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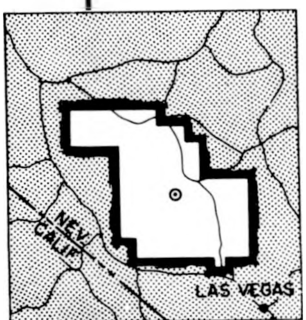
PRELIMINARY REPORT

OPERATION PLUMBBOB



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Project 30.2

RESPONSE OF DUAL-PURPOSE REINFORCED-CONCRETE MASS SHELTER

Issuance Date: November 8, 1957

CIVIL EFFECTS TEST GROUP

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This is a preliminary report based on all data available at the close of this project's participation in Operation PLUMBBOB. The contents of this report are subject to change upon completion of evaluation for the final report. This preliminary report will be superseded by the publication of the final (WT) report. Conclusions and recommendations drawn herein, if any, are therefore tentative. The work is reported at this early time to provide early test results to those concerned with the effects of nuclear weapons and to provide for an interchange of information between projects for the preparation of final reports.

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Operation PLUMBBOB Preliminary Report

Project 30.2

RESPONSE OF DUAL-PURPOSE REINFORCED-CONCRETE MASS SHELTER

By

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Approved by:	Hal J. Jennings Director Program 30	Approved by:	R. L. Corsbie Director Civil Effects Test Group

Ammann & Whitney

September 1957

ABSTRACT

Project 30.2 was conducted to test a reinforced-concrete dual-purpose underground parking garage and personnel shelter designed for a long-duration incident pressure of 30 psi. The estimated peak incident pressure at the structure was 42 psi.

To facilitate postshot analyses, soil borings were made to obtain undisturbed samples for determining soil characteristics.

Preshot and postshot field surveys were made to determine the total lateral and vertical displacement of the structure.

Blast instrumentation consisted of Wiancko pressure gauges, Carlson earth-pressure gauges, dynamic-pressure gauges, and a self-recording pressure gauge. Structural response was recorded by Ballistics Research Laboratory gauges.

Radiation measurements were taken using film dosimeters, gamma-radiation chemical dosimeters, and one gamma-rate telemetering unit.

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Chapter 1

INTRODUCTION

1.1 OBJECTIVES

The primary objective of Project 30.2 was to evaluate the capabilities of a reinforced-concrete dual-purpose underground parking garage and personnel shelter in providing protection against effects of a nuclear detonation. Secondary objectives were: to obtain additional information regarding blast load transmitted to underground structures, information regarding reflected and dynamic pressures in the ramp and on the entranceway door, data on nuclear-radiation attenuation characteristics of the structure, and to check assumptions used in design procedures.

Structure 30.2, a typical full-scale section of the prototype, was the largest shelter tested in Operation Plumbbob and was located at the predicted 35-psi peak overpressure level.

1.2 BACKGROUND

In the summer of 1956, the Federal Civil Defense Administration contracted with Ammann & Whitney, Consulting Engineers, to prepare a preliminary layout for a dual-purpose reinforced-concrete underground parking garage and shelter and to design a structurally representative portion of such a structure to be exposed to nuclear blast for test purposes.

Studies were made of prototype architectural layouts and various types of roof framing including (1) flat-slab system with drop panels, (2) two-way slab systems with girders of various depths, and (3) hipped-plate construction. After consulting with FCDA, the structure was designed using a flat-slab roof system. Figure 1.1 shows the prototype layout of the flat-slab type construction.

The blast requirements of the prototype structure specified by FCDA were to (1) resist the predicted effects of a 30-psi incident shock produced by a megaton-range weapon and (2) provide a minimum of 4 ft 6 in. of concrete or 2 ft 6 in. of concrete plus 3 ft of earth for protection against nuclear radiation.

To compensate for the relatively short duration of the test loading compared to design loading, the test structure was located at the predicted 35-psi level rather than the 30-psi design level. The peak incident pressure measured during the test was 42.0 psi.

1.3 TEST STRUCTURE DESCRIPTION

The test section, shown in Fig. 1.2 was a below-grade flat-slab structure with an interior floor area of 7569 sq ft (87 x 87 ft) and nine interior columns 29 ft on center (Fig. 1.3). Access was by a 14-ft-wide vehicular ramp along one side of the structure (Figs. 1.4 and 1.5). The roof slab was 3 ft below grade. Walls of the structure were 12 in. thick, except for the exposed wall along the ramp which was 4 ft 6 in. thick for radiation protection. The floor slab was 9 in. thick, and the roof slab, 30 in. thick with 14-in. drop panels 14 ft square. The vertical load was carried to the foundation by circular concrete columns 33 and 36 in. in diameter. The footings were two-way slabs 15 ft to 16 ft 9 in. square. However, the footing near the entranceway was a two-column continuous member. Maximum thickness of the square footings varied from 3 ft 11 in. to 4 ft 3 in. The entranceway to the structure (Figs. 1.6, 1.7 and 1.8) was protected by a reinforced-concrete rolling door 29 ft long by 10 ft high by 4 ft 6 in. thick, which was sealed around the perimeter by a 3-in.-wide inflatable rubber gasket.

Operating equipment for the door was not included in this test, and space for maintenance around the door was kept to a minimum. Although the operation of the test door required more effort than was anticipated in the design, this could have been greatly reduced by minor field adjustments to improve the as-built tolerances, alignment, smoothness, and lubrication of the door and frame.

A personnel escape exit had originally been included in the design but was deleted because personnel exits either were included in other projects or had been previously tested.

The structure was oriented with the center line of the ramp radial to Ground Zero as shown in Fig. 1.9.

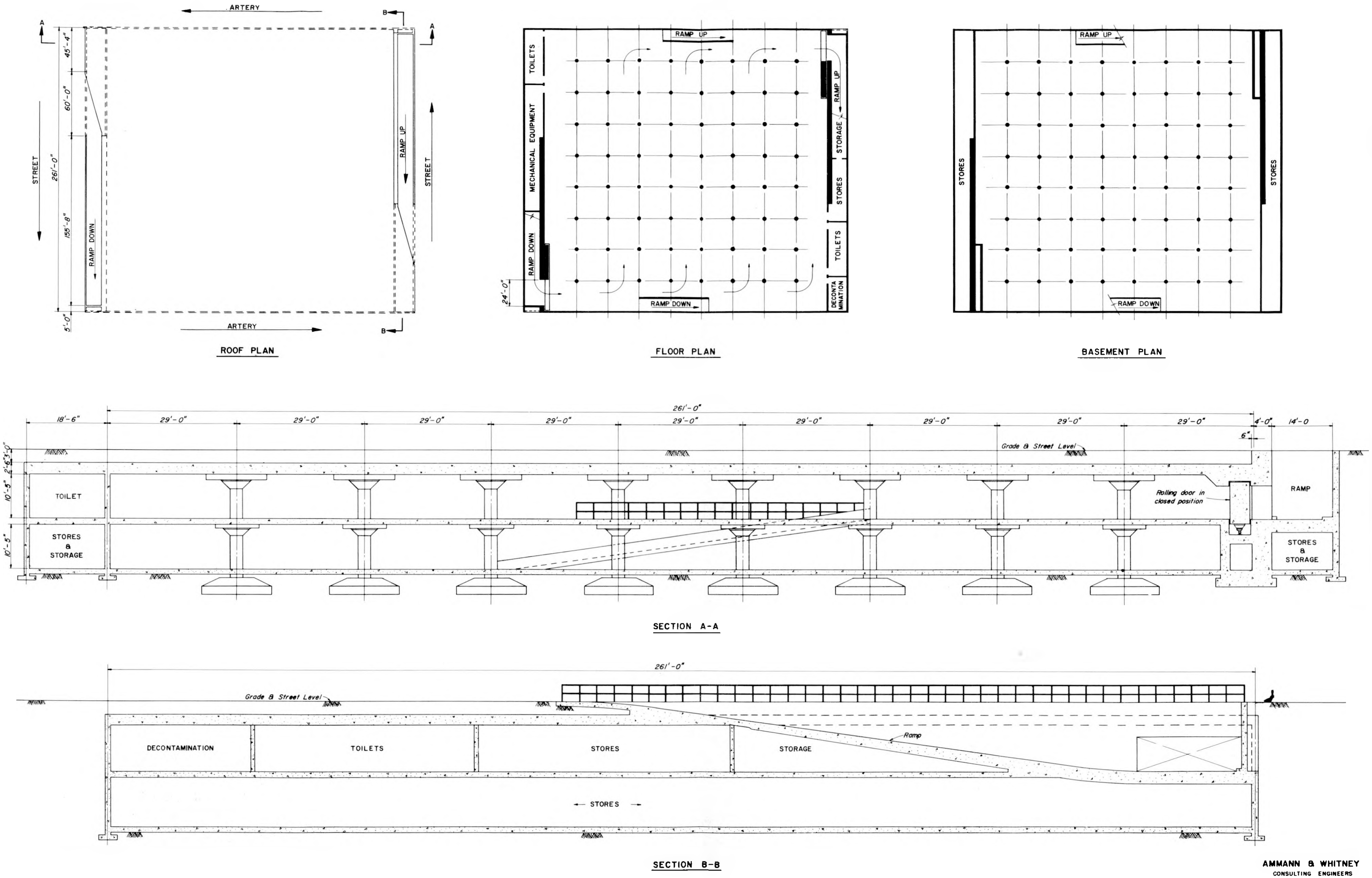


Fig. 1.1—Preliminary layout of prototype flat-slab structure.

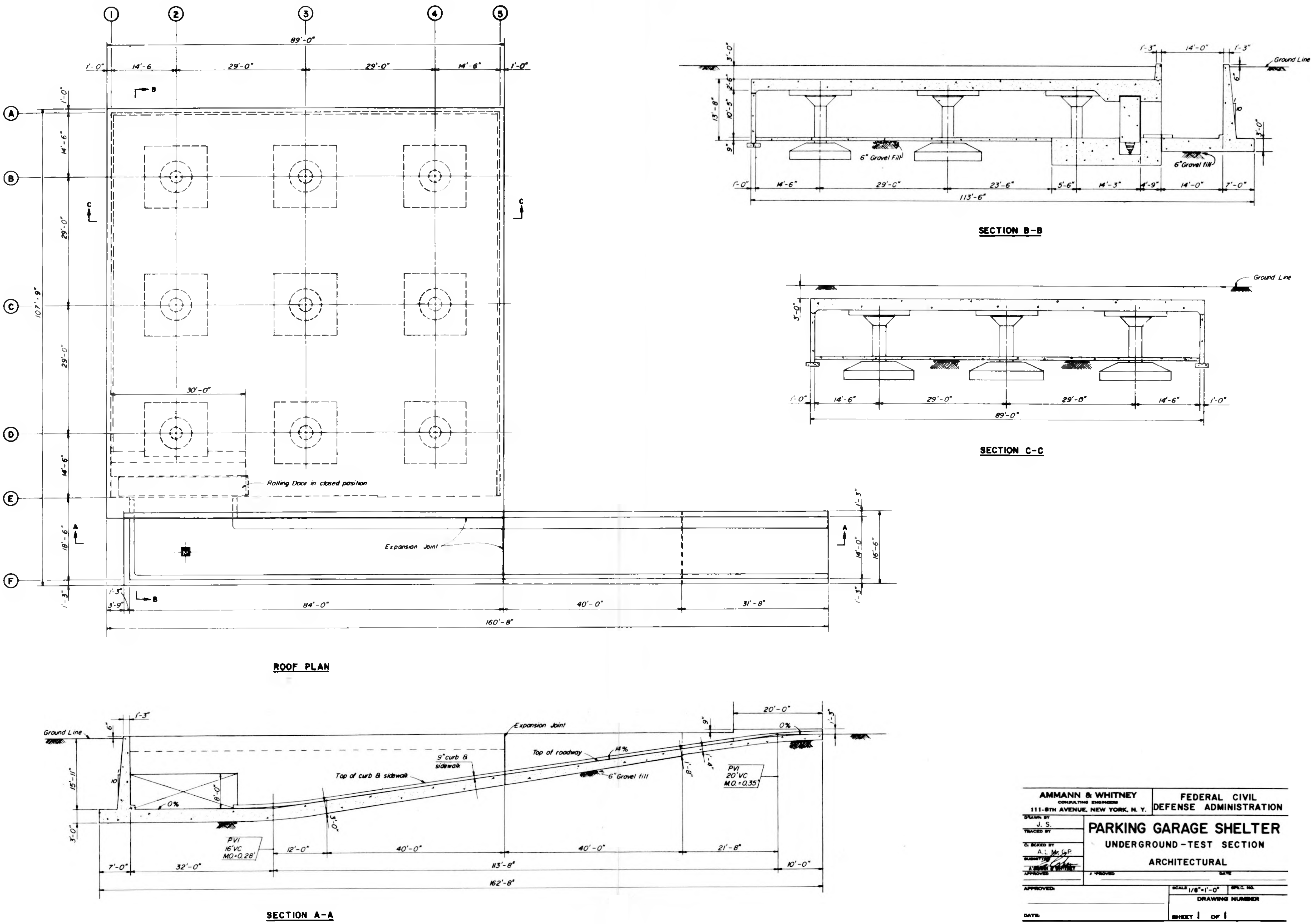


Fig. 1.2-Parking garage-shelter underground test section.

Fig. 1.3-Interior view showing columns.

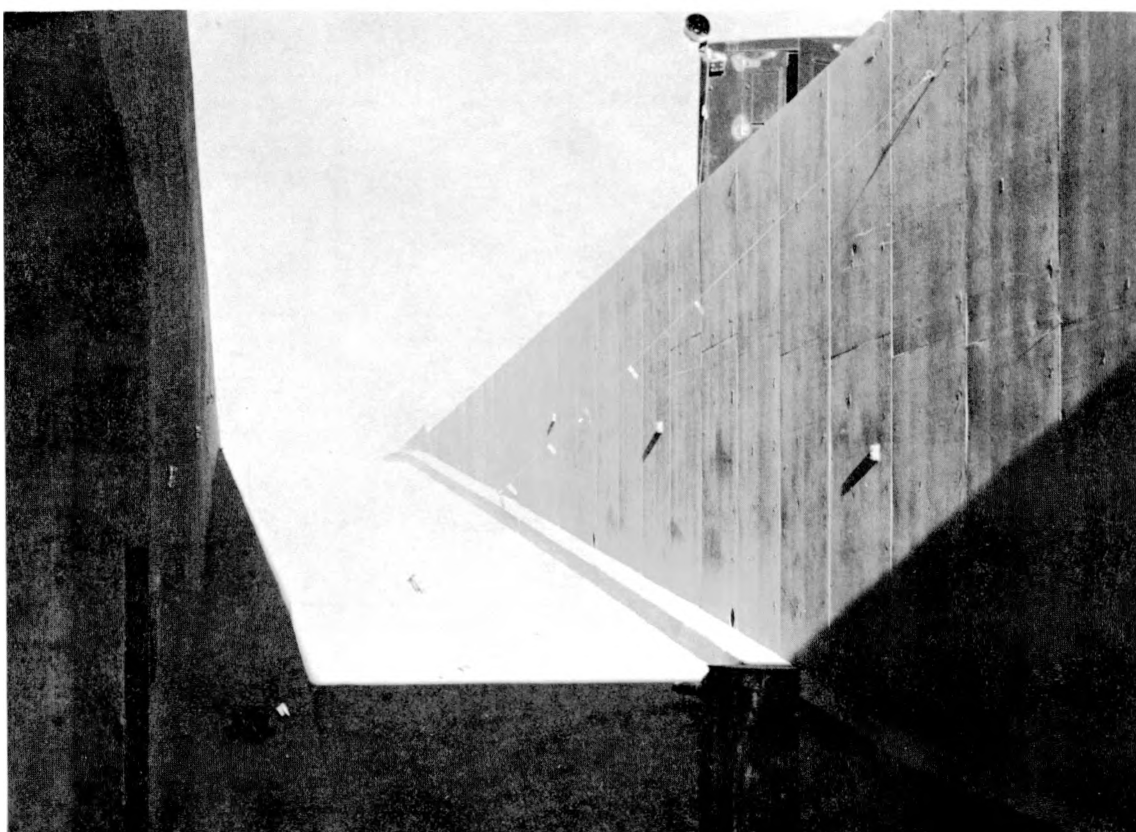


Fig. 1.4-View of ramp looking up from end wall.

Fig. 1.5-View of ramp looking down toward end wall.

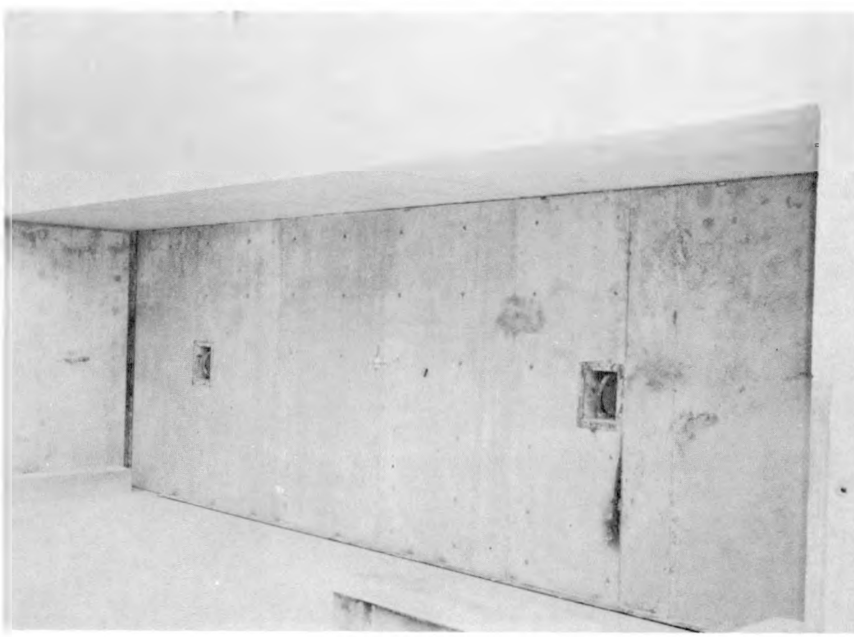


Fig. 1.6-Exterior view of door in closed position.

Fig. 1.7-Interior view of door partly open during installation of instruments.

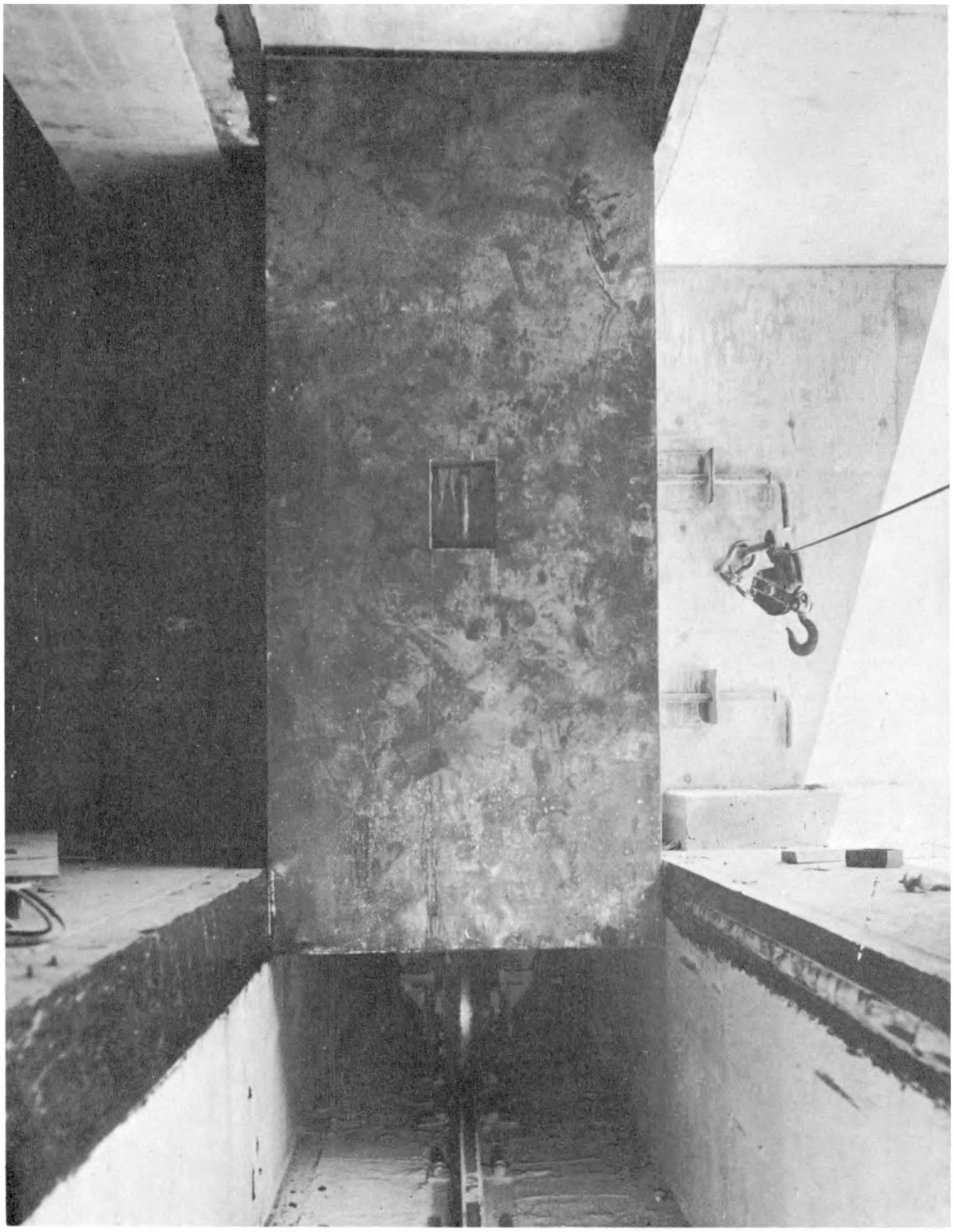


Fig. 1.8-End view of door from door pit.

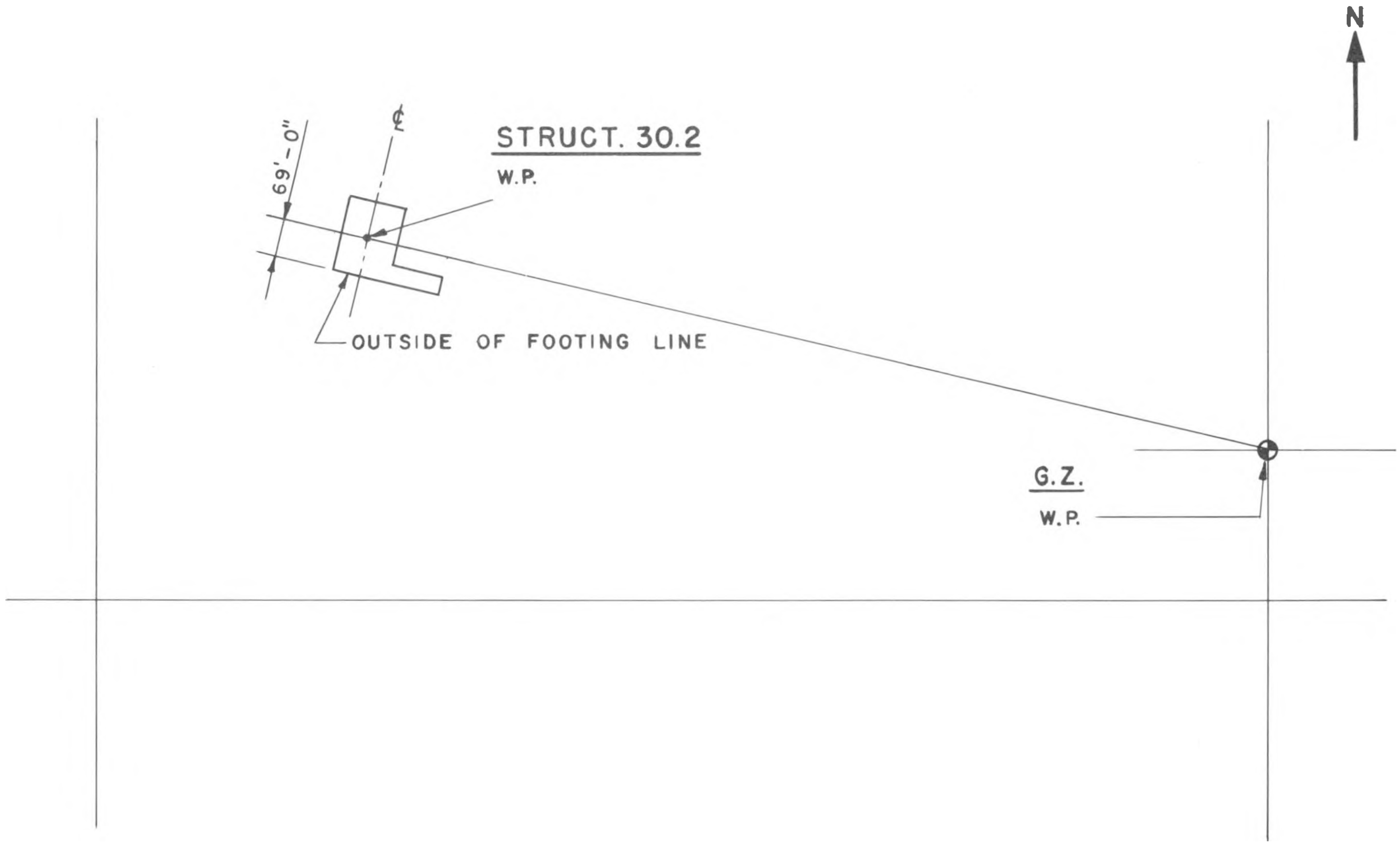


Fig. 1.9-Orientation of structure with respect to GZ.

The design drawings for the test structure are included in Appendix A.

1.4 THEORY

The shelter was designed for dynamic behavior using ultimate strength theory and theoretical loadings consistent with a peak incident shock of 30 psi of a megaton-range weapon.[1-5]

The roof slab, columns, footings, and earth-covered walls of the shelter were designed utilizing additional strain energy available in the elasto-plastic and plastic ranges. The exposed shelter wall and rolling door at the ramp were increased in thickness to provide radiation protection. This additional thickness provided sufficient strength so that only minimum reinforcement was required to produce elastic behavior.

The roof slab was designed for a 30-psi long-duration load as a flat slab using yield-line theory.[3] The earth-covered walls of the shelter were designed for a 15-psi long-duration load as one-way panels spanning vertically with the reinforcement continuous with the roof slab. The wall loading for this test was not expected to be greater than 20 per cent of the incident shock. The floor slab was designed for conventional loading plus blast-load reaction of the walls. The foundations were designed for a maximum bearing capacity of approximately 10 tsf.

Analysis of the consolidation and triaxial test data from undisturbed samples of soil removed from the 48-in.-diameter shafts at the garage structure and the nearby test vault indicates that, within the significant depth region, the soil possesses a natural prestress of about 10 tsf. Table 2.1 contains selected values for the test results. The high triaxial stresses and small strains at failure are especially noted as a peculiar characteristic of this soil in its natural state.

In view of the high prestress and state of over-consolidation, the probable natural static in-place stress-strain relations, failure strength, and shearing strength cannot be less than would be obtained under a lateral stress, p_3, of 40 psi (2.88 tsf). Theoretical studies and correlations of load-settlement relations from plate bearing tests and from full-scale footings with triaxial test stress-strain relations obtained from undisturbed soil samples were performed by D. M. Burmister of Columbia University. From these studies the estimated static failure stress under a footing can be at least 2.5 times the comparable laboratory triaxial failure stress. This results from the natural confinement and restraint conditions afforded to lateral displacements by the natural soil mass surrounding a footing, which cannot be duplicated by a simple stress restraint of the lateral stress in a triaxial test. In addition, the confining and restraint influences of the surrounding earth surcharge above the level of the base may increase this value to 3.0 or more. Using triaxial data for a lateral stress of 40 psi (2.88 tsf), the static failure stress on the center footing may be 16 tsf x 2.5 = 40 tsf.

Footing sizes could have been substantially reduced because dynamic soil strengths generally exceed static values. Foundation motions relative to general soil motion under design loading were expected to be less than 1 in.

The test structure was intended to be a typical section of an actual garage-shelter structure, and the design assumed it could be oriented in any direction with respect to GZ. Also, an actual garage-shelter would have at least two vehicular ramps oriented in opposite directions, as shown in Fig. 1.1, plus at least two emergency personnel exits.

Because of the alternate means of entrance and exit and the many possible critical orientations, it was decided to design the retaining walls for a nominal loading equal to one-half the incident pressure acting normal to the wall in either direction. The structure was tested with ramp center line oriented on a radial line with GZ; this was the most unfavorable orientation for the end wall and rolling door at the garage entrance. Because of the unusually high predicted dynamic load and nonideal nature of the incident shock wave, the load on the ramp end wall of the test structure was expected to be many times the peak incident pressure and to differ substantially from the theoretical design values.

The end wall, and to a lesser extent the side walls within the region of high reflected and stagnation pressure, was expected to undergo large plastic deformations and fully utilize the restraint afforded by passive resistance of the backfill.

The material strengths used for design of the dual-purpose underground garage structure were as follows:

Concrete	. 4,000 psi (ultimate)
Reinforcing steel (intermediate grade) .	.47,500 psi (yield)
Structural steel	.38,000 psi (yield)

The above stresses were increased [1,3] to account for rapid strain rates.

Flexural and/or thrust capacities were determined using data of reference 3, and shear capacity was computed using reference 1 and verified using recent laboratory data.

REFERENCES

1. Principles of Atomic Weapons Resistant Construction, Department of Civil and Sanitary Engineering, Massachusetts Institute of Technology, 1954. (Secret).

2. Design of Structures to Resist Atomic Blast, Ammann & Whitney, Consulting Engineers, New York, N. Y., 1954 (Confidential).

3. C. S. Whitney, B. G. Anderson, and E. Cohen, Design of Blast Resistant Construction for Atomic Explosions, Journal of the American Concrete Institute, March 1955 (Proceedings Vol. 51).

4. C. S. Whitney and E. Cohen, Guide for Ultimate Strength Design of Reinforced Concrete, Journal of the American Concrete Institute, November 1956.

5. C. S. Whitney, Plastic Theory of Reinforced Concrete Design, Transactions of the American Society of Civil Engineers, V. 107 (1940).

Chapter 2

PROCEDURE

2.1 SOIL TESTS

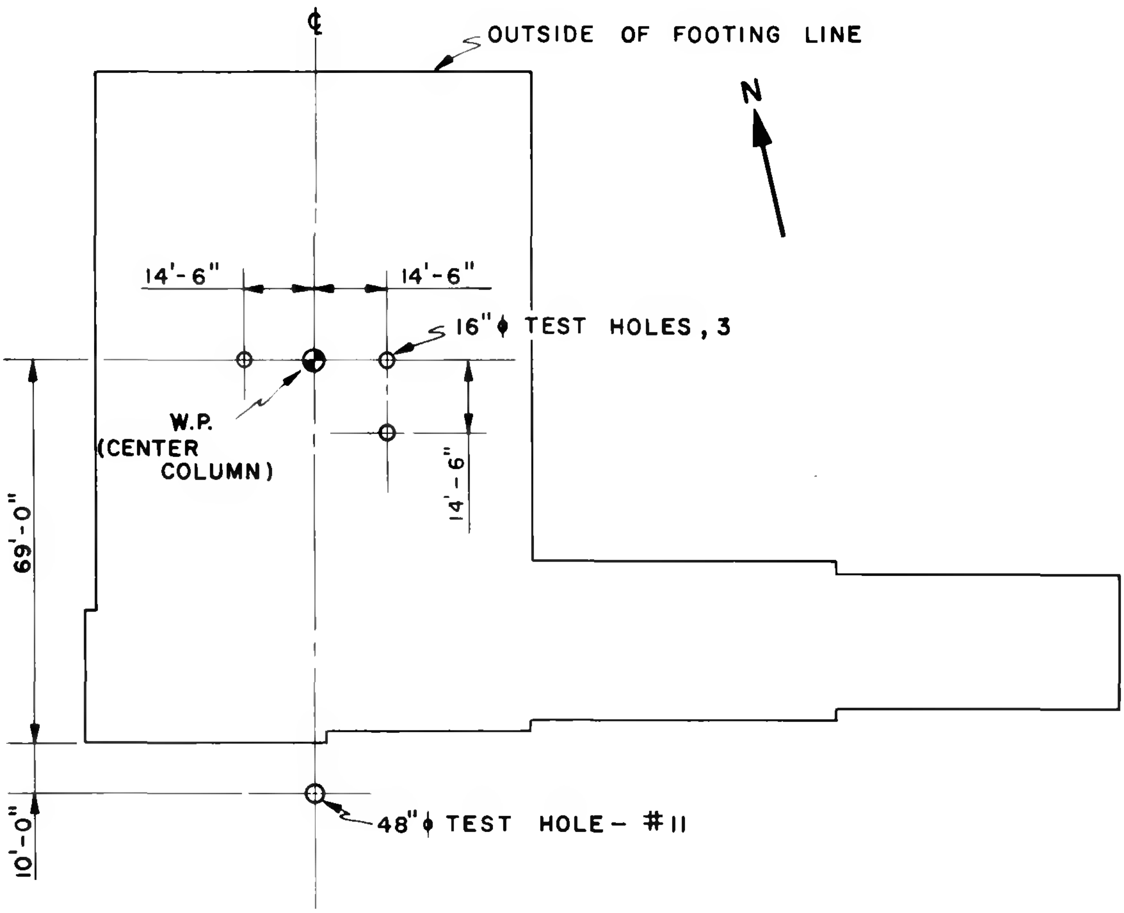
Soil tests were made by the International Testing Corporation under the direction of Holmes and Narver at the request of Ammann & Whitney. Three 16-in.-diameter borings 40 ft deep and one 48-in.-diameter shaft (Fig. 2.1) 40 ft deep were drilled at the site of the shelter, and one 48-in.-diameter shaft was drilled at the site of the nearby test vault, Project 30.4. The large-diameter shafts were used to obtain undisturbed soil samples at various depths.

The following tests were made on the samples from the 48-in. hole:

(1) Field density.
(2) Liquid and plastic limit.
(3) Sieve analysis.
(4) Unconfined compression tests.
(5) Consolidation tests to determine the natural vertical state of stress.
(6) Triaxial tests.

The soils were unusual in character and possessed remarkable properties, with a high state of high consolidation. As a result, these conditions are directly reflected in unusual stress-strain relations, failure stress conditions, and shearing strength of the soil, as indicated in Table 2.1, showing selected values. The high triaxial stress and relatively small strain at failure are noted.

Additional information is available as a result of the soil-testing program of the Waterways Experiment Station, Vicksburg, Miss., as reported in Project 3.8 report (ITR-1427).



PLAN VIEW

Fig. 2.1-Test holes for structure 30.2.

Table 2.1-- STRESS-STRAIN RELATIONS, FAILURE STRESS CONDITIONS, AND SHEARING STRENGTH OF SOILS BY TRIAXIAL TESTS AT DEPTH OF 17 FEET IN 48-IN. HOLE

Lateral stress (p_3) Psi	Tsf	Triaxial failure stress Psi	Tsf	Strain at failure	Max. shearing strength, tsf
20	1.44	154		0.06	
		140/147	10.6	0.04	5.3
40	2.88	217		0.02	
		238/228	16.4	0.06	8.2
50	3.6	276	19.8	0.07	9.9
61	4.1	197	14.2	0.08	7.1
80	5.8	303*	21.6	0.08	10.8

*No failure.

2.2 SURVEYS

To determine absolute and relative lateral and vertical displacements of the structure during the blast, preshot and postshot high-order field surveys of the horizontal and vertical coordinates of the structure were required. The survey points are shown in Fig. 2.2.

2.3 INSTRUMENTATION

Blast instrumentation was provided by Project 30.5 and is covered in detail in the Project 30.5 report (ITR-1452). Blast instrumentation consisted of Wiancko pressure gauges, a self-recording pressure gauge, Carlson earth-pressure gauges, and dynamic-pressure gauges.

Structural response was recorded by BRL deflection gauges. Over-all vertical motion of the central column was referenced to a 4-in.-diameter steel pipe in an oversized casing anchored in a concrete block 20 ft below the floor slab.

Instrumentation locations are shown in Fig. A.9.

Radiation measurements were made using film dosimeters, gamma-radiation differential chemical dosimeters, and one gamma-rate telemetering unit. These were supplied by Projects 39.1, 39.1a, and 39.9 and were located as shown in Fig. 2.3 and described below:

1. Points "a" through "y" inclusive have two film dosimeters at each point located 3 ft and 5 ft above the floor.
2. Points "l" and "u", in addition to the two film dosimeters, have one chemical dosimeter located 2 ft above the floor.

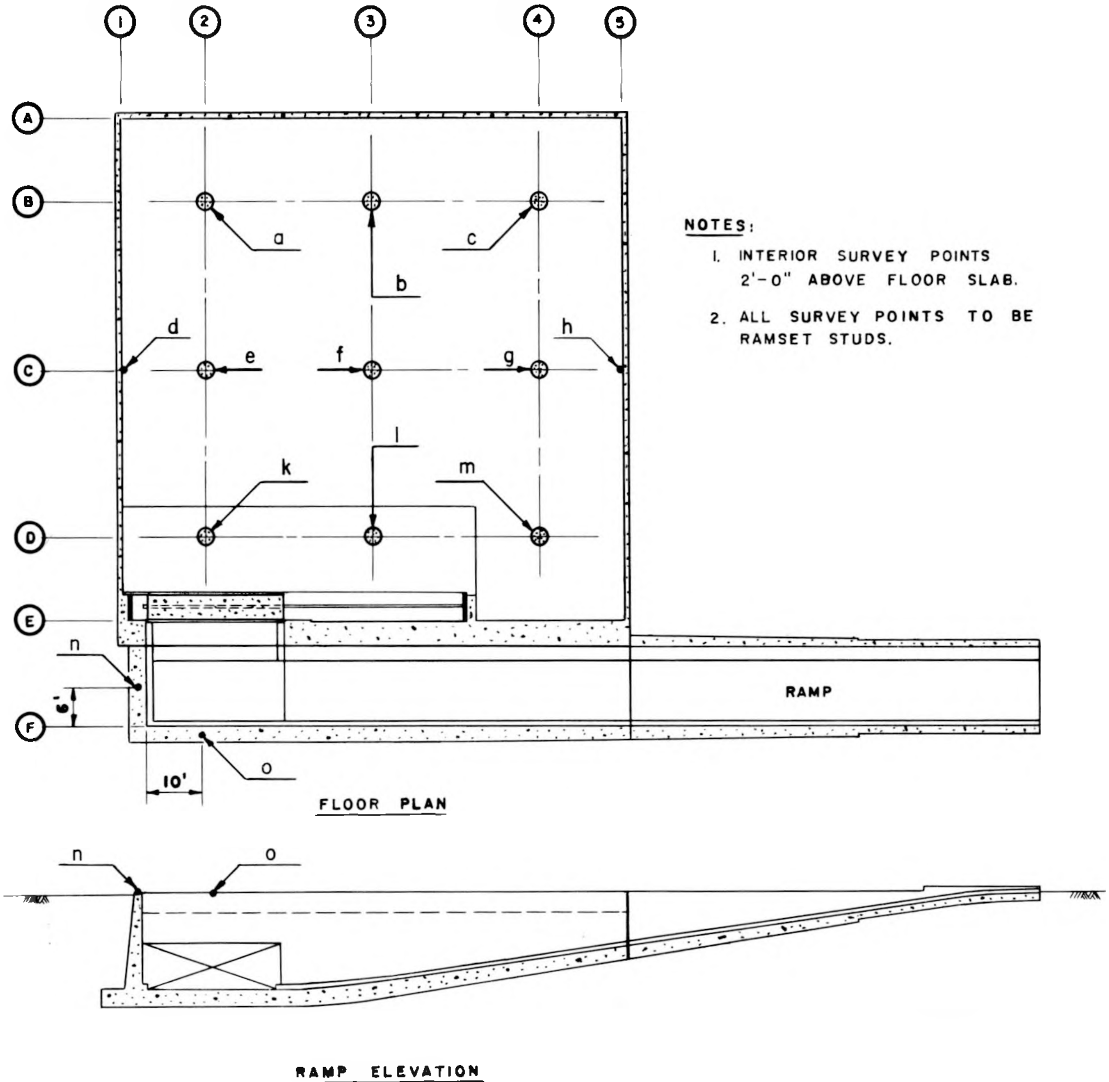


Fig. 2.2- Survey points for structure 30.2.

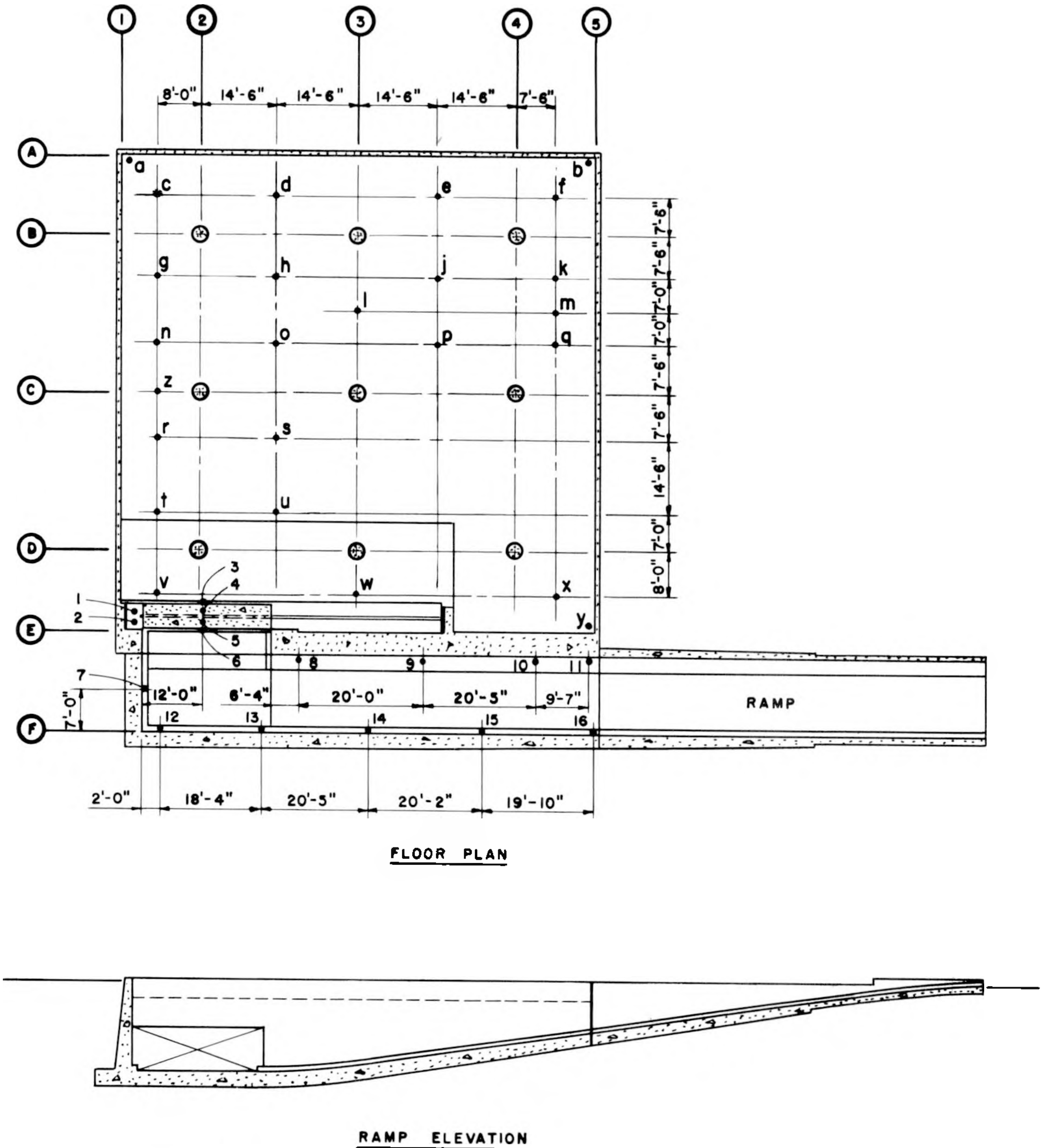


Fig. 2.3-Locations of dosimeters and neutron detectors for structure 30.2.

3. Point "z" is the telemetering unit.
4. Points 1 through 16 inclusive have one film dosimeter at each point located as follows:
   (a) Points 1 and 2 are on top of the concrete door bumper.
   (b) Point 3 is on the inside face of the door 4 ft 6 in. above the top of the floor slab.
   (c) Points 4 and 5 are on the bottom of the door pit on each side of the steel rail.
   (d) Point 6 is on the outside face of the door 4 ft 6 in. above the top of the ramp slab.
   (e) Points 7 through 16 inclusive are on the garage and ramp walls 5 ft above the top of the curb and sidewalk.

Chapter 3

RESULTS OF BLAST DAMAGE

The exposed wall of the garage withstood the blast with no damage. The ramp wall at column line F had several large cracks between the end wall and a point about 30 ft up the ramp (Fig. 3.1). The top of the ramp side wall opposite the door entrance was pushed into the earth about 1 ft at the end. The top edge of the ramp wall farther up the ramp showed no apparent displacement. Although there was no visible damage to the ramp slab, the slab, together with the ramp wall, was separated by 1/2 in. to 1 in. from the main garage structure at the expansion joint (Fig. 3.2). Gravel backfill was sucked through the weepers onto the ramp (Fig. 3.3). More gravel was found opposite the weepers at the mid-length and toward the top of the ramp than opposite the weepers at the lower end.

The end wall of the ramp was the only area badly damaged (Figs. 3.4 and 3.5). The top 8 ft of the end wall was broken off as a single unit on a nearly horizontal plane at the top of the splice of the vertical steel. It tipped into the backfill and slipped over the lower section until it wedged tightly between the garage wall and the longitudinal wall of the ramp. It was torn loose on a diagonal through the corner at its junction with the longitudinal retaining wall. In its final position, the top was displaced approximately 5 ft 6 in. into the earth backfill, which was pushed up and mounded. It is estimated that it may have been displaced approximately 8 ft before sliding back. The concrete cover had split off most of the lower section, and the bars had separated at the splice apparently without having developed their yield strength. Near the middle of the panel where the concrete had not split, three bars failed in tension and had fractured after necking down at the top of the splice (Fig. 3.6).

The bars in the lower section were bent away from the displaced concrete; the cover was deposited at the base (Fig. 3.7). The remaining concrete behind the bars was reduced to rubble, varying in size from 6 in. to 3 ft in diameter. Fractures, including the one at the

Fig. 3.1-Cracks in ramp side wall opposite door. View from interior of garage through door opening (postshot).

Fig. 3.2—Open joint between ramp slab and wall of main garage structure (postshot).

Fig. 3.3–Gravel backfill at weepers (postshot).

Fig. 3.4–View of ramp looking toward damaged end wall (postshot).

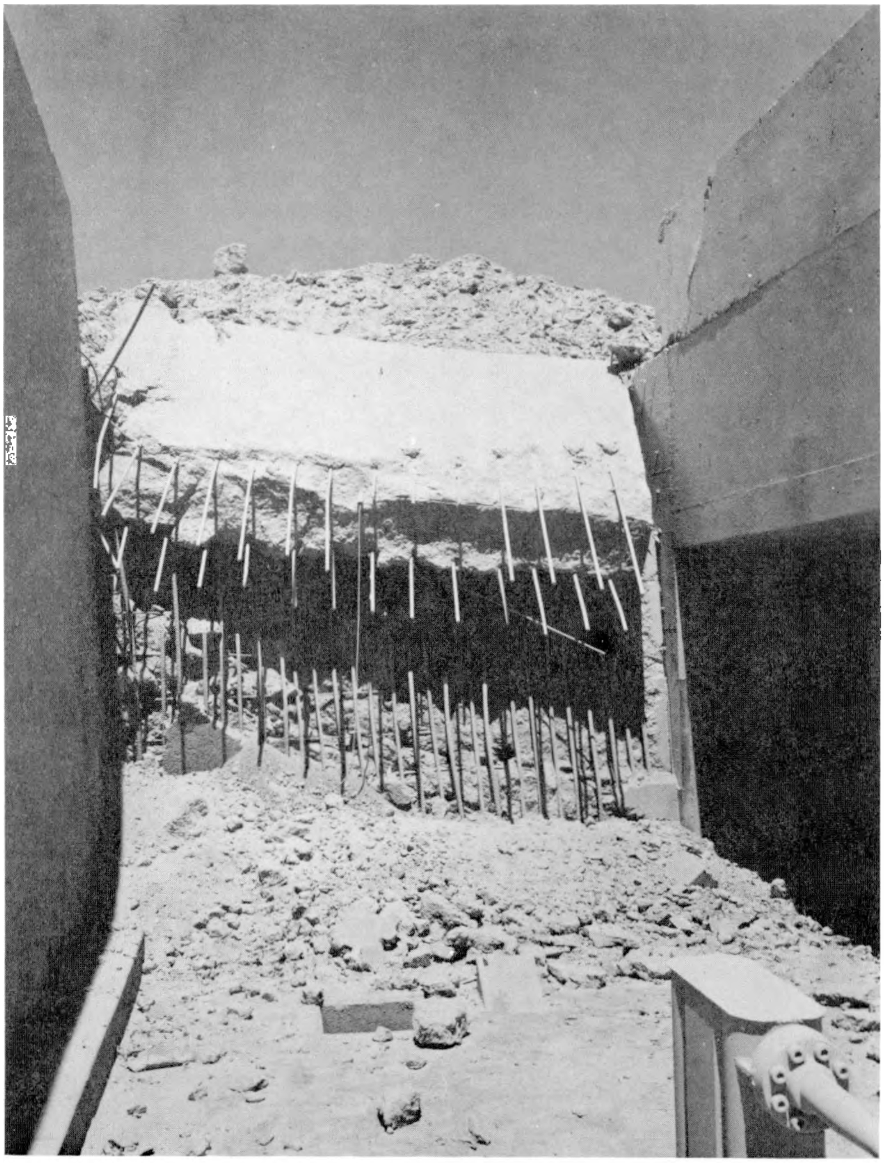


Fig. 3.5—Close-up of damaged end wall (postshot).

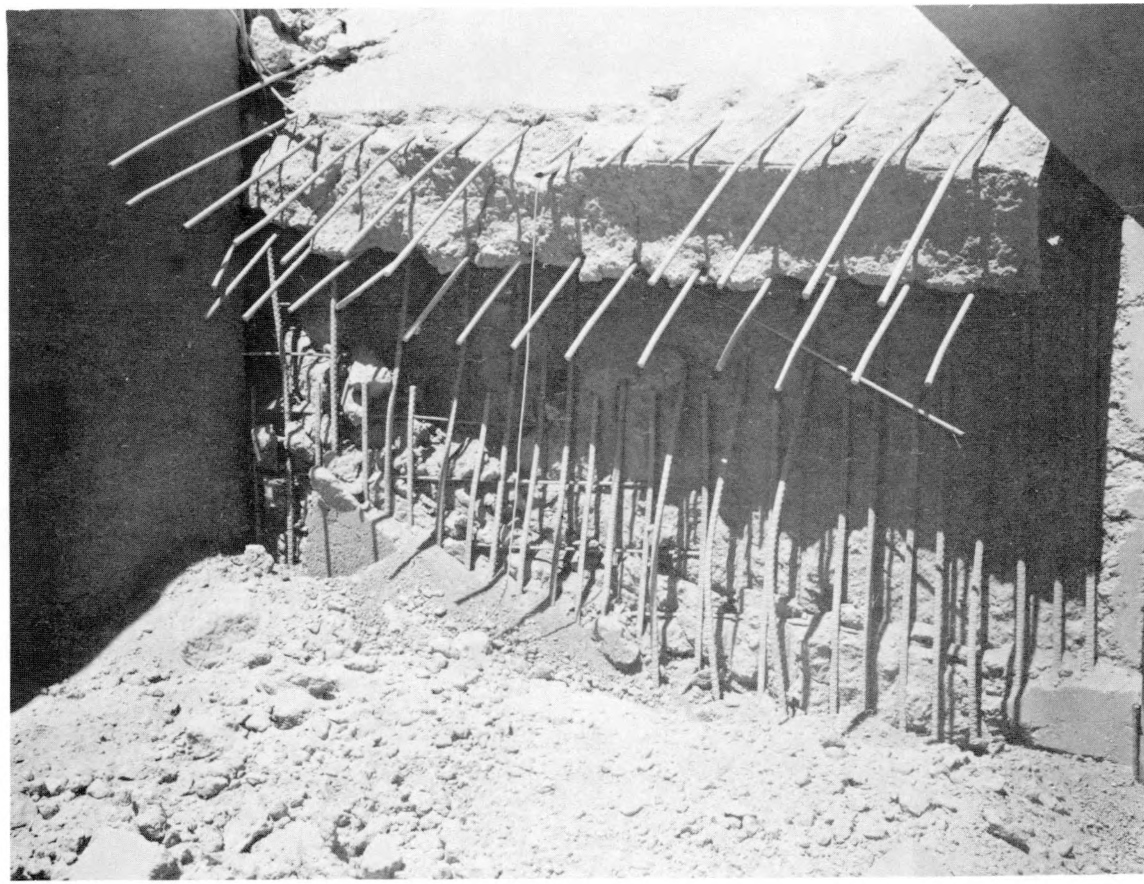


Fig. 3.6—Detail of failure showing fractured bars (postshot).

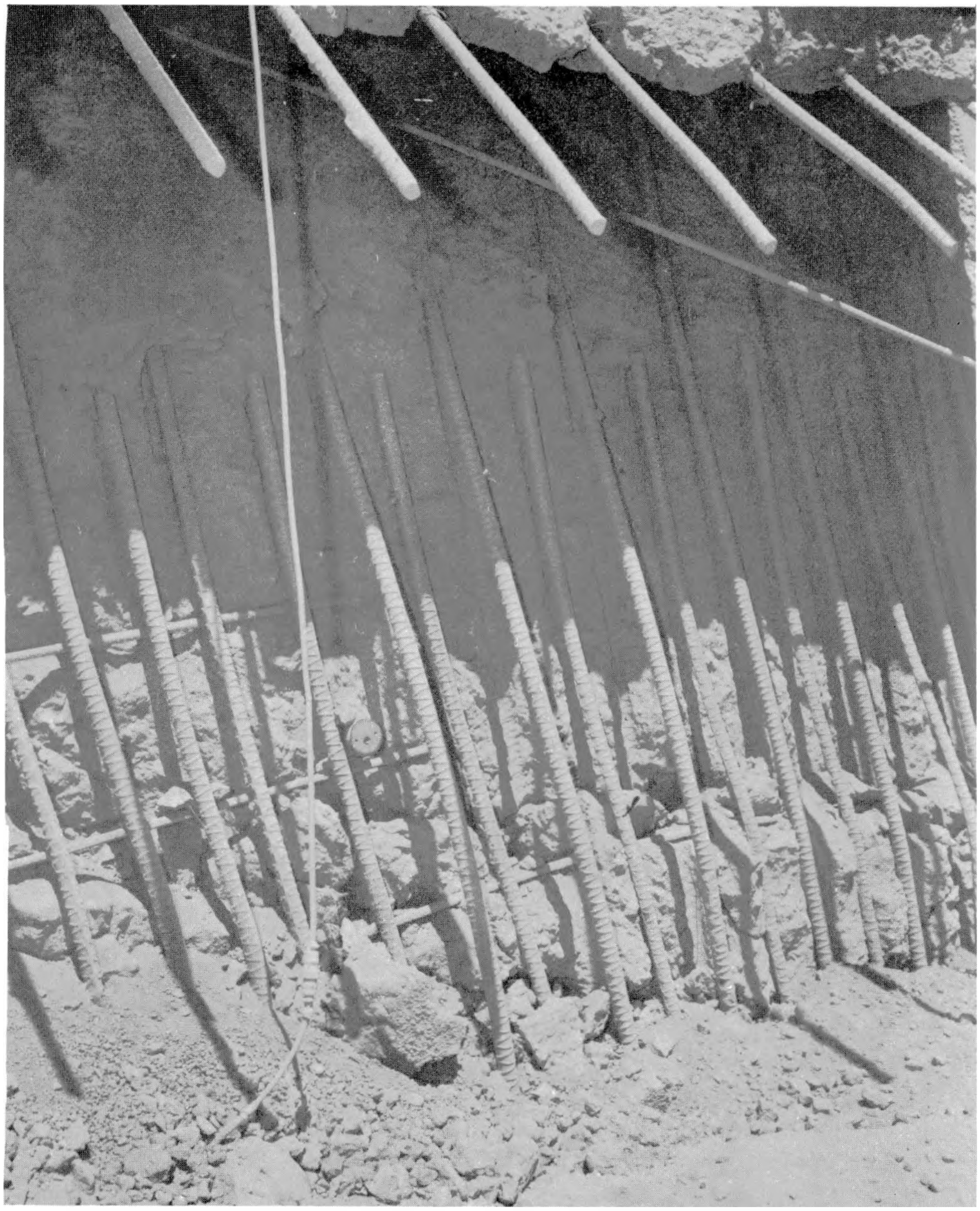


Fig. 3.7–Detail of lower portion of wall (postshot).

bottom of the top section of the damaged end wall revealed no breaks through the aggregate. The concrete had also separated on the plane of the rear-face reinforcement.

The end corner of the 1 ft 3 in. parapet wall at the lower end of the ramp was cracked as shown in Fig. 3.5.

The door withstood the blast without any evidence of shifting or disalignment. Locking bolts in the door were intact and freely retractable (Fig. 3.8). The 1/4-in. cover plate angle at the exposed top edge of the door frame was separated from the concrete at several locations. The 12-in. steel guide plate on the 4.5-ft wall was not made continuous as intended and was torn back by the door during the postshot opening (Fig. 3.9). The door wheels were not damaged, and the track was not displaced (Fig. 3.10). The door and end pilaster were partially blackened by the termal radiation.

The pneumatic gasket around the door frame was blown in and torn apart by the blast pressure (Fig. 3.11). The gasket had a slow leak prior to the shot, and a compressed-air cylinder was installed outside in the doorway recess to maintain the air pressure. The cylinder and air hose were sandbagged and were not damaged by the blast. No information is available regarding the condition of the gasket at shot time.

All dosimeters in the ramp fastened with a single wire and two ramset bolts were dislodged. Of two dosimeters fastened with two 1-in. light-gauge steel straps and four expansion bolts, one damaged dosimeter remained on the outside face of the door.

The two BRL deflection gauges on the ramp were intact, but the drum wires were snapped.

Pressure gauges and Q gauges in the ramp and at column line A were all intact.

No damage to the garage interior was observed. Lateral movement of the isolated columns was indicated by a small amount of concrete spalling and cracking of the floor slab around the perimeter of the column. The cracking occurred at the blast side of the columns, and the spalling occurred at the leeward side.

All pressure gauges on the roof and walls were intact, and all interior deflection gauges were in good condition.

There was no obvious soil settlement in the vicinity of the garage. However, surface soil cracks up to 2 1/2 in. wide were opened around the projected perimeter of the roof slab of the structure. Another surface crack was opened parallel to the ramp wall at column line E (Fig. 1.2) about 6 ft from the outside face of the parapet wall, extending from a position about 30 ft from the top of the ramp to the

Fig. 3.8-View of door showing locking bolts (postshot).

Fig. 3.9- Torn guide plate before removal
to allow opening of door (postshot).

Fig. 3.10—Wheel assembly and rails, door partly open (postshot).

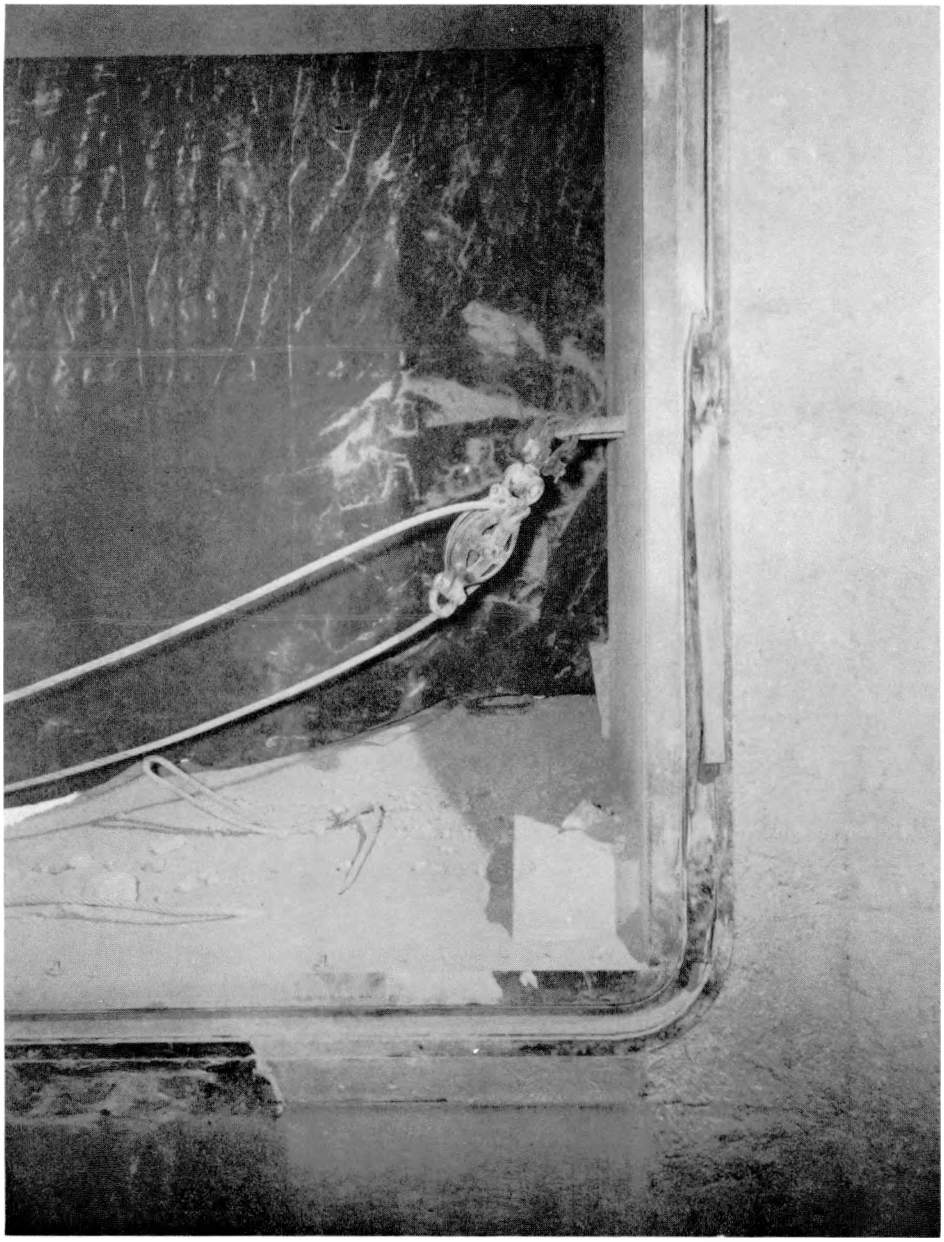


Fig. 3.11—Damaged gasket at lower corner of door opening (postshot).

crack along column line 5. A similar crack was observed about 6 ft away from the ramp wall at column line F extending as far as column line 3 along the approximate line of excavation.

Chapter 4

DISCUSSION OF BLAST DAMAGE

The only substantial damage noted in connection with this project was to the retaining walls at the end of the ramp where damage was expected (see Sec. 1.4). The loading on this wall was about 6 to 7 times the nominal design pressure, and a very low or negative surcharge could be expected on the backfill behind the wall because of high turbulence. The turbulence and diffraction patterns in this region are not well defined. Measurements made for this project indicate additional shock-tube and wind-tunnel data may be desirable.

There is evidence that the upper several feet of backfill behind the wall was not compacted in accordance with the specifications at the time of detonation because of excavation and backfill for an instrumentation trench after original backfilling had been completed. Such a condition would mean that a greater deflection would be required before developing the maximum passive resistance of the soil.

Although the damage to the end wall is not important in connection with this project, the mode of failure is technically interesting and important.

Because of its orientation and high pressure loading, the wall was expected to deflect into the backfill by yielding of the reinforcement, but the concrete and reinforcement were expected to remain banded together, although the concrete would be badly cracked. The complete separation of the top half of the end wall from the bottom, disintegration of the bottom portion into loose rubble, separations along the planes of the reinforcement, and failure of the splices without yielding of the steel can be attributed to the poor adhesive quality of the concrete in place. However, the mode of failure also may have been influenced by the rapid blast loading and the probability that the strength of normal splices under such load may be much lower than under static loading. Laboratory data to verify splice efficiencies under dynamic loading would be highly desirable. The ductility of the wall would be greatly increased by raising the splice point, welding the splices, or using full-length bars thus eliminating the splice.

Had the ramp been of a symmetrical through type without an end wall, high door pressures would have been avoided. However, a through type ramp, although desirable to minimize blast effects, is often not economical or practical. Damage would have been considerably less if the backfill had been compacted to specifications.

The side wall toward the top of the ramp was subjected to a less severe face load in combination with a positive surcharge load and consequently was not noticeably damaged. Although the ramp side wall opposite the garage door was loaded by the backup of reflected pressures, the dynamic pressure effect was less pronounced. Therefore, the ramp side wall was only moderately cracked, although it appears that there may have been some bond failure in the cracked area.

The exposed wall of the garage and the concrete door were more than adequate for the blast effects experienced because of the thickness required for radiation protection. No damage was evident.

Because the as-built clearances between the door and frame were as high as 13/16 in., more than three times that shown on the plans, the pneumatic seal would have been ineffective even in good working condition. To prevent excessive infiltration of the blast pressures, clearances at the base were reduced prior to the shot by 3 x 3 x 3/8 angles; one leg was inserted vertically into the opening and the other bolted to the frame. Although it is not known if the compressed-air cylinder placed prior to the test was adequate to maintain pressure in the gasket up to shot time, the blast pressures entered the joint and forced the gasket toward the interior of the shelter, stripping and tearing it along its entire length.

All film-badge dosimeters in the ramp attached to the walls by means of one-way wiring to two ramset studs were blown away. The one surviving dosimeter on the concrete door was bolted to four threaded ramset studs by two 1-in.-wide light-gauge metal strips. This method of attaching was adequate for the garage test structure. The dosimeter with the heavier fastening attached to the damaged end wall was lost.

The absence of cracks on the roof slab and walls indicates that the structure was capable of resisting the blast load without additional energy available in the plastic range. Because the 12-in. garage walls were designed to resist a blast pressure of 15 psi and the loading was probably much less, it is reasonable that the walls were not damaged.

The column deflections were small in magnitude. This was expected because the soil bearing stress used for design (10 tsf) was only 1/3 to 1/4 of the static ultimate bearing capacity indicated by the postdesign soils-test data.

This discussion is based on visual observations and may be revised when evaluated pressure and deflection vs. time records become available.

Chapter 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

1. The test structure would have provided adequate protection from the effects of the test weapon at the test GZ distance. Preliminary reports indicate that the structure provided a gamma attenuation of between 40,000 to 70,000 and calculations indicate that the neutron dose inside the shelter would be much too low to measure. The rise in pressure on the interior did not exceed 1.0 psi, despite failure of the door-sealing gasket.

2. The flat-slab roof and supporting structure are more than adequate to resist the 42 psi peak incident test loading. When the air and earth pressure-time records and results of material tests are available, a more detailed analysis will be made to determine the safe peak incident pressure for a megaton type loading. It should be greater than the long-duration peak incident pressure of 30 psi.

3. The shear stresses used for design were substantially in excess of values recommended by other sources and were conservative. They will be reviewed in light of recent laboratory tests when the actual concrete strength at test time has been determined from compression tests on cores taken from the region of critical shear or diagonal tension.

4. The door survived without damage.

5. The pneumatic seal around the door frame was unsatisfactory.

6. High pressures that acted on the end retaining wall were the result of the particular orientation of the structure relative to GZ and the site conditions. Even under these severe circumstances, the damage that occurred did not impair the usefulness of the ramp for vehicular use during the immediate postshot period. In addition, an

actual shelter-garage structure would have alternate vehicular and personnel entrances and exits. For this reason, the design strength of the retaining walls need not be increased. However, the brittle type failure is undesirable, and the problem should be studied further in an effort to modify the details to produce a more ductile retaining-wall structure.

With the given orientation, the high pressures and damage experienced by the end wall can be avoided by use of a through type ramp where such a layout is not prohibited by other factors.

5.2 RECOMMENDATIONS

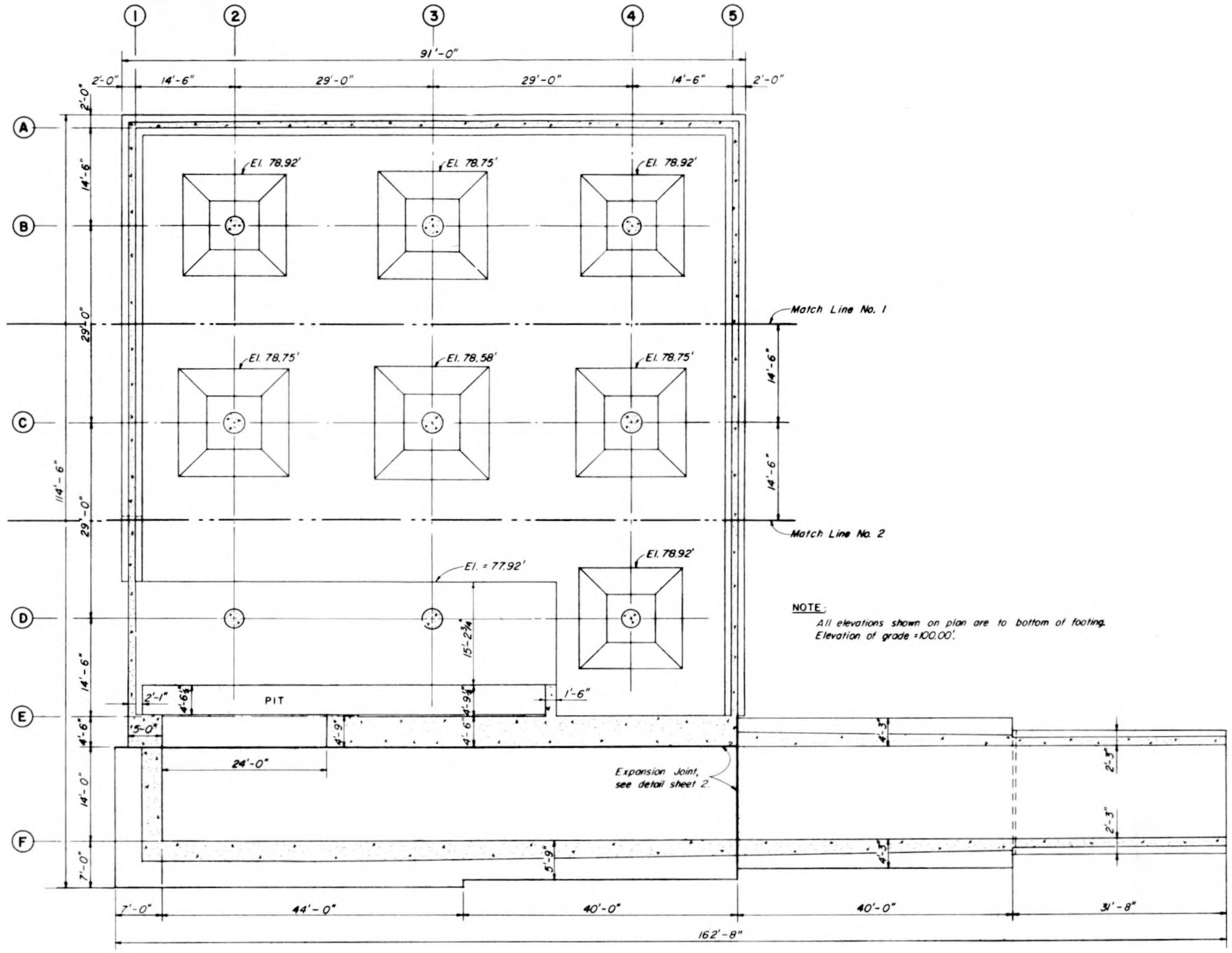
1. The pneumatic seal should be replaced with a rigid mechanically operated seal adequate to resist the maximum peak pressures in the region of the door.

2. Additional study should be given to the mechanical equipment, space, maintenance, and operating requirements in actual garage shelters for vehicular doors of the type proof tested.

3. Further laboratory tests and studies in connection with the reflected pressures and air flow in the ramp are desirable.

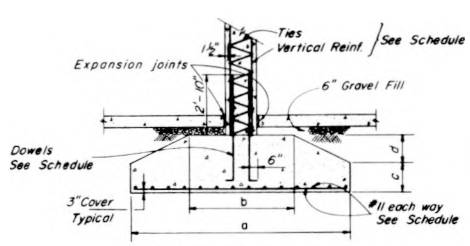
Appendix A

DESIGN DRAWINGS



FOUNDATION PLAN

SCALE: $\frac{1}{8}$" = 1'-0"



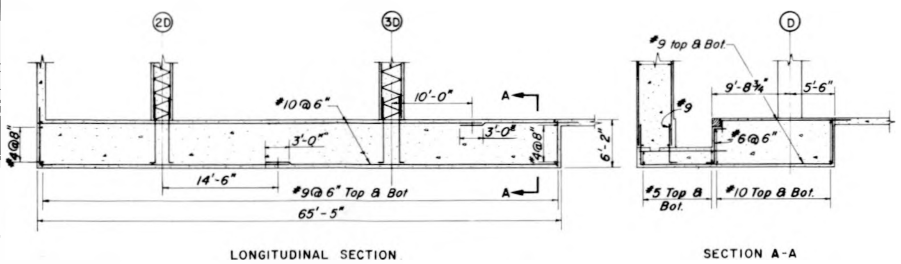
TYPICAL COLUMN FOOTING DETAIL

NTS

COLUMN & COLUMN FOOTING SCHEDULE				
MARK		3C	2B, 2D, 4B, 4D	2C, 3B, 3D, 4C
	Size	36 ϕ	33 ϕ	36 ϕ
	Core	33 ϕ	30 ϕ	33 ϕ
	Vertical Reinf.	20 #9	16 #9	12 #9
	Spiral	#5 @ 3"	#5 @ 3"	#5 @ 3"
	Dowels	20 #9	16 #9	12 #9
FOOTING	a	16'-9"	15'-0"	16'-0"
FOOTING	b	8'-3"	7'-2"	7'-10"
FOOTING	c	2'-2"	2'-0"	2'-1"
FOOTING	d	2'-1"	1'-11"	2'-0"
FOOTING	Reinf. each way	44 #11	36 #11	41 #11

Notes:

1. Spirals to be held firmly in place and true to line by a minimum of four vertical spacers.
2. 1½ extra turns of spiral rod at each end of spiral to be provided for anchorage
3. Spirals to extend from top of footing to a plane at which the dia. of the capital is twice that of the column
4. Column reinforcing to extend from top of footing to top reinforcing in roof slab.
5. For column footing 2B only, distribute 36 #11 reinforcing bars each way except as follows:
   Middle strip (b = 7'-2"): 22 #11
   Outside strip (7'-10"): 14 #11



COMBINED FOOTING

SCALE: $\frac{1}{8}$" = 1'-0"

GENERAL NOTES

Material:

1. Structural steel, (including welds and bolts), shall conform to A.S.T.M specifications designation A7-53T and to Federal specifications QQ-S-741a.
2. All concrete shall have a minimum compressive strength of 4,000 pounds per square inch at 28 days.
3. All reinforcing steel shall be intermediate grade. Deformation shall be in accordance with A.S.T.M. specification designation A305-53T and with Federal specification QQ-B-71a(3).

Miscellaneous

1. Waterproof as required.
2. All reinforcing laps and splices to be a minimun of 30 diameters except as noted.
3. Minimum cover for reinforcement is to be 2" except as noted.
4. For alternate structure, omit all construction between match lines 1 and 2. All other details remain unchanged.

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FEDERAL CIVIL DEFENSE ADMINISTRATION

DRAWN BY J.S.
TRACED BY
CHECKED BY G.P. & A.L.M.
SUBMITTED AMMANN & WHITNEY
APPROVED

PARKING GARAGE SHELTER
UNDERGROUND-TEST SECTION
FOUNDATION PLAN

APPROVED Hal H. Jennings DIRECTOR PROGRAM 30 DATE 2-7-57
SCALE AS NOTED
APPROVED: DIRECTOR ENGINEERING OFFICE DATE: 2-7-57
DRAWING NUMBER
SHEET 1 OF 11
Rec'd 2/9/57 with letter of 2/9/57

Fig. A.1- Foundation plan.

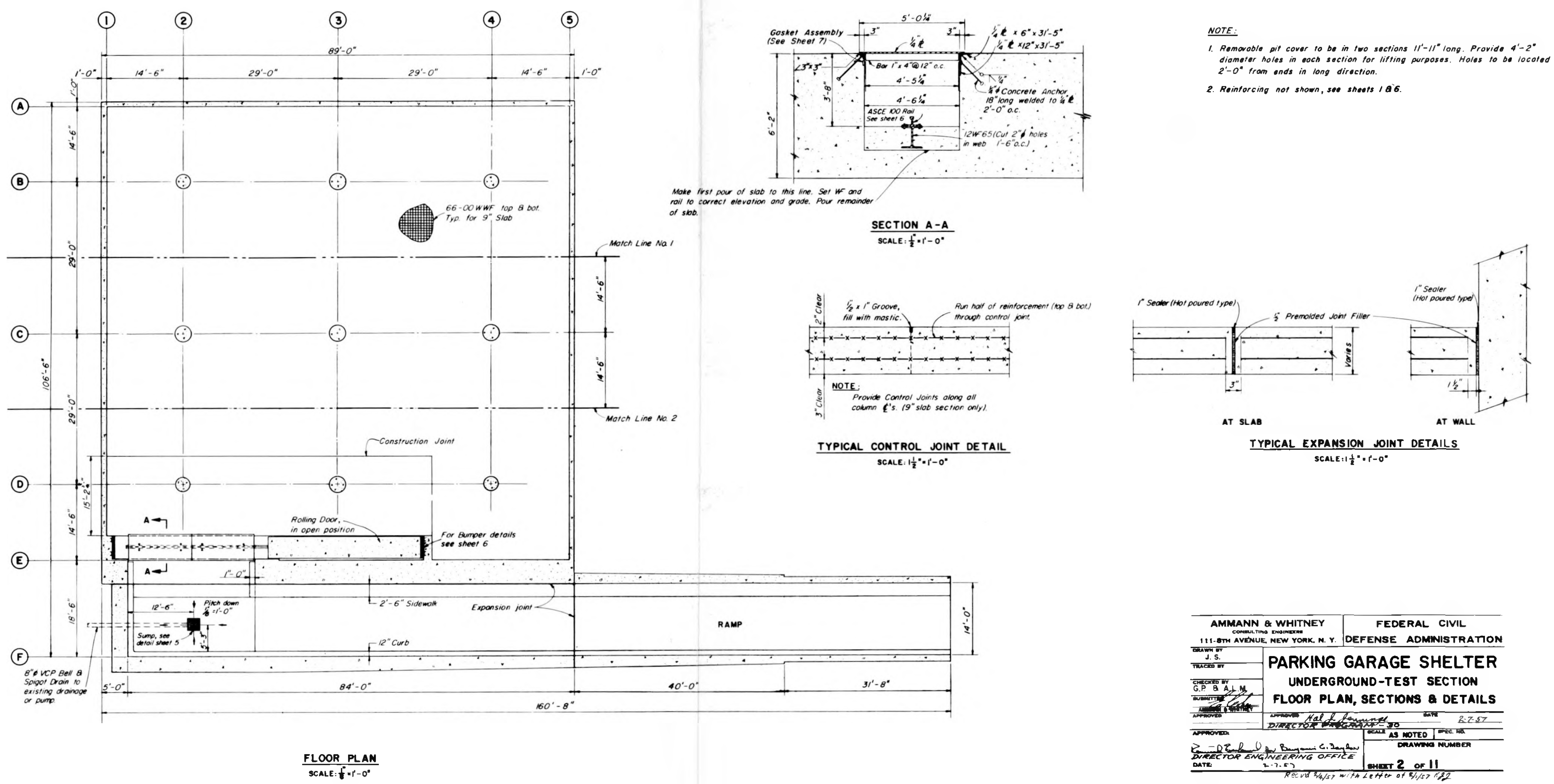

Fig. A.2-Floor plan, sections, and detail.

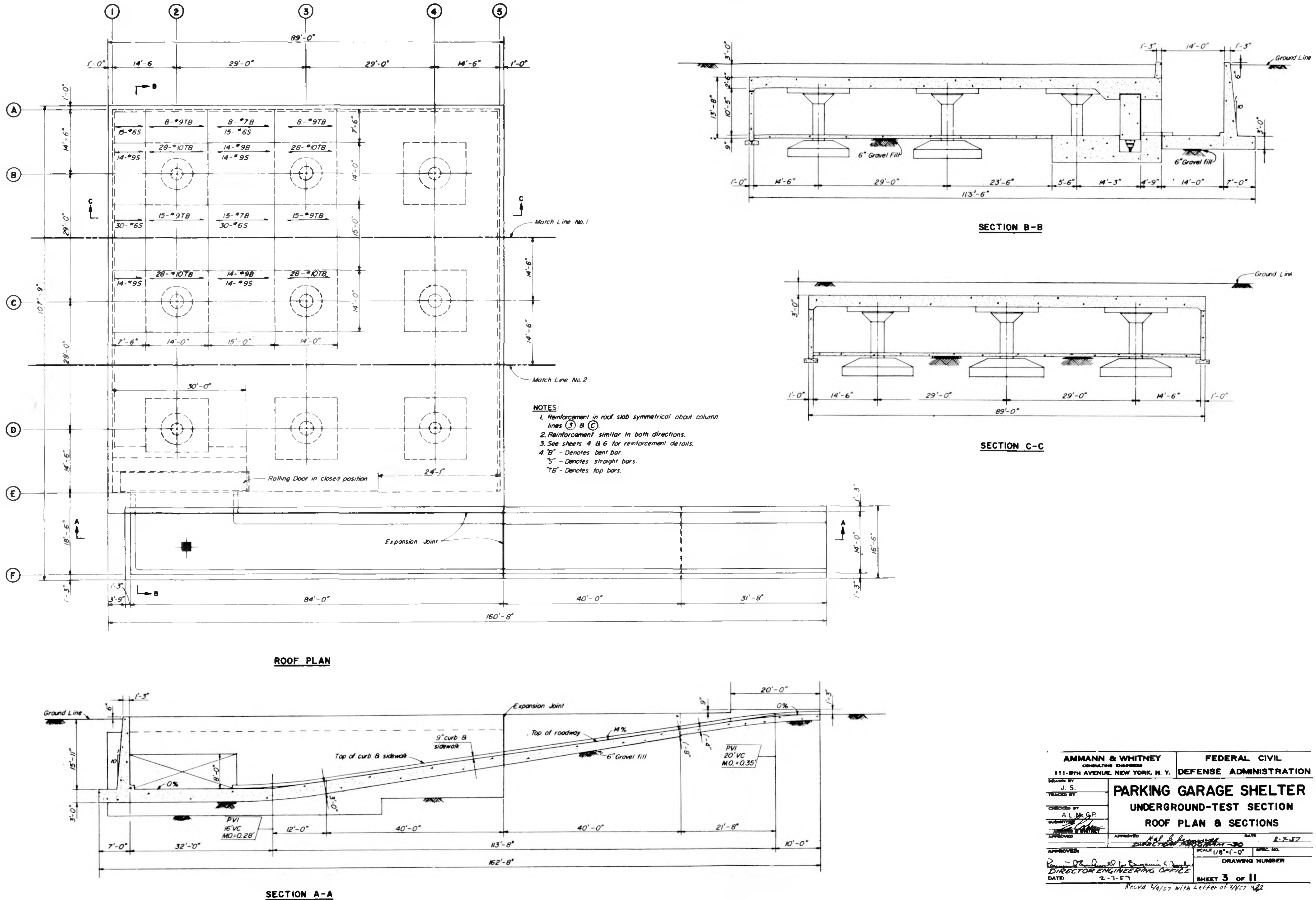

Fig. A.3-Roof plan and sections.

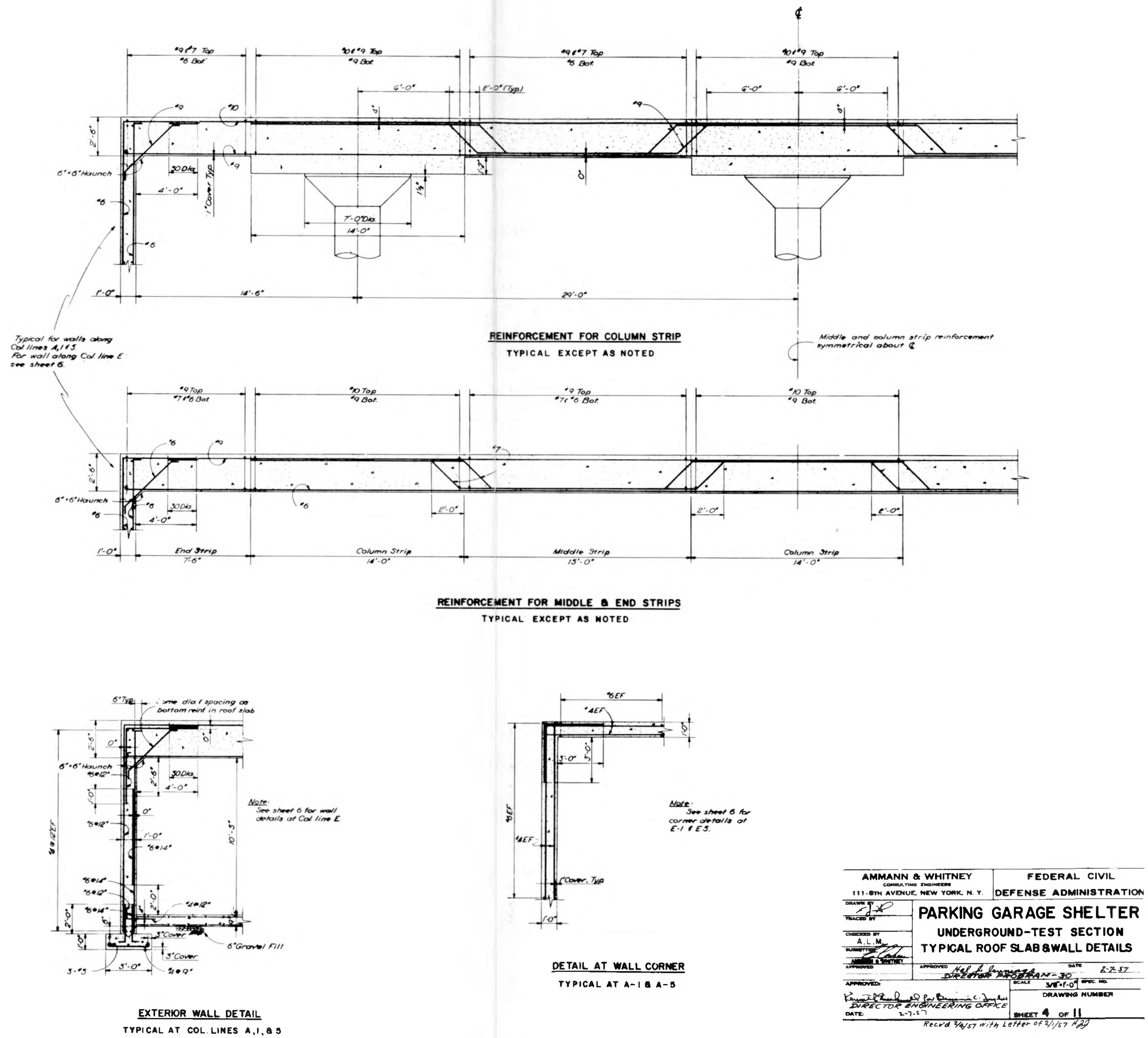

Fig. A.4—Typical roof slab and wall details.

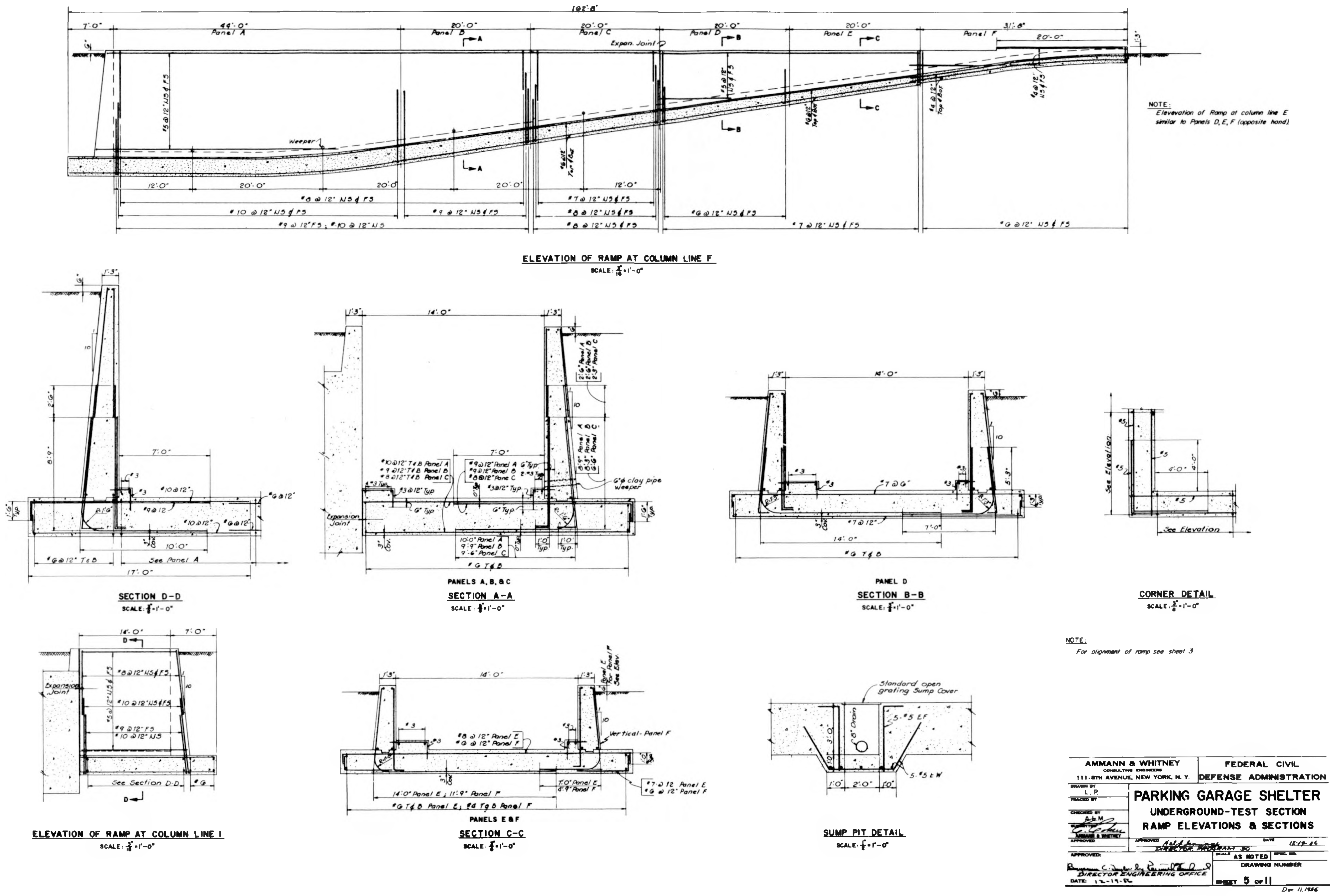

Fig. A.5—Ramp elevations and sections.

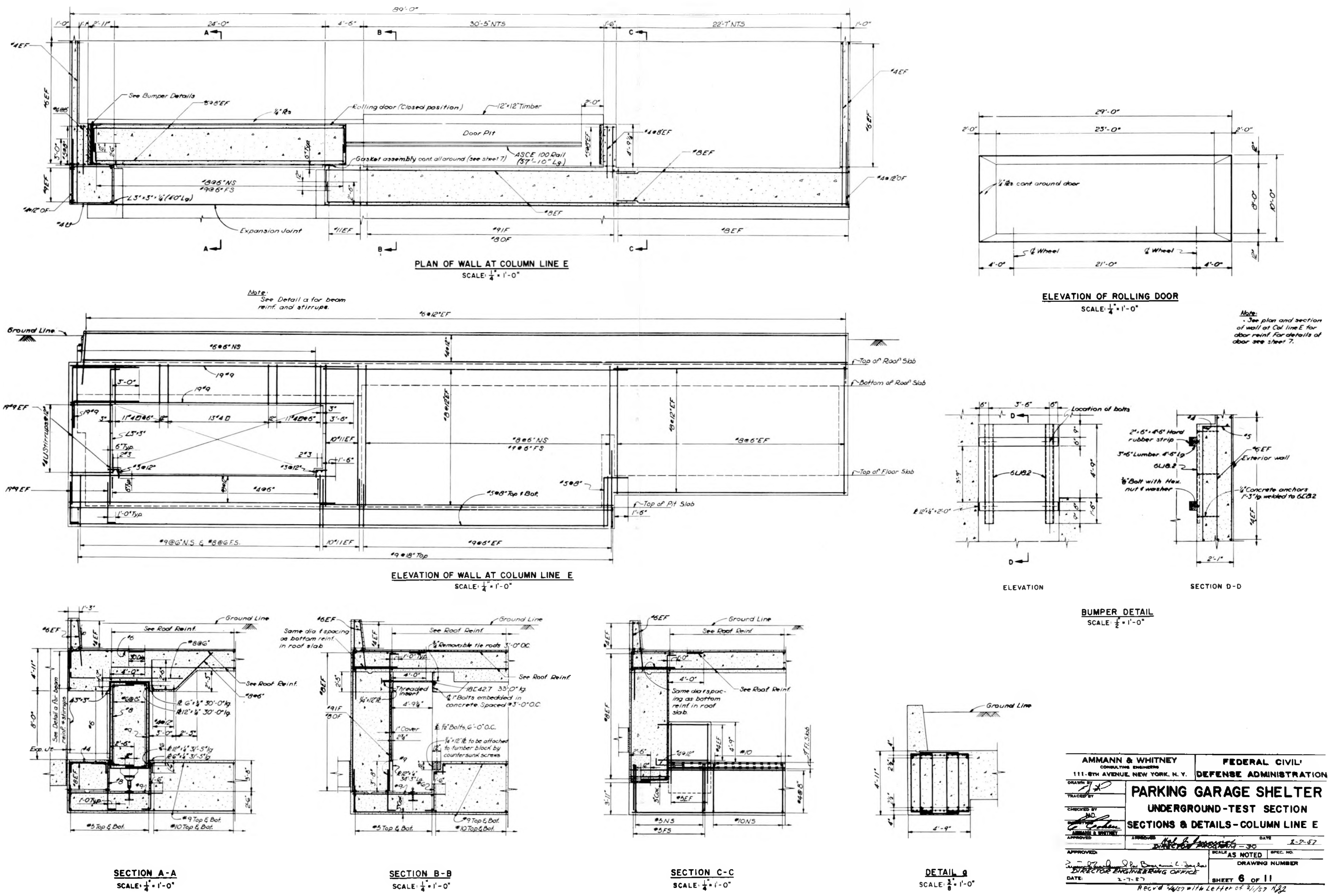

Fig. A.6—Column line E, sections and detail.

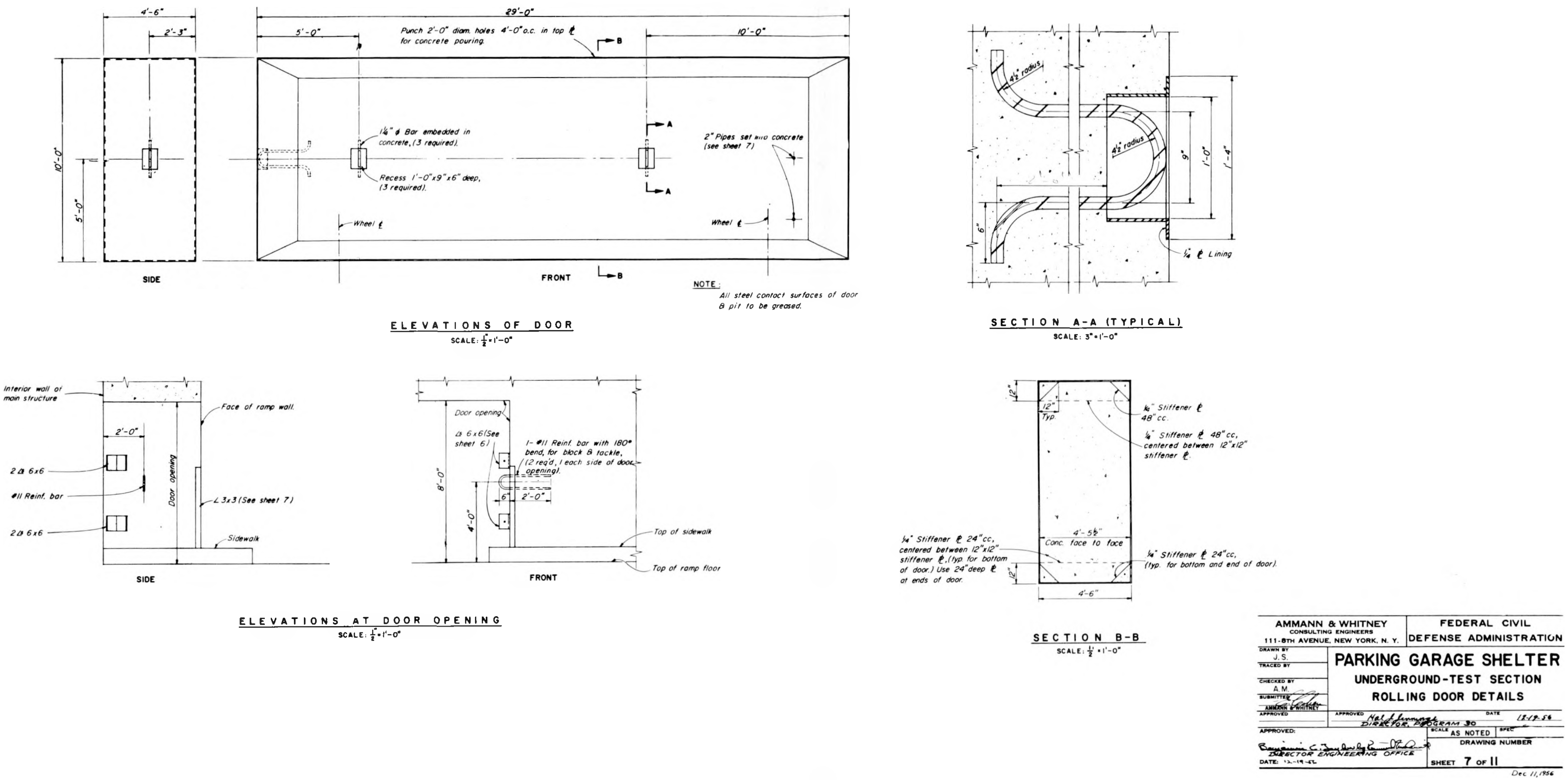

Fig. A.7a-Rolling-door details.

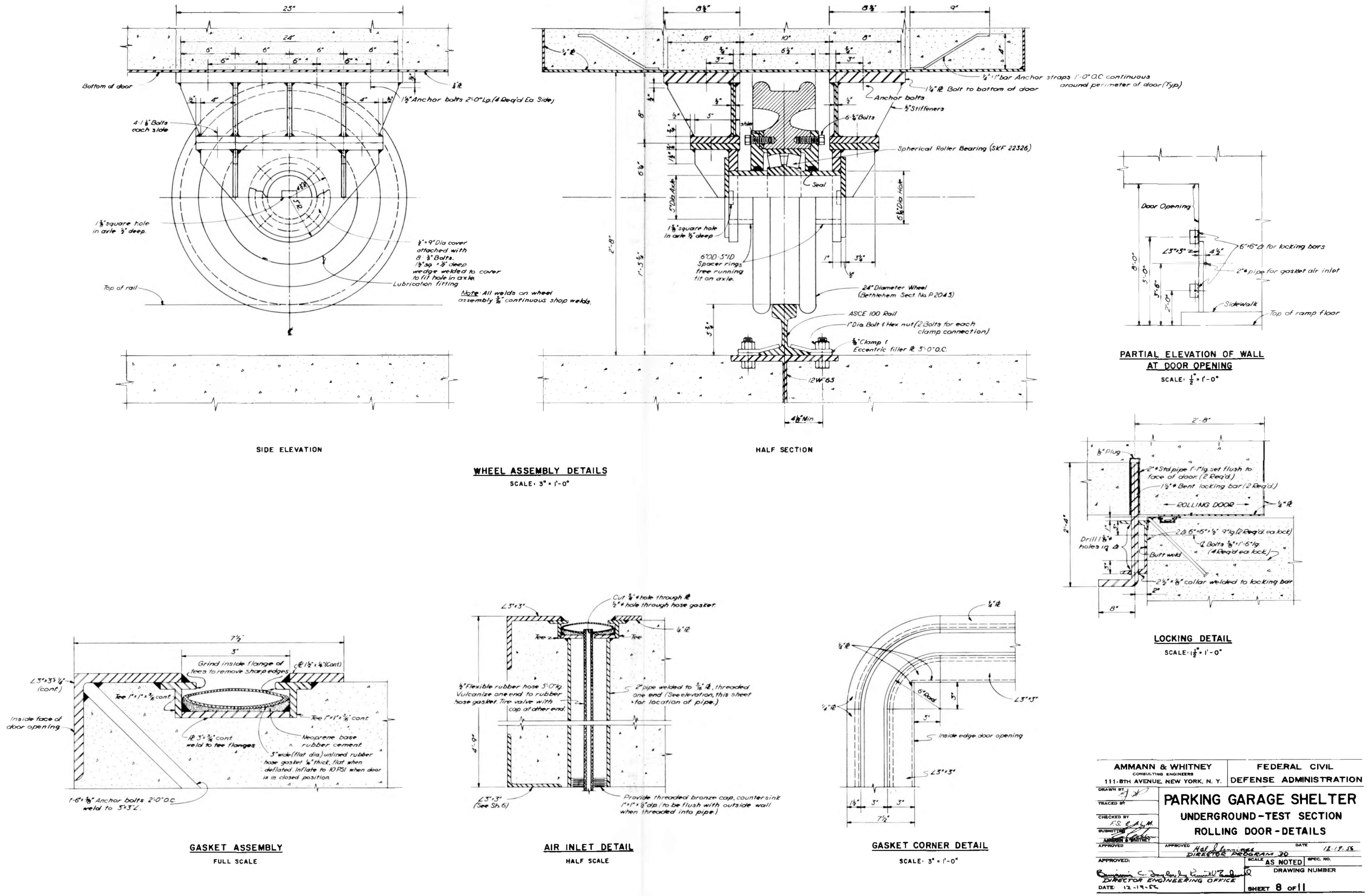

Fig. A.7b—Rolling-door details.

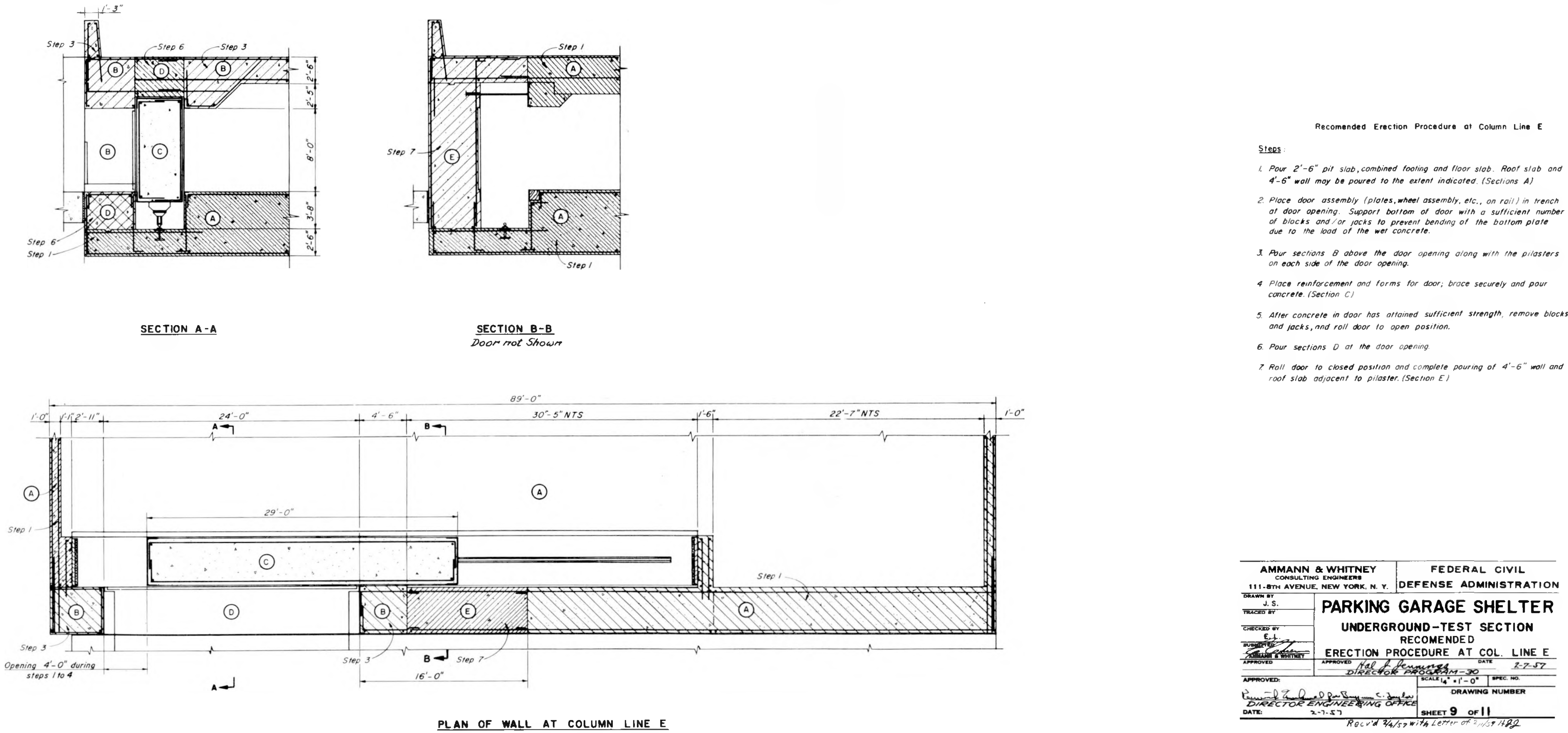

Fig. A.8—Erection procedure at column line E.

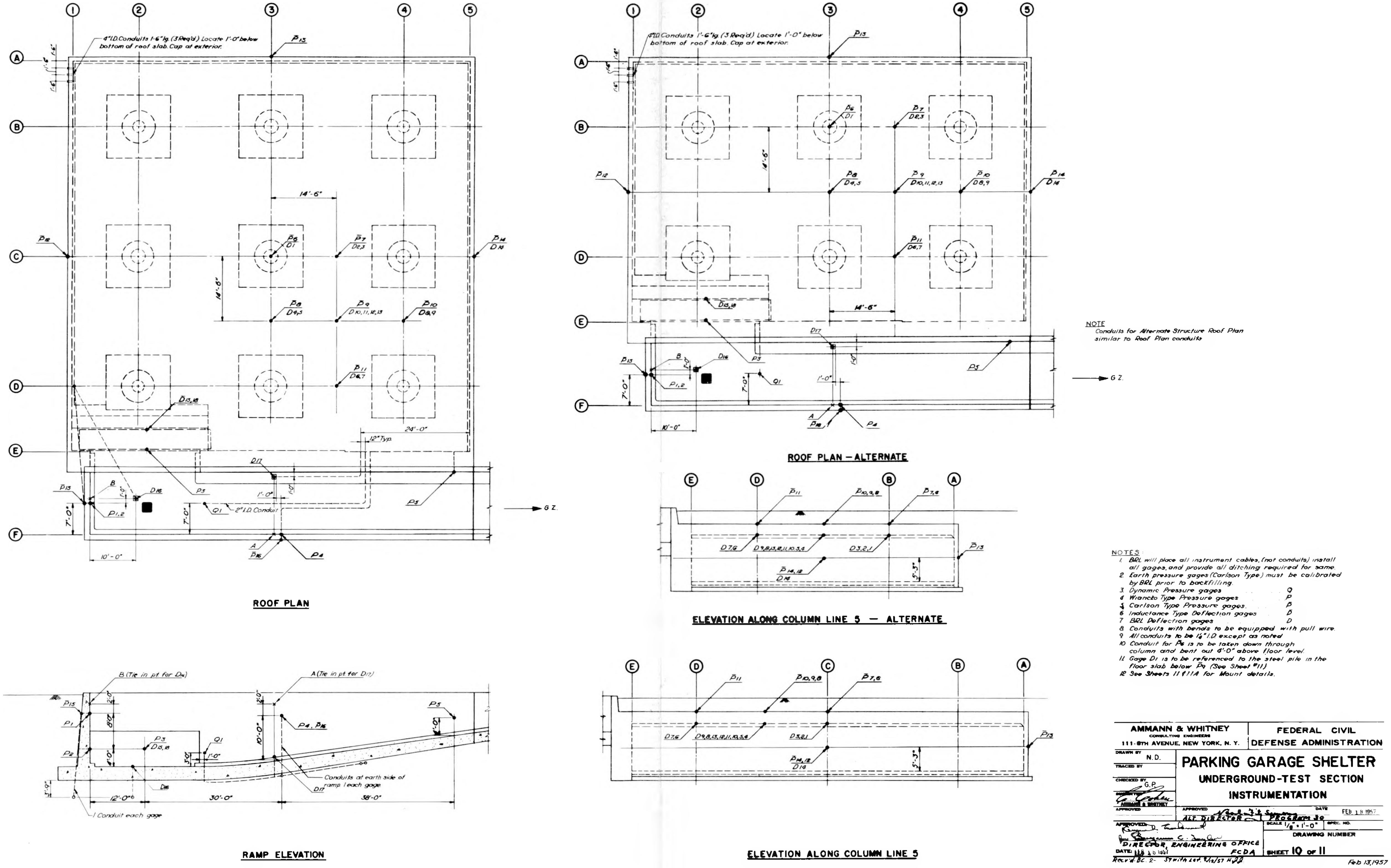

Fig. A.9-Instrumentation.

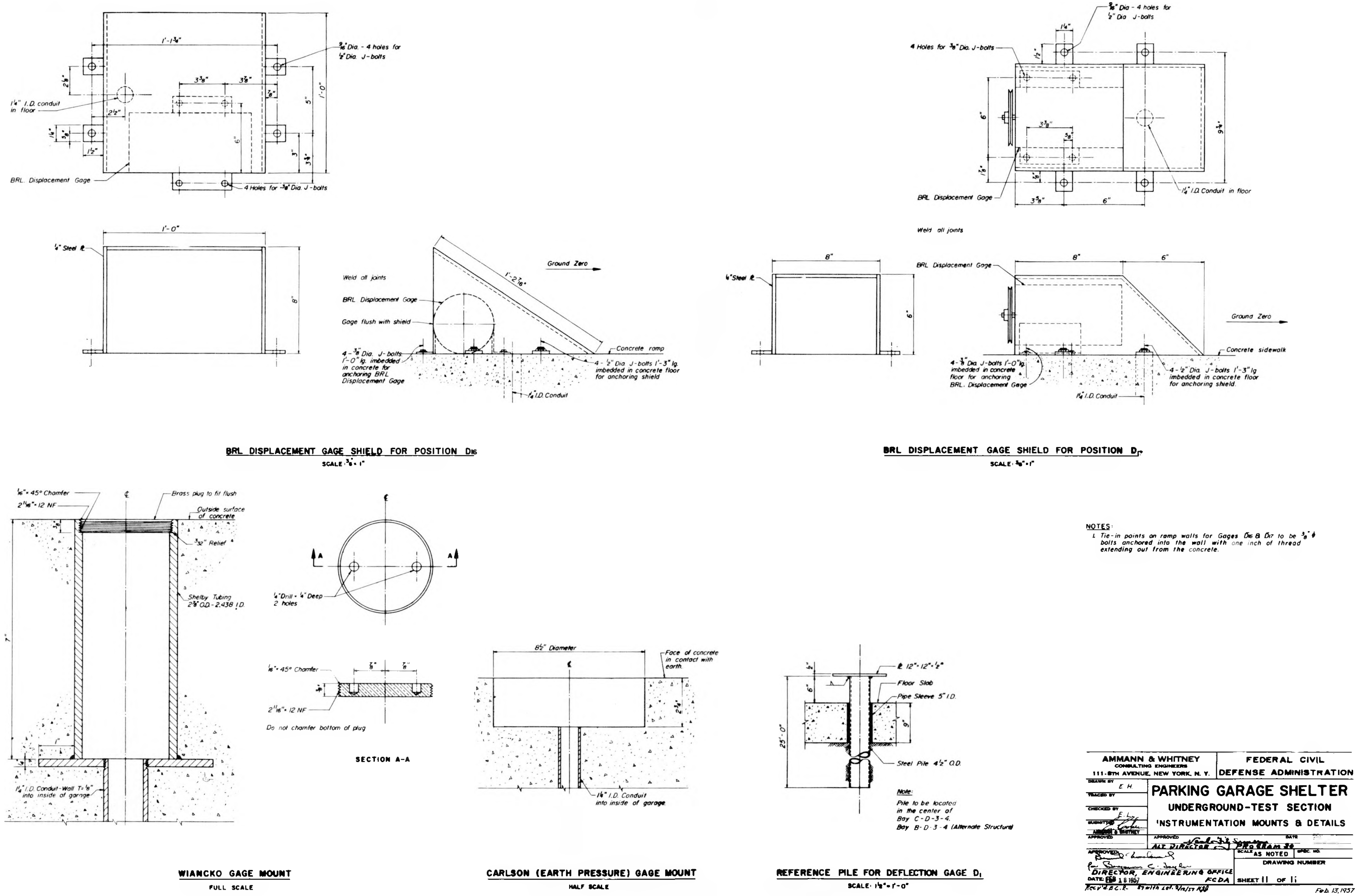

Fig. A.10a—Instrumentation mounts and details.

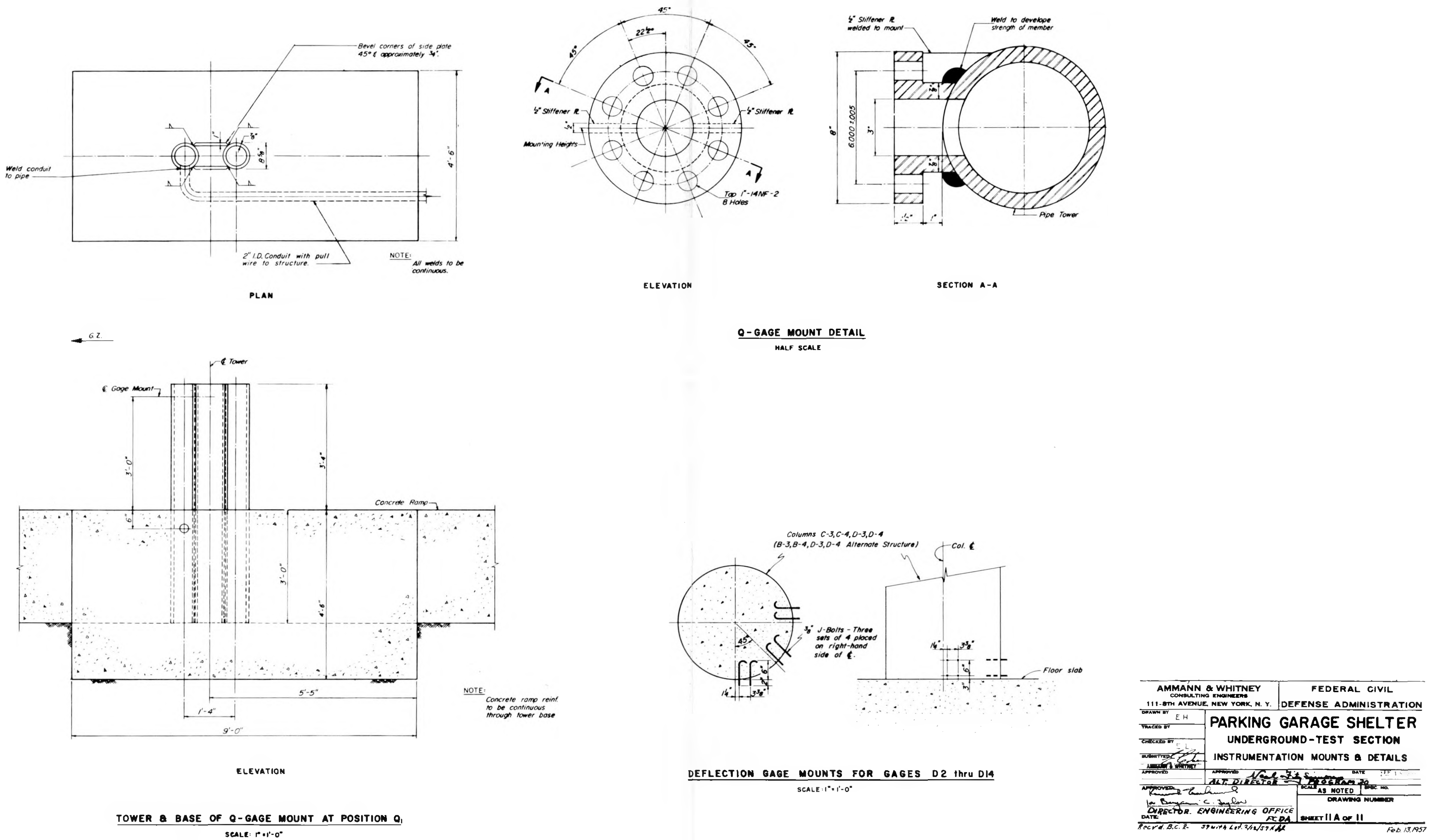

Fig. A.10b—Instrumentation mounts and details.