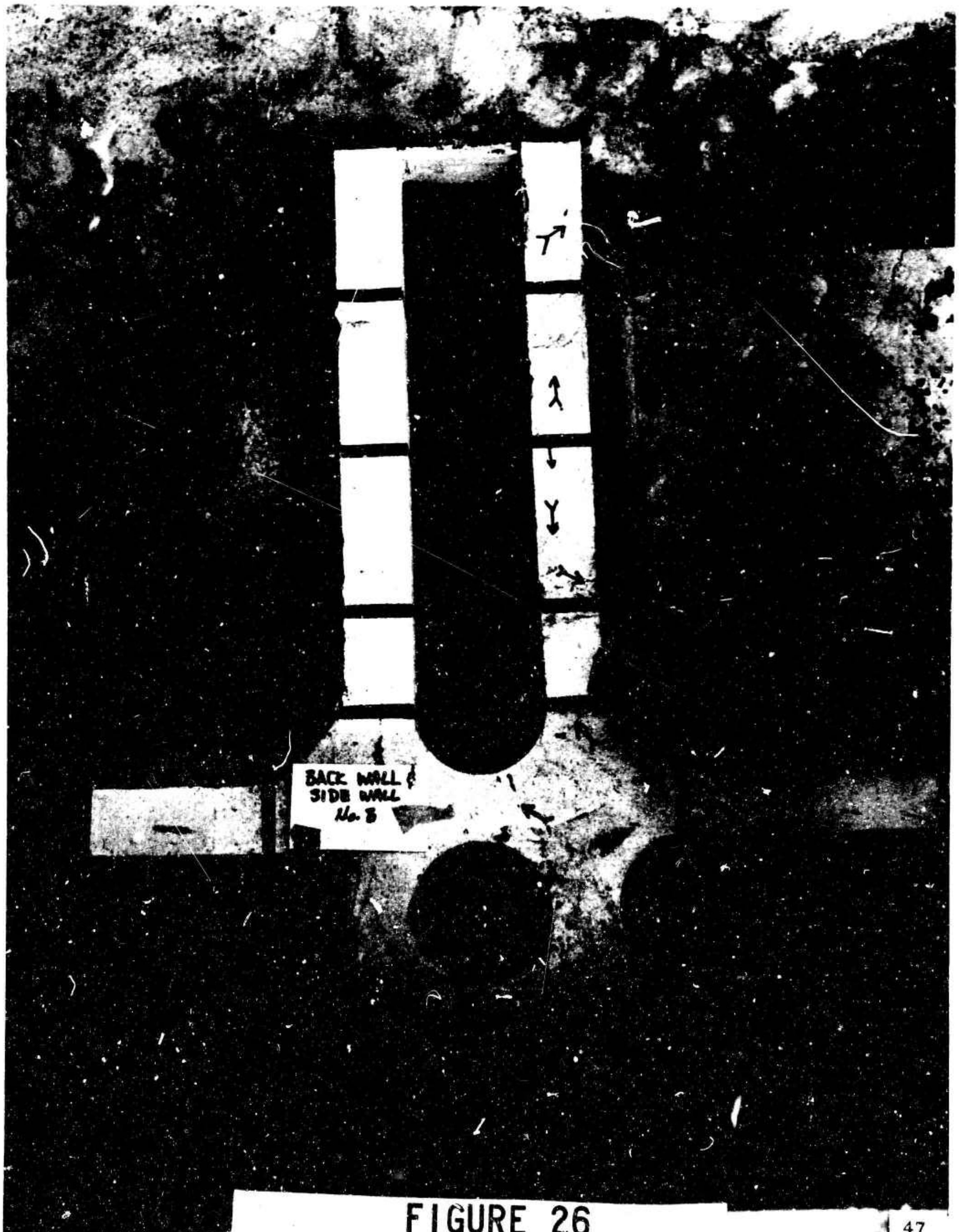

FIGURE 26
TOP OF WALL 3 - ROUND 1

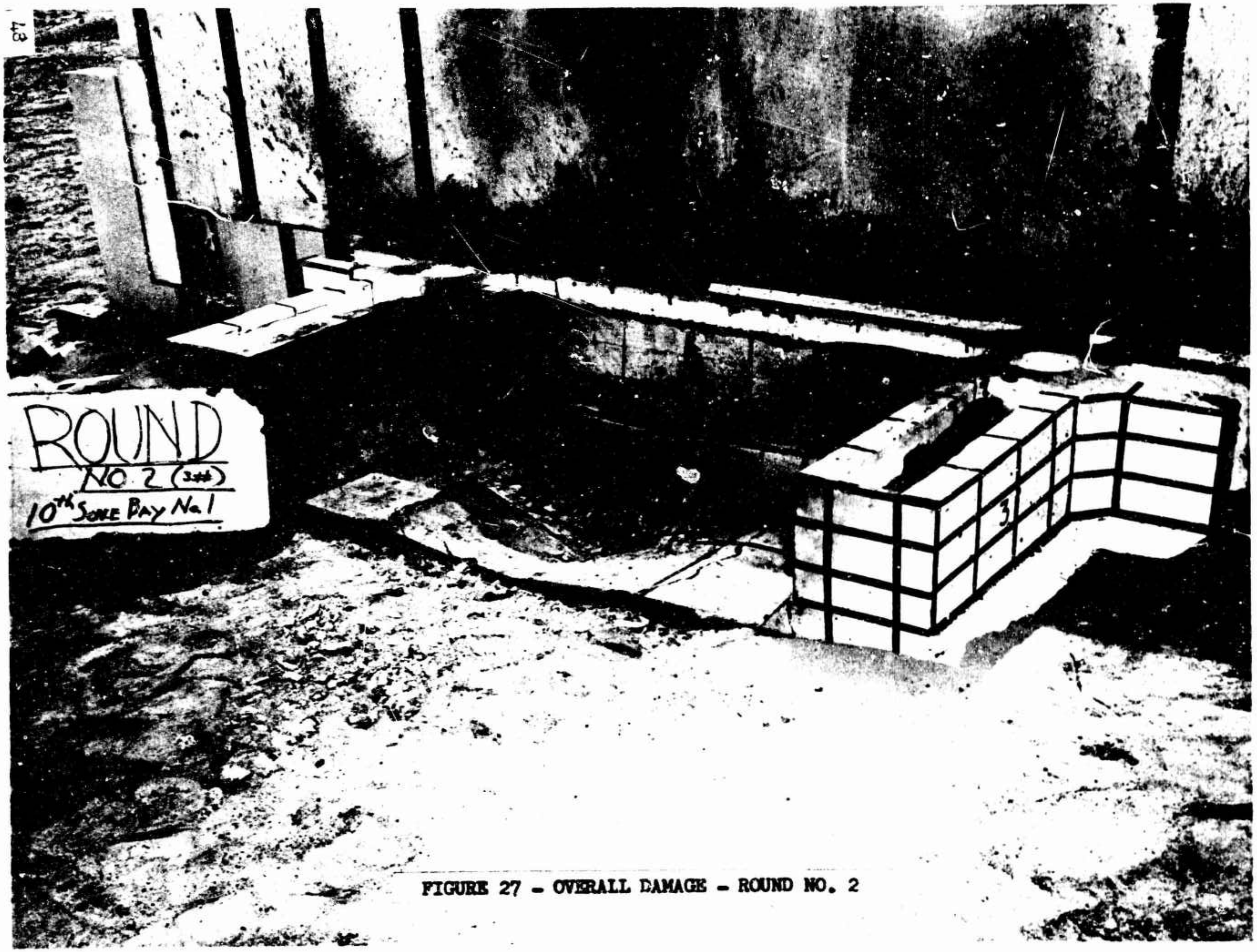

FIGURE 27 - OVERALL DAMAGE - ROUND NO. 2

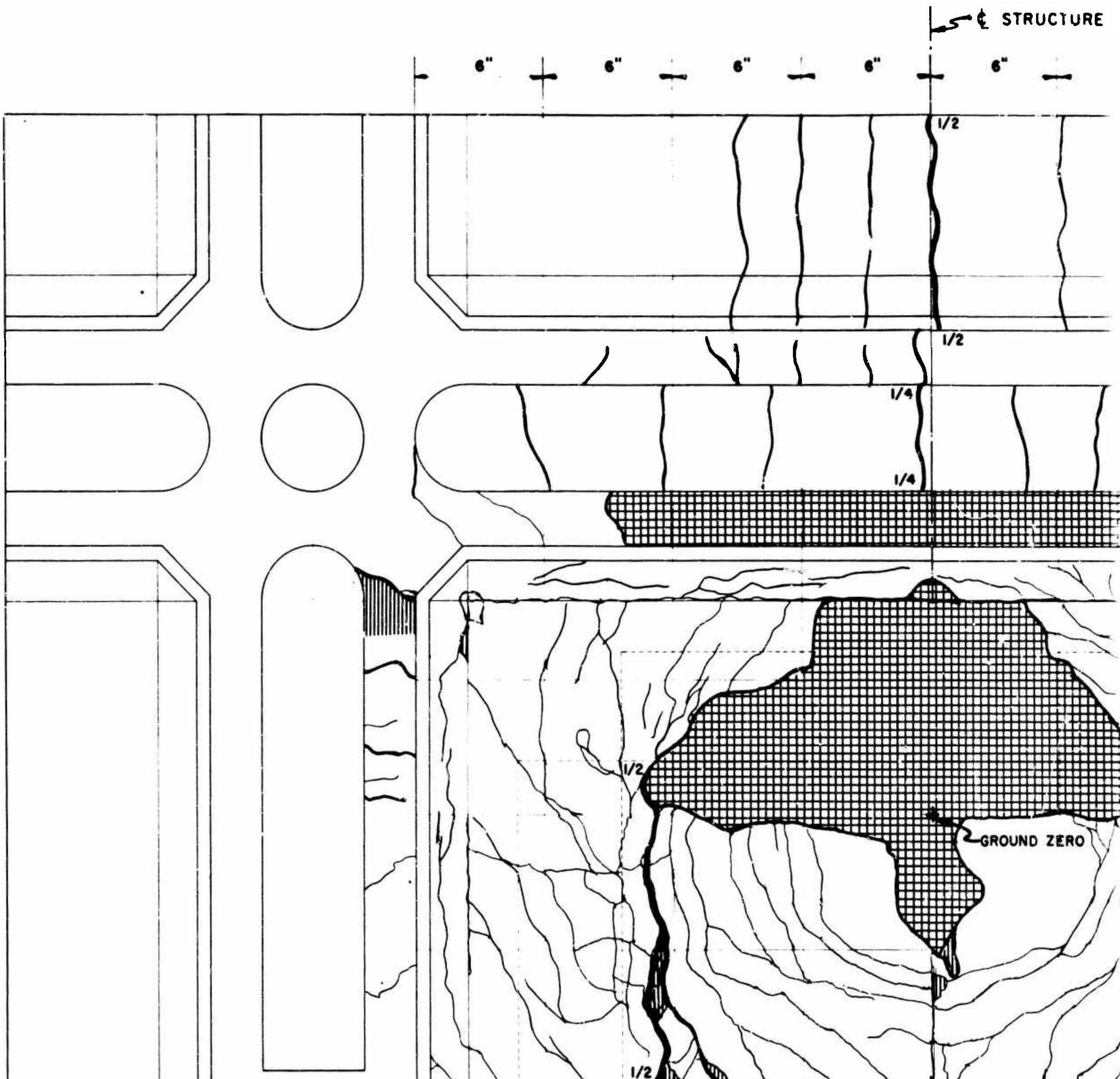

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FIGURE 28
DAMAGE TO STRUCTURE - PLAN VIE

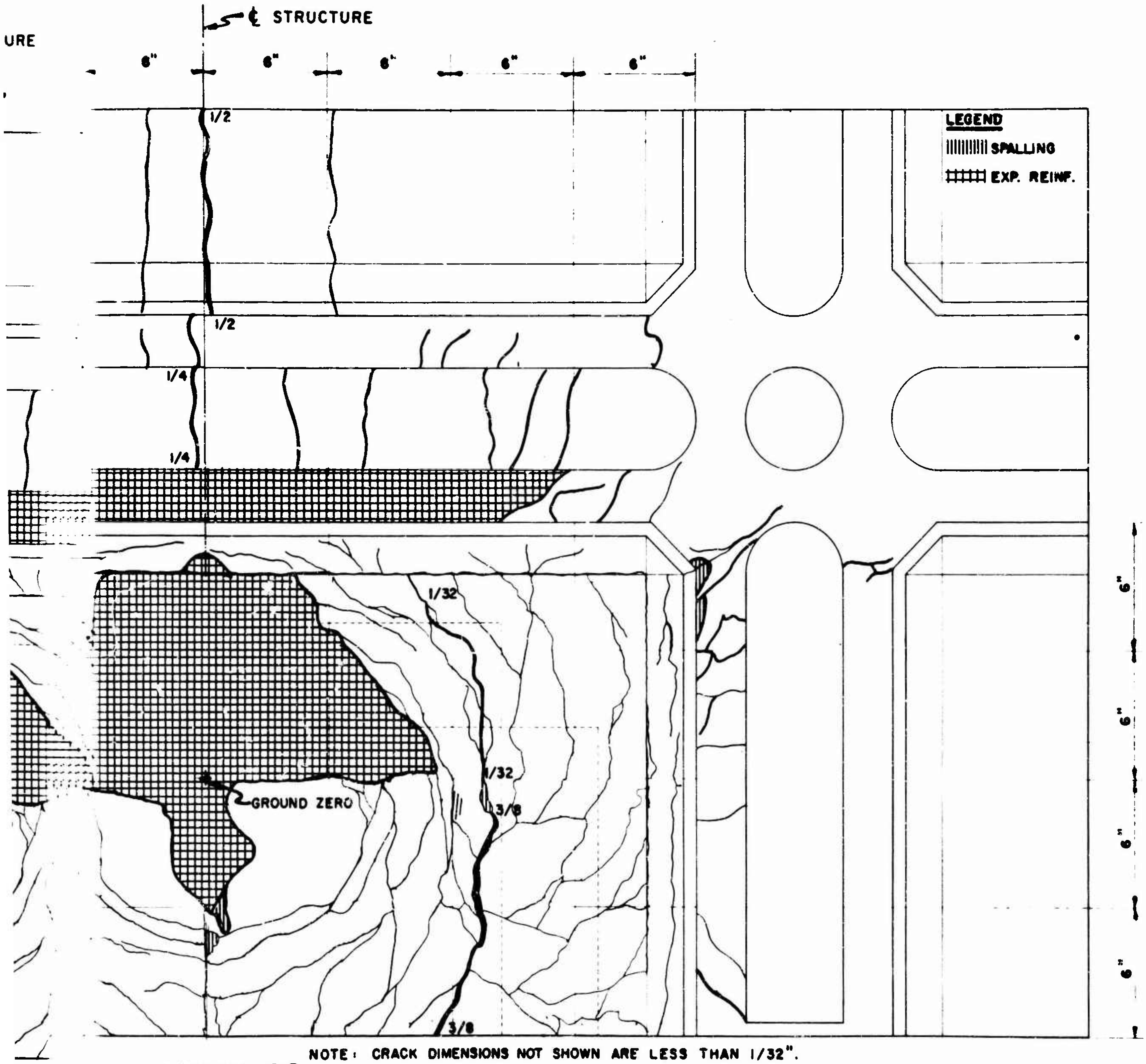


FIGURE 28

IE ...CTURE - PLAN VIEW - ROUND 2

2

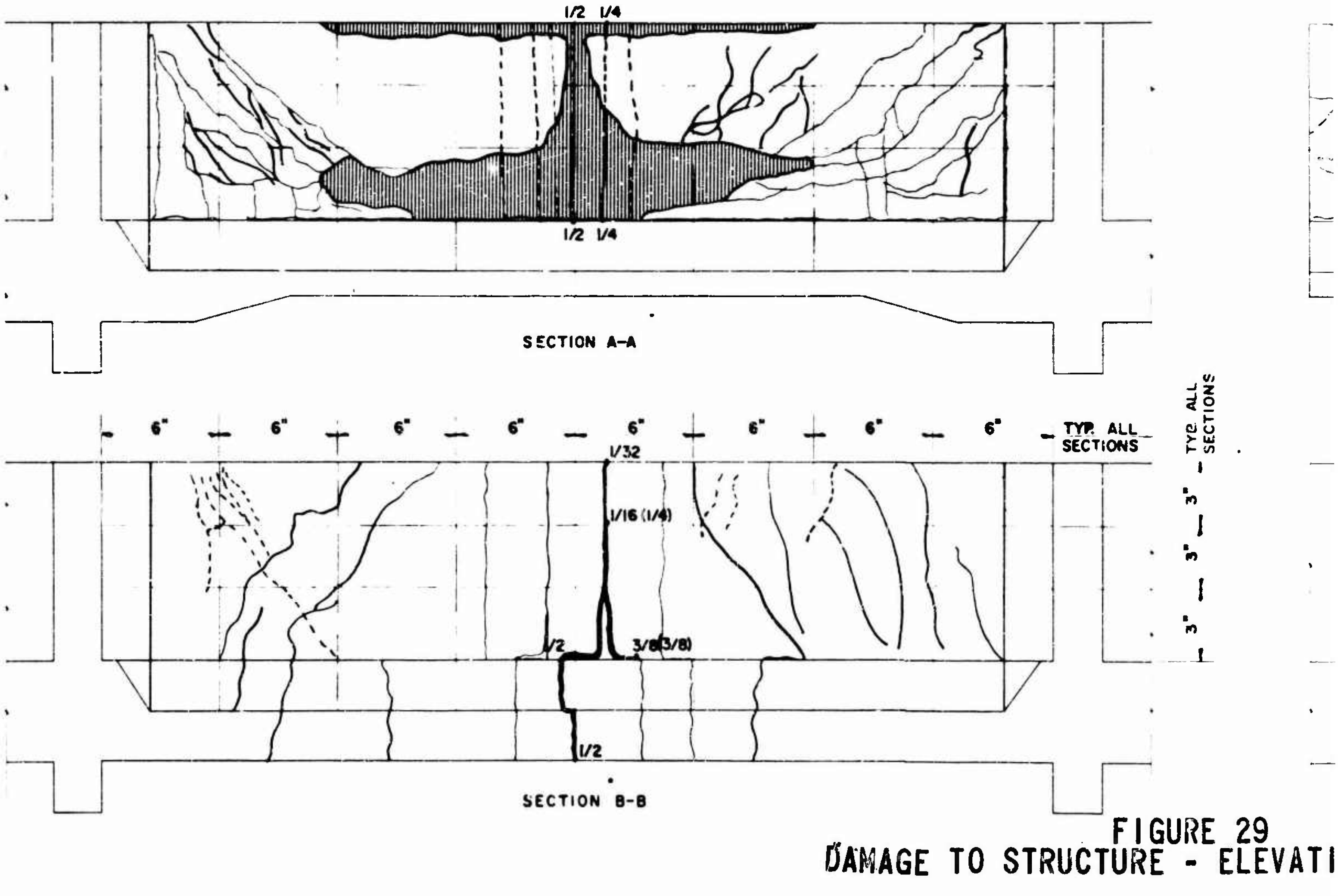


FIGURE 29
DAMAGE TO STRUCTURE - ELEVATI

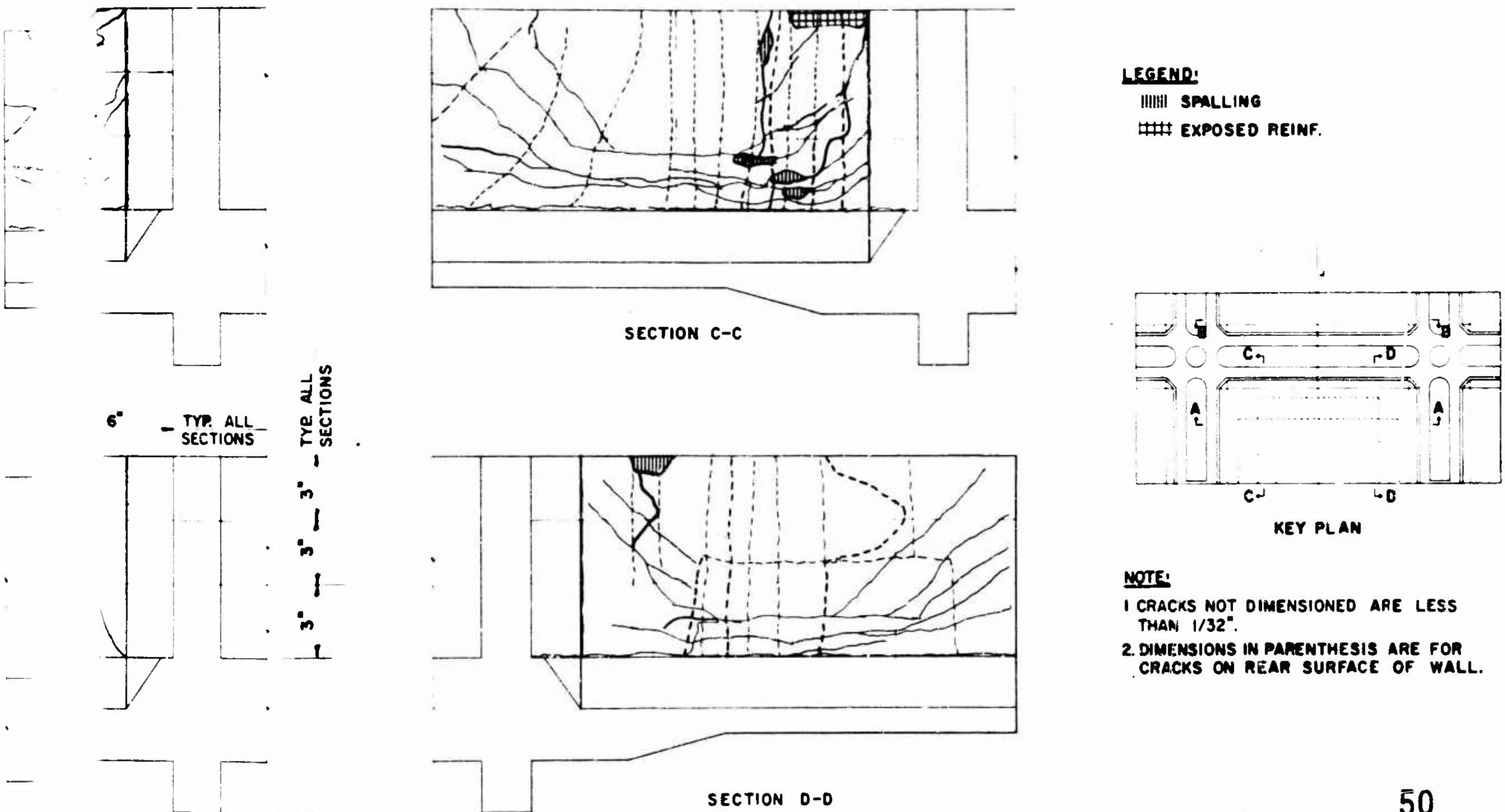


FIGURE 29
'ATI(·O STRUCTURE - ELEVATIONS - ROUND 2

2

FIGURE 30
DAMAGE TO FLOOR SLAB - CENTRAL PORTION - ROUND 2

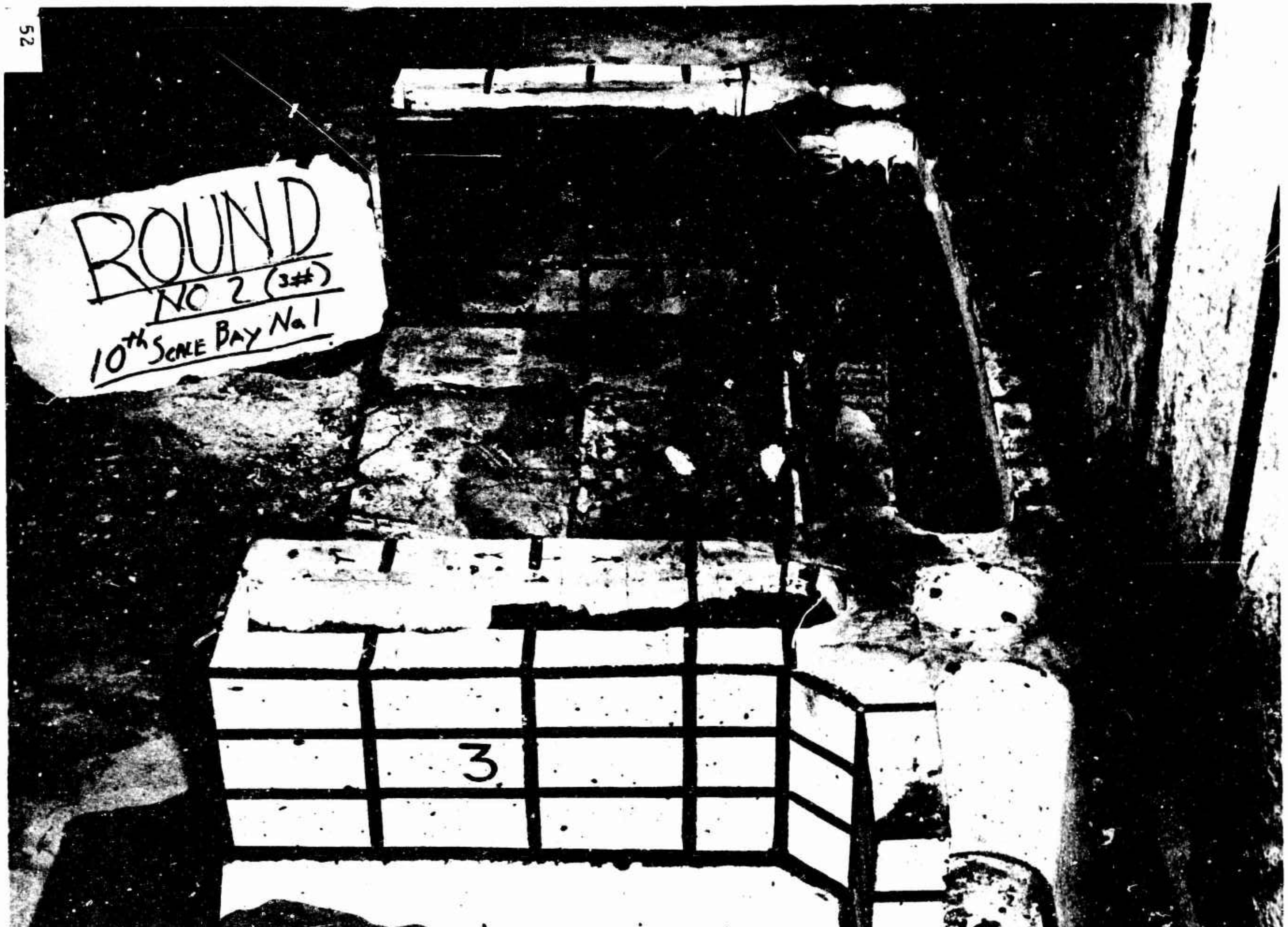


FIGURE 31
DAMAGE TO SIDE WALLS - ROUND 2

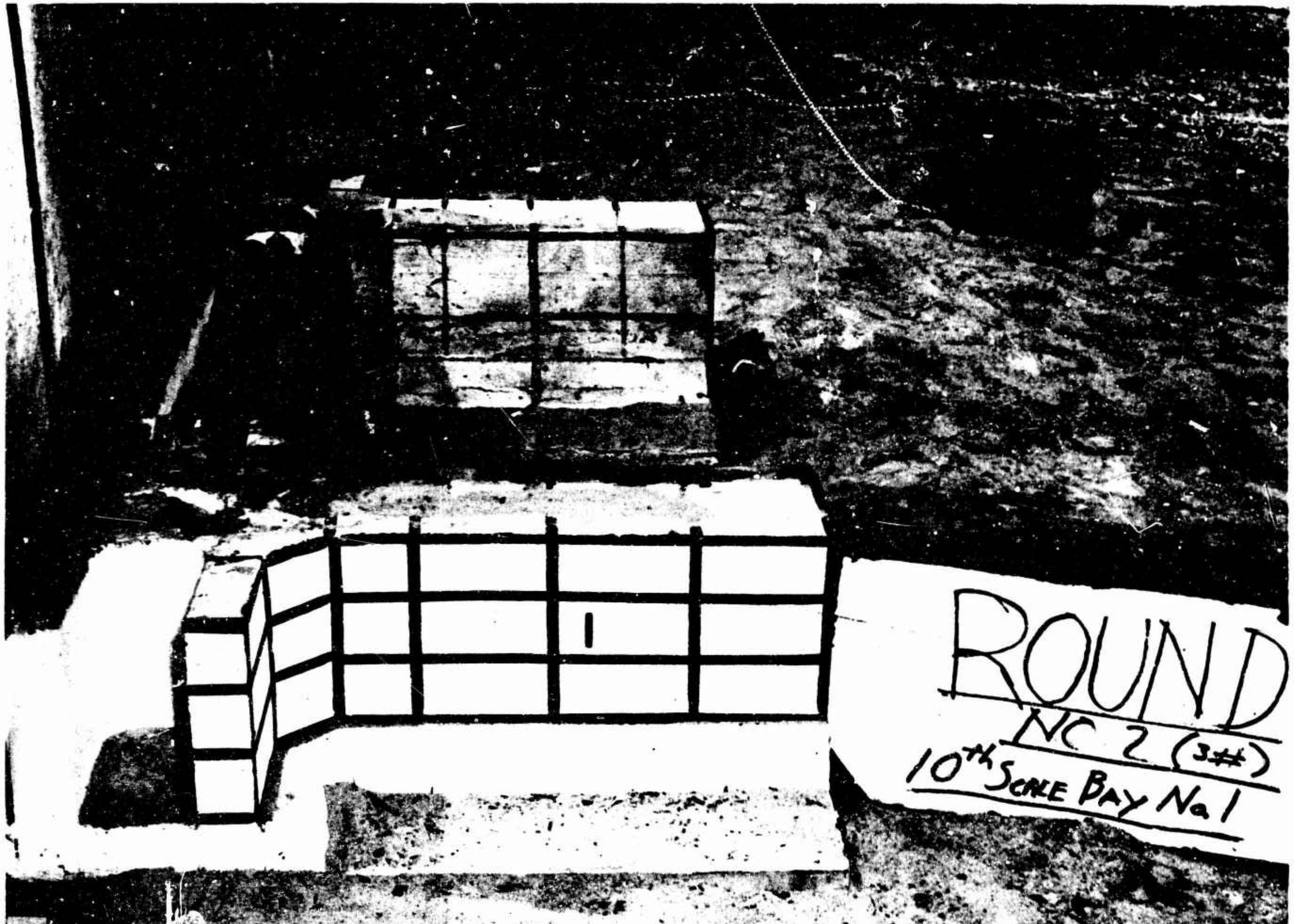


FIGURE 32
DAMAGE TO SIDE WALLS - ROUND 2

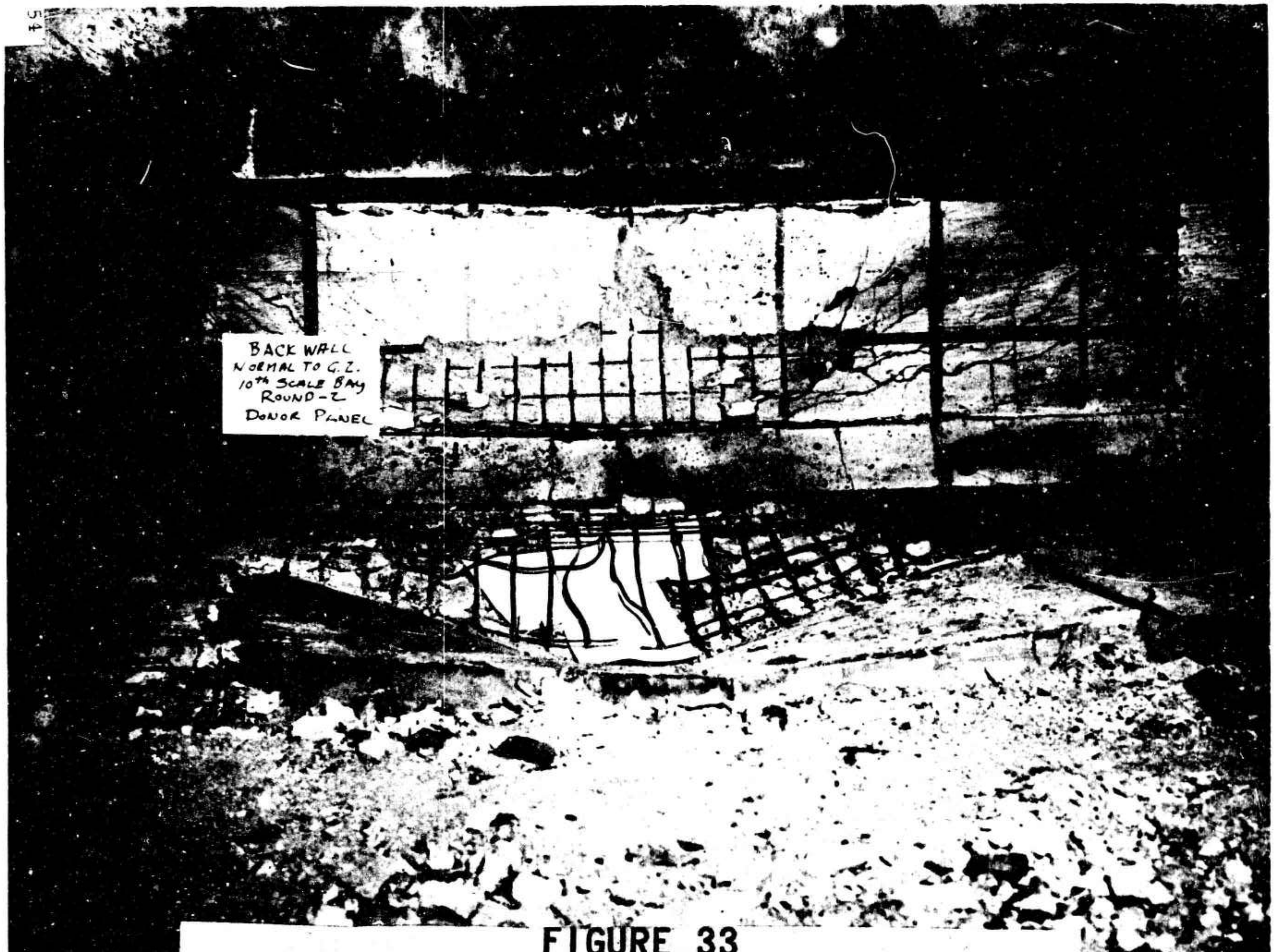


FIGURE 33
WALL 2 - DAMAGE TO DONOR PANEL - ROUND 2

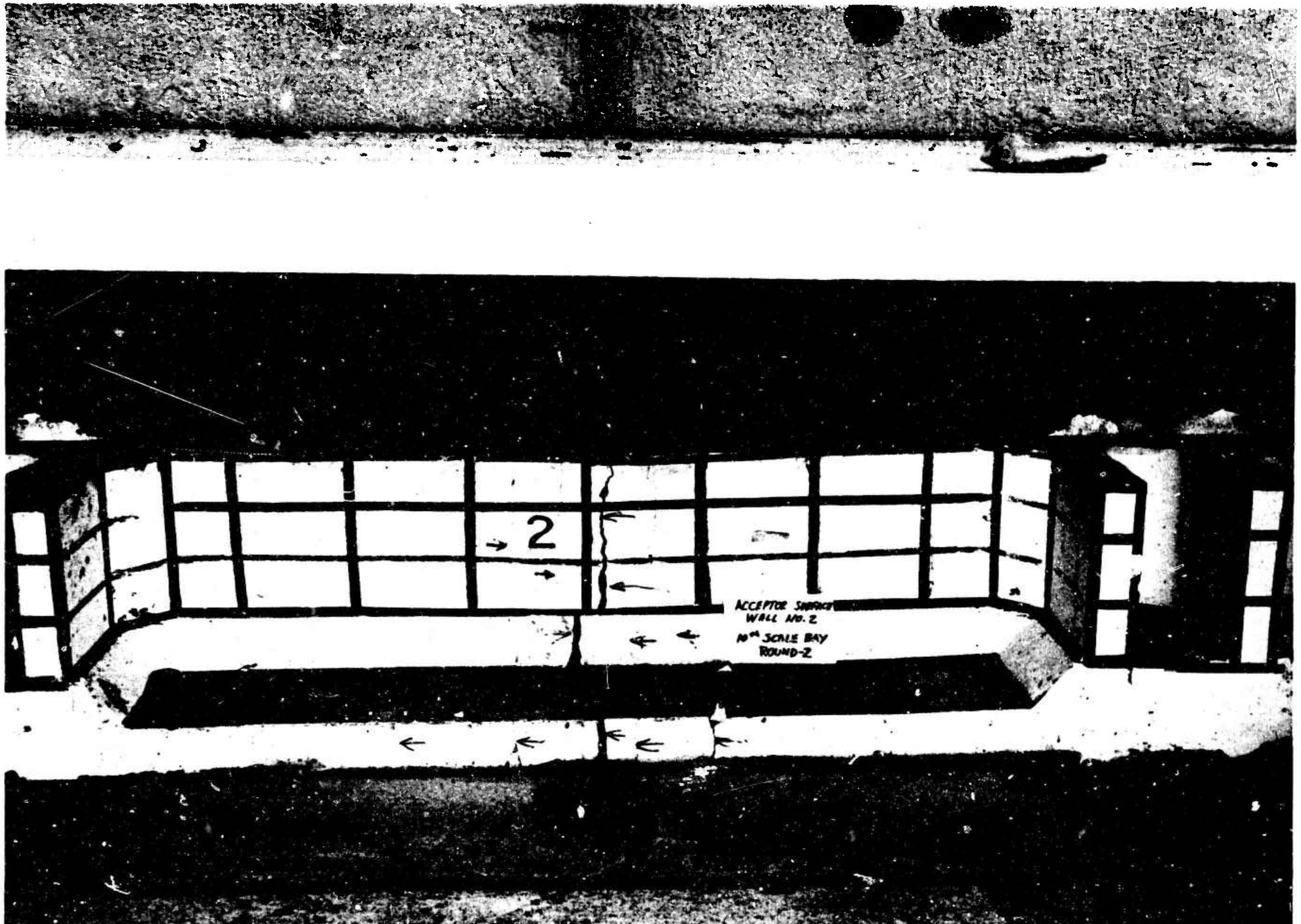


FIGURE 34
WALL 2 - DAMAGE TO ACCEPTOR PANEL - ROUND 2

donor panel but only one principal crack appeared on the acceptor panel; however, the latter crack which had a width of 1/2-inch not only traversed the haunch but also extended through the full thickness and length of the floor slab of the acceptor cell.

Deflection of the back wall was proportionately much greater than that of the side walls since the deformation of the acceptor panel exceeded even that of the donor panels of the side walls. A pronounced displacement of 1-5/16-inch was displayed by the back wall donor panel (Figure 35) but deflection of the acceptor panel was only 1/2 inch (Figure 36).

Round 3 (W = 4.24 lbs.) -- This round greatly amplified the relatively minor fissuring observed in the preceding rounds particularly in the back wall acceptor panel and in the end panel of Wall 3 (Figure 37). Crumbling of the floor slab occurred in this round as did severing of the back wall donor panel.

Further fragmentizing of concrete and fracturing of reinforcing occurred in the central portion of the floor slab and in the contiguous haunch to such an extent that a crater was created in the soil supporting the structure (Figure 38). On the other hand, except for some superficial spalling, little additional damage was sustained by the remainder of the slab..

Although deflection of side Wall 1 was greater than of side Wall 3, the damage suffered by Wall 3 was significantly more serious in that 50% of the end panel was demolished (Figure 37) and extensive spalling was evident at the top of the donor panel (Figure 39 and 40). Despite the fact that the donor panels of both side walls manifested considerable cracking the acceptor panels remained entirely free from any indication of degradation (Figure 39 and 40).

However, the opposite phenomenon prevailed in Wall 2 both in the donor panel (Figure 41) and in the acceptor panel (Figure 42) -- each of which was breached along the vertical center line. In this round, the deflection progressed more than five inches (Figure 41). At the top of the acceptor panel, the slight 1/2-inch deflection of the preceding round (Figure 36) was increased to 4-1/2 inches (Figure 43). Spalling and ruptured reinforcing were widespread in both panels.

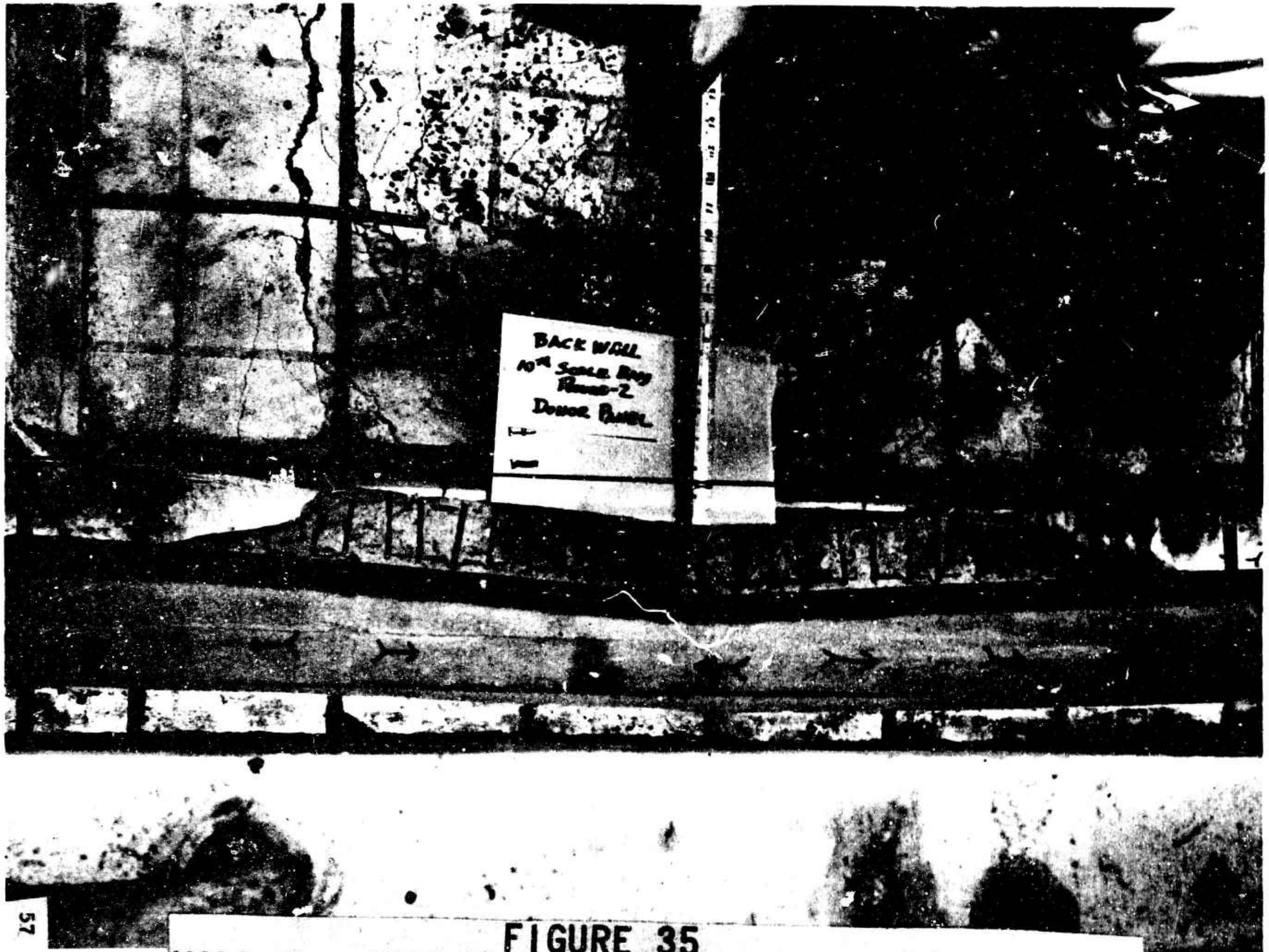


FIGURE 35
WALL 2 - DEFLECTION OF DONOR PANEL - ROUND 2

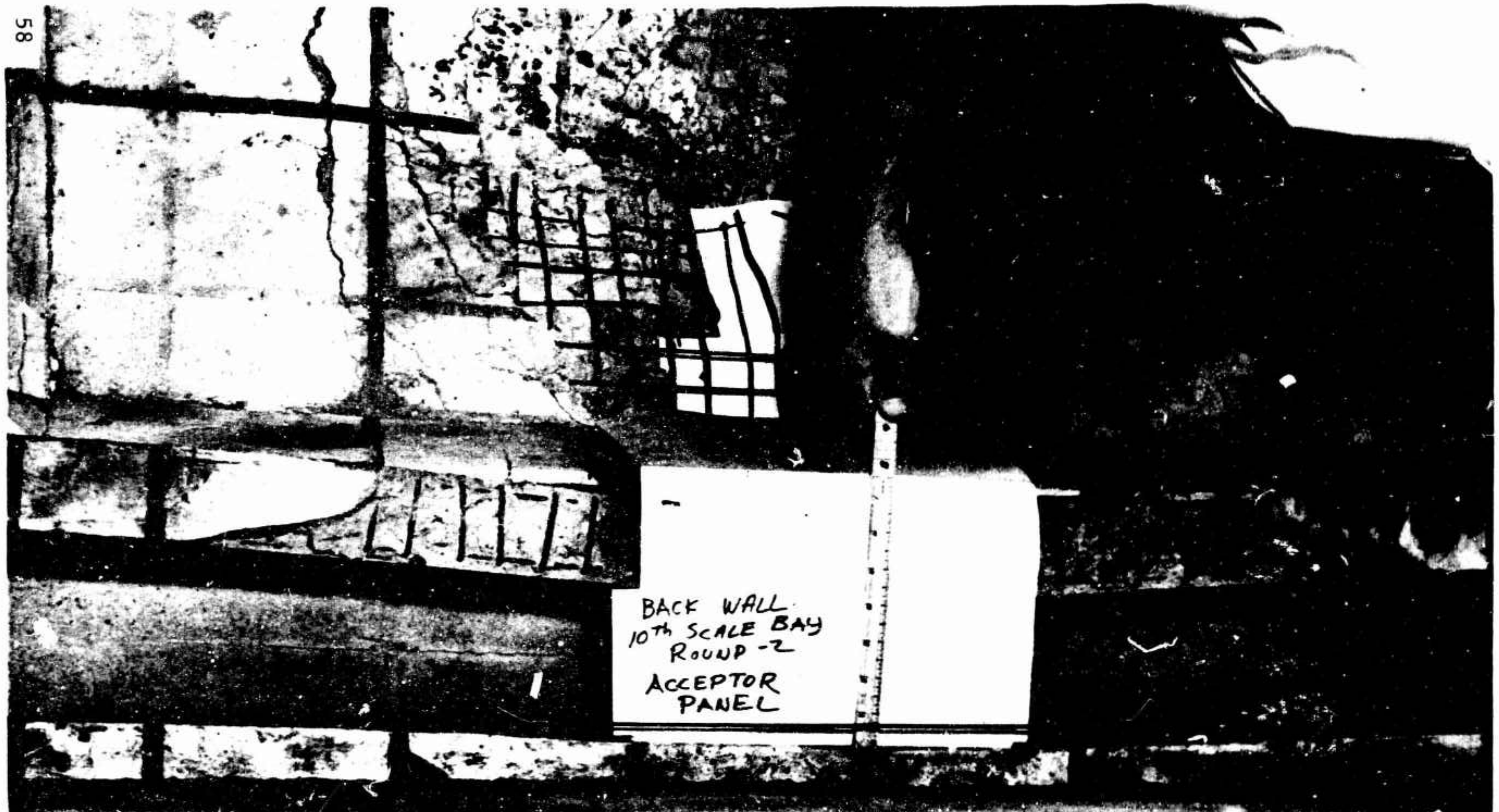


FIGURE 36
WALL 2 - DEFLECTION OF ACCEPTOR PANEL - ROUND 2

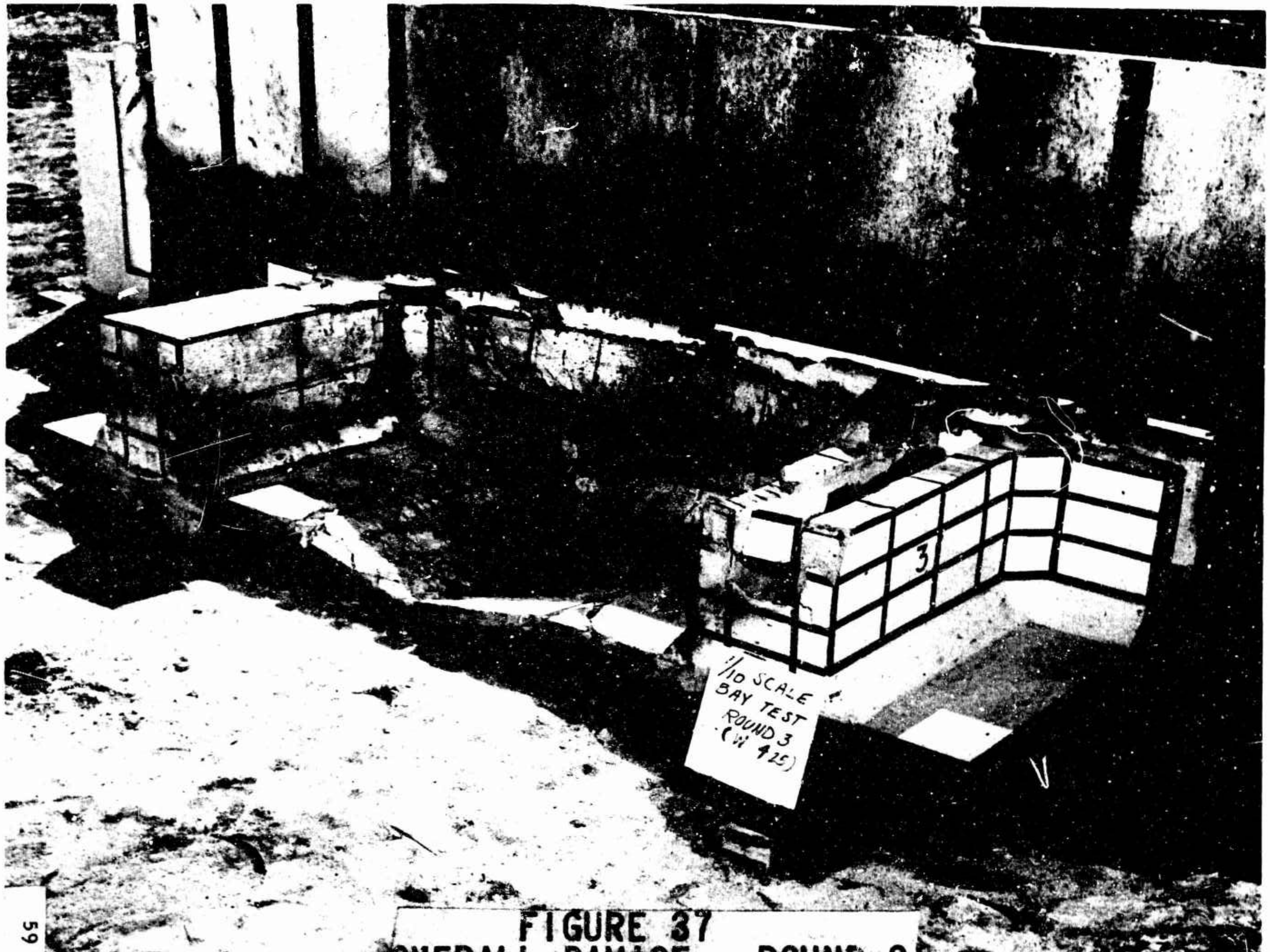


FIGURE 37
OVERALL DAMAGE - ROUND 3

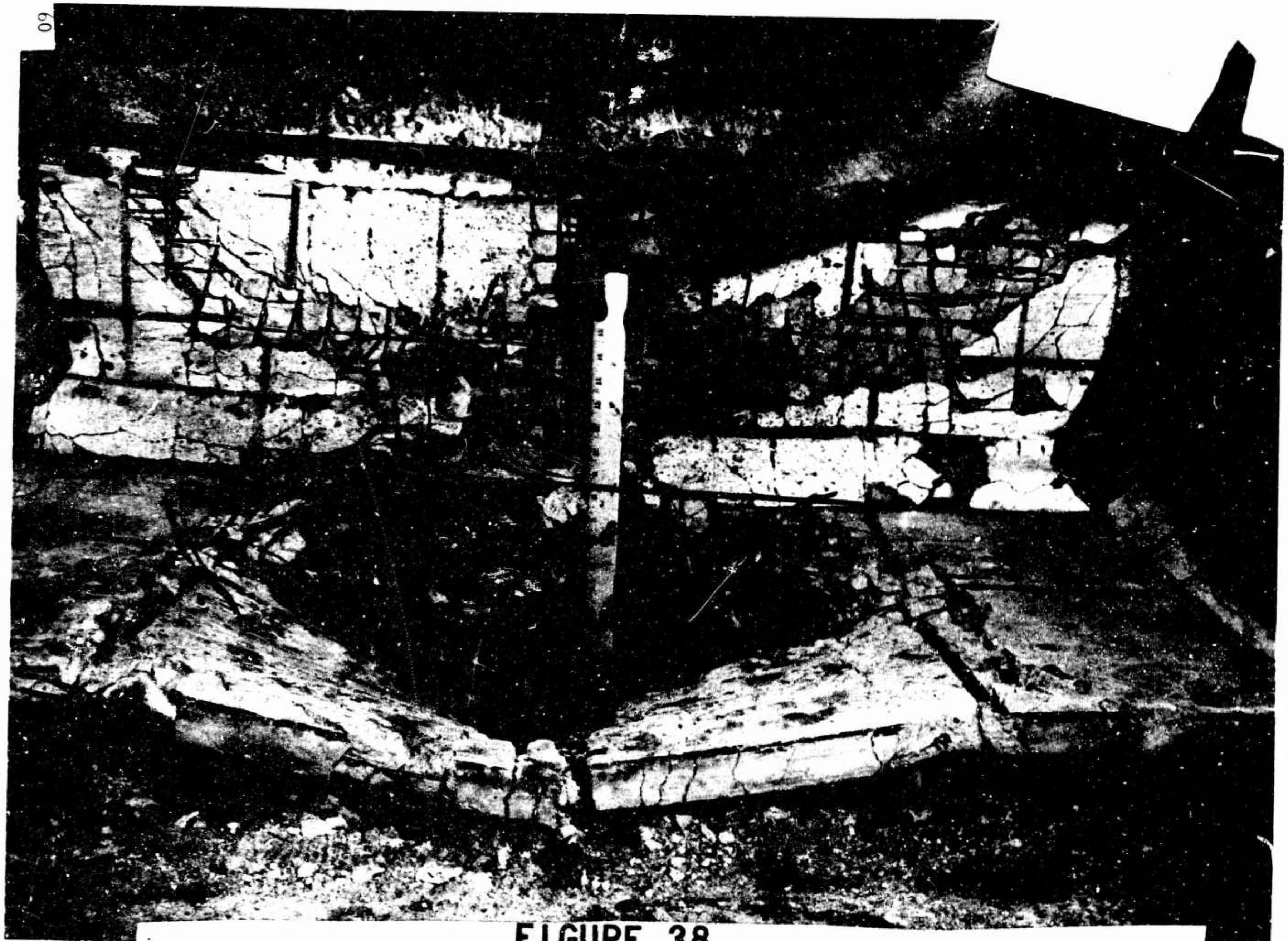


FIGURE 38
DAMAGE TO FLOOR SLAB - CENTRAL PORTION - ROUND 3

FIGURE 39
SIDE WALLS - ROUND 3

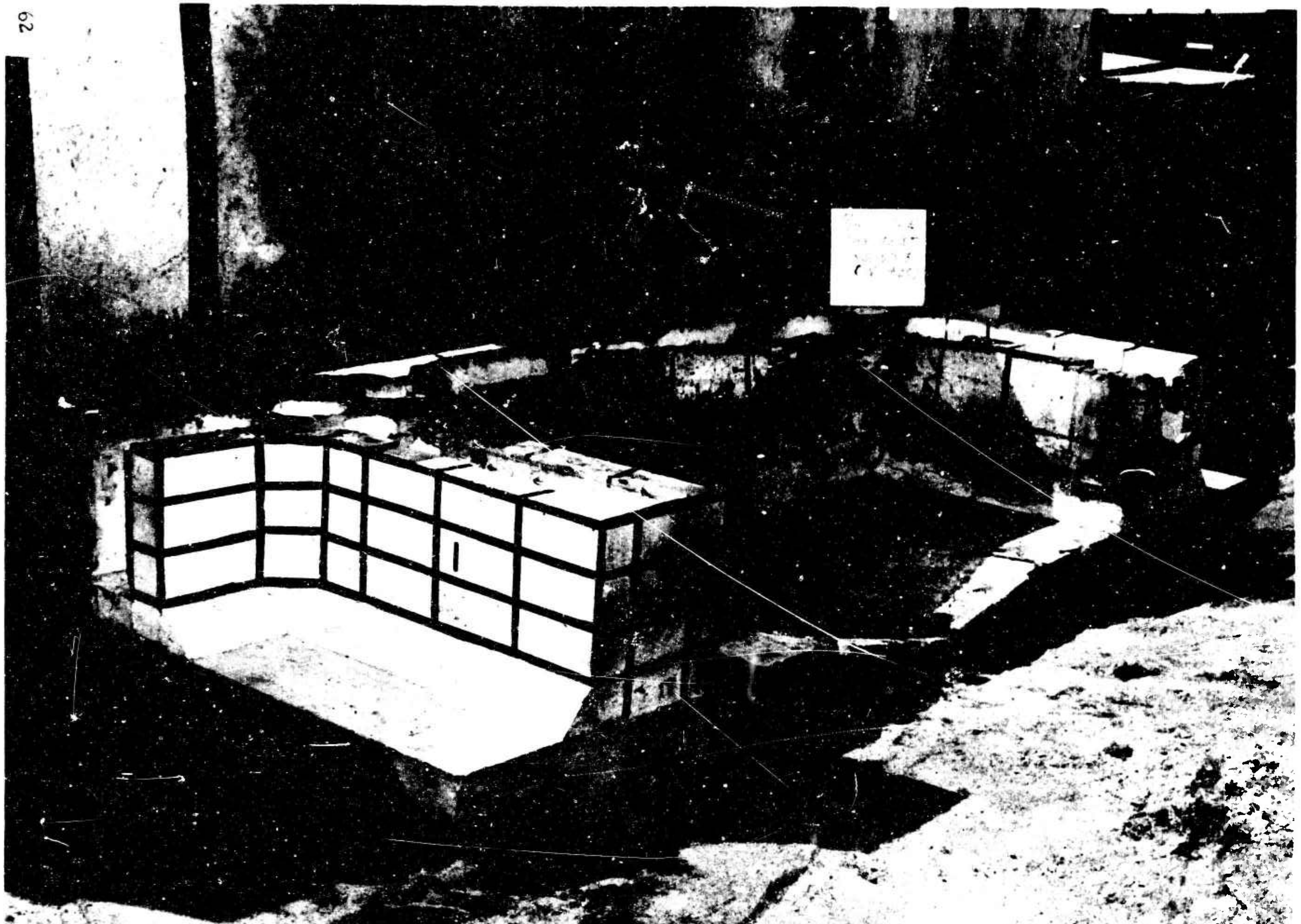


FIGURE 40
SIDE WALLS - ROUND 3

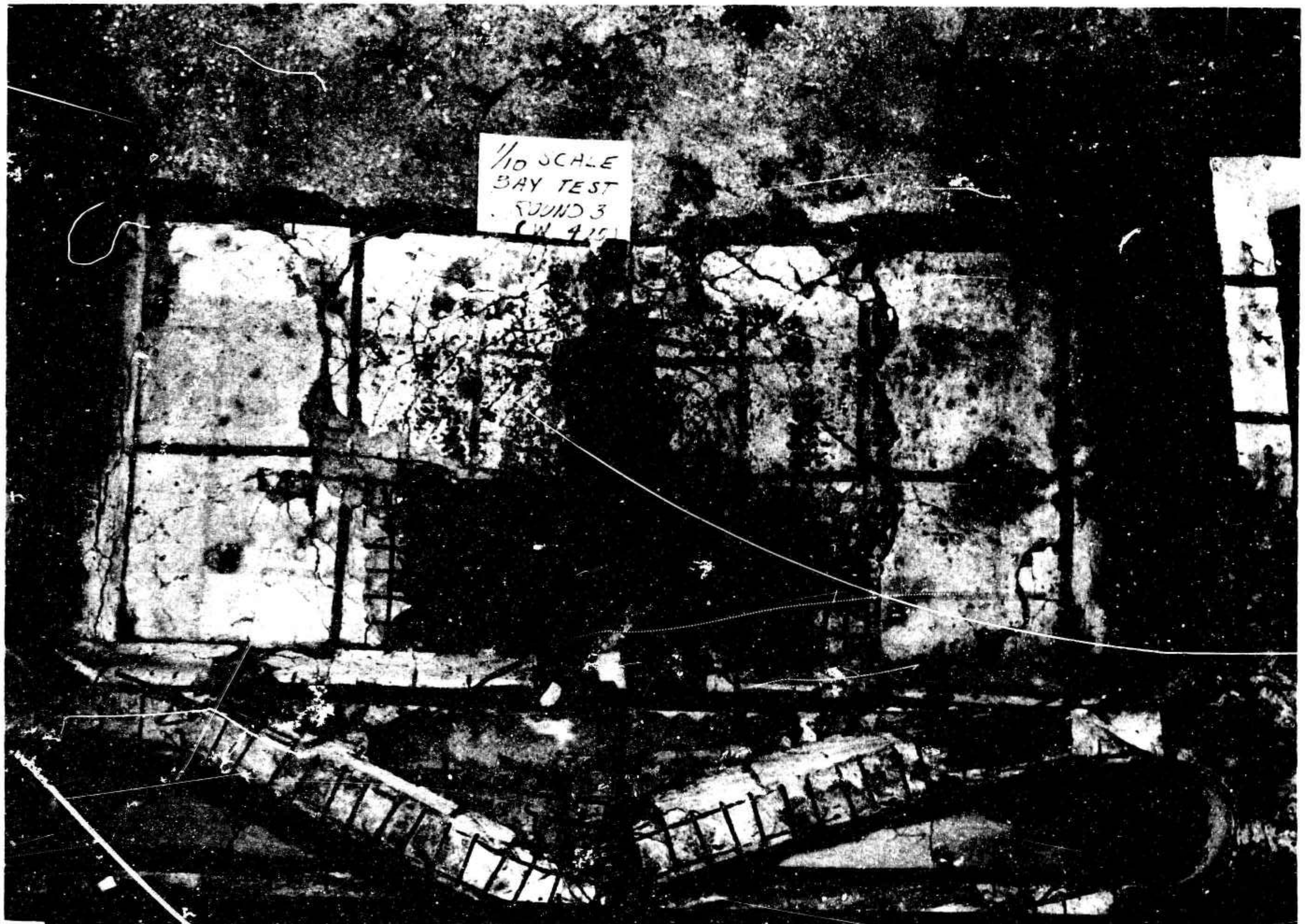


FIGURE 41
WALL 2 - DAMAGE TO DONOR PANEL - ROUND 3

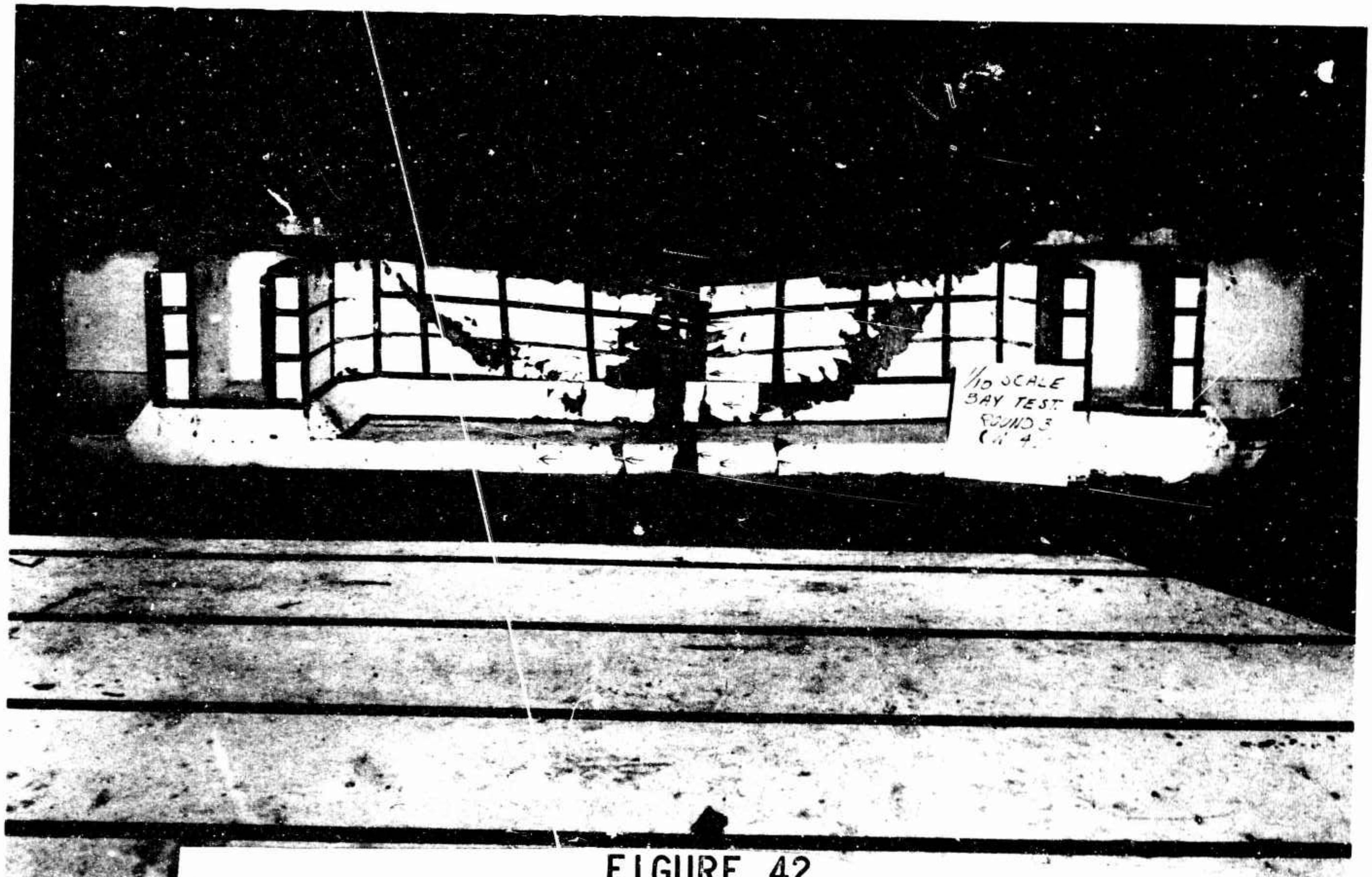


FIGURE 42
WALL 2 - DAMAGE TO ACCEPTOR PANEL - ROUND 3

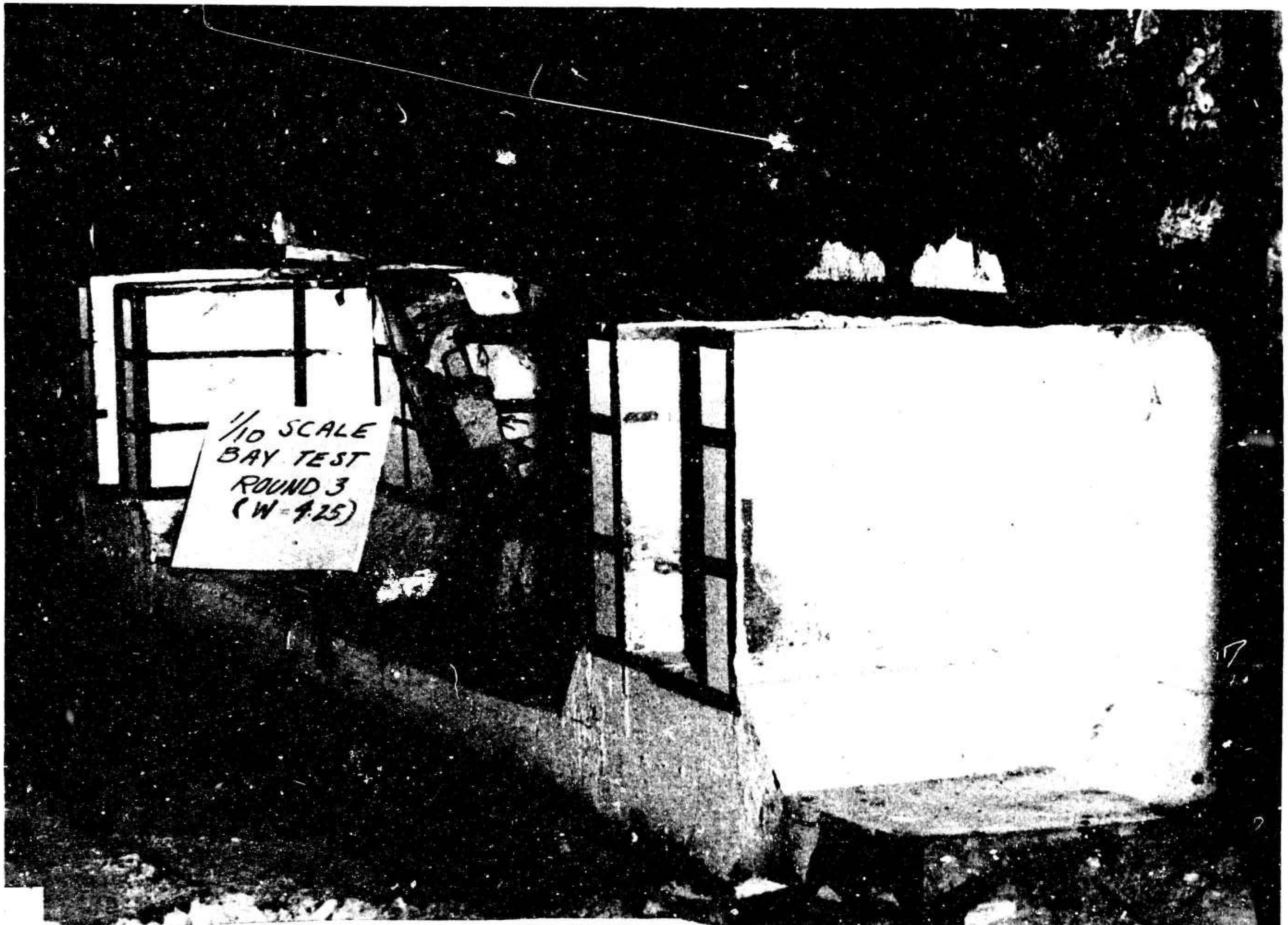


FIGURE 43
WALL 2 - DAMAGE TO ACCEPTOR PANEL - ROUND 3

Deflections of Walls and Floor

Floor slab deflections and back wall deflections were increasingly severe as testing progressed and were generally symmetrical about the transversal center line of the structure. Side wall deflections were relatively minor throughout the testing and were not symmetrical with respect to both the transverse and the longitudinal center lines of the structure.

Figure 44 is a plot of the permanent deflections recorded along the longitudinal center line of the floor slab. Plotted in Figure 45 are the permanent deflections measured along the top of the donor and acceptor panels of the back wall. Relative donor panel deflections of the side walls are plotted in Figure 46. Contours of all recorded deflections are indicated in the various views of Figures 47-49.

In Round 1, deflections were limited to the central floor slab and to the donor panel of the back wall and the donor panel of one side wall. Although deflection of the floor slab attained 1-7/8 inches, deflection of the walls was less than 1/4 inch.

In Round 2, deflection of the central floor slab increased twofold, and the peripheral floor slab exhibited slight deflection. Displacement of the back wall extended to the acceptor panel where a deflection of 1/2 inch was recorded. Side wall deflections were more widespread but nevertheless limited to 1/4 inch.

In Round 3, deflection of the central floor slab increased considerably whereas the peripheral floor slab remained substantially unchanged. Both the donor and the acceptor panels of the back wall were severly deflected. However, deflection of the side wall donor panels augmented only slightly and deflection of the acceptor panels remained imperceptible.

Fragment Velocities

Although fragmentation of the floor slab occurred at the outset of testing it was not until Round 3 that any wall surface was significantly fragmentized. Wall fragmentation occurred chiefly in the back wall between the vertical reinforcing bars situated at the section of failure. Table 2 indicates velocities attained by fragments propelled from the back wall during Round 3. Maximum fragment velocity was 81 fps and overall average velocity was 74 fps.

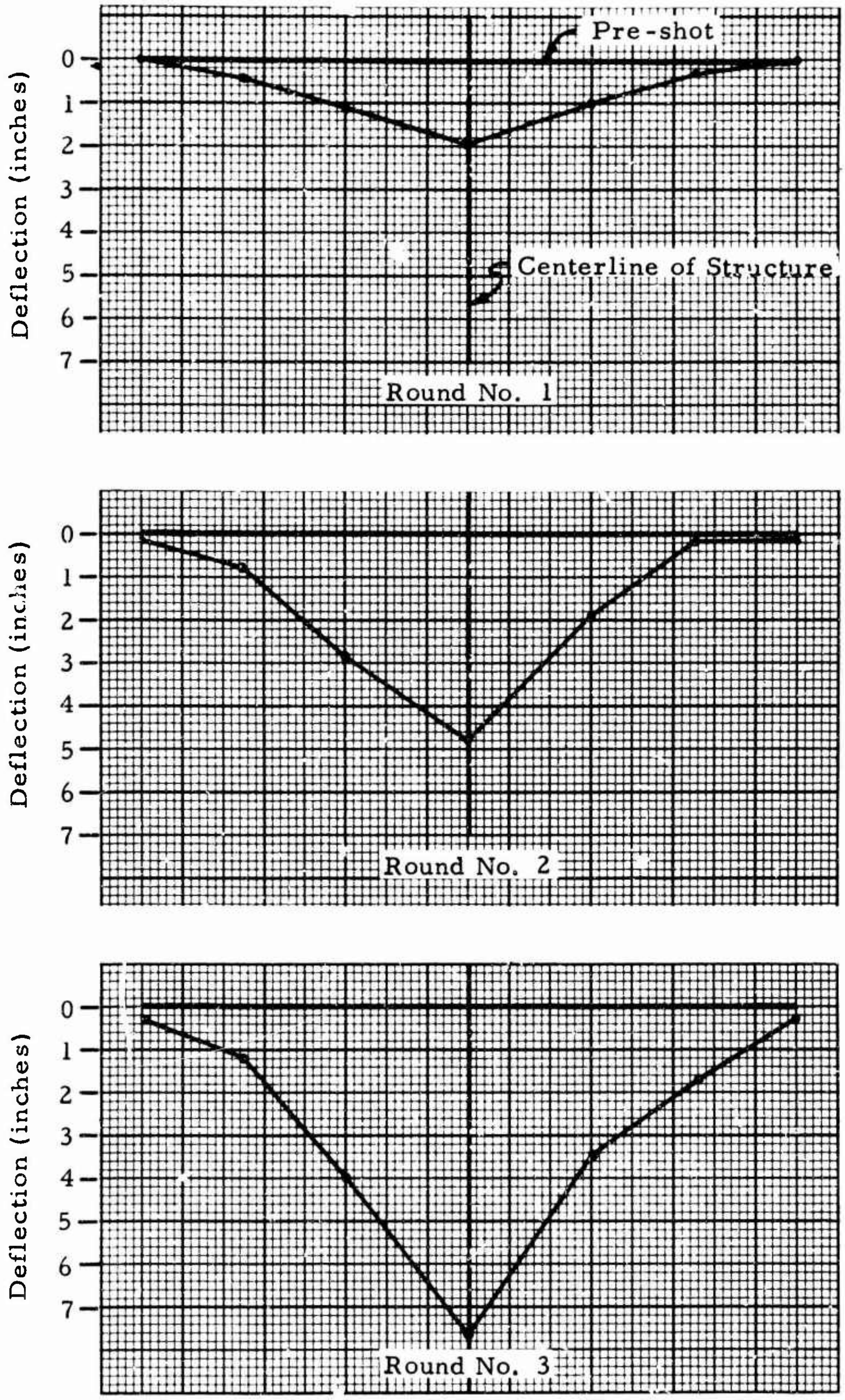


FIGURE 44
FLOOR SLAB -- PERMANENT DEFLECTIONS MEASURED ALONG LONGITUDINAL CENTER LINE

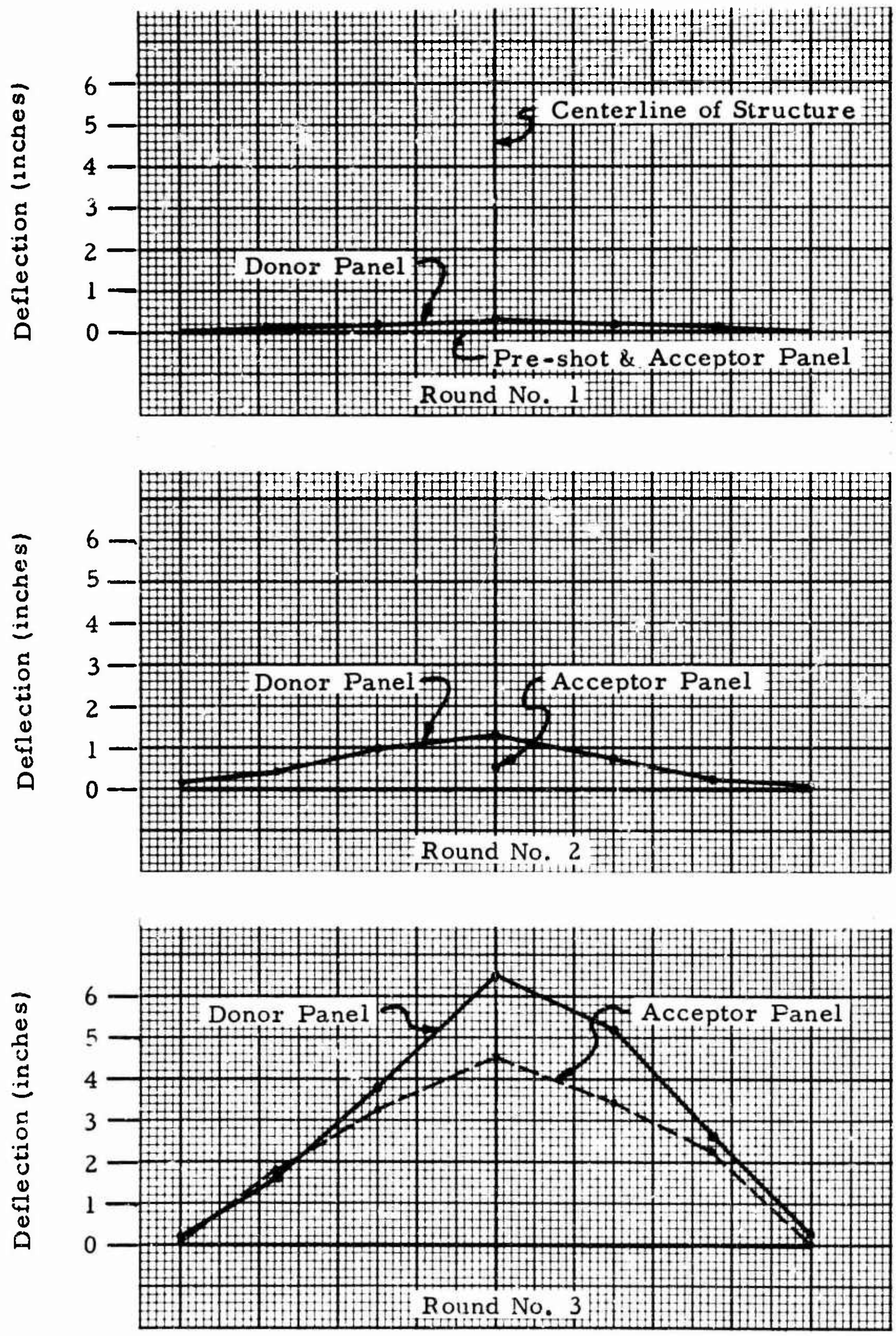


FIGURE 45
BACK WALL -- PERMANENT DEFLECTIONS
MEASURED ALONG TOP OF PANELS

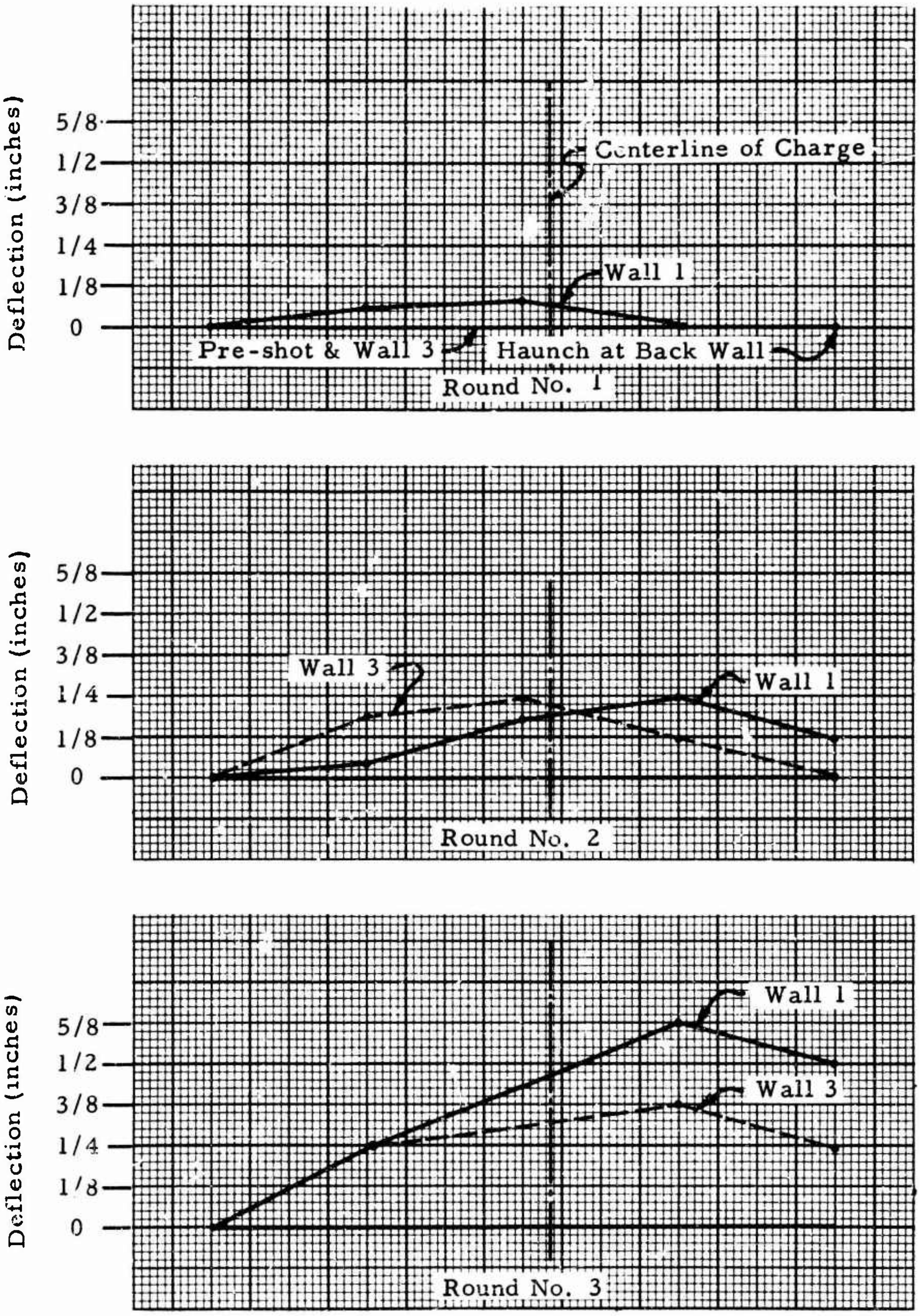


FIGURE 46
SIDE WALLS -- RELATIVE PERMANENT DEFLECTIONS
MEASURED ALONG TOP OF DONOR PANELS

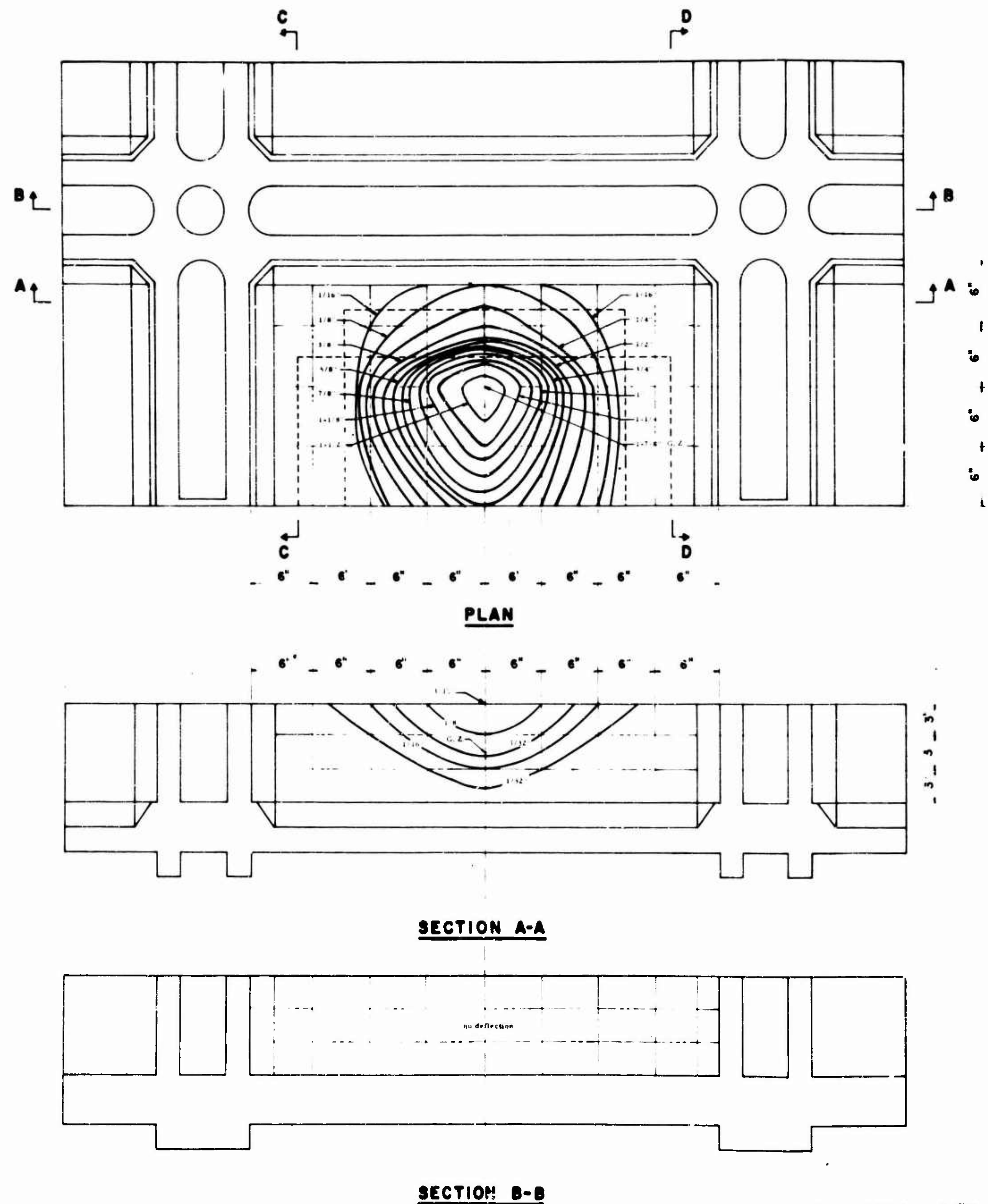


FIGURE 47
DEFLECTION CONTOURS

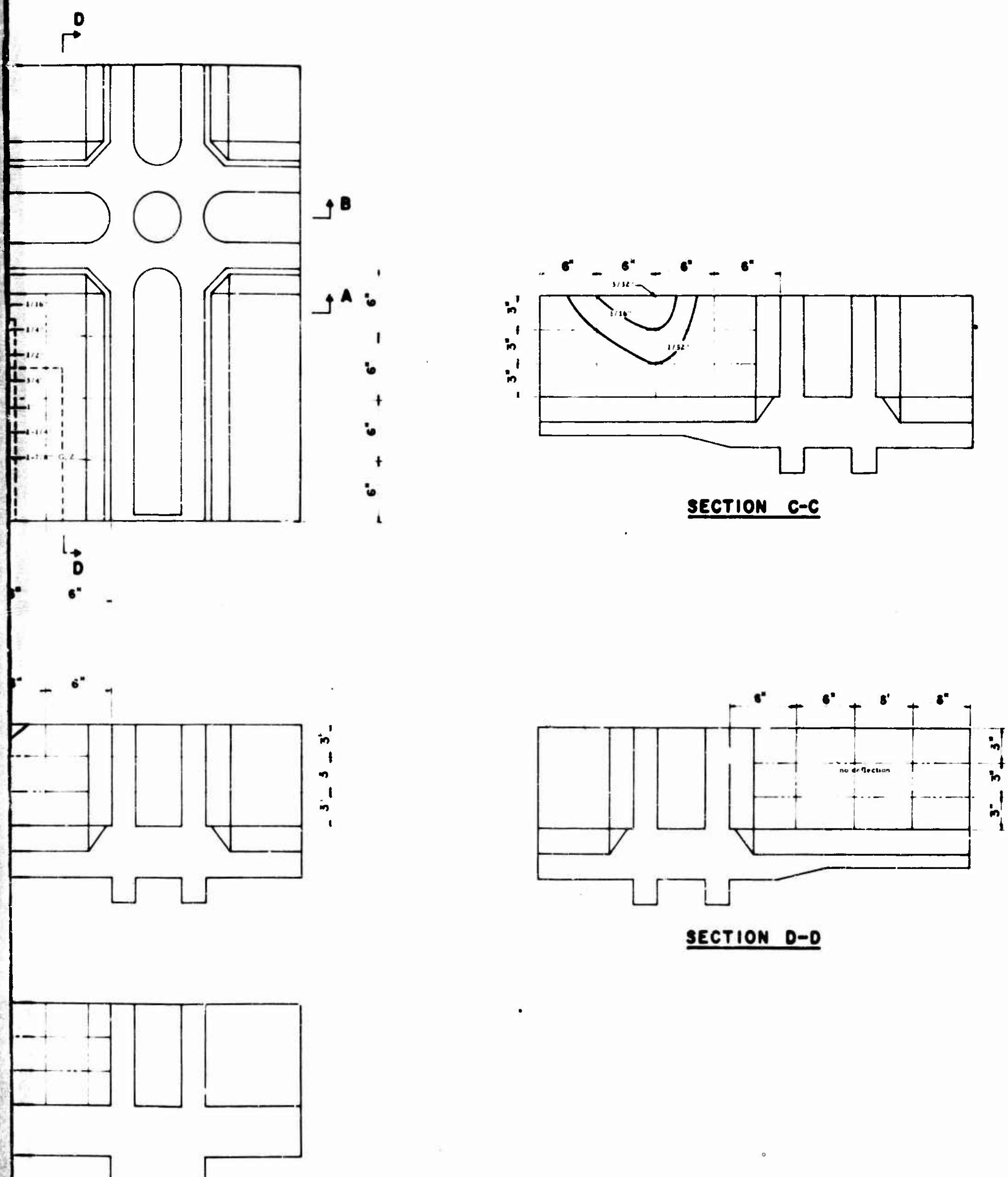
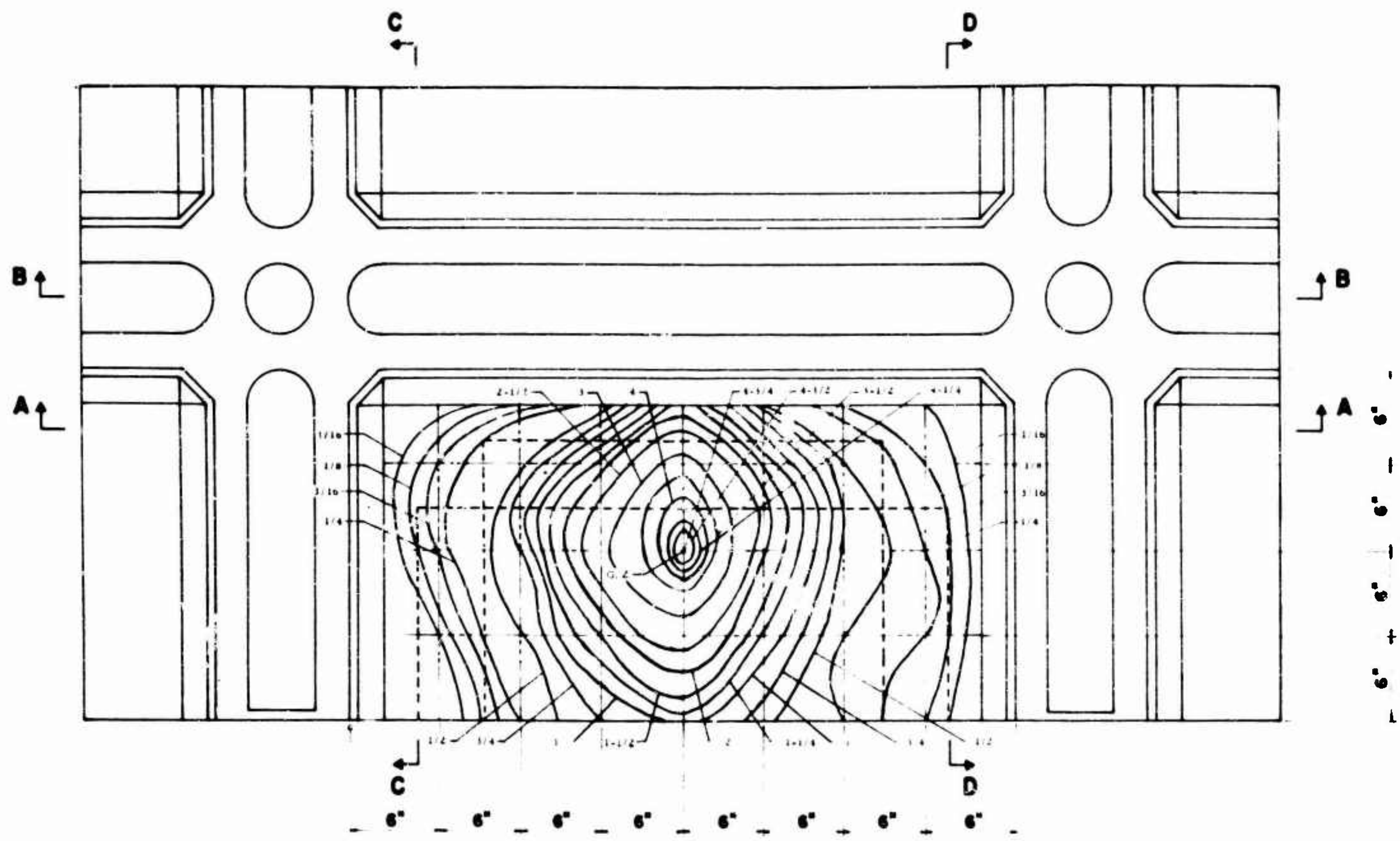


FIGURE 47
-ECTION CONTOURS - ROUND 1



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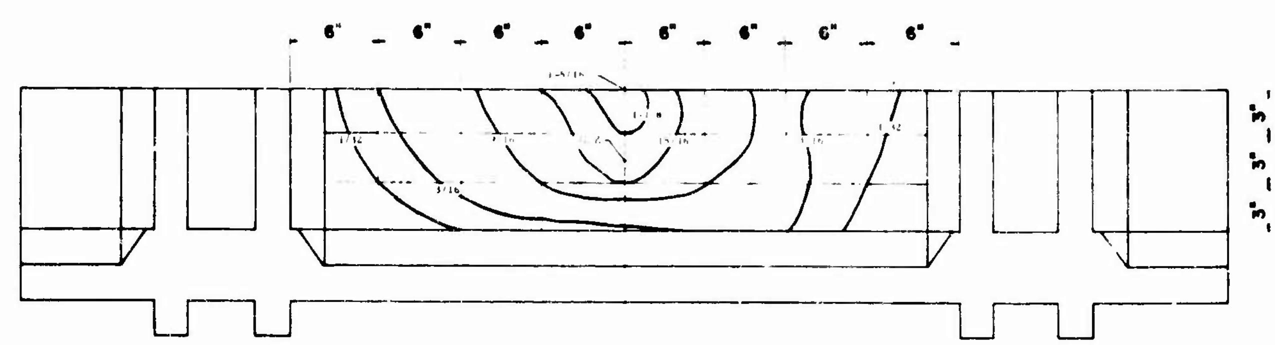
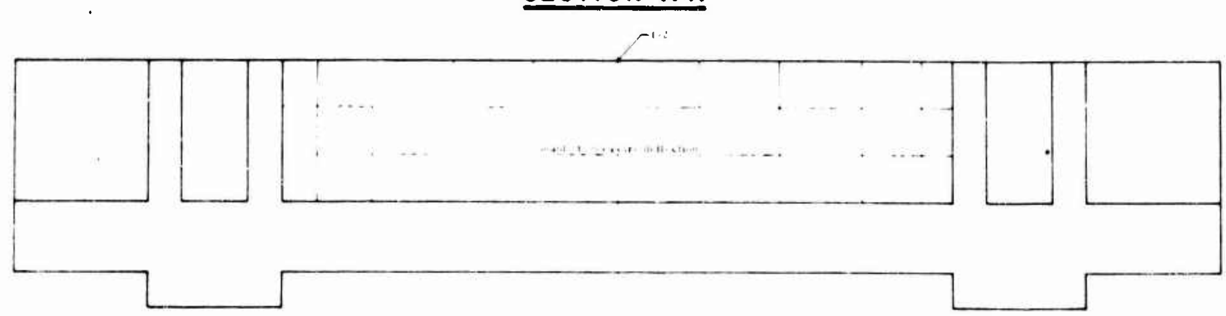


SECTION A-A



SECTION B-B

FIGURE 48
DEFLECTION CONTOURS - ROU

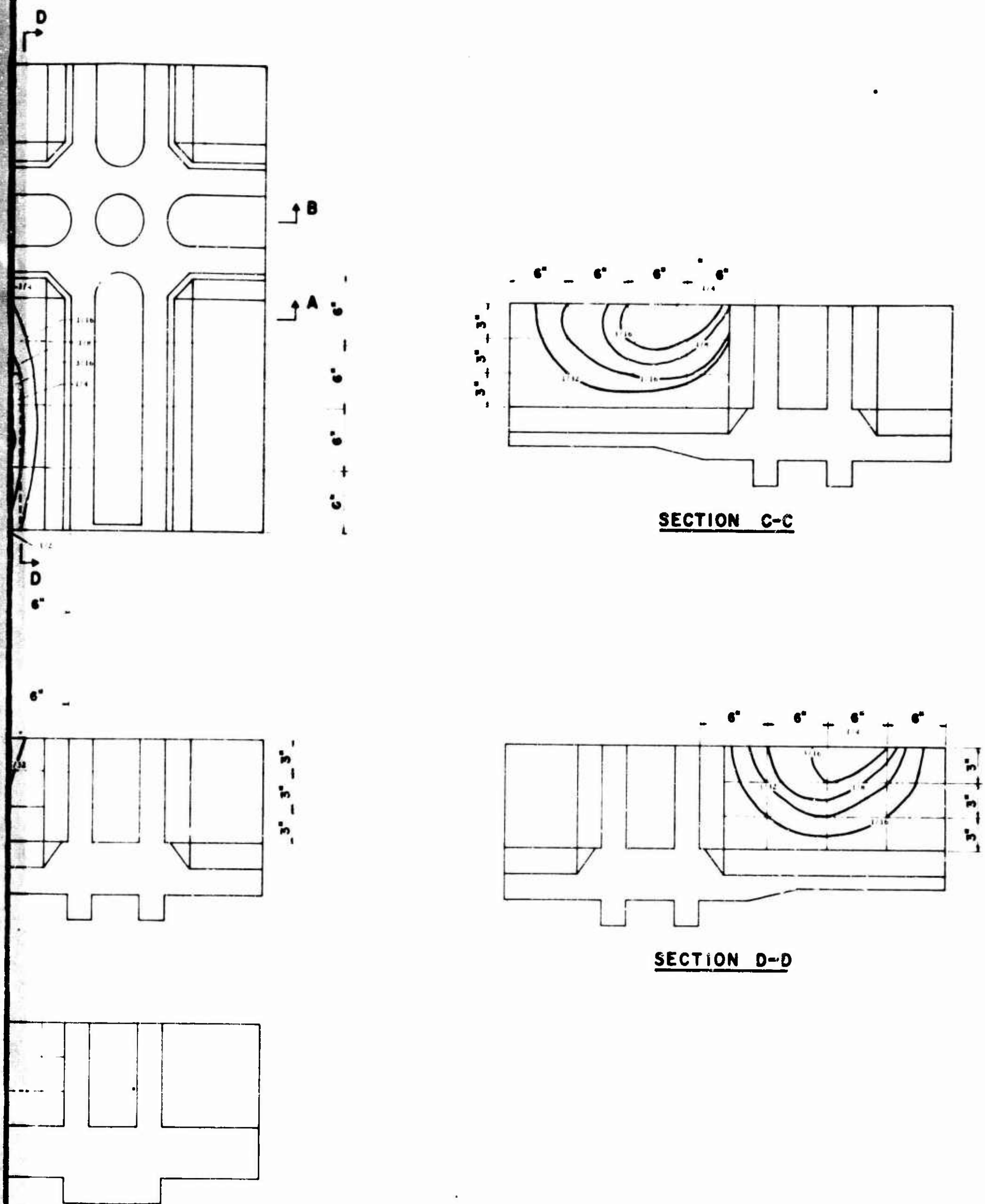


FIGURE 48
OUWN CONTOURS - ROUND 2

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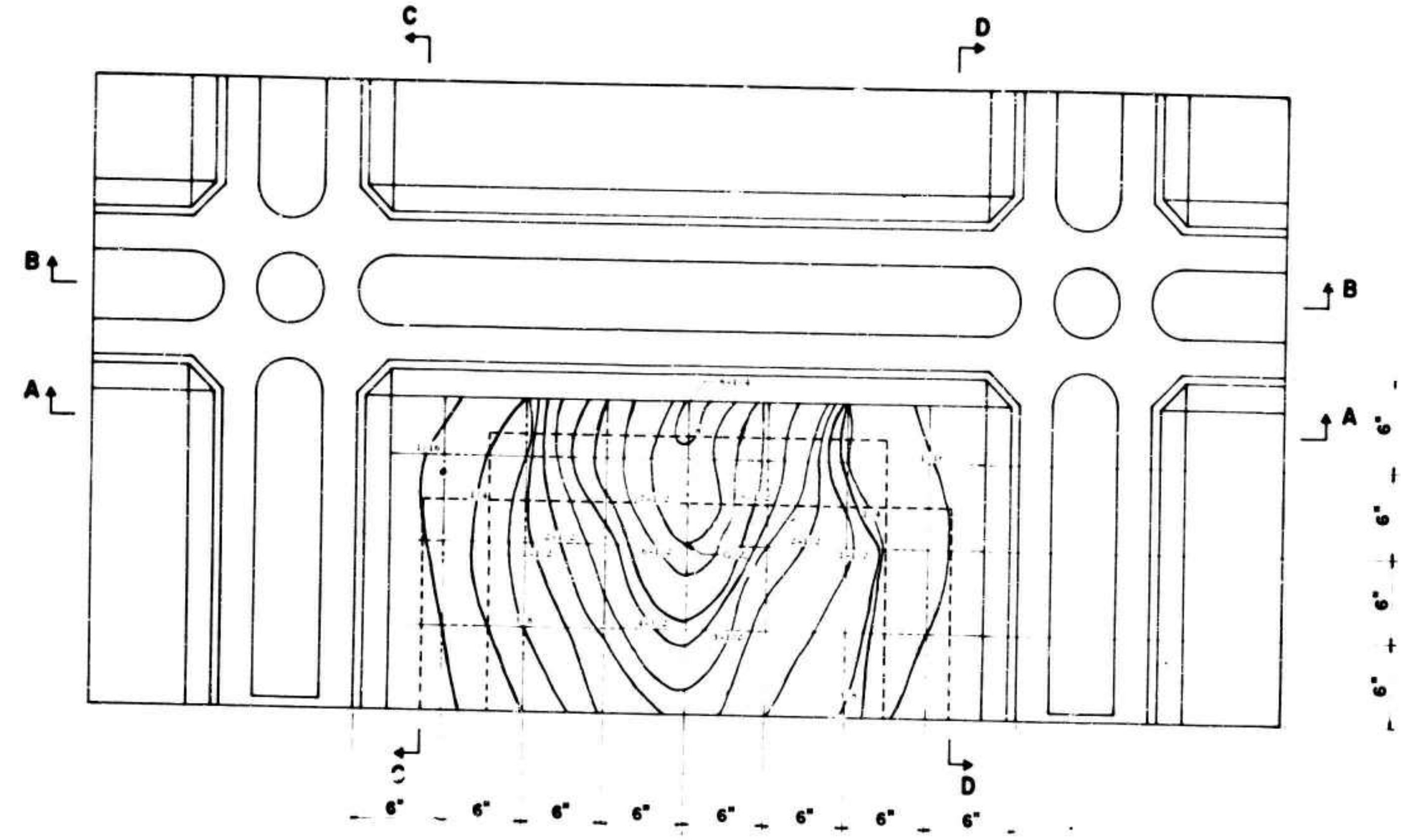

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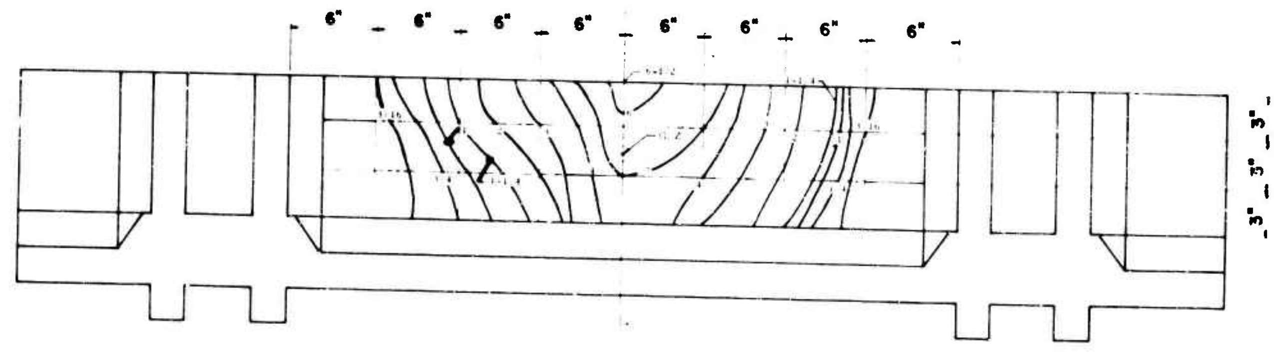
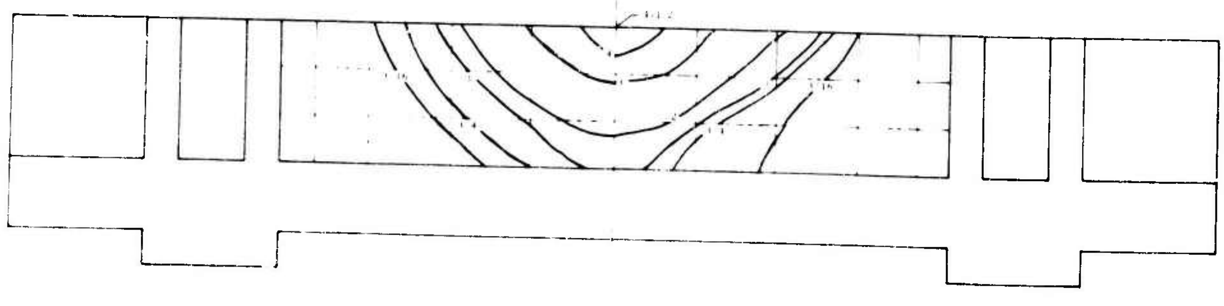
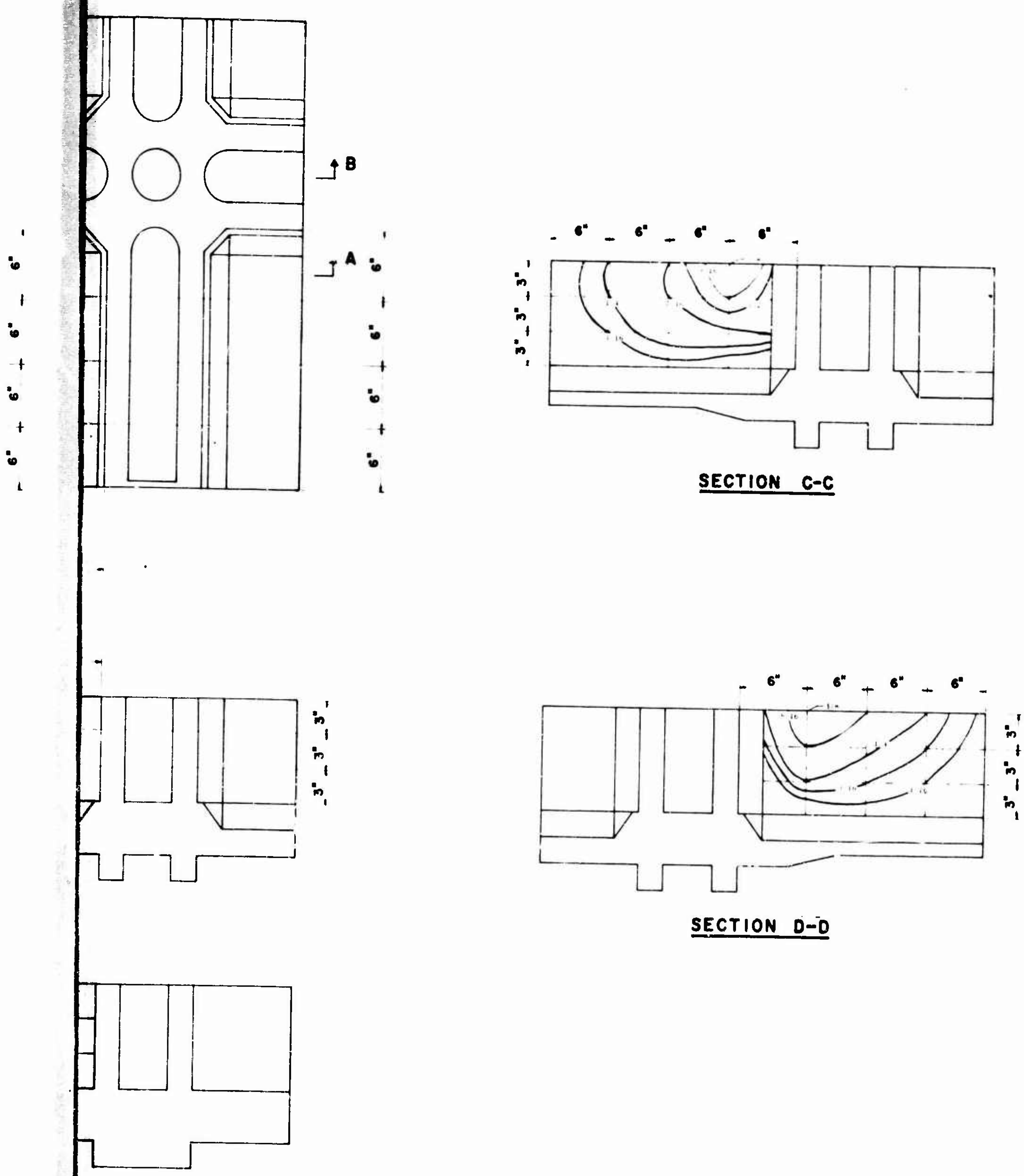


SECTION A-A



SECTION B-B

FIGURE 49
DEFLECTION CONTOURS - ROUN



GURE 49
UNDONTOURS - ROUND 3

2

TABLE 2

BACK WALL FRAGMENT VELOCITIES (Round 3)

Fragment Number	Velocity (feet per second)					Overall Average
	Space 1	Space 2	Space 3	Space 4	Average	
1	78	78	78	78	78	
2	78	78	70	66	73	
3	81	71	71	71	74	74
4	76	73	73	71	73	
5	76	71	71	71	72	

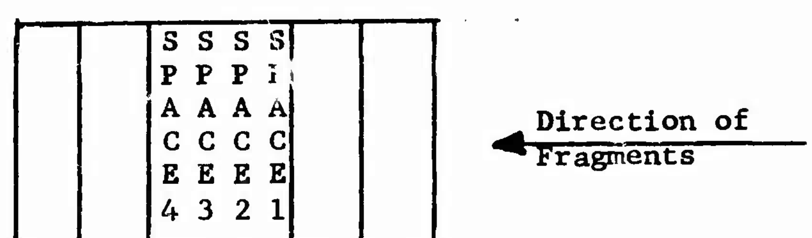


Diagram of Backboard

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APPENDICES

APPENDIX A

CONSTRUCTION

CONSTRUCTION

General

Construction of 1/10 scale bay Structure 1 was accomplished in three phases:

Procurement of materials
Fabrication in the shop of the entire model
Transportation of the completed model and its installation at the testing site

The model, including procurement of all materials except reinforcing steel, was fabricated by the Civil Engineering Department of Columbia University. Specially treated and finished reinforcing steel was procured by Ammann & Whitney from the United States Steel Corporation. Installation of the model was accomplished by Picatinny Arsenal.

Procurement of Materials

Other than the reinforcing steel all materials were obtained through normal sources of supply. Following are the chief characteristics of the materials used in the fabrication and installation of the model:

Cement -- High early strength cement conforming to ASTM Standard C150, Type III.

Aggregate -- Tested according to ASTM Standard D422-63. Material is sand, coarse to medium to fine, grain sizes ranging from 4.0mm through 0.05mm (Figure A-1).

Water -- Potable, as supplied by city distribution system.

Admixtures -- None

Formwork -- Plywood, 3/4-inch-thick, oiled.

Sand (for Wall Cavities) -- Tested according to ASTM Standard D422-63. Material is sand, medium to fine, grain sizes ranging from 0.6mm through 0.06mm (Figure A-1).

Paint -- Latex, flat, white and black.

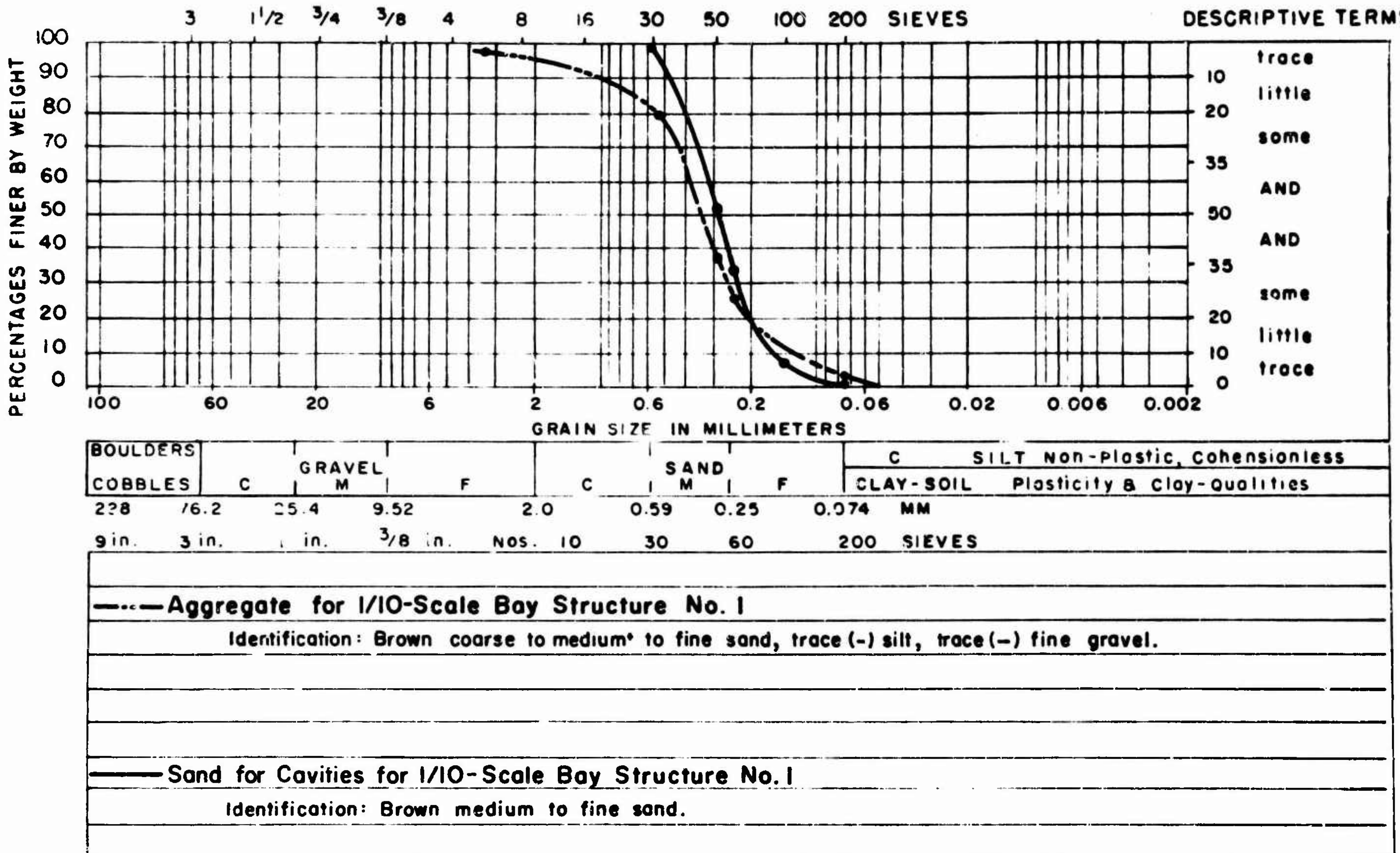


FIGURE A-1
GRAIN SIZE ANALYSIS: AGGREGATE AND SAND FOR CAVITIES

Reinforcing Steel -- Because the small diameter reinforcing rods required were not available, it was necessary to provide specially heat-treated bars to obtain the requisite mechanical properties. In collaboration with the United States Steel Corporation, it was decided to:

1. Utilize cold drawn steel wire conforming to AISI Standard C1040.

2. Subject the wire to heat treatment so conceived as to impart those properties prescribed for the prototype -- high strength billet steel bars conforming to ASTM Standard A432.

Accordingly, during a period of 22 hours the coiled wire was annealed by the normalizing process as follows:

1. Steel gradually heated to 1090°F during eight hours.

2. Steel maintained at 1090°F during six hours.

3. Steel gradually cooled to room temperature during eight hours.

Indicated in Table A-1 are the desired properties of the bars and those actually attained after heat treatment. Each size of wire was subjected to three tests, all of which yielded tangible results except for two 16-gage specimens that failed at the gage marks.

Fabrication

All operations of cutting, bending, placing and testing of the reinforcement were performed by the Civil Engineering Department of Columbia University, as was the subsequent concreting of the structure including cylinder tests.

The coiled reinforcing wire was straightened by stressing suitable lengths to a predetermined load (below the yield point) in a tension testing machine. Bending of bars was accomplished by means of specially designed jigs (Figure A-2).

TABLE A-1

PROPERTIES OF REINFORCING WIRE

Nominal Size		Diameter (in.)	Area (in. ^{2})	Yield Strength (psi)		Tensile Strength (psi)		Elongation in 8 in. (%)	
Theoretical	Actual			Theoretical	Actual	Theoretical	Actual	Theoretical	Actual
11	10-1/2	0.127	0.0127	60,000* minimum	81102 80394 81260 (80919)	90,000* minimum	91102 91102 90157 (90787)	11.11* minimum	7.25 6.87 7.12 (7.08)
12	11-1/2	0.112	0.0099		74747 81414 81919 (79360)		94141 91616 92525 (92761)		6.00 7.12 7.12 (6.74)
13	13	0.0915	0.0069		89855 87391 87681 (88309)		94203 94348 94203 (94251)		3.00 1.87 3.00 (2.62)
14	13-3/4	0.0084	0.0055		90000 89818 90363 (90060)		102182 102545 103455 (102727)		7.37 6.87 6.87 (7.04)
15	14-1/2	0.077	0.0047		93191 94043 89362 (92199)		105532 105957 100213 (103901)		12.12 12.25 10.62 (11.66)
16	16-1/4	0.061	0.0029		120000		130000		10.25

* According to ASTM Standard A432.
Average values are indicated in parentheses

FIGURE A-2

JIG FOR BENDING OF REINFORCING BARS

Placement of reinforcement was restricted by the small scale of the structure and by the intricacy of the reinforcing at certain intersectional areas (Figures A-3 to A-7).

The formwork presented no exceptional difficulty although extra caution was exercised to obviate any excessive deflection.

Batching and mixing were accomplished very efficiently inasmuch as all ingredients and equipment were arranged accessibly in the same shop. All ingredients were measured and charged manually; all batches were mixed in a three cubic foot portable mixer. A slump test was performed for each batch and cylinders were removed for compressive tests (Table A-2).

Depositing of concrete was much more difficult because of limited accessibility and working space. Although the slump of the concrete was somewhat greater than that normally recommended the high water content was more than offset by the high cement content thereby resulting in a water-cement ratio suitable for high compressive strength concrete (Table A-3). Concreting began at 10 a.m. and was completed at 2 p.m. Mechanical vibrators were utilized continuously for consolidating the concrete.

Curing of the concrete was accomplished by means of fabric coverings kept continuously wet following completion of concreting After removal of the ferms, all dimensions of the structure were verified for conformance with the drawings and no anomalies were noted.

Installation

Upon completion of fabrication, the model was transported to the testing site and installed ready for the test setup. Handling and transportation were facilitated by the arrangement of lifting beams consisting chiefly of two-inch channels positioned back-to-back and by the lifting ring situated at the centroid of the structure (Figure A-8).

Before placing the model in its final location for testing at Picatinny Arsenal, the existing soil (comprised of sandy fill) was removed to a depth of six inches and the ensuing excavation was back filled with compacted sand to the level of the adjacent terrain, the compaction being effected by means of hand tamping. After final positioning of the model, the voids located between the underside of the floor slab and the soil were backfilled with hand-tamped sand. Lastly, all exposed surfaces of the model were coated with two applications of white latex paint onto which was superimposed a grid of black lines to facilitate future identification and measurements.

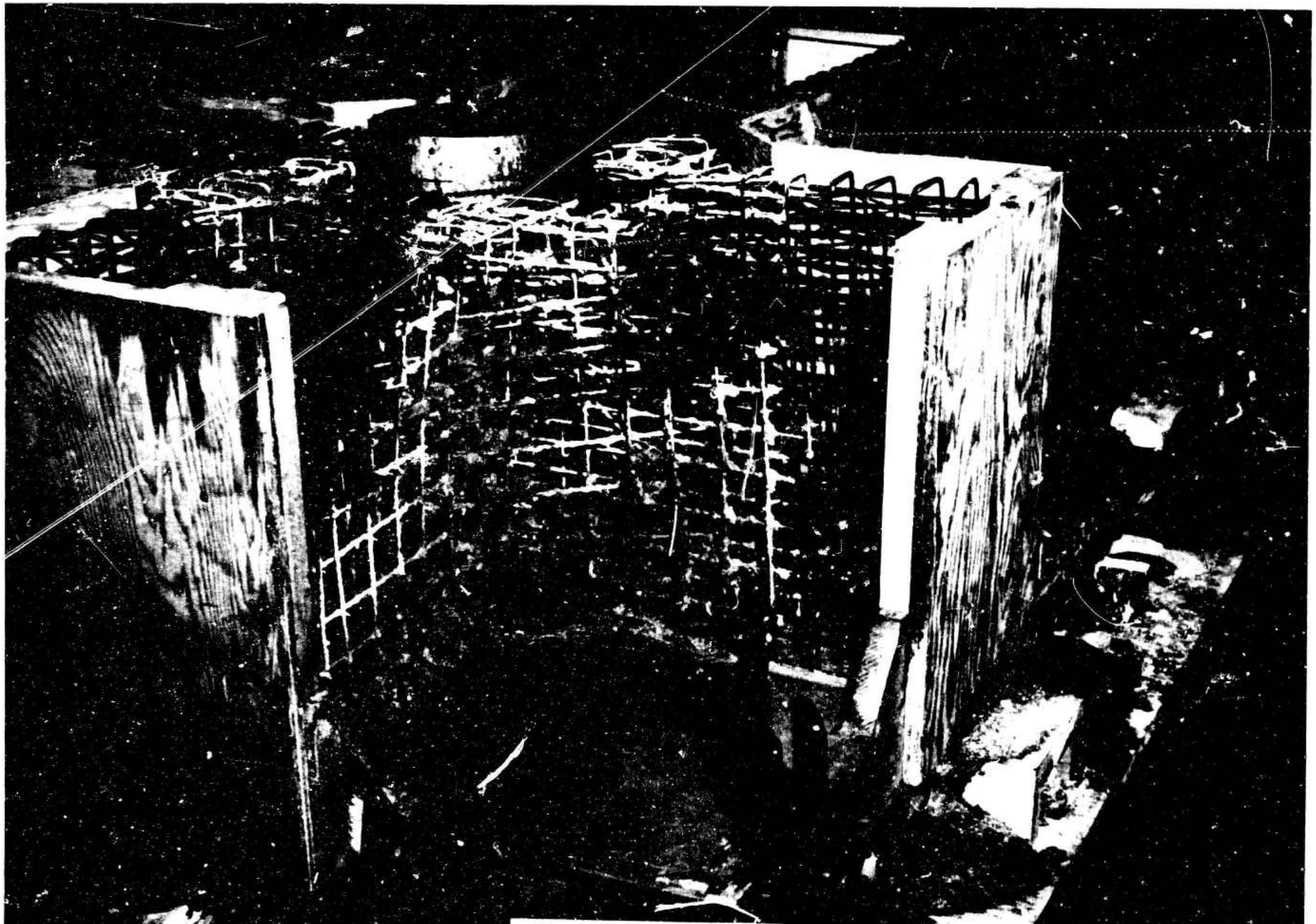


FIGURE A-3

REINFORCEMENT AT WALL INTERSECTION - ELEVATION

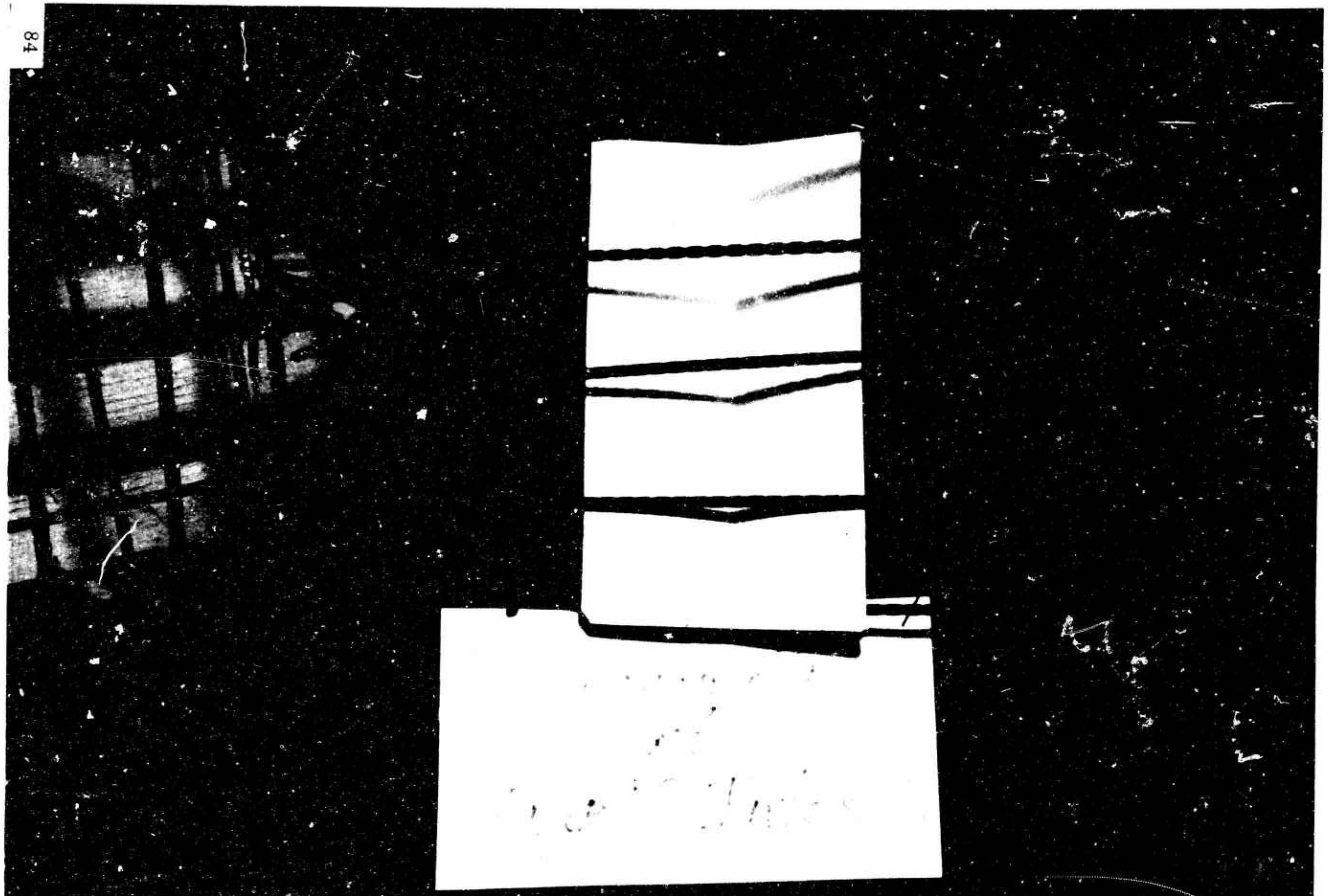


FIGURE A-4

REINFORCEMENT AT WALL INTERSECTION - DETAIL

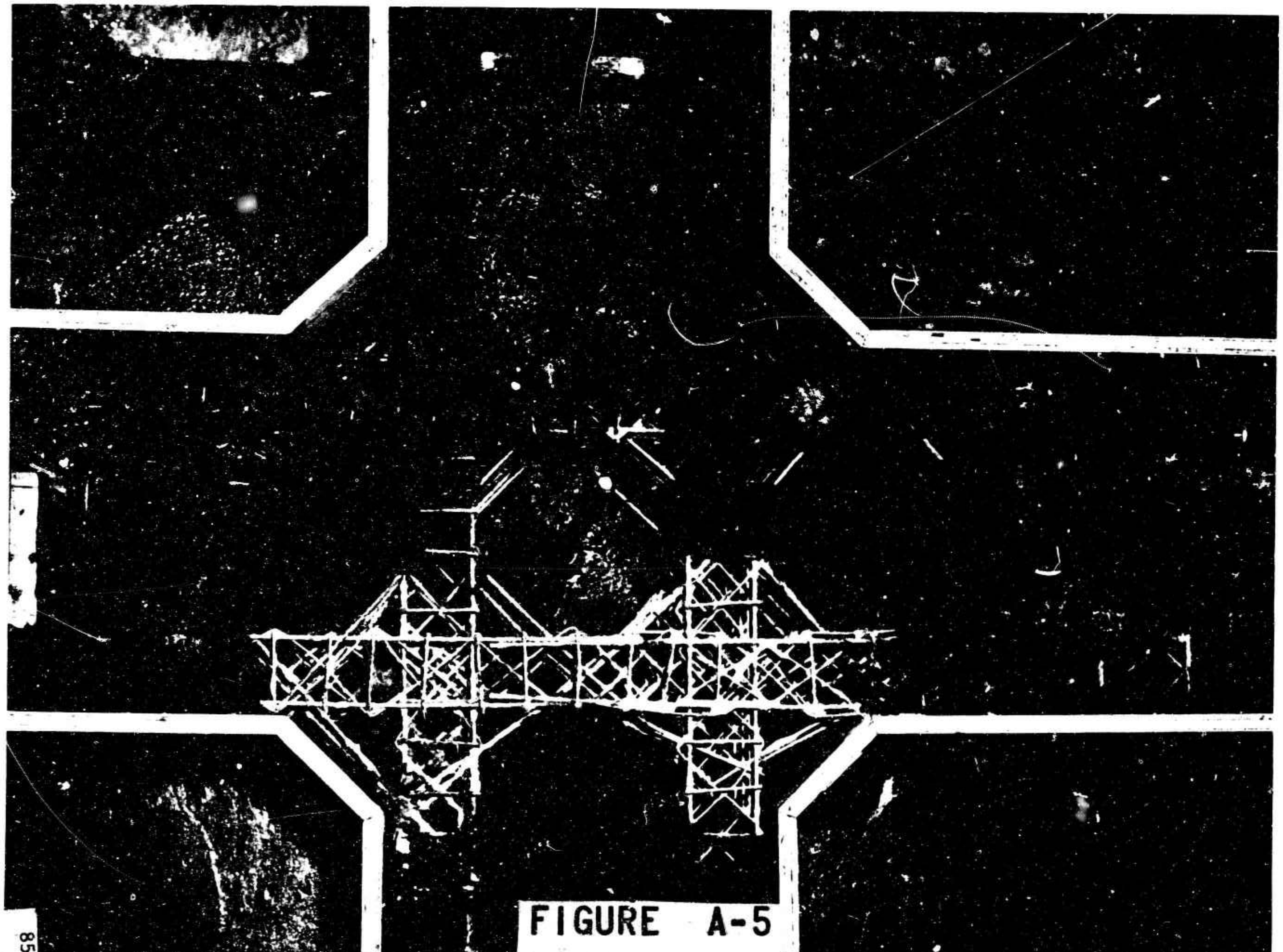


FIGURE A-5

REINFORCEMENT AT WALL INTERSECTION - PLAN

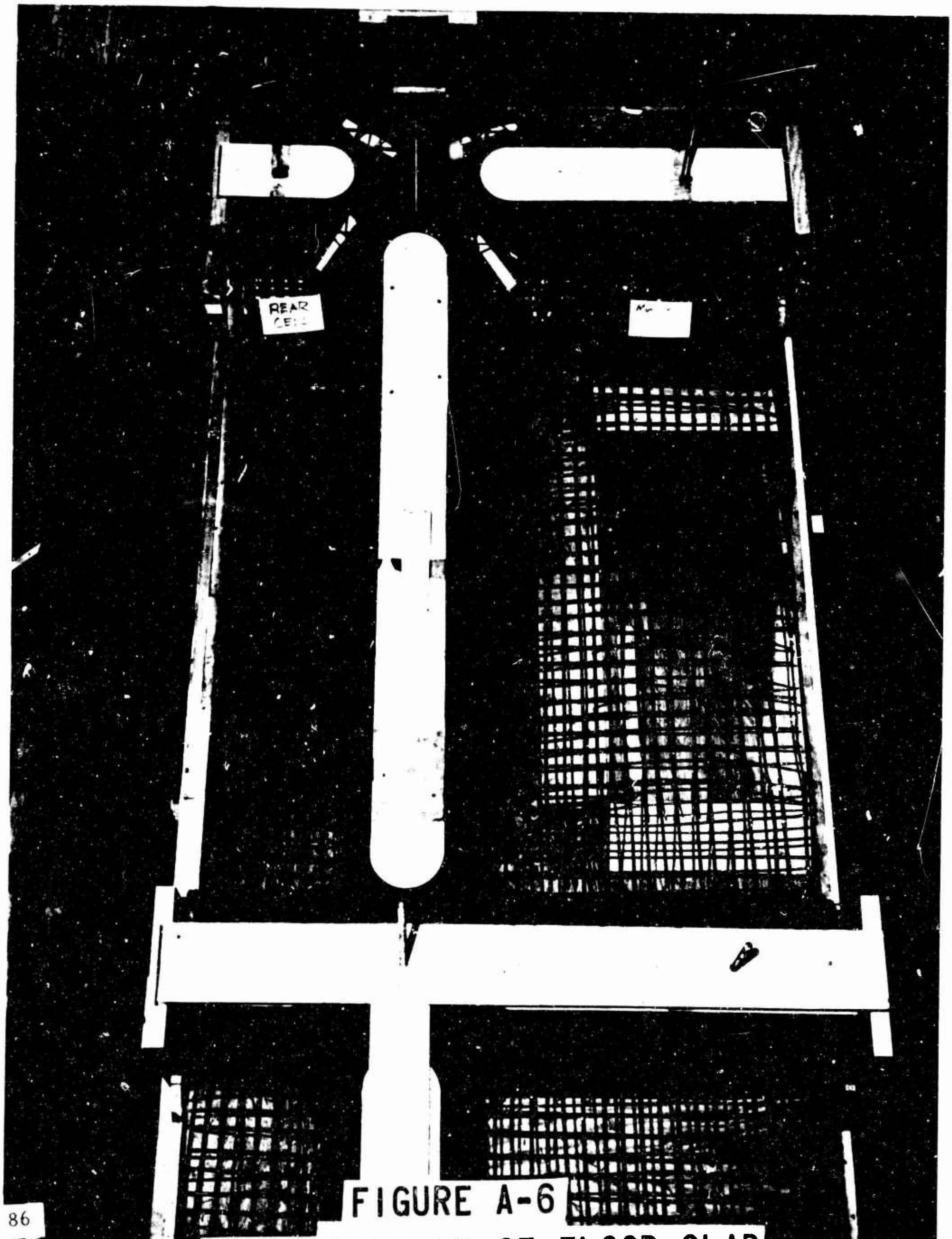


FIGURE A-6
REINFORCEMENT OF FLOOR SLAB

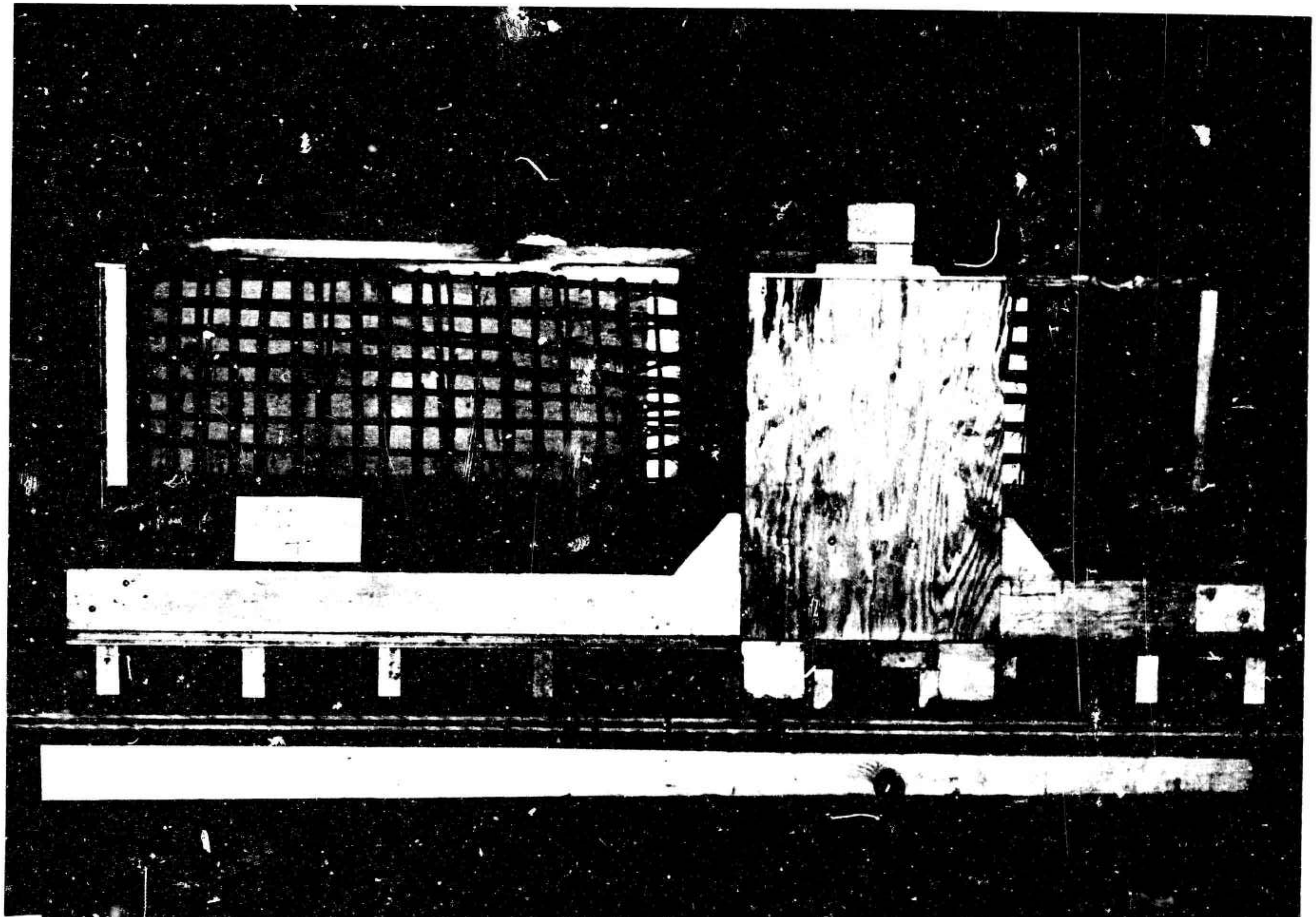


FIGURE A-7
REINFORCEMENT OF WALL

TABLE A-2

CONCRETE MIXES AND TEST RESULTS

Batch No.	Date Mixed	Cement (lbs.)	Aggregate* (lbs.)	Water (lbs.)	Slump (in.)	Compressive Strength	
						7 days	43 days
1	24 June 1965	103.4	215	50	4.75	4750	-
2	24 June 1965	103.4	215	50	4.0	-	5620
3	24 June 1965	103.4	215	50	4.5	-	6010
4	24 June 1965	103.4	215	50	4.25	-	5680
5	24 June 1965	103.4	215	50	4.25	-	6060
6	24 June 1965	103.4	215	50	6.0	4750	-

* See Figure A.1.

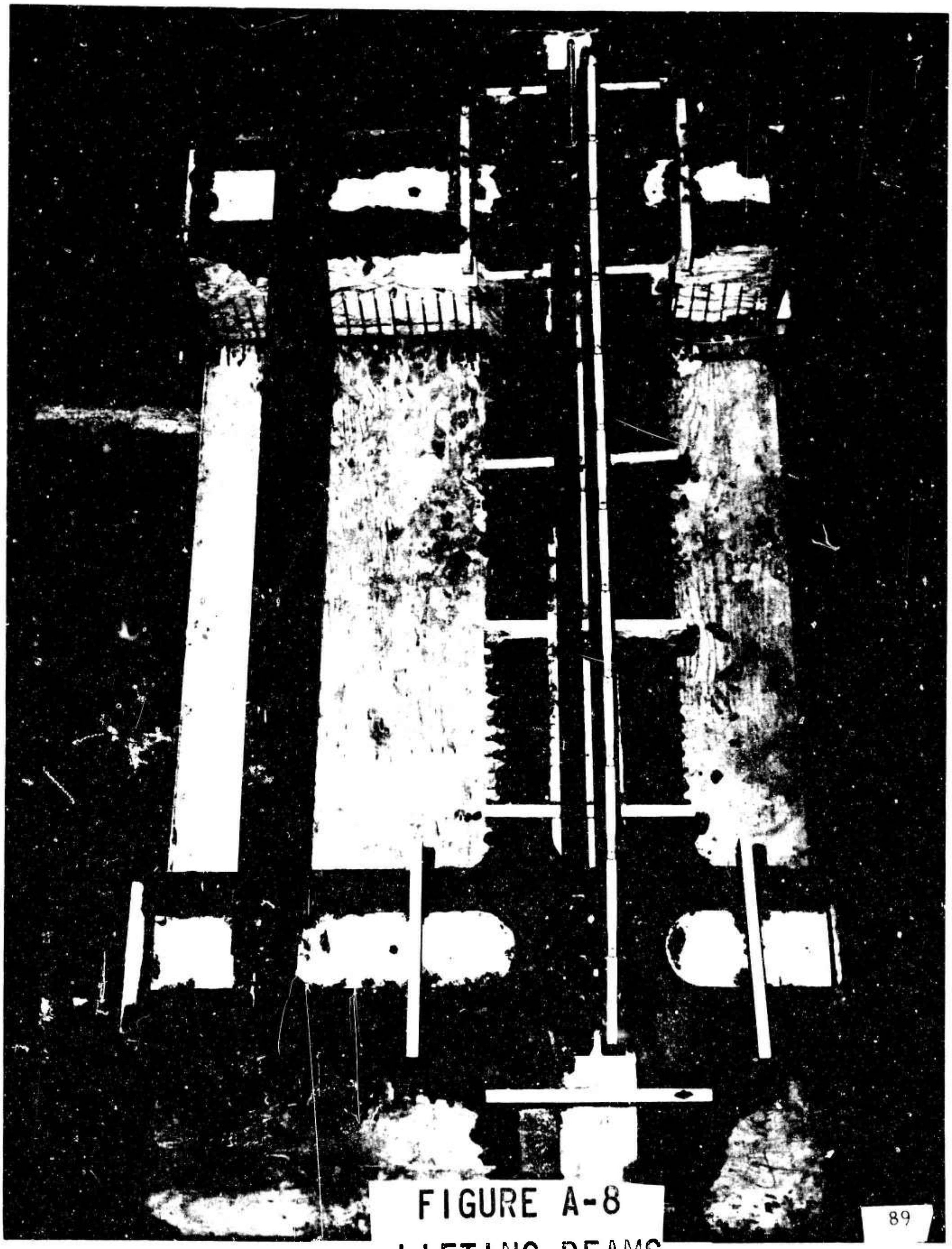


FIGURE A-8
LIFTING BEAMS

APPENDIX B

ANALYSIS OF MAXIMUM EXPLOSIVE CAPACITY OF BACK WALL OF 1/10 SCALE BAY STRUCTURE

ANALYSIS OF MAXIMUM EXPLOSIVE CAPACITY OF BACK WALL OF 1/10 SCALE BAY STRUCTURE

General

This structure was designed in connection with a scaling investigation of bay-type explosive facilities which included a full scale prototype as well as 1/3, 1/5, 1/8 and 1/10 scale models. The 1/10 scale bay structure was originally designed for an explosive capacity of two lbs. (2,000-lb equivalent in the full-scale prototype) on the basis of criteria presented in Reference 3. However, as demonstrated by test results, the design was found to be conservative. Two subsequent tests were performed on the same model. The combined weight of the charges in the three tests was equal to about 9-1/2 lbs. or 9,500 lbs. full-scale equivalent. This analysis is based on information obtained from empirical data from tests performed subsequent to the publishing of Reference 3. This data verified the greater capacity of the structure.

Blast Loading

In this report, the analysis utilizes the theoretical blast impulse loads included in Reference 4. These blast loads include the increased impulse loads produced by the multiple reflections of the blast pressures during an explosion in cubicle type structures.

Strength Criteria

The back wall was analyzed for dynamic behavior by use of the ultimate strength theory (Reference 3).

Compressive strength of concrete was 5,840 psi and was determined as the average value obtained from post-shot cylinder tests. Reinforcing steel was of annealed deformed wire having a static unit stress of 87,000 psi (average of the yield and ultimate strengths as obtained from test specimens). The increase in strength under dynamic loads was taken into account with the use of dynamic increase factors obtained from Reference 6.

Analysis

In general, the analysis of the structure is based on the solution of the equation of motion $F - R = m\ddot{X}$ where F is the applied blast force, R is the internal resistance of the structural member, m is the mass of an equivalent single degree of freedom system and $\ddot{X}$ is the acceleration of the mass.

This equation of motion can be readily solved by any of several numerical integration methods. The numerical method illustrated in this Appendix for the analysis of the back wall is the semi-graphical method of analysis described in Reference 5.

In this analysis the structural dynamic properties of the proposed wall are determined and the maximum loading calculated. The method of calculation utilized includes these steps:

1. Establish structural properties of the wall (Figure B-1).
2. Calculate the dynamic resistance of each panel (composite wall) based on ultimate bending moment capacity.
3. Check shear capacity at the supports of each panel.
4. Calculate other pertinent dynamic properties of each panel (including maximum deflection and effective mass).
5. Calculate pertinent dynamic properties of the wall.
6. Calculate the impulse load capacity of the wall.
7. Determine the explosive charge that would produce a blast impulse load equal to the impulse capacity of the wall.

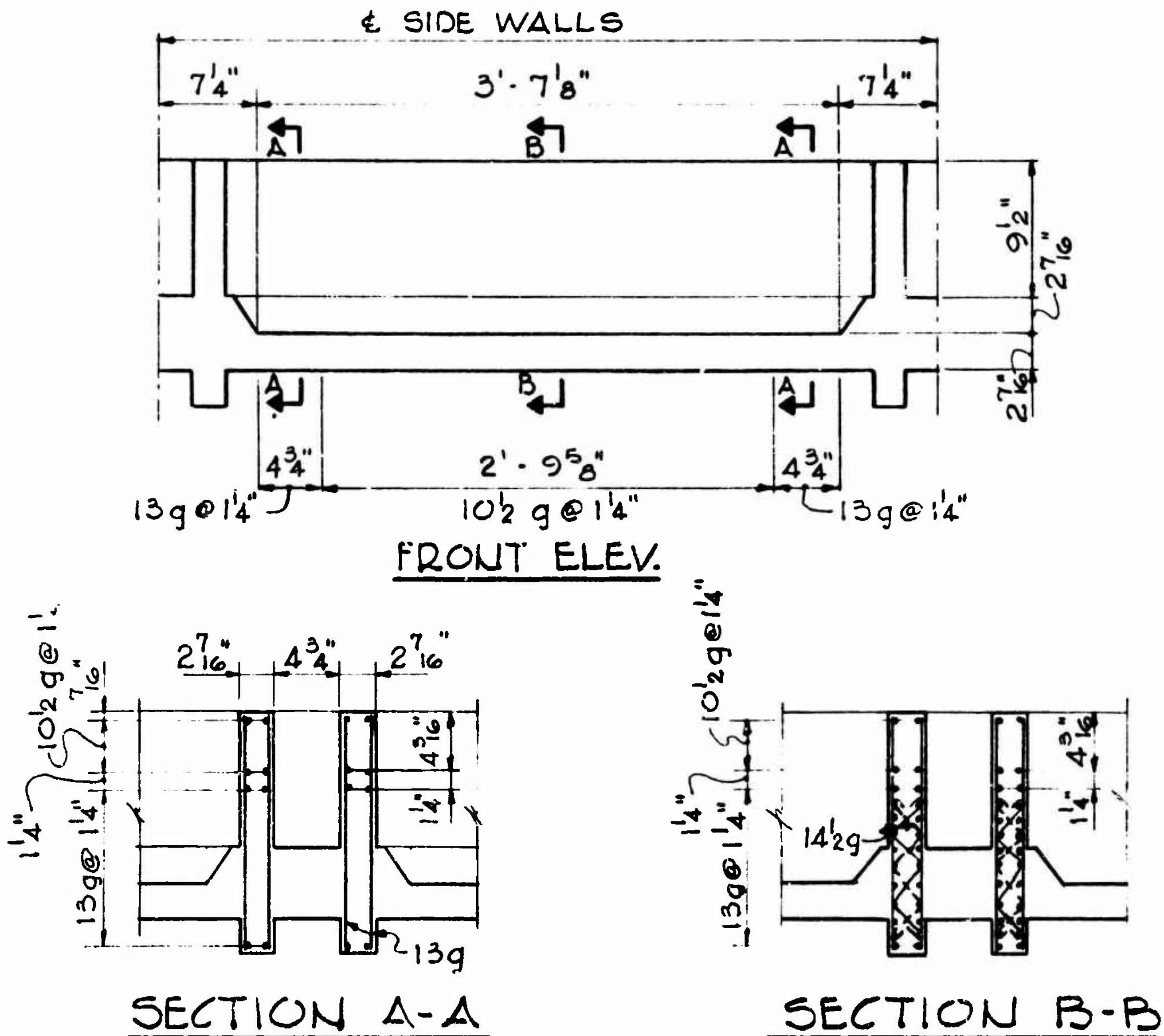


FIGURE B-1

STRUCTURAL PROPERTIES OF BACK WALL

Structural Properties of Wall (Figure B-1)

Properties of reinforcement

Steel Wire (AS&W Gage No.)	Diameter (in.)	Area (in.^{2})	Spacing (in.)	Area Per Foot (in.^{2}/ft.)
10-1/2 g	0.127	0.0127	1.25	0.122
13 g	0.094	0.0069	1.25	0.066
14-1/2 g	0.077	0.0047	1.25	--

Effective slab depth (d) (Figure B-2)

Total Thickness T = 2.436 inches
Concrete Cover = 0.188 inches

d (Section I - 10-1/2 g) = 2.436-0.188-0.063= 2.185 in.
d (Section I - 13 g) = 2.436-0.188-0.047= 2.201 in.
d (Section II - 10-1/2 g) = 2.436-0.188-0.126-0.063
= 2.058 in.
d (Section II - 13 g) = 2.436-0.188-0.127-0.047
= 2.074 in.

Ultimate stresses

f'_c = 5,840 psi - Average of cylinder tests of concrete
f_s = 87,000 psi - Average static stress reinforcement
f_v = 96,000 psi - Average static stress of shear reinforcement

Ultimate dynamic stresses (Reference 6)

$f'_{dc} = 1.25\, f'_c$ = 7,300 psi - Dynamic stress in concrete
$f_{ds} = 1.1\, f_s$ = 95,700 psi - Dynamic stress in reinforcement

Ultimate dynamic resistance of each panel of the wall

The ultimate resistance of a structure is a maximum total force which it can sustain at the point of ultimate failure.

Each panel of the back wall is considered as a plate fixed on three adjoining sides and free on the fourth side (Figure B-2).

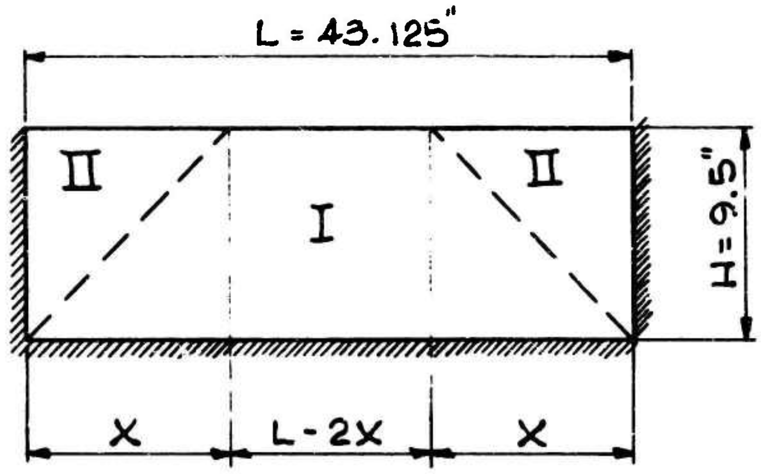


FIGURE B-2
ELEVATION OF BACK WALL SHOWING
CRACK PATTERN

Therefore, the ultimate resistances of Sections I and II are governed by the moment capacity which is developed by the vertical and horizontal reinforcement, respectively. The reinforcement near the donor surface of the panel and at the panel supports (floor slab and side walls) produces the ultimate negative bending moment capacity of the wall whereas the reinforcement near the acceptor face of the panel and crossing the positive crack lines (dashed lines in crack pattern diagram) produces the ultimate positive bending moment capacity.

Slope of crack lines

$\phi = 36.5^{\circ}$ (assumed by trial and error solution)

Check of Ultimate Dynamic Resistance of Section I (Figure B-2)

Moment capacity per foot of Section I of back wall panel

Ultimate moment capacity is a function of the depth of compression stress block "a". Therefore,

$$m_{ULT} = A_s f_{ds}\left[d - \frac{a}{2}\right], \quad \text{(Ref. 7)}$$

where

$a = \dfrac{A_s f_{ds}}{0.85 b f'_{dc}}$ - depth of compression stress block (inches)

A_s = area of reinforcement (square inches)

f_{ds} = dynamic stress of reinforcement (psi)

b = width of one-foot-wide strip (inches)

Also, if the negative and positive reinforcement are the same, then,

$$m_{N\text{-}ULT} = m_{P\text{-}ULT}$$

where

$m_{N\text{-}ULT}$ = Ultimate moment capacity of negative reinforcement per foot

$m_{P\text{-}ULT}$ = Ultimate moment capacity of positive reinforcement per foot

Moment capacity per foot of 10-1/2 gage wire (Section I)

$$a = \frac{A_s f_{ds}}{0.85 b f'_c} = \frac{0.122(95.7)}{0.85(12)(7.3)} = 0.157 \text{ inches}$$

$$m_{N\text{-}ULT} = m_{P\text{-}ULT} = 0.122(95.7)\left[2.185 - \frac{0.157}{2}\right]$$

$$= 24.6 \text{ Kip-in/ft.}$$

Moment capacity per foot of 13 gage wire (Section I)

$$a = \frac{(0.066)(95.7)}{0.85(12)(7.3)} = 0.085 \text{ inches}$$

$$m_{N\text{-}ULT} = m_{P\text{-}ULT} = 0.066(95.7)\left[2.201 - \frac{0.085}{2}\right]$$

$$= 13.6 \text{ Kip-in/ft.}$$

Total moment capacity of wall (Section I)

The total moment capacity of a section of a wall is equal to the sum of the moment capacities of all the reinforcement crossing the crack lines and acting perpendicular to the axis of rotation (Figure B-3).

$$\Sigma M_{N\text{-}ULT} = \left[2(13.6)(4.75) + 24.6(33.625)\right](1/12)$$

$$= 79.7 \text{ Kip-inches}$$

$$\Sigma M_{P\text{-}ULT} = \left[2(13.6)(4.75) + 2(24.6)(12.8 - 4.75)\right](1/12)$$

$$= 43.7 \text{ Kip-inches}$$

$$\Sigma M_{\triangle ULT} = \Sigma M_{N\text{-}ULT} + \Sigma M_{P\text{-}ULT}$$

$$= 79.7 + 43.7 = 123.4 \text{ Kip-inches}$$

Ultimate dynamic resistance of Section I

The centroidal distance (C) of a section is equal to the distance from the centroid of the section to the axis of rotation (Figures B-3 and B-4).

$$C = \frac{2(1/6)(12.8)(9.5)^2 + 1/2(17.525)(9.5)^2}{2(1/2)(12.8)(9.5) + 17.525(9.5)} = 4.083 \text{ inches}$$

$$R_{\triangle ULT} = M_{\triangle ULT} / C = 123.4 / 4.083 = 30.22 \text{ Kips}$$

Ultimate dynamic unit resistance of Section I

$$r_{\triangle ULT} = \frac{R_{\triangle ULT}}{\text{Area} \triangle} = \frac{30.22(144)}{289} = 15.11 \text{ Kips/ft.}^2$$

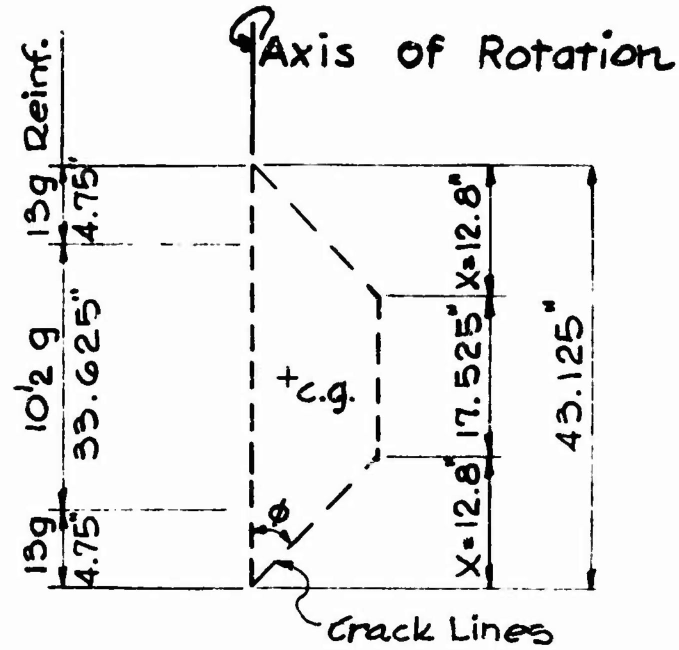


FIGURE B-3
LAYOUT OF SECTION I OF THE BACK
WALL PANEL

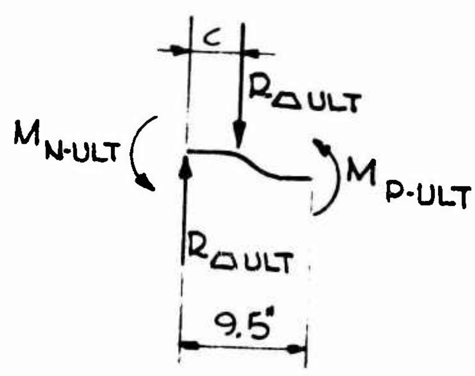


FIGURE B-4
FREE BODY DIAGRAM OF WALL PANEL

SECTION I

Check of Ultimate Dynamic Resistance of Section II (Figure B-2)

Moment capacity of 10-1/2 gage wire (Section II)

$$M_{N\text{-}ULT} = M_{P\text{-}ULT} = 0.122(95.7)\left[2.058 - \frac{0.157}{2}\right]$$
$$= 23.1 \text{ Kip-in/ft.}$$

Moment capacity of 13 gage wire (Section II)

$$M_{N\text{-}ULT} = M_{P\text{-}ULT} = 0.066(95.7)\left[2.074 - \frac{0.085}{2}\right]$$
$$= 12.83 \text{ Kip-in/ft.}$$

Total moment capacity of wall (Section II - Figure B-5)

$$\Sigma M_{N\text{-}ULT} = \left[23.10(4.812) + 12.83(4.688)\right](1/12)$$
$$= 14.27 \text{ Kips-in}$$
$$\Sigma M_{P\text{-}ULT} = \Sigma M_{N\text{-}ULT} = 14.27 \text{ Kip-in}$$
$$\Sigma M_{\Delta ULT} = \Sigma M_{N\text{-}ULT} + \Sigma M_{P\text{-}ULT} = 2(14.27)$$
$$= 28.5 \text{ Kip-in}$$

Ultimate dynamic resistance of Section II (Figures B-5 and B-6

$$C = 1/3\,(12.8) = 4.27 \text{ inch}$$

$$R_{\Delta ULT} = \frac{\Sigma M_{\Delta ULT}}{C} = \frac{28.5}{4.27} = 6.69 \text{ Kips}$$

Ultimate dynamic unit resistance of Section II

$$r_{\Delta ULT} = \frac{R_{\Delta ULT}}{\text{Area }\Delta} = \frac{6.69(144)}{1/2(9.5)(12.8)}$$
$$= 15.84 \text{ Kips/ft}^2 \approx r_{\square ULT} = 15.11 \text{ Kips/ft}^2$$

Note: The ultimate unit resistance of Section I and II should be equal.

Ultimate dynamic resistance of wall panel

Average unit resistance of total panel

$$r_{av} = \frac{R_{\square ULT} + 2R_{\Delta ULT}}{\text{Area (Panel)}} = \frac{\left[30.22 + 2(6.69)\right]144}{9.5(43.125)} = 15.32 \text{ Kips/ft}^2$$

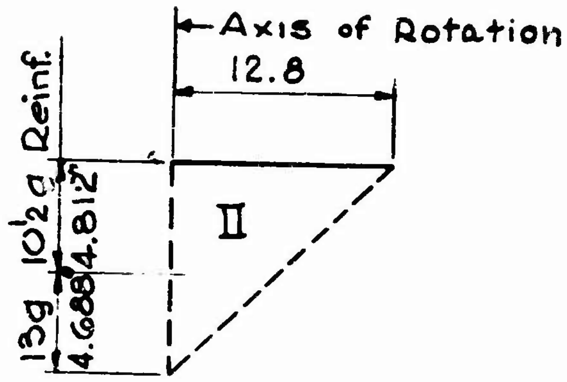


FIGURE B-5
LAYOUT OF SECTION II OF BACK WALL

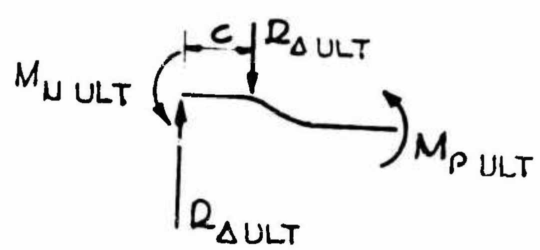


FIGURE B-6
FREE BODY DIAGRAM OF WALL PANEL

SECTION II

Ultimate dynamic resistance of total panel

$$R_{ULT} = 30.22 + 2(6.69) = 43.6 \text{ Kips}$$

Check of Shear Capacity of Wall Panels at Supports (Figure B-7)

Both Sections I and II are subdivided into strips, namely: the outer and midstrip. The strips for Sections I and II extend the full height and length of the panel, respectively. The width of the outer strip is governed by the length over which the smaller reinforcement (13 g) is effective whereas the width of the midstrip corresponds to the effective length where the larger reinforcing bars (10-1/2 g) are effective. In this analysis, the unit shear forces acting across the section supports are assumed to be proportional to the moment capacities of the individual strips. For diagonal tension, the critical section is located at a distance "d" from the edge of the section support (Reference 8), where d is a weighted value, allowing for the variation in effective depth along the support edge.

Check of Shear Capacity of Section I

The maximum load that the panel can resist at ultimate flexural failure, "$r_{av.}$" previously determined, is applied as a shear force to check for diagonal tension failure at supports. If the design is subject to shear failure, then shear reinforcement is added.

Weighted effective depth - d_w (Figure B-3).

$d = 2.201$ at outer strip of section
$d = 2.185$ at midstrip of section

$$d_w = \frac{2(4.75)(2.201) + 33.625(2.185)}{43.125} = 2.189 \text{ inches}$$

Total effective shear

A plane ⓪–⓪ , is taken a distance d_w above the support. This is the critical section for shear (Reference 8). The dimensions of Section I above this plane is shown in Figure B-8.

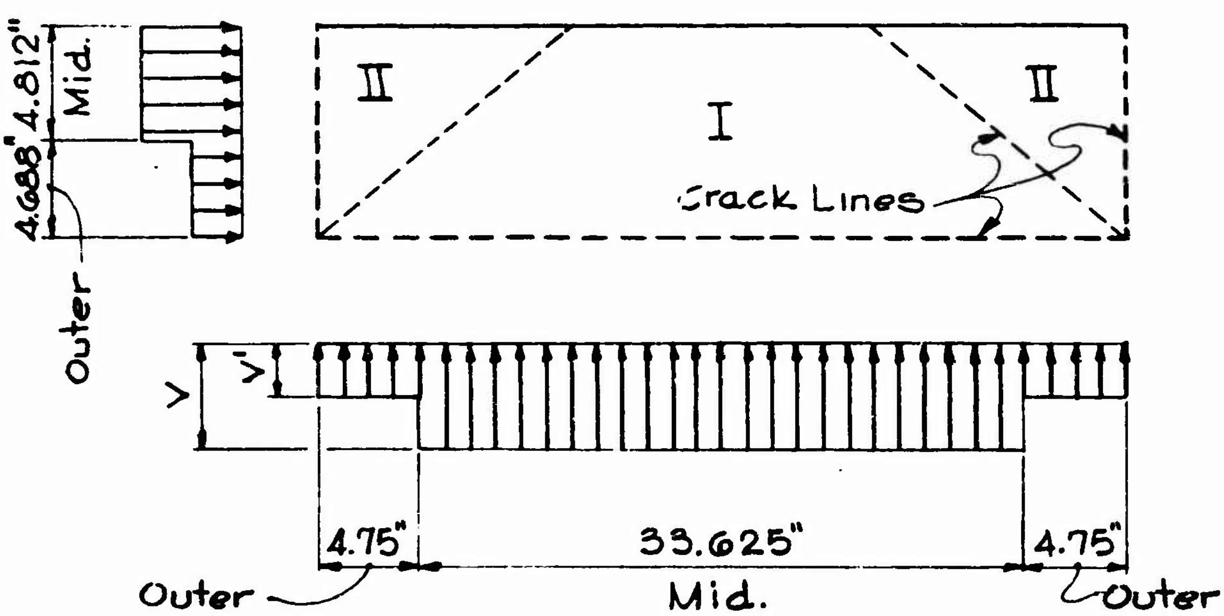


FIGURE B-7
DISTRIBUTION OF SHEAR FORCE ON WALL PANEL

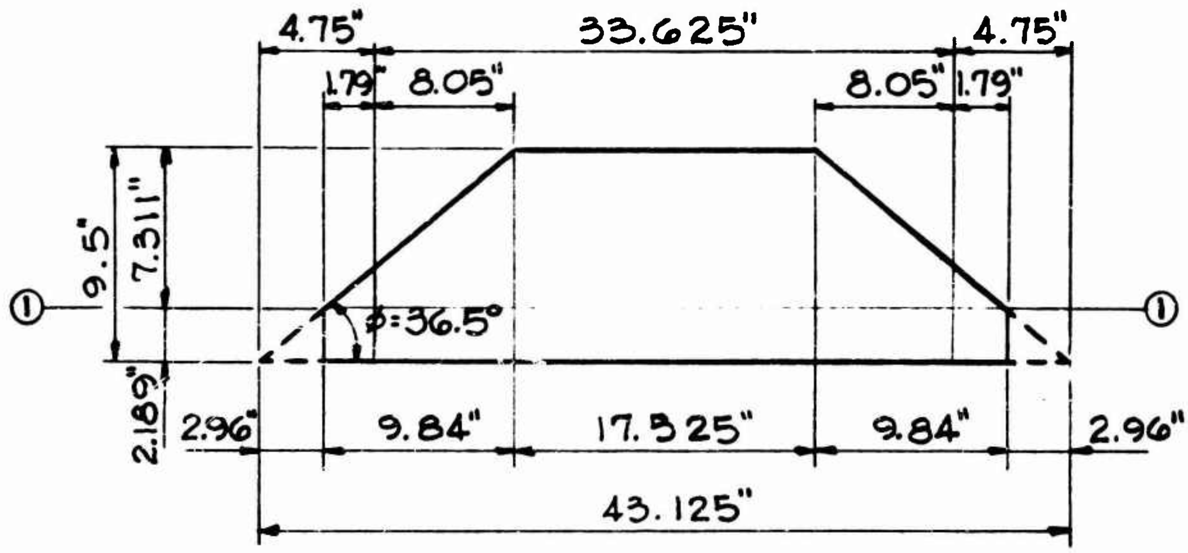


FIGURE B-8

LAYOUT OF WALL SECTION I SHOWING DIMENSIONS FOR SHEAR ANALYSIS

$$\text{Area above plane } ⓪\text{-}⓪ = 2(1/2)(9.84)(7.311) + 17.525(7.311)$$
$$= 200 \text{ inches}$$

$$\sum V_{⓪\text{-}⓪} = \text{Area above } ⓪\text{-}⓪ \times r_{av}$$

$$= 200\left[\frac{15.32}{144}\right] = 21.28 \text{ Kips}$$

<u>Shear per foot</u>

The ratio of the shear at the outer strip to the shear at the mid strip is assumed to be equal to the ratio of the unit moment at the outer strip to the unit moment at the mid strip.

$$\frac{V(\text{outer strip})}{V(\text{mid strip})} = \frac{m_{ULT}(\text{outer strip})}{m_{ULT}(\text{mid strip})} = \frac{13.6}{24.6} = 0.554$$

$$V(\text{outer strip}) = 0.554 V(\text{mid strip})$$

$$\sum V = \sum V(\text{outer strip}) + \sum V(\text{mid strip})$$
$$= 2(1.79)(0.554V) + 33.625V = 35.61 V$$

$$V = 21.28(1000)/35.61 = 598 \text{ lb/in}$$

<u>Maximum shear stress along plane ⓪-⓪</u>

$$v = \frac{V}{bd} \quad \text{(Reference 6)}$$

where

v = unit shear stress (psi)
V = unit shear of mid strip (lb. /in.)
b = width of one-inch-wide strip (inch)
d = d_w = weighted effective depth (inches)

$$v = \frac{598}{1(2.189)} = 274 \text{ psi}$$

<u>Total moment capacity of Section I about plane ⓪-⓪</u>

The total moment capacity of Section I about plane ⓪-⓪ is equal to the area of Section I above plane ⓪-⓪ times the distance between its centroid and the plane 0-0 times the average ultimate resistance of the panel.

$$\Sigma M_{ULT} = \left[2(1/2)(9.84)(1/3)(7.311)^2 + 1/2(17.525)(7.311)^2\right]\frac{15.32}{144}$$
$$= 68.5 \text{ Kip-in}$$

Total moment along plane 0-0

$$\Sigma M_{P\text{-}ULT} = \left[2(13.6)(1.79) + 24.6(33.625)\right](1/12) = 73.0 \text{ Kip-in}$$

$$\Sigma M_{N\text{-}ULT} = \Sigma M_{ULT} - \Sigma M_{P\text{-}ULT}$$
$$= 68.5 - 73.0 = -4.5 \text{ Kip-in}$$

where

13.6 = moment capacity of 13 gage reinforcement
24.6 = moment capacity of 10-1/2 gage reinforcement

Weighted percent reinforcement

The weighted percentage of reinforcement is to allow for variation in the reinforcement in the mid and outer strips of Section I.

$$p_w = \frac{A_s}{bd_w}$$

where

A_s = total area of reinforcement acting along plane square inches
b = length of plane 0-0 (inches)
d_w = weighted effective depth (inches)

$$p_w = \frac{2(4.75)(0.066) + 33.625(0.122)}{12(43.125)(2.189)} = 0.418\,\%$$

Allowable shear stress in reinforced concrete

$$v_c = 1.9\sqrt{f'_c} + 2{,}500 p_w \frac{Vd}{M} \quad \text{where } Vd \le M \text{ (Reference 8)}$$

where

v_c = shear stress in reinforced concrete (psi)
f'_c = ultimate static stress in concrete (psi)
V = total effective shear
d = d_w - weighted effective depth (inches)

$p = p_W$ = weighted percentage of reinforcement
M = moment capacity along plant ⓪–⓪

and

$$Vd = 21.28\,(2.189) = 46.8\ \text{Kip-in} > M = 4.5\ \text{Kip-in}$$

Therefore:

$$v_c = 1.9\sqrt{5840} + 2{,}500(.00418) = 155.6\ \text{psi}$$

Note: The allowable shear stress is less than the shear stress produced by the resistance of the panel. Therefore, the shear reinforcement is required.

Shear stress resisted by shear reinforcement (Figure B-9)

The shear reinforcement must resist the excess shear stress (v') above the allowable stress

$$v' = v - v_c = 274 - 155.6 = 118.4\ \text{psi}$$

Cross-sectional area of shear reinforcement

$$A_v = \frac{v'\,b\,s}{f_v(\sin\alpha + \cos\alpha)} \quad \text{(Reference 8)}$$

where

v' = excess shear stress (psi)
b = spacing of longitudinal reinforcement (inches) (Figure B-1)
f_v = average static stress in shear reinforcement (psi)
s = spacing between bends in shear reinforcement measured parallel to the longitudinal reinforcement (inches) (Figure B-1)
$\alpha = \tan^{-1}\frac{d'}{s}$ – angle of inclination of shear reinforcement with longitudinal reinforcement (Figure B-9)
d' = clear distance between longitudinal reinforcement (inches) (Figure B-1)

$$\tan\alpha = \frac{d'}{s} = \frac{2.436 - 2(0.188 + 0.127 - 0.039)}{2.5} = 0.754$$

$$\alpha = 37.0°$$

$$A_v = \frac{118.4\,(1.25)\,(2.5)}{98{,}000(0.062+0.79)} = 0.00269\ \text{in}^2$$

Note: 14-1/2 gage wire is used as shear reinforcement and has a cross-sectional area of 0.0047 in.^{2} which is greater than A_v. Therefore, adequate reinforcement is provided.

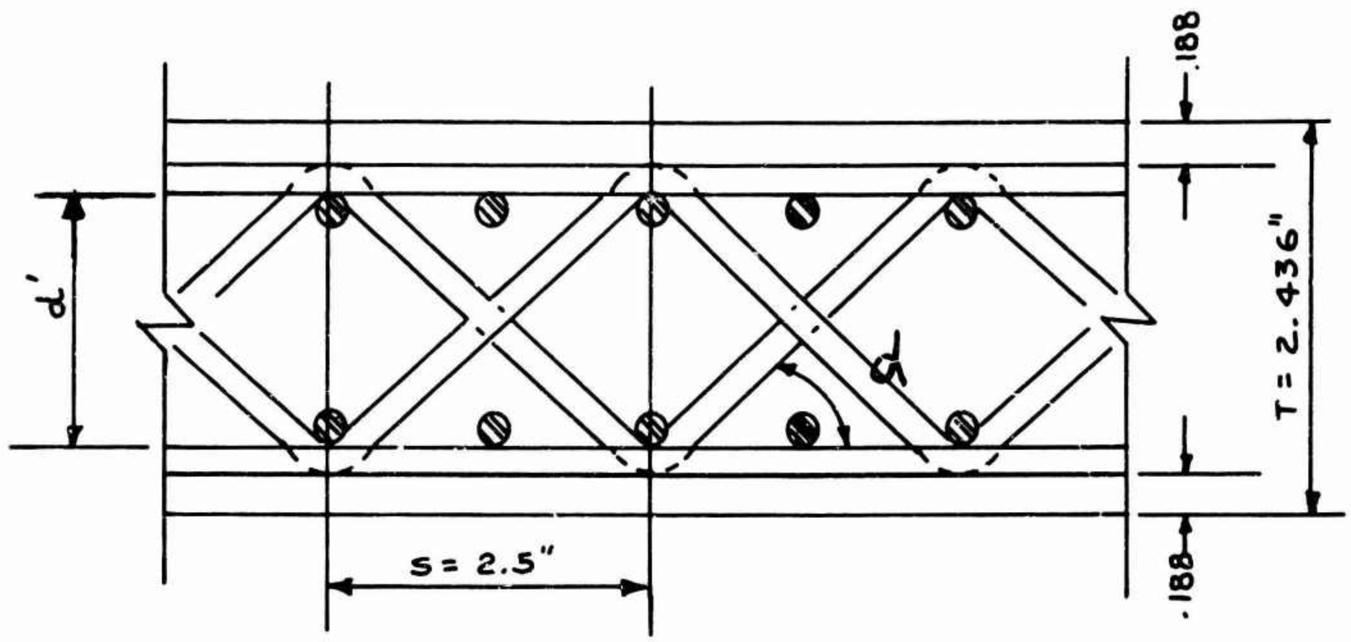


FIGURE B-9

CROSS SECTION OF WALL SHOWING SHEAR REINFORCEMENT

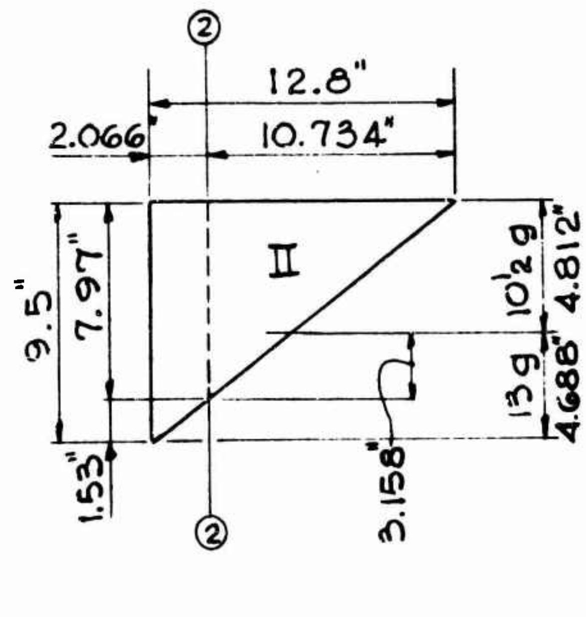


FIGURE B-10
LAYOUT OF WALL SECTION II SHOWING DIMENSIONS
FOR SHEAR ANALYSIS

Check of Shear Capacity of Section II (Figure B-10)

The analysis for determining the shear capacity of Section II (Figure B-10) is similar to that of Section I.

Weighted effective depth (Figure B-10)

$$d_w = \frac{4.688(2.074) + 4.812(2.058)}{9.5} = 2.065 \text{ in.}$$

Total effective shear of Section II

Area above plane ②-② $= 1/2(10.73)(7.97) = 42.9 \text{ in}^2$

$$\Sigma V_{②-②} = 42.9\left[\frac{15.32}{144}\right] = 4.55 \text{ Kips}$$

Shear per foot of Section II

$$V(\text{outer strip}) = \frac{12.83}{23.10} V(\text{mid strip}) = 0.555 V(\text{mid strip})$$

$$\Sigma V = 3.158(0.555V) + 4.812 V = 6.585 V$$

$$V = \frac{4.555(1000)}{6.585} = 693 \text{ lb/in}$$

Maximum shear stress along plane ②-② , Section II

$$v = \frac{693}{1(2.066)} = 335 \text{ psi}$$

Total moment capacity of Section II about plane ②-②

$$\Sigma M_{ULT}\left[1/6(7.97)(10.73)^2\right]\frac{15.32}{144} = 16.3 \text{ Kip-in}$$

Total moment along plane ②-②

$$\Sigma M_{P\text{-}ULT} = \left[12.83(3.158) + 23.1(4.812)\right]1/12 = 12.66 \text{ Kip-in}$$

$$\Sigma M_{N\text{-}ULT} = 16.3 - 12.66 = 3.64 \text{ Kip-in}$$

Weight percent reinforcement

$$p_w = \frac{4.688(0.066) + 4.812(0.122)}{12(9.5)(2.066)} = 0.38\,\%$$

Allowable shear stress in reinforced concrete

$$Vd = 4.55(2.066) = 9.5 \text{ Kip-in} > 3.64 \text{ Kip-in}$$

$$v_c = 1.9\sqrt{5840} + 2{,}500(.0038) = 154.7 < 335 \text{ psi}$$

Shear stress resisted by shear reinforcement

$$v' = 335 - 154.7 = 180.3 \text{ psi}$$

Cross-sectional area of shear reinforcement

$$\text{Tan}\,\alpha = \frac{2.436 - 2(0.188 - 0.039)}{2.5} = 0.85$$

$$\alpha = 40.4°$$

$$A_v = \frac{180.3(1.25)(2.5)}{98{,}000(0.65 + 0.76)} = 0.0041 \text{ in.}^2$$

$$A_v \text{ (Provided)} = 0.0047 > A_v \text{ (Required)} = 0.0041 \text{ in.}^2$$

Other Dynamic Properties of Each Panel

To determine the capacity of the wall in terms of resisting impulse loading, it is necessary to determine other dynamic properties of the wall. These include pertinent elastic properties of the wall and the resistance-deflection relationship prior to wall failure.

Determination of Resistance-Deflection Characteristics of Wall Panel (Figure B-11)

Each panel is originally supported by fixed edges at three sides and is free at the top. As the wall deflects, yielding occurs at certain locations which changes the support conditions when subject to further loading. Figure B-11 shows critical locations P_1, P_2 and P_3 where yielding first occurs. M_x indicates moment capacity in the vertical direction, the maximum being first developed in the reinforcement in the donor face at P_3. M_y indicates moment capacity in the horizontal direction, the maximum being first developed in the reinforcement in the donor face at P_2 and the acceptor face at P_1. Table B-1 which is based on information from Figures B-12-14 and Reference 9, Chapter 7, gives values for moment and deflection at the critical points for various support conditions.

Ultimate Moment Capacity in x and y Directions

$$M_{xULT} = 24.6 \text{ Kips-in/ft.} \quad , \quad M_{yULT} = 23.1 \text{ Kips-in/ft.}$$

Dynamic Properties at First Yield

Unit resistance "r" expressed in terms of wall height "H"

$$M_y(P_2) = M_{yULT} = 23.1 = 0.57\, r(P_2)\, H^2 ,$$
$$\therefore r(P_2) = 40.5/H^2$$

$$M_x(P_3) = M_{xULT} = 24.6 = 0.415\, r(P_3)\, H^2 ,$$
$$\therefore r(P_3) = 59.[illegible]/H^2$$

NOTE: Because r (P_2) < r (P_3) the reinforcement will yield at location 2 first. Therefore, the panel will them assume simple-simple-fixed-free support conditions.

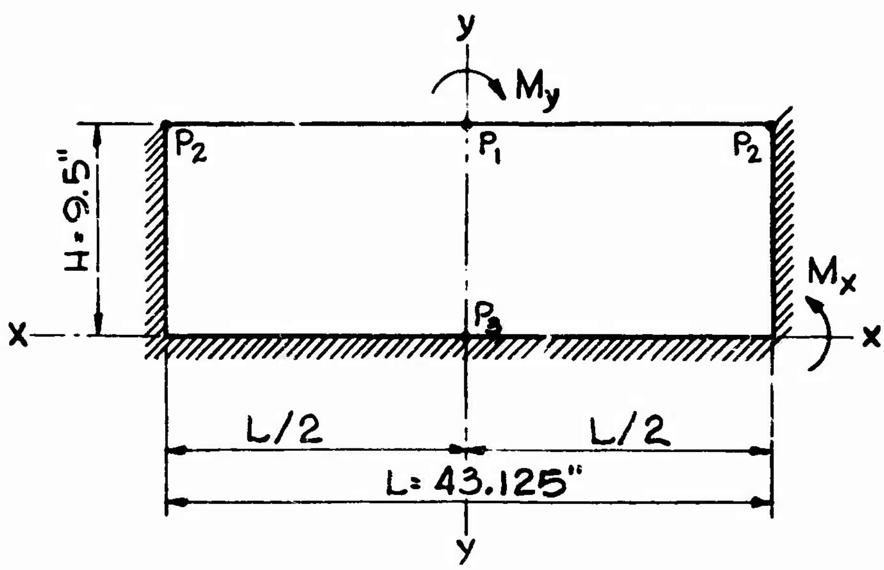


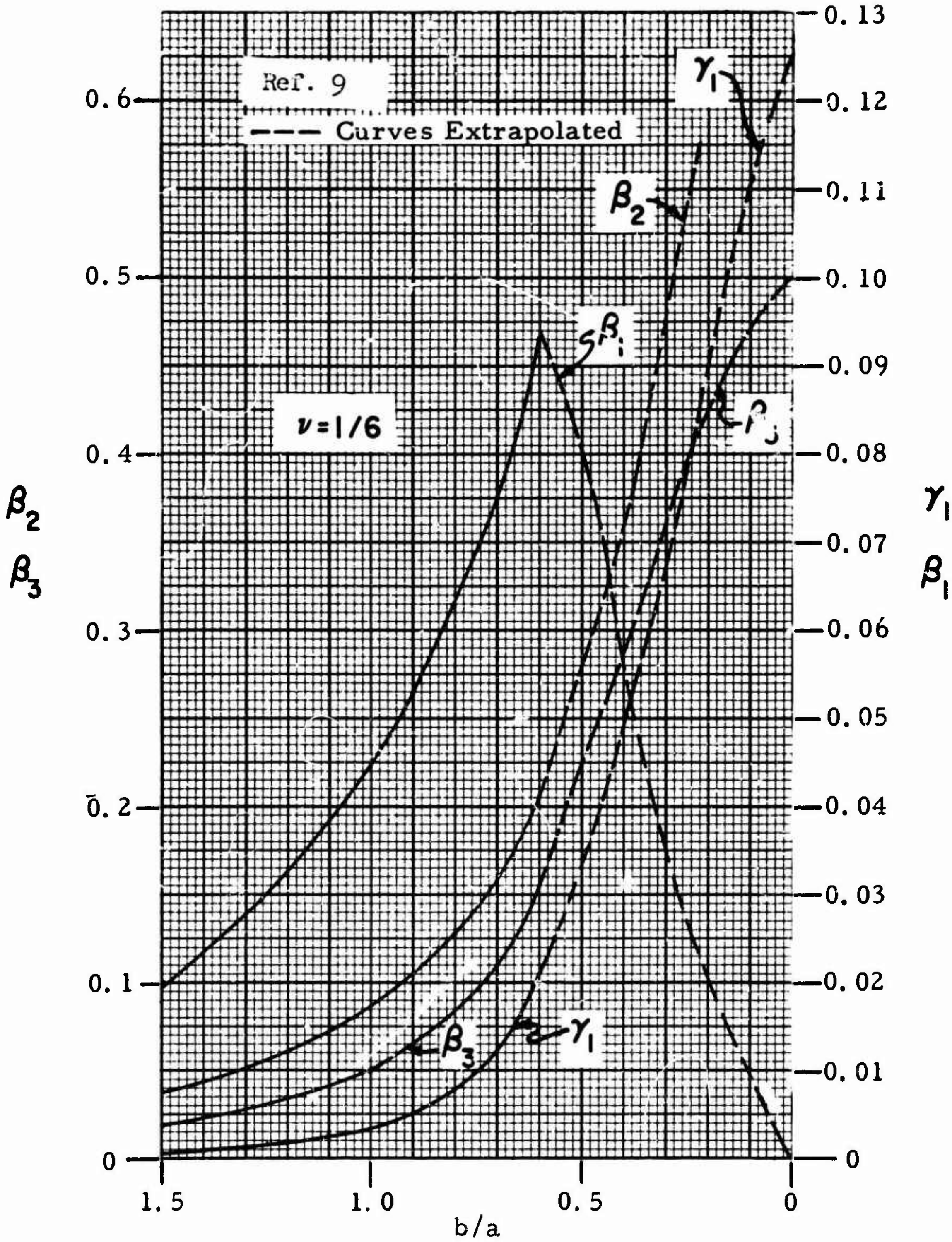
FIGURE B-11

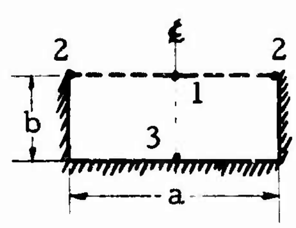
ELEVATION OF WALL PANEL SHOWING LOCATION OF CRITICAL POINTS IN ULTIMATE BENDING FAILURE

TABLE B-1 DYNAMIC PROPERTIES OF RECTANGULAR PLATES FOR VARIOUS SUPPORT CONDITIONS

Moment and Deflection	Three Sides Fixed and One Side Free	One Side Fixed, Two Sides Simple and One Side Free	Three Sides Simple and One Side Free
$M_y(P_1)$	$0.030\ r\ H^2$	$0.039\ r\ H^2$	$0.315\ r\ H^2$
$M_y(P_2)$	$0.570\ r\ H^2$	---	---
$M_x(P_3)$	$0.415\ r\ H^2$	$0.468\ r\ H^2$	---
$XEI(P_1)$	$0.085\ r\ H^4$	$0.110\ r\ H^4$	$0.800\ r\ H^4$

Note: Poisson's ratio for reinforced concrete is usually taken to vary between 0.10 and 0.167. In this analysis, Poisson's ratio has been varied between 0.167 and 0.3 depending on the wall support condition considered. Previous experience has indicated that the effect of Poisson's ratio may be considered negligible over the entire range from elastic to plastic behavior of the structural member.





$M = \beta r b^2$

$XD = \gamma r b^2$

FIGURE B-12

DEFLECTION AND MOMENTS FOR UNIFORMLY LOADED PLATE WITH THREE EDGES BUILT IN, ONE FREE

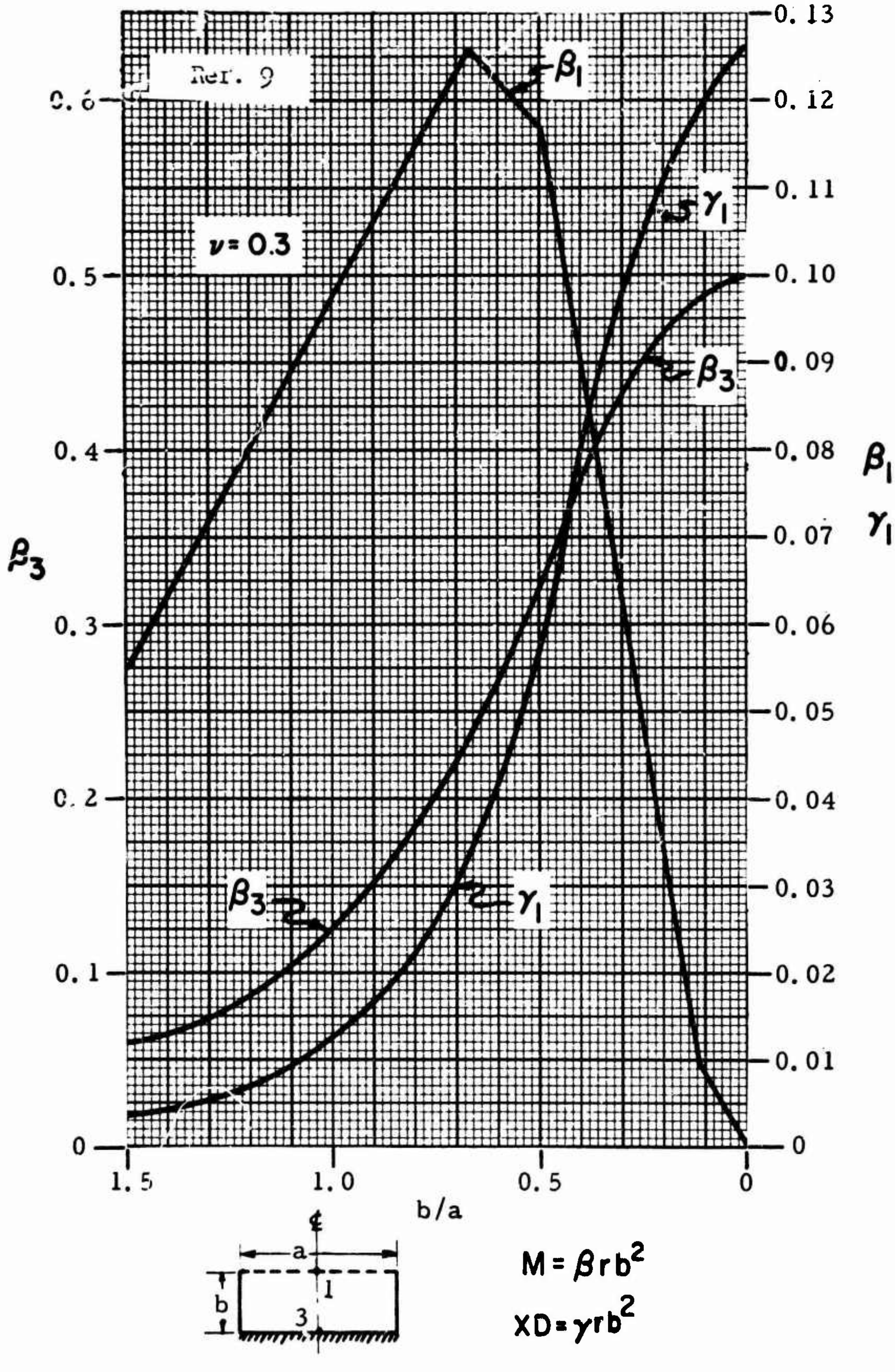


FIGURE B-13

DEFLECTION AND MOMENTS FOR UNIFORMLY LOADED PLATE WITH TWO OPPOSITE EDGES SIMPLY SUPPORTED, ONE FREE, ONE BUILT IN

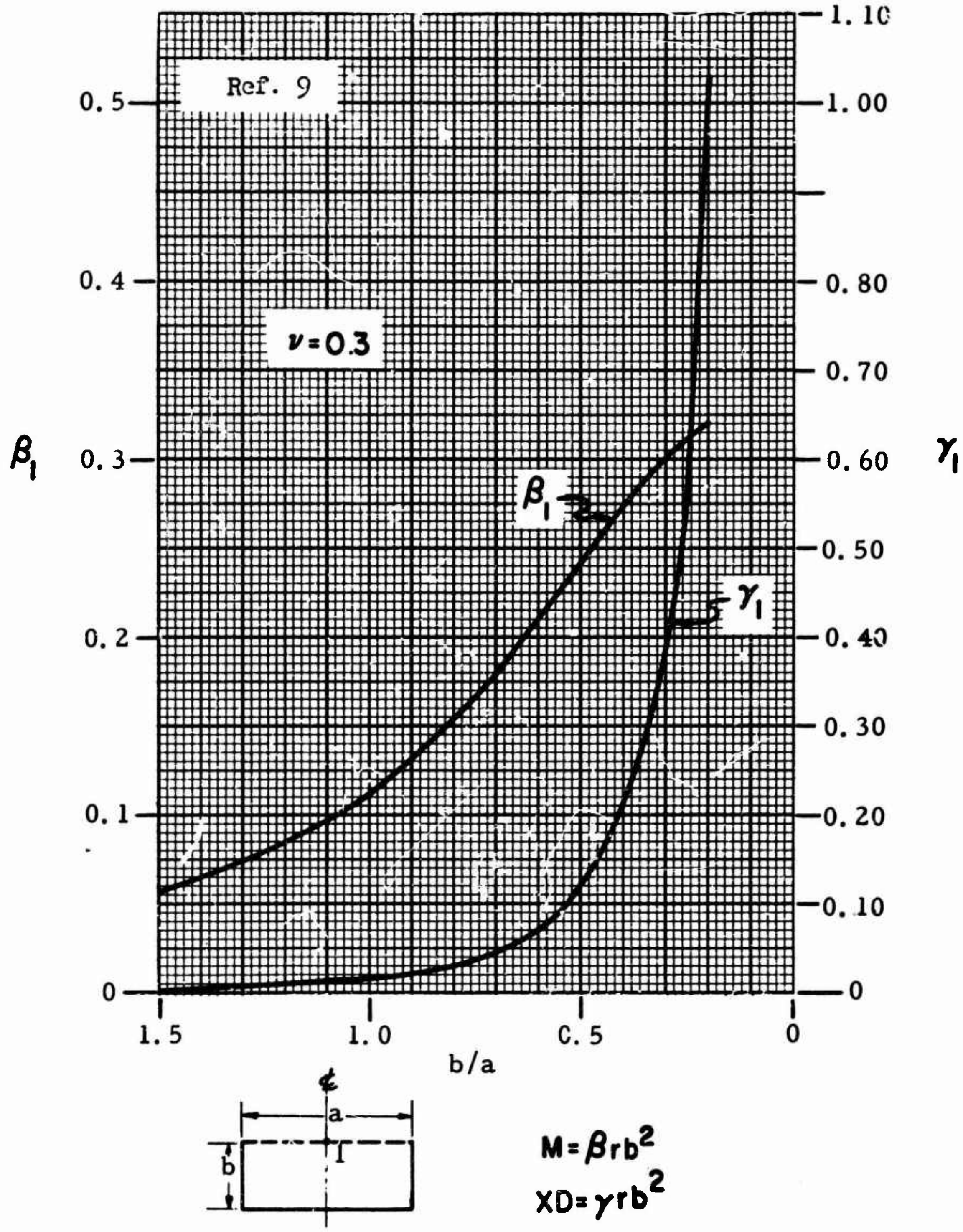


FIGURE B-14

DEFLECTION AND MOMENT FOR UNIFORMLY LOADED PLATE WITH THREE EDGES SIMPLY SUPPORTED, ONE FREE

Unit resistance at first yield (P_2)

$$r_1 = r(P_2) = 40.5\,(12)\,/\,(9.5)^2 = 53.9\ \text{Kips/ft}^2$$

Positive moment at P_1 at first yield

$$M_p(r_1) = 0.030\, r_1\, H^2 = 0.030\,(5.39)\,(9.5)^2 \frac{1}{12}$$
$$= 1.22\ \text{Kips-in/ft. (Table B-1)}$$

Negative moment at P_3 at first yield

$$M_N(r_1) = 0.415\, r_1\, H^2 = 0.415\,(5.39)\,(9.5)^2 \frac{1}{12}$$
$$= 16.8\ \text{Kips-in/ft. (Table B-1)}$$

Deflection at P_1 at first yield

$$X_1\, EI(P_1) = 0.085\, r_1\, H^4 = 0.085\,(5.39)\,(9.5)^4 \frac{1}{144}$$
$$= 25.91\ \text{Kip-in}^2\ \text{(Table B-1)}$$

Total resistance at first yield

$$R_1 = 5.39\,(9.5)(43.125)\frac{1}{144} = 15.3\ \text{Kips}$$

Properties at Second Yield

Change in unit resistance " Δ r" expressed in terms of "H"

$$M_y(P_1) = M_{yULT} - M_p(r_1) = 23.1 - 1.22 = 21.88$$
$$= 0.039\,\Delta r\,(P_1)\,H^2,\ \therefore\ \Delta r\,(P_1) = 561/H^2$$
$$M_x(P_3) = M_{xULT} - M_N(r_1) = 24.6 - 16.8 = 7.8$$
$$= 0.110\,\Delta r(P_3)H^2,\ \therefore\ \Delta r(P_3) = 71/H^2$$

NOTE: Because $\Delta r\,(P_3) < \Delta r\,(P_1)$, the reinforcement will yield at location 3 next. Therefore, the panel will then assume a simple-simple-simple-free support condition.

Change in unit resistance between first and second yield

$$\Delta r_2 = \Delta r\,(P_3) = 71\,(12)\,/\,9.5^2 = 9.44\ \text{K/ft}^2$$

Change in deflection at Location 1 between first and second yield

$$\Delta X_2 EI(P_1) = 0.110\,\Delta r_2 (H)^4 = 0.110\,(9.44)(9.5)^4 \frac{1}{144}$$
$$= 58.7 \text{ Kip-in}^2$$

Total deflection at Location 1 at second yield

$$X_2 EI(P_1) = 25.91 + 58.7 = 84.61 \text{ Kip-in}^2$$

Total resistance at second yield

$$R_2 = 15.3 + \frac{9.44}{5.39}(15.3) = 42.1 \text{ Kips}$$

Properties at Final Yield

Change in unit resistance " Δ r" between second and final yield

$$\Delta r_3 = r_{ULT} - r_1 - \Delta r_2 = 15.32 - 5.39 - 9.44 = 0.49 \text{ Kip/ft}^2$$

Deflection at P_1 at the final yield

$$\Delta X_3 EI(P_1) = 0.80\,\Delta r_3\, H^4 = 0.80(0.49)(9.5)^4 \frac{1}{144}$$
$$= 22.10 \text{ Kip-in}^2$$
$$X_3 EI(P_1) = 84.61 + 22.10 = 106.71 \text{ Kip-in}^2$$

Total resistance at final yield

$$R_3 = R_{ULT} = 43.6 \text{ Kips}$$

Normalize-Resistance-Deflection Curve

A plot of the normalize-resistant-deflection curve is given in Figure B-15. Also shown is a normalize-idealized resistant-deflection curve which represents an equivalent stress-strain condition for elastic distribution of the reinforcement.

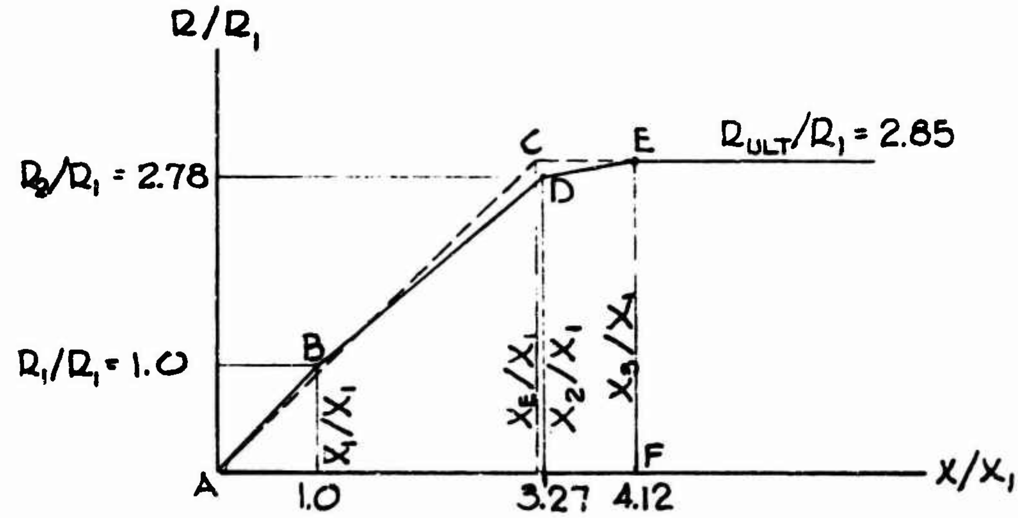


FIGURE B-15

ACTUAL AND IDEALIZED RESISTANCE -- DEFLECTION CURVE FOR WALL PANEL

Normalized values of R and X (Figure B-15)

$$R_2/R_1 = 42.1/15.3 = 2.78, \quad R_3/R_1 = 43.6/15.3 = 2.85$$
$$X_2/X_1 = 84.61/25.61 = 3.27, \quad X_3/X_1 = 106.71/25.91 = 4.12$$

The value of the deflection (X_E) of the idealized resistance-deflection curve can be determined by equating the areas ABDEF and ACEF, which represents the energy absorbed in the elasto-plastic range of action of the wall.

$$\text{Area ABDEF} = \frac{1}{2}(1)^2 + 1(2.27) + \frac{1}{2}(1.78)(2.27) + 2.78(0.85) + \frac{1}{2}(0.07)(0.85) = 7.18$$

$$\text{Area ACEF} = \frac{1}{2}(2.85)X_E/X_1 + 2.85(4.12 - \frac{X_E}{X_1})$$
$$= 11.74 - 1.43\,X_E/X_1$$

Therefore,

$$X_E/X_1 = \frac{11.74 - 7.18}{1.43} = 3.19$$

Determination of Effective Elastic Deflection

Modulus of elasticity

E_c - modulus for concrete
E_s - modulus for steel

$$E_c = 2{,}000{,}000 + 470\,f'_{dc}$$
$$= 2.0 \times 10^6 + 470(7{,}300) = 5.43 \times 10^6 \text{ (Ref. 10)}$$
$$E_s = 30 \times 10^6 \text{ (Ref. 11)}$$
$$n = E_s/E_c \text{ (Ref. 11)} = 30/5.43 = 5.5$$

Weighted percent reinforcement for entire panel

$$p_w = \frac{.00418(43.125) + 2(.0038)(9.5)}{43.125 + 2(9.5)} = .00406$$

Weighted value of d_w for entire panel

$$d_w = \frac{2.189(43.125) + 2(2.066)(9.5)}{62.125} = 2.15 \text{ in.}$$

Cracked moment of inertia (Reference 3, Chapter 3)

$F = .0152$ (See Fig. B-16)

$$I_c = Fbd^3 = 0.152(1)(2.15)^3 = 0.151 \text{ in.}^4/\text{in.}$$

Effective elastic deflection

$$X_E = 3.19 X_1 = \frac{3.19(25.91)}{5.43(0.151)} = 0.101 \text{ in.}$$

Effective Mass of Each Panel

Effective mass of Section I (Figure B-4 and 4) (Reference 5)

$$\frac{I_{\triangle}}{CL'} = \frac{\left[2(1/12)(12.8)(9.5)^3 + 1/3(17.525)(9.5)^3\right]m_{\triangle}}{\left[2(1/12)(12.8)(9.5) + (17.525)(9.5)\right]9.5(4.083)} = 0.61\, m_{\triangle}$$

where

$I_{\triangle}$ = area moment of inertia of Section I
$L' = H = 9.5$ in., $C = 4.083$ in.
$m_{\triangle}$ = unit mass of Section I

Effective mass of Section II (Figure B-5 and 6)

$$\frac{I_{\Delta}}{CL'} = \frac{\left[1/2(9.5)(12.8)^3\right]m_{\Delta}}{1/2(9.5)(12.8)(12.8)(4.27)} = 0.5\, m_{\Delta}$$

where

I_{Δ} = area moment of inertia of Section II
$L' = X = 12.8$ in.
$C = 4.27$ in.
m_{Δ} = unit mass of Section II

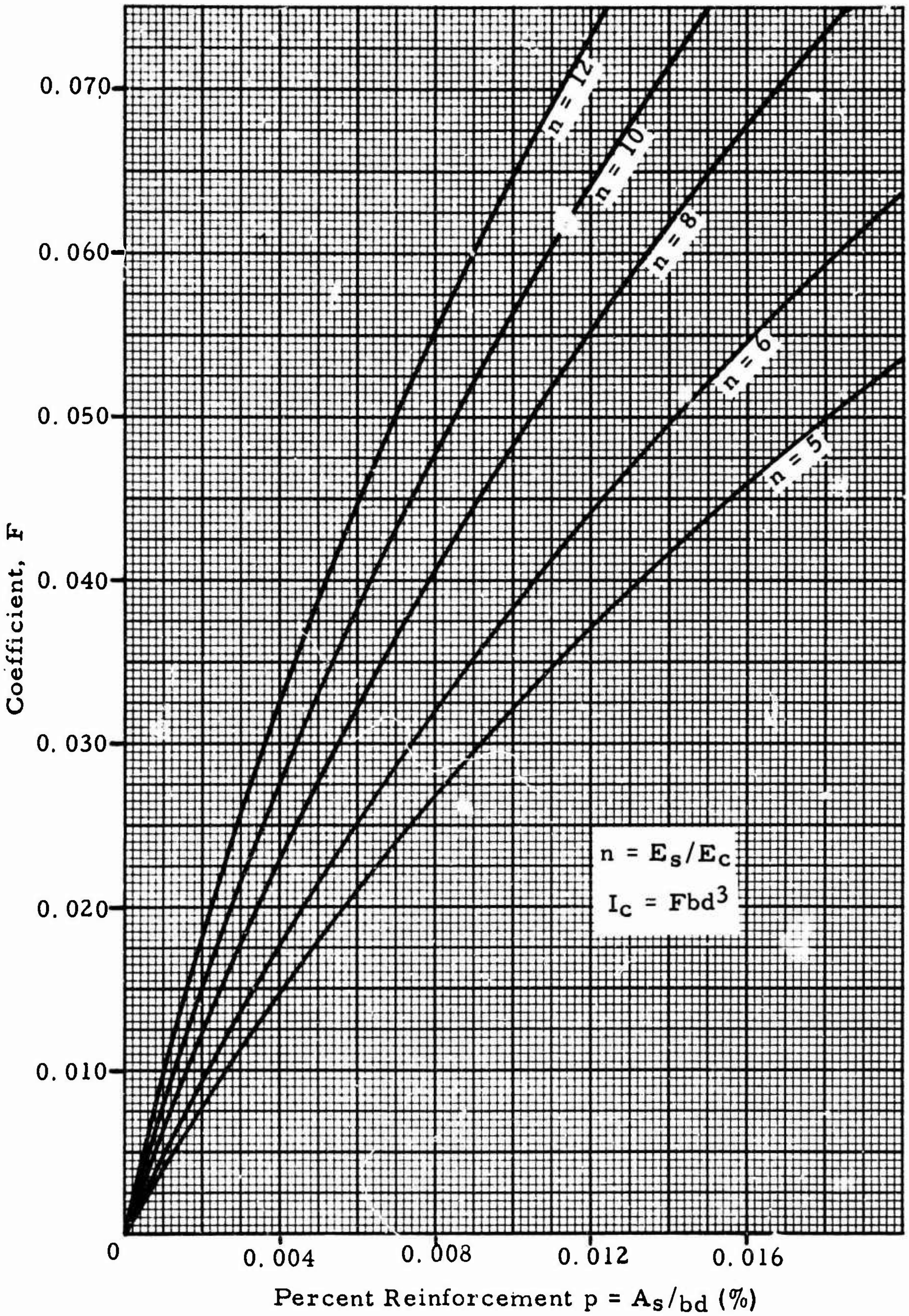


FIGURE B-15 COEFFICIENTS FOR MOMENT OF INERTIA

(CRACKED SECTIONS)

Weight per unit area of concrete and sand

$$W_c = 1 \times 1(2.436)(150)(1/12) = 30.4 \text{ psf of wall area}$$

$$W_s = \frac{100}{150}\left[\frac{4.75}{2.436}\right] 30.4 = 39.6 \text{ psf of wall area}$$

where;

Thickness of concrete panel	$= T_c$	= 2.436 in.
Thickness of sand	$= T_s$	= 4.75 in.
Density of concrete		= 150 lb./ft.^{3}
Avg. density of sand (after being compressed by motion of concrete)		= 100 lb./ft.^{3}

Total effective mass of the concrete portion of each panel

$$m_c = 0.61\, m_{\square} + 0.50\, m_{\Delta}$$

$$= \frac{[0.61(289) + 0.5(2)(61)]\,30.4}{32.2\,(144)} = 1.55\ \frac{\text{lb-sec}^2}{\text{ft.}}$$

Total effective mass of each panel

In the forthcoming analysis the mass of each panel of the wall is equal to the mass of each concrete panel plus the mass of one-half the sand separating the concrete.

$$m_D = m_A = m_C\left[\frac{W_c + 0.5\,W_s}{W_c}\right] = 1.55\left[\frac{30.4 + 0.5\,(39.6)}{30.4}\right]$$

$$= 2.56\ \frac{\text{lb-sec}^2}{\text{ft.}}$$

m_D = effective mass of donor panel (plus sand)

m_A = effective mass of acceptor panel (plus sand)

Dynamic Properties and Analysis of Wall

When explosive charges are detonated close-in to a protective barrier, the time required for the wall to reach yield (t_y), and the fictitious positive duration (t'_o) (Figure B-17A and 17B) are small in comparison to the time required to reach maximum deflection (t_m) of the wall. The elastic deflection (X_E) will usually be small in comparison with the maximum deflection of the wall (X_M) at incipient failure. Therefore, in this analysis, the values of t_y, t'_o and X_E are generally assumed to be equal to zero. One exception is in the determination of maximum deflection (X_M), where it is necessary to predict the exact point at which this maximum deflection occurs. In this case the value of X_E previously determined is multiplied by 35 to determine X_M. This factor is based on analysis of test results given in References 12 and 13.

Another observation from these references is the loss of resistance of the wall when rotations greater than 12^o occurred at the supports. In Figure B-17A the value of R_1 represents resistance before this point is reached and R_2 is the resistance after this point is reached and before ultimate failure.

The basic relationships for this analysis are:

$$F - R = m\ddot{X} \quad (1)$$

$$(F - R)\,dt = m\,dv \quad (2)$$

$$I = mv \quad (3)$$

$$K.E. = 1/2\ mv^2 \quad (4)$$

$$P.E. = \Sigma\ (RX) \quad (5)$$

Where

F = applied load in lbs.

R = potential resistance in the wall in opposing the load in lbs.

m = effective mass of the wall

$\ddot{X}$ = acceleration of the wall movement in ft./sec.^{2}

t = time in seconds

v = velocity of wall movement in ft./sec.

I = impulse load on the wall in lb.-sec.

K.E. = kinetic energy of applied blast loading lb.-ft.

P.E. = potential energy of wall in resisting loading lb.-ft.

X = deflection of wall in inches

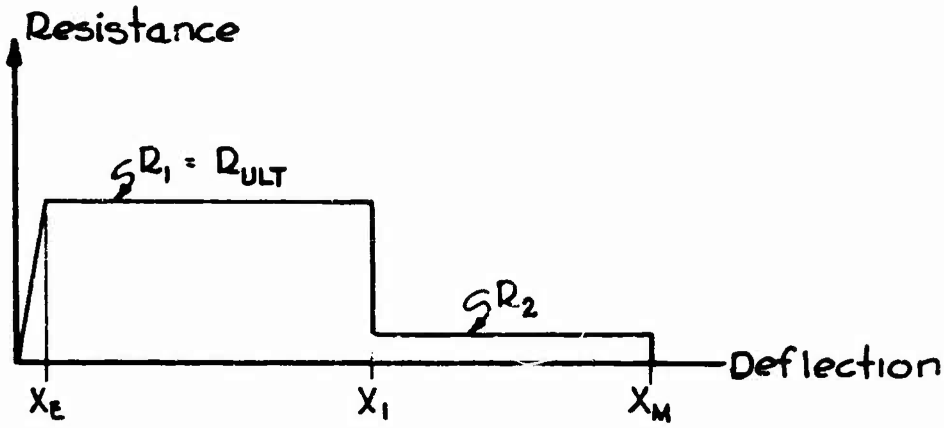


FIGURE B-17A

RESISTANCE-DEFLECTION CURVE

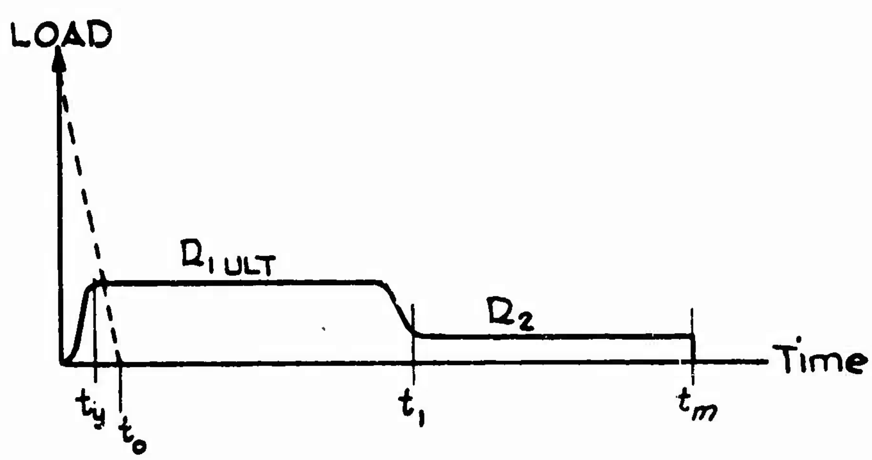


FIGURE B-17B

RESISTANCE TIME CURVE FOR BACK WALL
CARRIED TO ULTIMATE FAILURE

Determination of Impulse Capacity of the Wall

Combining Equations (3), (4) and (5)

K. E. = P. E. at incipient failure

$$\frac{I^2}{2m_p} = K.E.$$

and

$$I^2 \doteq 2m_p \, \Sigma(RX) = 2\,m_p\left[R_1 X_1 + R_2(X_M - X_1)\right]$$

where

m_p = effective mass of panel in plastic range
$\Sigma(RX)$ = area under resistance-deflection curve

$X_M = 35X_E = 35(.101) = 3.54$ in.
$R_1 = 43.6$ Kips , $r_1 = 15.32$ Kips/ft^2

Deflection of 12° rotation of each section

Section I . . . . X_1 = Tan 12° (9.5) = 2.02 in. < X_M
Section II . . . . X_1 = Tan 12° (43.125/2) = 4.58 in. > X_M

Reduced resistance of each panel due to excessive rotation at base

After failure at their base, each panel will span between the side walls as a one-way spanning member (Reference 1).

$$r_2 = \frac{16\,\Sigma M_{N\text{-}ULT}}{L^2 H} = \frac{16(14.27)(12)}{(43.125)^2(9.5)} = 0.0156 \text{ K/in}^2$$
$$= 2.25 \text{ K/ft}^2$$

$$R_2 = R_1\left[\frac{r_2}{r_1}\right] = 43.6\left[\frac{2.25}{15.32}\right] = 6.40 \text{ K}$$

Impulse capacity of each panel

$$I_{CD}^2 = I_{CA}^2 = 2(2.56)(1/12)\left[43{,}600(2.02)+6{,}400(3.56-2.02)\right](10^6)$$
$$= 4.25\times10^{10}\ (\text{lb-ms})^2$$

$$I_{CD} = I_{CA} = \sqrt{4.25\times10^{10}} = 2.10\times10^5\ \text{lb-ms}$$

Where

I_{CD} = impulse capacity of donor panel
I_{CA} = impulse capacity of acceptor panel

Unit impulse capacity of each panel

$$i_{CD} = i_{CA} = 2.10\times10^5/(9.5)(43.125) = 514\ \text{lb-ms}$$

Scale unit impulse capacity of each panel

Assume charge weight of 7 lb. (trial).

$$\bar{i}_{CD} = \bar{i}_{CA} = i/W^{1/3} = 514/(7)^{1/3} = 270\ \frac{\text{psi-ms}}{\text{lb}^{1/3}}$$

Scaled thickness of sand and concrete

$$\bar{T}_c = \frac{T_c}{W^{1/3}} = 2.436/(12)(1.92) = 0.106\ \frac{\text{ft}}{\text{lb}^{1/3}}$$

$$\bar{T}_s = \frac{T_s}{W^{1/3}} = 4.75/(12)(1.92) = 0.208\ \frac{\text{ft}}{\text{lb}^{1/3}}$$

Attenuated impulse load through sand

With the use of the values of $\bar{i}_{CD}$, $\bar{T}_c$ and $\bar{T}_s$, the impulse load which is attenuated through the sand is determined from Figure B-18.

$$\bar{i}_a = 400\ \frac{\text{psi-ms}}{\text{lb}^{1/3}}$$

Where

$\bar{i}_a$ = permissible scale impulse load acting on the sand at the acceptor surface of the donor panel. $\bar{i}_a$ includes the impulse attenuated by the sand and the impulse absorbed by the acceptor panel.

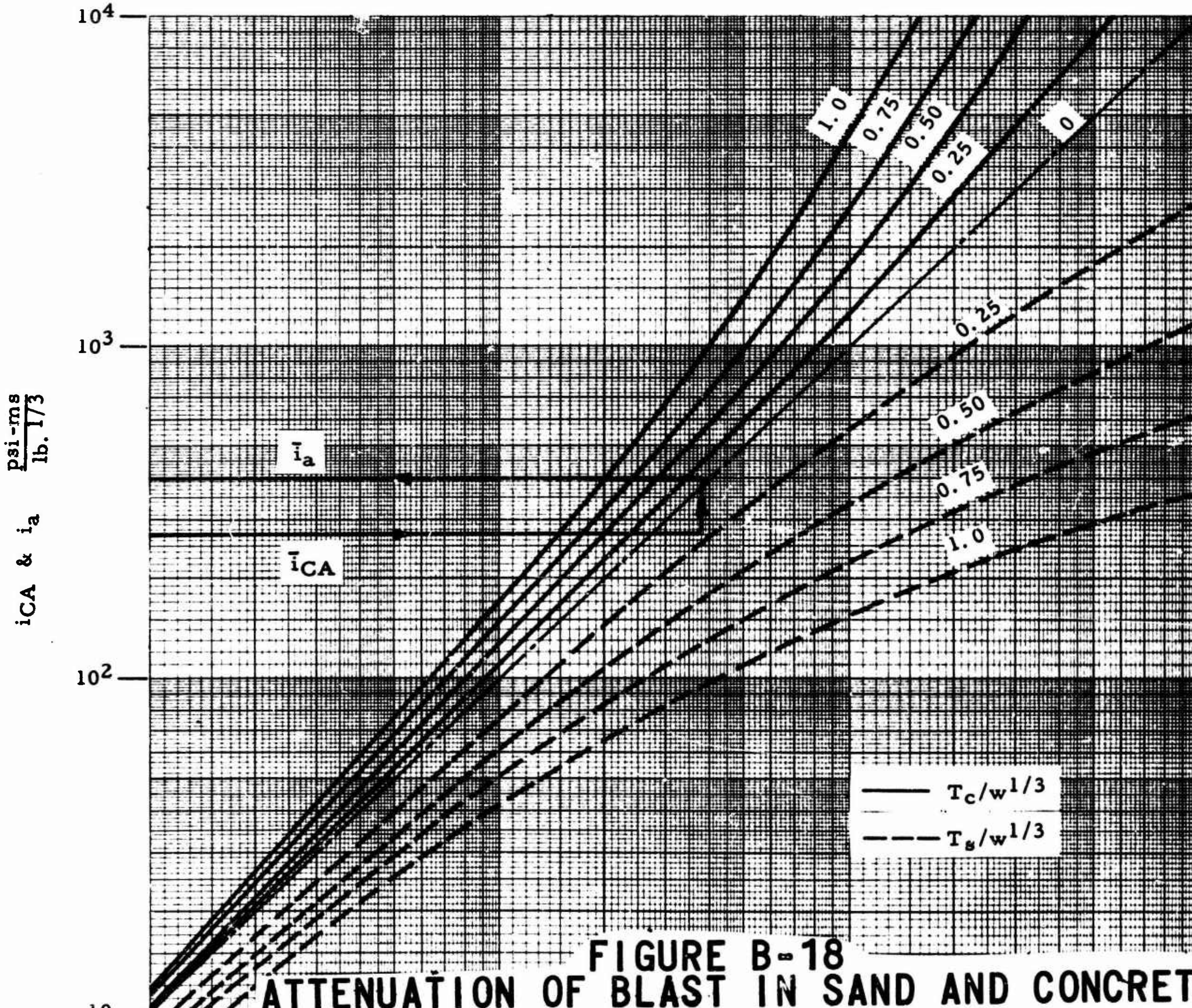


FIGURE B-18
ATTENUATION OF BLAST IN SAND AND CONCRETE

Total scale unit impulse capacity of wall

$$\bar{i}_T = \bar{i}_{CD} + \bar{i}_a = 270 + 400 = 670 \frac{\text{psi-ms}}{\text{lb}^{1/3}}$$

Determination of Blast Impulse Load Acting on Back Wall (B-19)

The blast impulse load acting on the back wall is determined from Figure B-20 using the following:

L = 4 = length of back wall in feet
H = 1 = height of back wall in feet
h = 0.5 = height of charge above floor slab in feet
ℓ = 2 = location of charge relative to side wall in feet
R = 1 = location of charge relative to back wall in feet
$Z_A = R/W^{1/3}$ = normal scaled distance of charge relative to back wall in ft./lb.$^{1/3}$
$Z_A = L/2W^{1/3}$ = scaled distance of center of back wall relative to side wall in ft./lb.$^{1/3}$

Therefore,

$L/H = 4/1 = 4$

$\ell/L = 2/4 = 0.5$

$h/H = \frac{0.5}{1} = 0.5$

$Z_A = \frac{1}{7^{1/3}} = 0.52 \text{ ft/lb}^{1/3}$

$Z_B/Z_A = L/2R = 4/2 = 2$

Unit scaled impulse load (Figure B-20)

$$\bar{i}_b = 680 \frac{\text{psi-ms}}{\text{lb}^{1/3}}$$

Check of impulse load and capacity

For incipient failure condition the scale impulse capacity of the wall is equal to the scale impulse load of the blast.

$$\bar{i}_T = 670 \approx \bar{i}_b = 680 \frac{\text{psi-ms}}{\text{lb}^{1/3}}$$

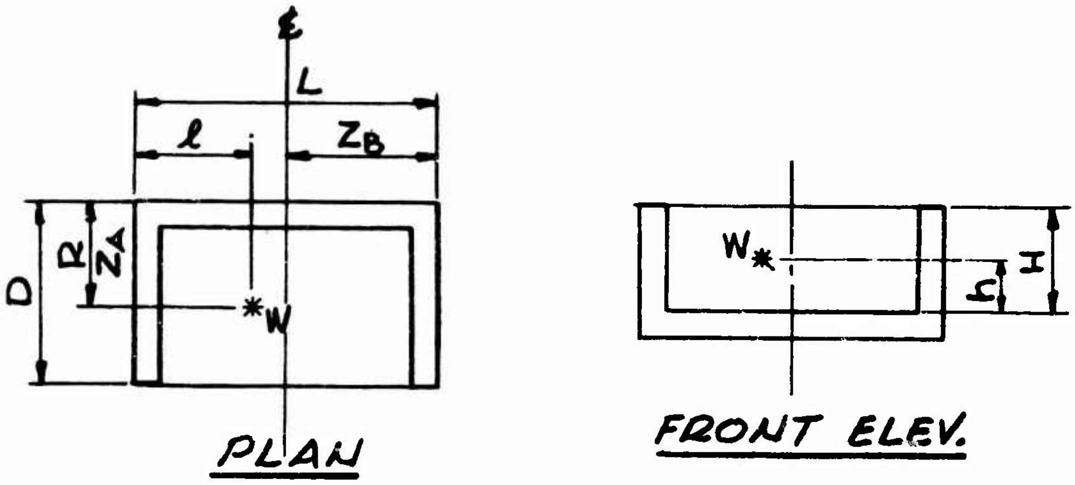


FIGURE B-19

LAYOUT OF BAY STRUCTURE SHOWING VARIOUS PARAMETERS THAT AFFECT IMPULSE

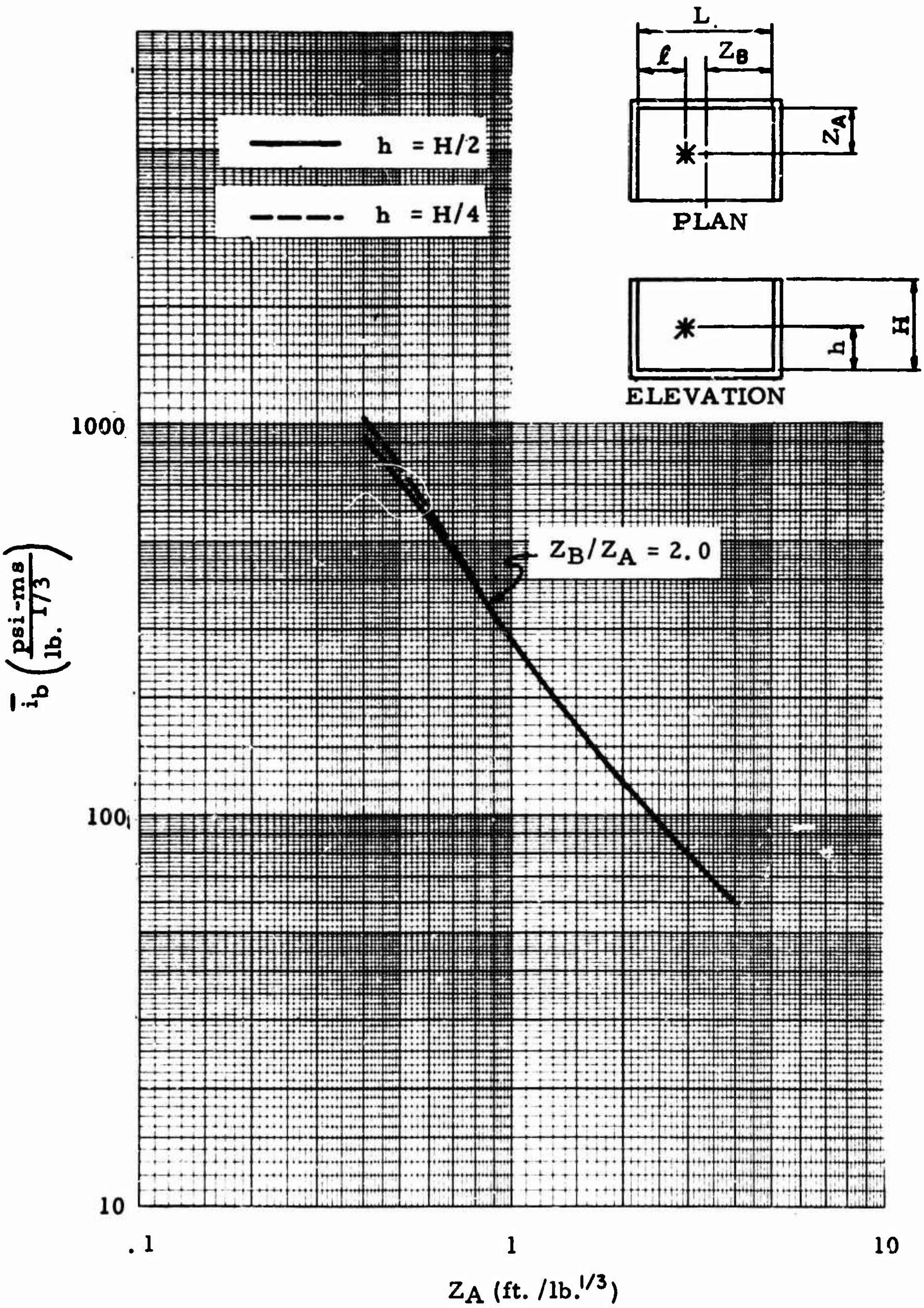


FIGURE B-20

AVERAGE IMPULSE ON BACK WALL DUE TO REFLECTIONS OFF FLOOR SLAB AND TWO SIDE WALLS (FOR L/H = 4 AND ℓ/L = 1/2)

NOMENCLATURE

A_s	Area of flexural steel
A_v	Area of shear reinforcement
a	(Figures B-12-14) also depth of compression stress block of flexural member
α	Angle between inclined web bars (shear reinforcement) and longitudinal axis of member
β	Moment coefficient (Figures B-12-14)
γ	Deflection coefficient (Figures B-12-14)
b	(Figures B-12-14) and width of compression stress block of flexural member
C	Centroidal distance, distance from centroid of a section of a panel to axis of rotation
D	$\frac{EI}{(1-\nu^2)}$; also depth of cell
d	Distance from extreme compressive fiber to centroid of tension force in tensile reinforcement
d_w	Weighted value of d
E	Modulus of elasticity
E_c	Modulus of elasticity of concrete
E_s	Modulus of elasticity of steel
F	Parameter used in Figure B-16; also force acting on a member
f_{ds}	Dynamic unit stress of flexural reinforcement (average of yield and ultimate stresses)
f_s	Static unit stress of flexural reinforcement (average of yield and ultimate stresses)

f_v	Static unit stress of shear reinforcement (average of yield and ultimate stresses)
f'_c	Static ultimate compressive stress of concrete
f'_{dc}	Dynamic ultimate compressive stress of concrete
H	Height of wall
h	Height to center of charge above floor slab
I	Moment of inertia and impulse
I_c	Moment of inertia of cracked section
I_{CA}	Total impulse capacity of acceptor panel
I_{CD}	Total impulse capacity of donor panel
i_{CA}	Unit impulse capacity of acceptor panel
$\bar{i}_{CA}$	Scaled unit impulse capacity of acceptor panel
i_{CD}	Unit impulse capacity of donor panel
$\bar{i}_{CD}$	Scaled unit impulse capacity of donor panel
I_T	Total impulse capacity of wall
$\bar{i}_a$	Scaled impulse attenuated by sand plus impulse capacity of acceptor panel
$\bar{i}_T$	Total scaled unit impulse capacity of the wall
$\bar{i}_b$	Scaled unit impulse of blast load
K. E.	Kinetic energy
L	Length of wall
ℓ	Location of charge
M_{ULT}	Ultimate capacity of a section of a panel

m_{ULT}	Ultimate capacity of a section of a panel per foot
$M_{N\text{-}ULT}$	Ultimate capacity of negative reinforcement
$m_{N\text{-}ULT}$	Ultimate capacity of negative reinforcement per foot
$M_{P\text{-}ULT}$	Ultimate capacity of positive reinforcement
$m_{P\text{-}ULT}$	Ultimate capacity of positive reinforcement per foot
M_x	(Table B-1)
M_y	(Table B-1)
m	Mass
m_A	Mass of acceptor panel
m_c	Mass of concrete panel (plastic mass of equivalent single degree of freedom system)
m_D	Mass of donor panel
n	E_s/E_c
ν	Poisson's ratio
P	Coordinate of a point on a panel
P. E.	Potential energy
p_w	Weighted percent of reinforcement
R	Total resistance of member
R_{ULT}	Total ultimate resistance of member
r	Unit resistance of member
r_{av}	Average unit resistance of total wall
r_{ULT}	Unit ultimate resistance of member

s	Spacing of bends in shear reinforcement in a direction parallel to the longitudinal reinforcement
T_c	Thickness of concrete panel
$\bar{T}_c$	Scaled thickness of concrete panel
T_s	Thickness of sand
$\bar{T}_s$	Scaled thickness of sand
t	Time
t_m	Time to reach maximum deflection
t_y	Time to reach yield
t'_o	Fictitious duration of positive phase of blast load
V	Total shear force
v	Unit shear force
v_c	Unit shear stress capacity of the concrete
v'	Unit shear stress capacity of the stirrups
W	Weight of spherical charge
x	Deflection
$\ddot{X}$	Acceleration of the mass
X_E	Effective elastic deflection
X_M	Maximum deflection
Z_A	Normal scaled distance between the charge and the wall in question
Z_B	Scaled distance between the center of the wall in question and the adjacent reflecting surface(s)

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13. ABSTRACT

Three tests of 1/10 scale specially reinforced concrete test bay were performed to evaluate the explosive capacity of a specific cubicle arrangement and as a first step in establishing the validity of scaled testing of concrete structures. The tests consisted of firing 2.00, 3.24 and 4.25-lb. charges of Composition B in the center of the test bay and evaluating the extent of damage to the bay to estimate its ultimate explosive capacity.

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