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AIRCRAFT SHELTER CONCEPTUAL STUDY

Civil/Nuclear Systems Corporation
Albuquerque, New Mexico 87102

January 1977

Final Report

Distribution limited to US Government agencies only because test and evaluation information is discussed in the report (Jan 77). Other requests should be referred to AFWL/DES, Kirtland AFB, NM 87116.

AIR FORCE WEAPONS LABORATORY
Air Force Systems Command
Kirtland Air Force Base, NM 87117

This final report was prepared by the Civil/Nuclear Systems Corporation, Albuquerque, New Mexico under Contract F29601-75-C-0128, Job Order 21011015, with the Air Force Weapons Laboratory, Kirtland Air Force Base, New Mexico. Captain Choate (DES) was the Laboratory Project Officer-in-Charge.

This technical report has been reviewed and is approved for publication.

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Concepts for shelters to protect aircraft from severe nuclear and nonnuclear weapon threats are developed and evaluated. Three of the more promising of the preliminary concepts considered are developed to the point where cost and merit can be evaluated and compared to select the optimum shelter system. The optimum system, the Covered Corridor Concept, is developed to the preliminary design stage. The concept is defined in detail; design requirements and criteria are presented; calculations and analyses of primary →

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structural elements subjected to the design loads are documented; and preliminary design drawings are prepared. Ground shock and shelter motions due to the specified threat are estimated and the potential effect of such motions on sheltered aircraft are discussed. Primary areas of uncertainity which should receive further study prior to proceeding with more detailed design are indentified.

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SECTION I

INTRODUCTION

1. BACKGROUND

Current generation aircraft shelters for tactical aircraft have been designed and constructed to defeat a specific conventional weapons threat level. If that threat level increases, upgrade of existing shelters and design of entirely new shelters become imminent. Research is currently underway to develop techniques for upgrading existing TAB VEE aircraft shelters. However, investigation has shown that upgrade of the large second and third generation aircraft shelters (82-ft and 71-ft arch spans, respectively) and closure systems to defeat significantly higher threat levels is economically unattractive. There is, therefore, a need for totally new aircraft shelter concepts which can provide effective protection against high level threats.

2. OBJECTIVE AND SCOPE

The objective of this study is to develop an optimum concept for an aircraft shelter system which will provide protection against a high level threat. The scope of the study includes analysis, evaluation, definition, and preliminary design. Preliminary concepts were developed for several systems capable of satisfying the operational and protection level requirements. From the preliminary concepts, three promising concepts were selected for further investigation.

Each of the selected concepts was subsequently developed to the point where system cost and merit could be evaluated and weighed against one another to select the most optimum system.

The most promising of the three candidate concepts was selected and developed to the preliminary design stage. This selected concept was defined in detail; design criteria were developed; calculations and analyses which verify protection against the specified weapon threat were documented; and preliminary engineering drawings from which a typically qualified Architect-Engineer firm could develop final design working drawings were prepared.

3. ORGANIZATION OF REPORT

The following section summarizes the operational requirements which must be provided to make a concept usable. Next, the protection requirements which must be satisfied to assure survival of a concept against the given threat are presented in section III. Section IV discusses the candidate concepts selected for evaluation. Evaluation of these concepts is presented in section V. The features of the most promising concept chosen for further development are described in section VII. Section VIII presents design calculations for the various shelter components and section IX, the results of analyses of designs. Section X presents ground shock and shelter motion predictions and discusses their effect on sheltered aircraft. A cost estimate for the selected concept is included in section

XI. Section XII identifies areas of uncertainty which should receive further study before the shelter design is finalized.

SECTION II

OPERATIONAL REQUIREMENTS

1. GENERAL

All shelter concepts considered are required to provide adequate clearance for the following aircraft.

- A-7, all versions
- A-10
- F-4, all versions
- F-15
- F-100
- F-101
- F-105
- F-111 with fully extended wings

At least 1 meter horizontal and 0.5 meter vertical clearance from the top of all parts of the aircraft is required. It is not necessary, however, to provide space for turn-around of the aircraft inside the shelter.

2. AIRCRAFT CHARACTERISTICS

Information on aircraft dimensions was obtained primarily from the Aeronautical Systems Division of the Air Force Systems Command and from official Air Force publications. General plan view dimensions of interest are illustrated in figure 1. Figures 2 through 9 show plan view envelopes for each of the aircraft of interest. All envelopes for aircraft with folding wings are shown with their wings in flight position.

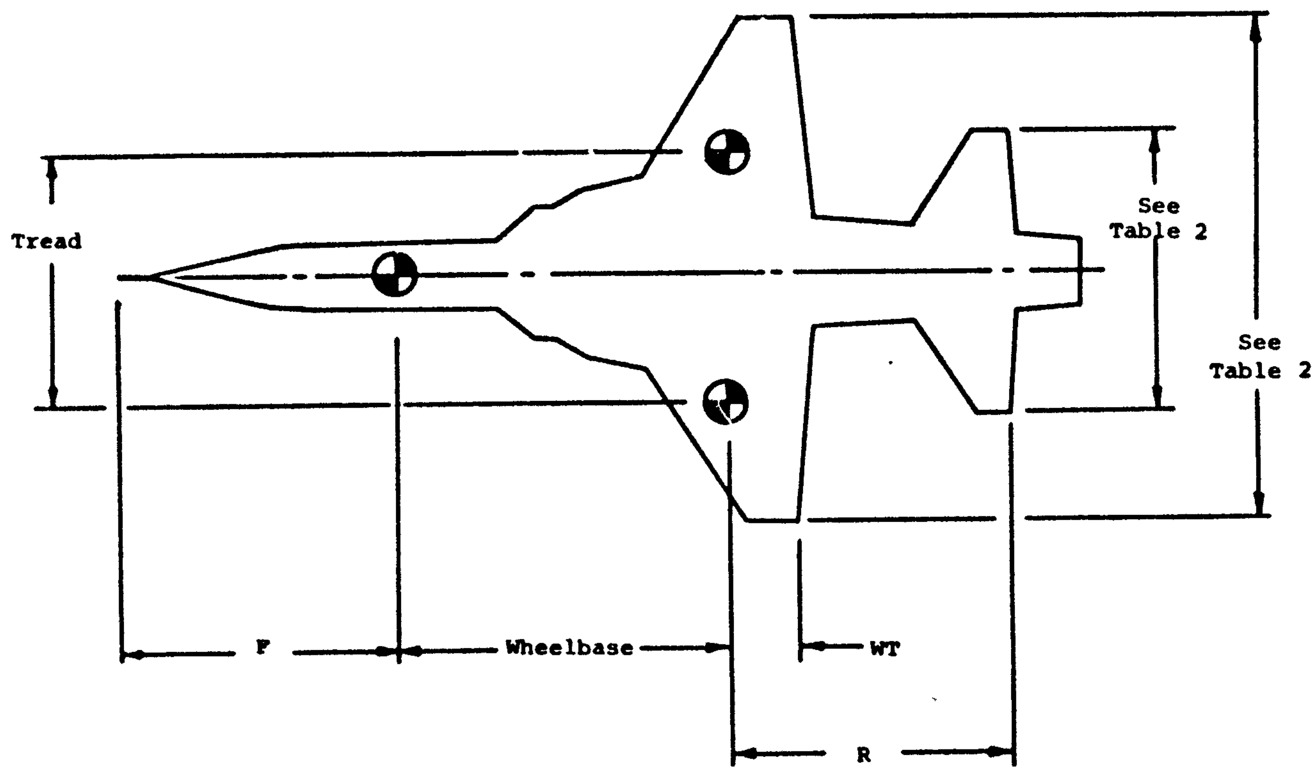


Figure 1. Characteristic Aircraft Dimensions

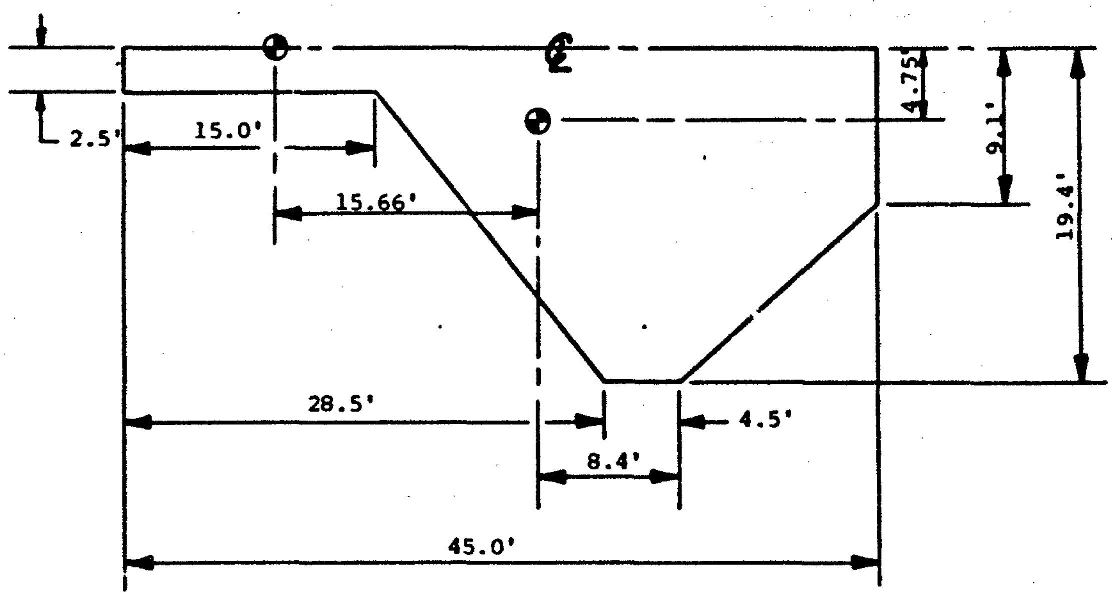


Figure 2. A-7 Plan View Envelope

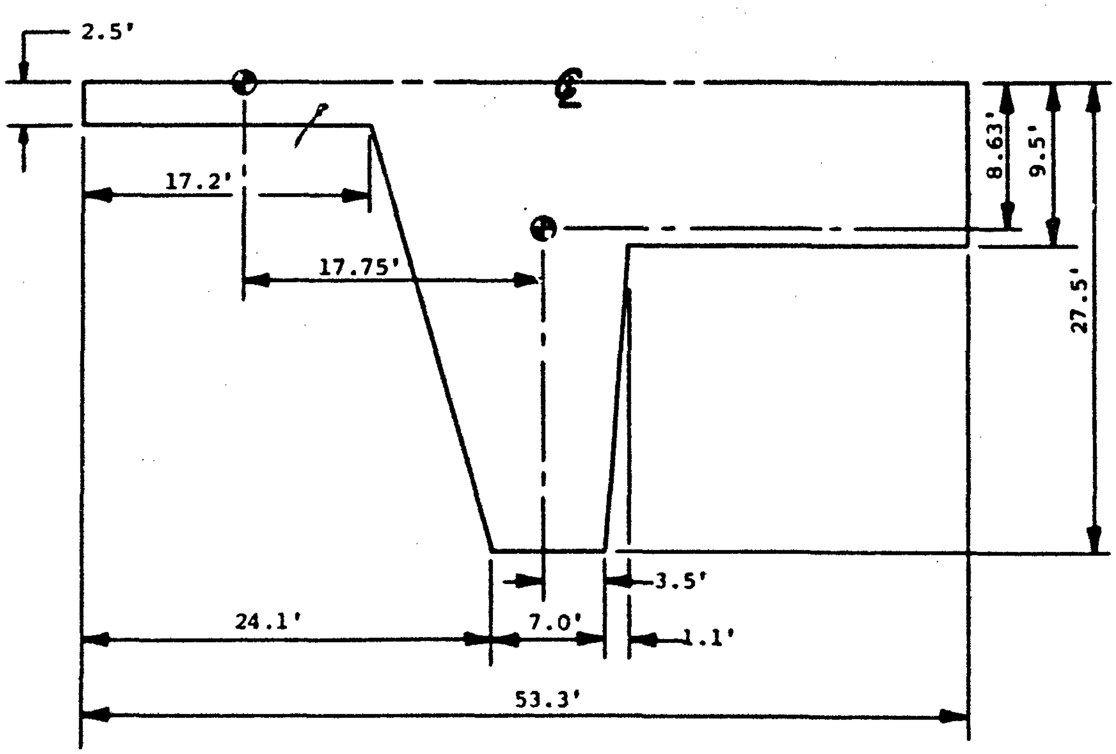


Figure 3. A-10 Plan View Envelope

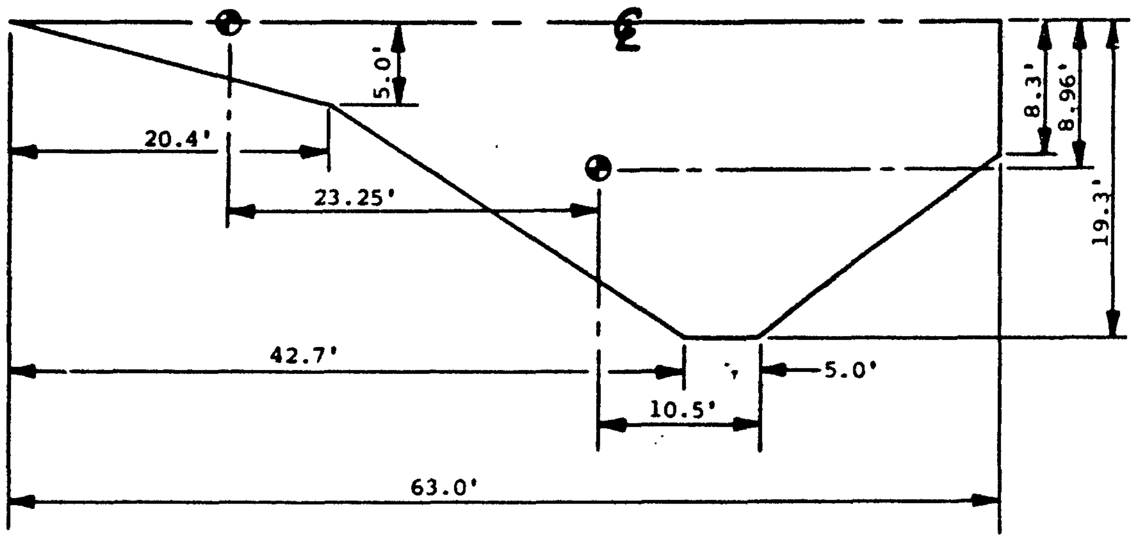


Figure 4. F-4E and RF-4 Plan View Envelope

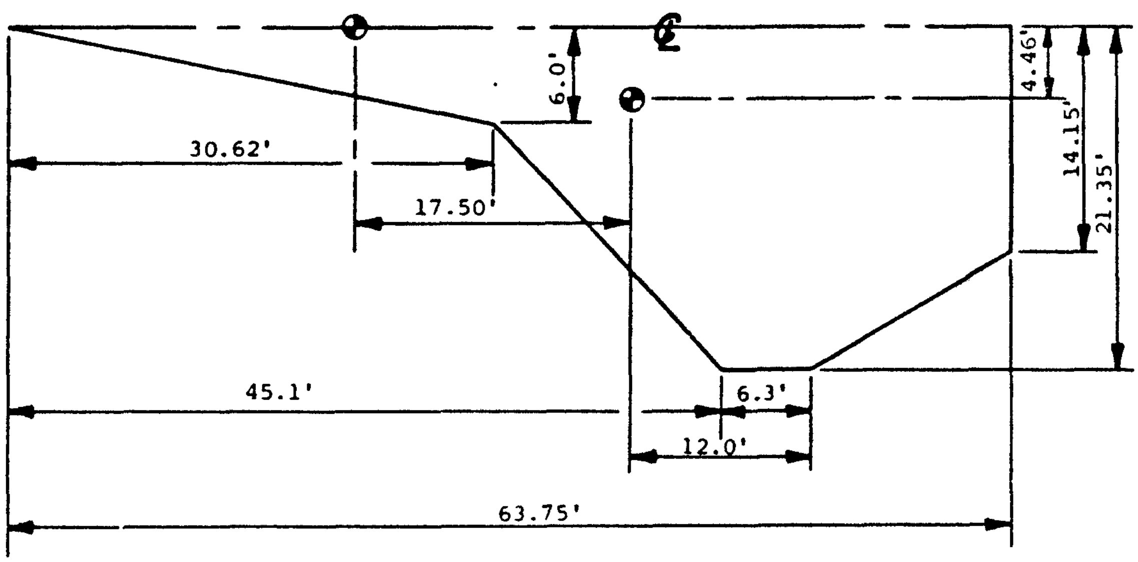


Figure 5. F-15 Plan View Envelope

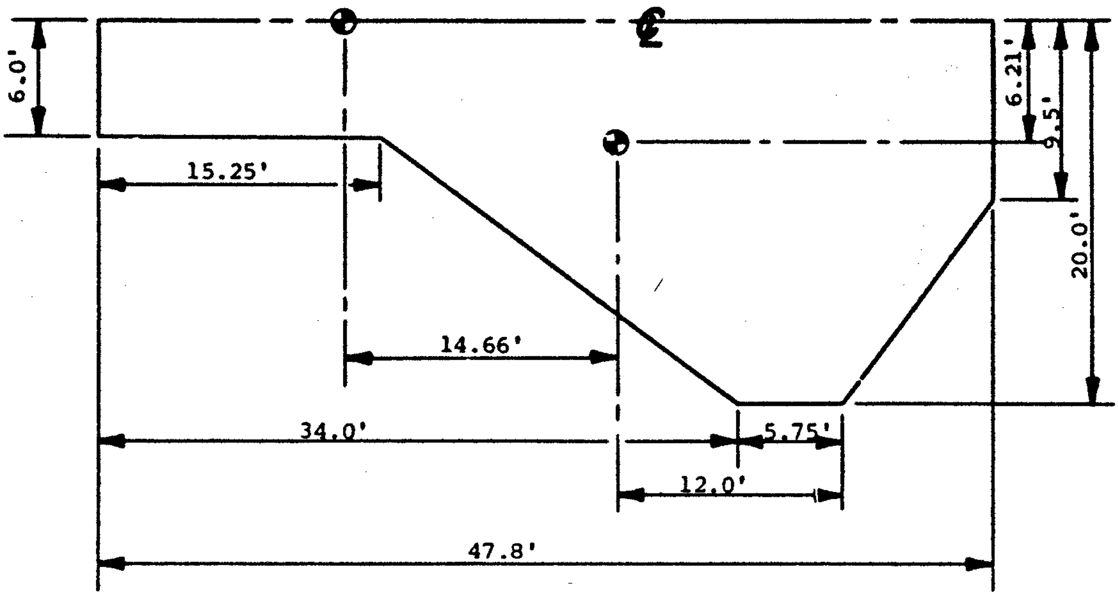


Figure 6. F-100D Plan View Envelope

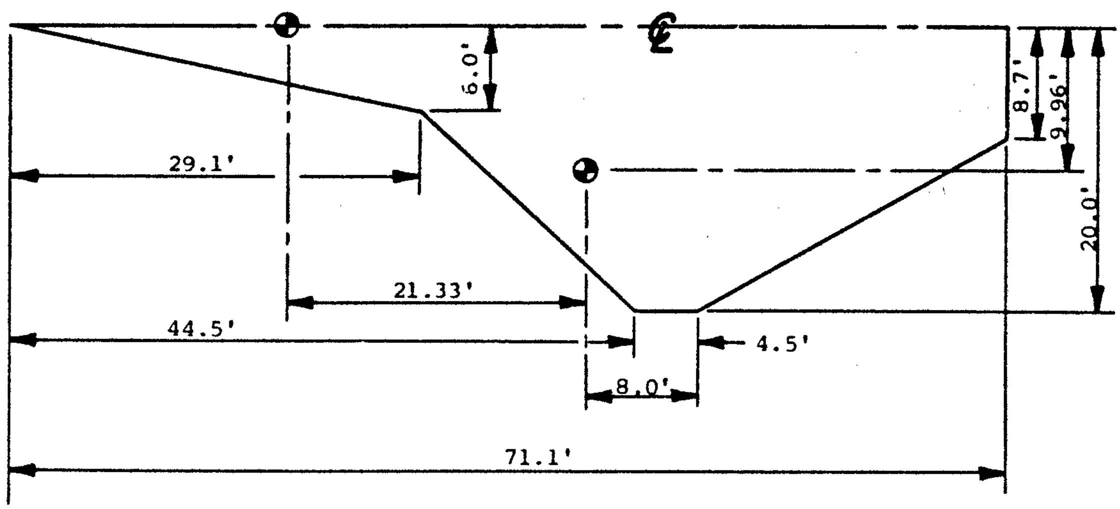


Figure 7. F-101 Plan View Envelope

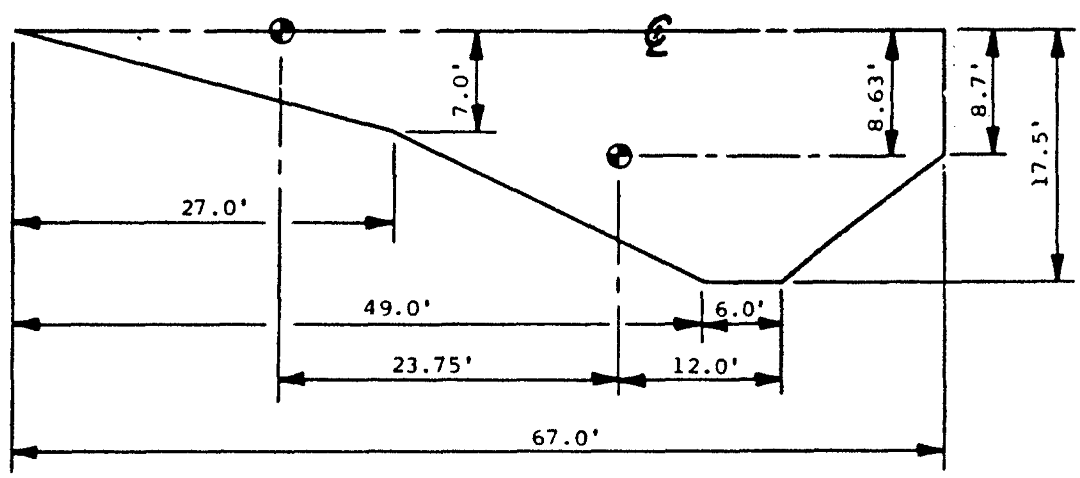
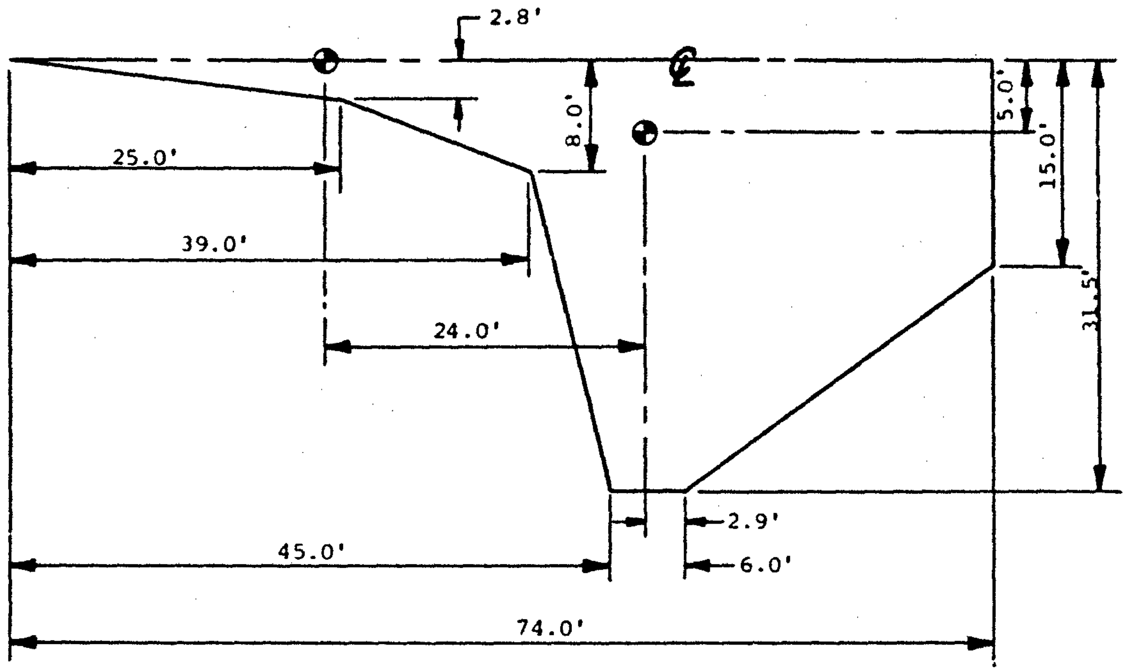
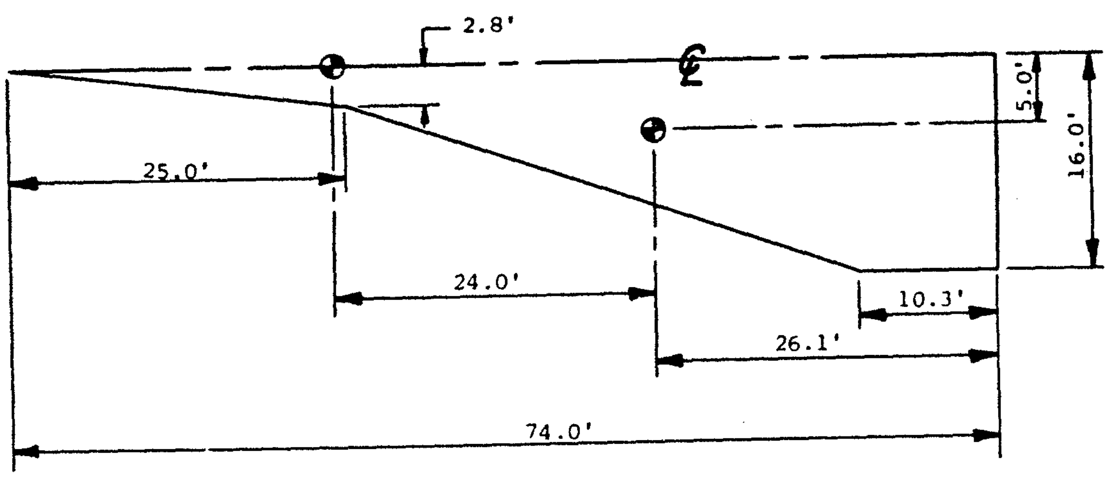


Figure 8. RF-105 Plan View Envelope



a. Wings Fully Extended



b. Wings Fully Swept

Figure 9. F-111 Plan View Envelope

Pertinent aircraft plan view dimensions are summarized in table 1. Table 2 lists governing front view dimensions for these same aircraft. The aircraft weights and wheel loads utilized in this study are shown in table 3.

3. CLEARANCE ENVELOPES

a. General

Internal dimensions of the shelter concepts must be sufficient to allow easy movement of aircraft within the interior. Movement can occur in a straight line or in a circular arc when turning from one region of a shelter to another. Since aircraft may be required to turn through angles of up to 90 degrees, consideration must be given to the required clearances for nose tips, wing tips, and horizontal and vertical stabilizers.

b. Straight Line Clearance

Clearance requirements for the various aircraft of interest moving in a straight line are plotted in figure 10. The minimum opening envelope shown includes the 0.5 meter vertical and 1 meter horizontal required clearances from outermost aircraft surfaces.

c. Turning Clearance

Clearance requirements for turning aircraft are different from those shown in figure 10. Two different turning conditions were studied. The first case is for an aircraft turning between two perpendicular pairs of vertical

Table 1

AIRCRAFT PLAN VIEW DIMENSIONS

Aircraft	Turn* Radius ft	Wheelbase in	Tread in	F ft	R ft	WT ft	Length ft
A-7	18.7 (NG)	188	114	9.2	20.3	8.4	46.1
A-7 (folded wings)	18.7 (NG)	188	114	9.2	20.3	8.4	46.1
A-10	-	213	207	10.2	25.5	3.5	53.3
F-4 E	24.9 (NG)	279	215	14.6	25.1	10.5	63.0
F-4 E (folded wings)	24.9 (NG)	279	215	14.6	25.1	14.6	63.0
RF-4	24.9 (NG)	279	215	14.6	25.1	14.6	63.0
RF-4 (folded wings)	24.9 (NG)	279	215	14.6	25.1	14.6	63.0
F-15	-	210	107	21.7	24.7	12.0	63.75
F-100 D	27.4 (NG)	176	149	20.0	20.0	12.0	47.8
RF-101 C	32.8 (NG)	256	239	20.0	30.5	8.0	67.9
F-105 D/G	36.0 (NG MG)	285	207	20.5	25.0	12.0	67.0
F-111 (fully extended)	37.5 (NG)	288	120	21.8	26.1	2.9	75.5
F-111 (26° swept)	37.5 (NG)	288	120	21.8	26.1	-	75.5
F-111 (fully swept)	37.5 (NG)	288	120	21.8	26.1	18.0	75.5

* (NG) = Nose Gear
(MG) = Main Gear

Table 2

AIRCRAFT FRONT VIEW DIMENSIONS

Aircraft	TRANSVERSE DISTANCE FROM AIRCRAFT CENTERLINE					
	Wing Extreme (ft)		Horizontal Stabilizer Extreme (ft)		Vertical Stabilizer Extreme (ft)	
	Horiz	Vert	Horiz	Vert	Horiz	Vert
A-7	19.4	6.0	9.1	4.0	0.	16.1
A-7 (folded wings)	11.9	14.0	9.1	4.0	0.	16.1
A-10	28.8	9.3	10.0	7.1	9.5	14.9
F-4 E	19.2	6.0	8.2	5.6	0.	16.5
F-4 E (folded wings)	13.8	11.3	8.2	5.6	0.	16.5
RF-4	19.2	6.1	8.2	5.6	0.	16.5
RF-4 (folded wings)	13.8	11.3	8.2	5.6	0.	16.5
F-15	21.4	8.2	14.1	7.9	5.6	18.9
F-100 D	19.4	4.2	9.5	3.0	0.	16.2
RF-101 C	19.9	5.6	8.7	17.0	0.	18.0
F-105 D/G	17.5	7.6	8.7	5.1	0.	20.5
F-111 (fully extended)	31.5	8.4	14.5	7.9	0.	17.3
F-111 (26° swept)	30.2	8.3	14.5	7.9	0.	17.3
F-111 (fully swept)	16.0	7.5	14.5	7.9	0.	17.3

Table 3

AIRCRAFT WEIGHTS AND WHEEL LOADS

Aircraft	Maximum Take-off Load kips	Equivalent Single Wheel Load kips	Tire Pressure psi	Contact Area sq in	Footprint Width in
A-7	42.0	17.5	280	62	6.9
A-10	45.6	22.8*	-	-	-
F-4E	61.7	27.0	265	102	8.9
F-15	40.0	20.0*	-	-	-
F-100	39.1	19.5*	290	62	6.9
RF-101	51.0	21.1	290	73	7.5
F-105	54.6	23.4	220	106	9.0
F-111	98.9	47.0	150	313	15.5

*1/2 maximum take-off load

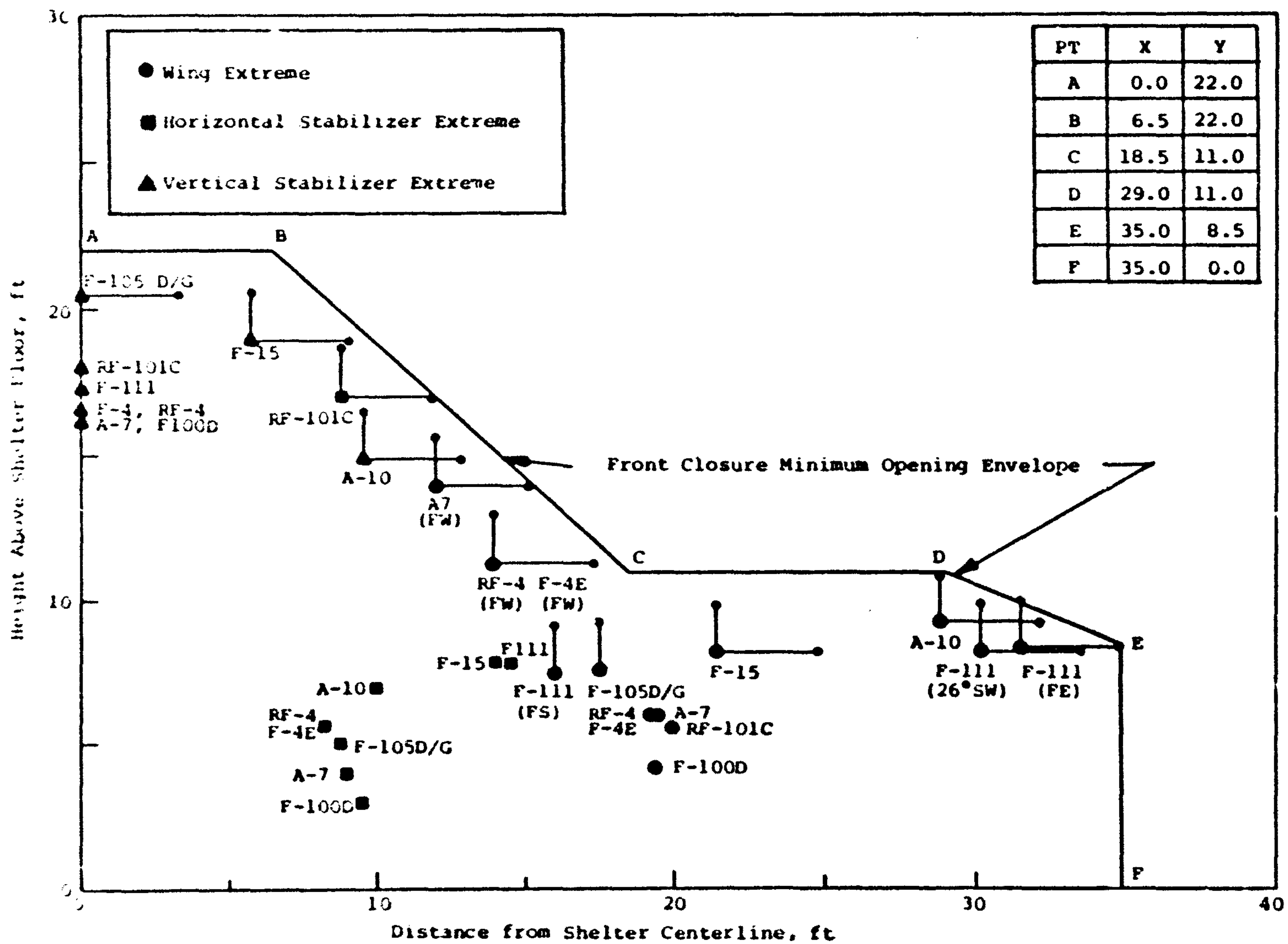


Figure 10. Straight Line Clearance Requirements

sidewalls such as revetments. In this case, there is no cover over the corridor to interfere with stabilizers. The second case involves turning of an aircraft from one fully covered corridor to another where there is a possibility that the stabilizers can contact the top of the corridor.

The first study was done as a preliminary exercise to determine the absolute minimum spacing that parallel vertical walls could be set apart and oriented at right angles to each other and still enable the applicable aircraft to negotiate perfect 90-degree turns. The most ideal turning radius was used for each of the aircraft considered. The results were that all but the A-10 and F-111 with wings fully extended could make the turn if the parallel walls were at least 46 feet apart. The A-10 would require about 58 feet between walls, and the F-111 with its wings fully extended would require about 63 feet. In view of the fact that aircraft cannot be expected to make perfect turns, some clearance would have to be added to the dimensions indicated for the wings and/or nose tips to safely clear the sidewalls.

The second turning condition involves the determination of the clearance required when the paths of travel are oriented as above but fully enclosed overhead. Assumptions made for this condition are

- Aircraft begin and complete their turns on corridor centerlines.

- The turn is made by swinging at a constant turning radius with the point of rotation located at the intersection point of the two corridors.
- One-half meter vertical and two meters horizontal clearances from outermost surfaces of the aircraft will be necessary.

Clearance requirements for the various aircraft are summarized in table 4. These clearances were obtained through application of a graphical solution technique and represent realistic approximations. Note that this table looks very similar to table 2, since only the horizontal dimensions for the wings and stabilizers have been altered.

The point coordinates in table 4 which define a minimum clearance envelope for turning aircraft are plotted in figure 11 with the 0.5 and 2 meter vertical and horizontal allowances added to the basic aircraft extremes. This figure indicates that a width of 76 feet at a height of 7.75 feet is required to provide reasonable clearances for turning. Since an arch will be wider at the base, a nominal diameter of 80 feet is indicated to provide the required clearances.

Examination of the clearance requirements in figures 10 and 11 indicates that the size of the shelter is primarily controlled by the F-111 and A-10. If these two aircraft were deleted from the set being considered, the 80-foot dimension could be reduced to 60 feet. The result would be a considerable

Table 4

CRITICAL TURNING AIRCRAFT DIMENSIONS

Aircraft	TRANSVERSE DISTANCE FROM ARC OF 40-FT RADIUS					
	Wing Extreme (ft)		Horizontal Stabilizer Extreme (ft)		Vertical Stabilizer Extreme (ft)	
	Horiz	Vert	Horiz	Vert	Horiz	Vert
A-7	19.4	6.0	13.5	4.0	5.0	16.1
A-7 (folded wings)	11.9	14.0	13.5	4.0	5.0	16.1
A-10	28.8	9.3	16.0	7.1	16.0	14.9
F-4 E	19.2	6.0	14.5	5.6	7.0	16.5
F-4 E (folded wings)	13.8	11.3	14.5	5.6	7.0	16.5
RF-4	19.2	6.1	14.5	5.6	7.0	16.5
RF-4 (folded wings)	13.8	11.3	14.5	5.6	7.0	16.5
F-15	21.4	8.2	19.5	7.9	12.0	18.9
F-100 D	19.4	4.2	14.0	3.0	5.0	16.2
RF-101 C	19.9	5.6	17.5	17.0	10.5	18.0
F-105 D/G	17.5	7.6	15.0	5.1	7.0	20.5
F-111 (fully extended)	31.5	8.4	21.0	7.9	8.0	17.3
F-111 (26° swept)	30.2	8.3	21.0	7.9	8.0	17.3
F-111 (fully swept)	16.0	7.5	21.0	7.9	8.0	17.3

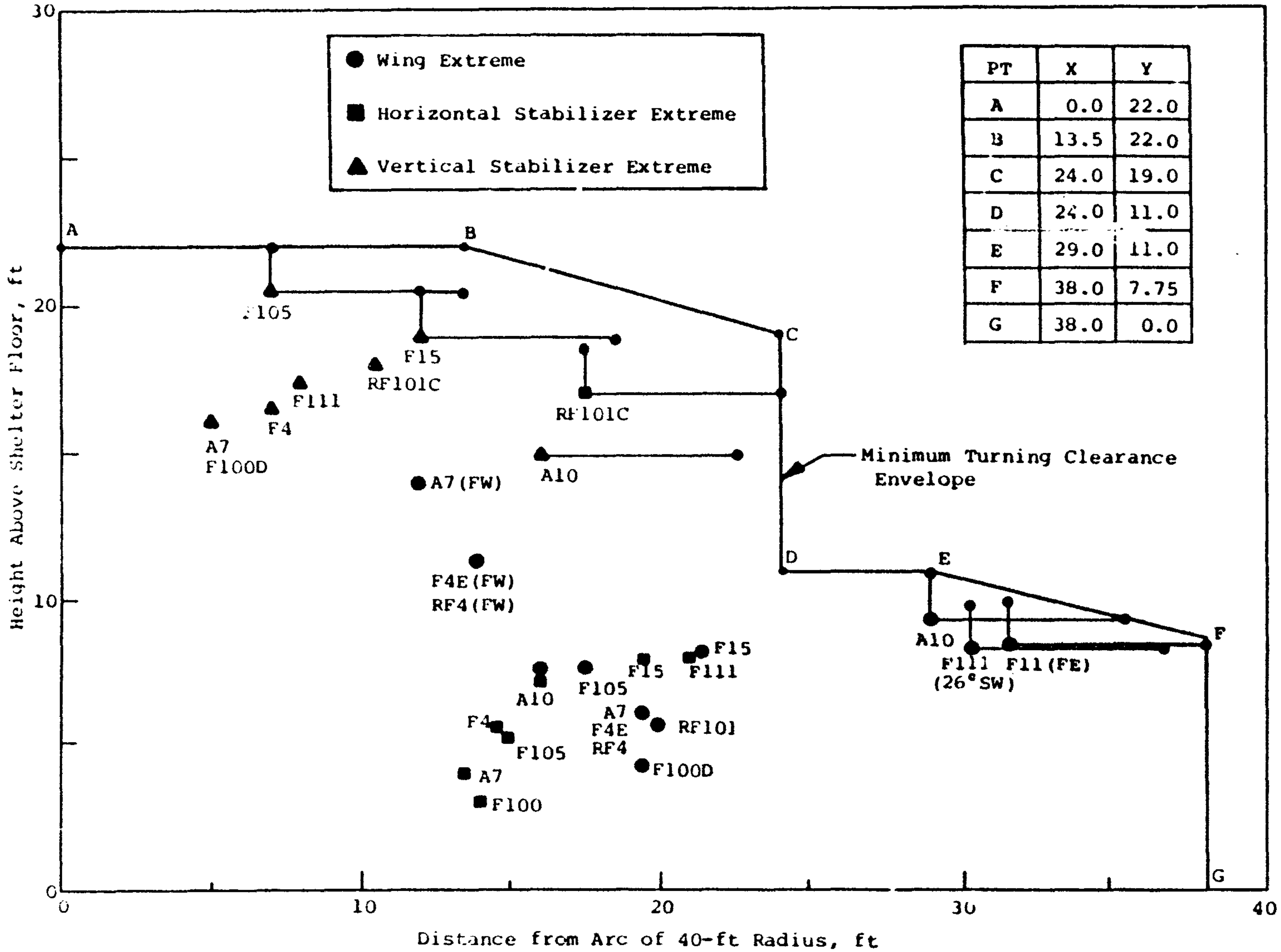


Figure 11. Turning Aircraft Clearance Requirements

cost reduction. However, several of the larger shelters would still be required at each installation where the larger aircraft are stationed. Consequently, the 80-foot span was selected as the baseline configuration for this study. This dimension will be used consistently henceforth for both corridors and shelters. The study results could be applied to smaller shelters by scaling down to the appropriate size.

4. MAXIMUM RAMP SLOPE

In shelter concepts involving sloping ramps, the grade of the ramp is determined by the ability of the aircraft to ascend the ramp under power or the tractive force of a towing vehicle. Based upon a compilation of all data received from various sources on the set of aircraft being considered, a value of 2.5 percent has been chosen as the maximum ramp slope for use in this study.

5. INSIDE START-UP CAPABILITY

It was found very early in the study that a requirement for inside start-up capability for each aircraft could easily drive the concept selection, since provision of total inside start-up capability in some shelter configurations is totally impractical. For example, in those configurations where aircraft are positioned one behind the other, a removable exhaust deflector and exhaust duct would be required for each of the forward aircraft. The exhaust problem is more tractable for the last aircraft in line, but even here, a stationary port in the endwall would have to be very large or fitted with a movable adapter to accommodate the range of exhaust locations involved with the set of aircraft being considered.

If inside start-up were required for maintenance purposes only, it would be possible to designate at least one storage position in the shelter where an aircraft engine could be operated. The large exhaust port/adapter problem would also exist here. Rough calculations indicate that the exhaust duct to the outside of the shelter would have to be at least 12 feet in diameter. The duct itself, therefore, would constitute a hardened structure of significant magnitude. In addition, a rather massive blast closure and associated movement mechanism would be required at the outlet to prevent airblast effects from entering the shelter proper.

The above considerations, coupled with the likelihood that tow vehicles or tow lines in the shelter floor may be necessary in any event to safely ingress and egress aircraft, led to the decision not to require total inside start-up capability during the preliminary concept development and evaluation phase. It was also decided, however, that both the requirement for and the feasibility of providing inside start-up capability for at least one location in the shelter would be more thoroughly evaluated during later stages of the study and development of the optimum concept.

6. AEROSPACE GROUND EQUIPMENT

All shelter concepts must include provisions for storage of essential aerospace ground equipment (AGE) along with the aircraft. A description of the AGE normally provided in conjunction with TAB VEE shelters (ref. 1) is presented in appendix A. These representative items have been used in this study as the basis for estimating the amount of AGE storage required. Examination of the space requirements for the various AGE components listed in appendix A indicates that all the equipment can be stored in an area of approximately 400 square feet.

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1. Naugle, D.F., Haney, J.T., Carroll, G.B., Environmental Testing in Aircraft Shelters, AFWL-TR-73-96, Air Force Weapons Laboratory, Kirtland AFB, New Mexico, July 1973.

SECTION III

PROTECTION REQUIREMENTS

1. INTRODUCTION

The weapon threat spectrum which candidate aircraft shelter concepts must survive is specified in a classified attachment to the statement of work for contract number F29601-75-C-0128. When particular weapons of the threat spectrum are mentioned herein, they are referred to as simply weapon no. 1, 2, etc., with the numbers corresponding to the order in which the weapons are listed in the classified document. Following paragraphs address the various weapon effects resulting from the overall threat and summarize minimum protection parameters or thicknesses determined by preliminary analyses for concept development and evaluation.

2. CRATER EFFECTS

Crater effects have been established utilizing the procedures of references 2 and 3. In so doing, it was found that all weapons were insignificant in terms of crater effects except for weapon nos. 1 and 4.

The shelters are not proposed to be designed to be within nor immediately adjacent to a nuclear crater. Consequently,

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2. The Air Force Manual for Design and Analysis of Hardened Structures, AFWL TR 74-102, Air Force Weapons Laboratory, Kirtland AFB, N.M., Oct. 74.

3. Protection from Nonnuclear Weapons, AFWL TR 70-127, Air Force Weapons Laboratory, Kirtland AFB, N.M., Feb. 71.

the only nuclear crater effects of concern are missiles and ejecta. Of these two, crater missile effects can be shown to be insignificant with respect to more severe impact threats from other aspects of the threat spectrum.

The most probable ejecta depth to be expected has been established as 1 foot. While such depth of ejecta poses no particular structural problems, measures must be taken to ensure that post-attack operational capability, particularly of closures, is not impaired.

The most severe craters produced by the conventional weapon threats due to direct hits on reinforced concrete and soil are expected to be

	Reinforced Concrete	Soil
Apparent Crater Depth	1.3 ft	4.0 ft
Apparent Crater Radius	4.0 ft	13.0 ft

Missiles and ejecta effects from these craters are overshadowed by other effects in the threat spectrum.

3. PENETRATION

The most severe kinetic energy penetration effects will be due to weapon no. 2. A weapon with very similar characteristics and impact conditions can be expected to perforate about 1.3 feet of 5000 psi concrete with the perforation thickness being limited by case breakup according to reference 3. With armor piercing case modifications, the weapon could possibly perforate 3.5 feet of 5000 psi concrete.

Impacting on soil, weapon no. 2 could conceivably penetrate to a depth of 15 to 21 feet as an inert projectile. Considering typical delay fuze options, however, the weapon would be expected to detonate at a depth of from 3 to 8 feet.

It will be necessary, then, to keep two aspects of the penetration phenomenon in mind; the effects from impact on exposed structural members of concrete and steel and the effects from impacting on soil.

4. BREACHING

Weapon no. 2 with a delay fuze option poses a severe breaching threat when impacting on a soil cover overlying shelter structural elements. After taking account of case effects, but assuming fully tamped placement, weapon no. 2 employed as a breaching charge could presumably breach 5 feet of concrete based on charge weight according to reference 3.

A 6-foot thickness of concrete is considered to be adequate for protection against breaching of a buried structural element by weapon no. 2. Penetrating weapon and breaching charge characteristics are almost mutually exclusive. An elongated shape, sharp ogive, and a thick, strong case are desirable characteristics of a penetrating weapon. These characteristics are directly contradictory to the compact charge shape, no case, and zero standoff features desired for a breaching charge. The practically negligible possibility of the CG of a penetrating weapon charge coming to rest against a buried structural element in a fully tamped condition at the time of detonation led to the conclusion that 6 feet of concrete is adequate protection against breaching in a buried configuration.

In considering breaching of exposed structural elements, the crater effects summarized in paragraph 2 are of interest. A direct hit by weapon no. 1 on massive reinforced concrete is expected to cause a crater about 1.3 feet deep. Weapon no. 2 with a delay fuze option impacting exposed massive 5000 psi structural concrete could be expected to cause a crater 2 feet deep if the weapon detonates with its CG at the surface of the concrete. For a detonation with the CG 2 feet below the surface, a crater 4 feet deep could be expected. Consequently, a 6-foot thickness of reinforced concrete is considered sufficient protection from breaching of exposed structural elements.

5. AIRBLAST

After evaluating the airblast effects from all weapons in the threat spectrum, the following parameters have been selected as a reasonable envelope for the airblast loading functions which the shelter structural elements must withstand.

a. Pure Impulse

The most severe loading applied as pure impulse is estimated to be approximately 12 psi-sec due to weapon no. 1.

b. Incident Overpressure

A peak incident overpressure of 250 psi resulting from weapon no. 4 has been selected as a representative value for this parameter. Effective times of duration for equivalent triangular loading functions utilized in the preliminary analyses are determined as described in reference 2.

c. Incident Dynamic Pressure

The peak dynamic pressure chosen is 430 psi with corresponding effective times of duration also determined with reference 2.

d. Reflected Pressure

Variations of the reflected overpressure and dynamic pressure drag coefficient with the angle of incidence of the structural element under consideration are as shown in figure 12.

e. Structural Requirements

(1) Shelter

Since reflection enhances the applied pressure due to airblast by almost an order of magnitude, it can easily be shown that the only effective manner for defeating airblast is to bury the structures, shield them, or impose the requirement that the angle of incidence be less than 40 degrees. Thus, the preliminary calculations were performed with the assumption that the structure was totally buried so that

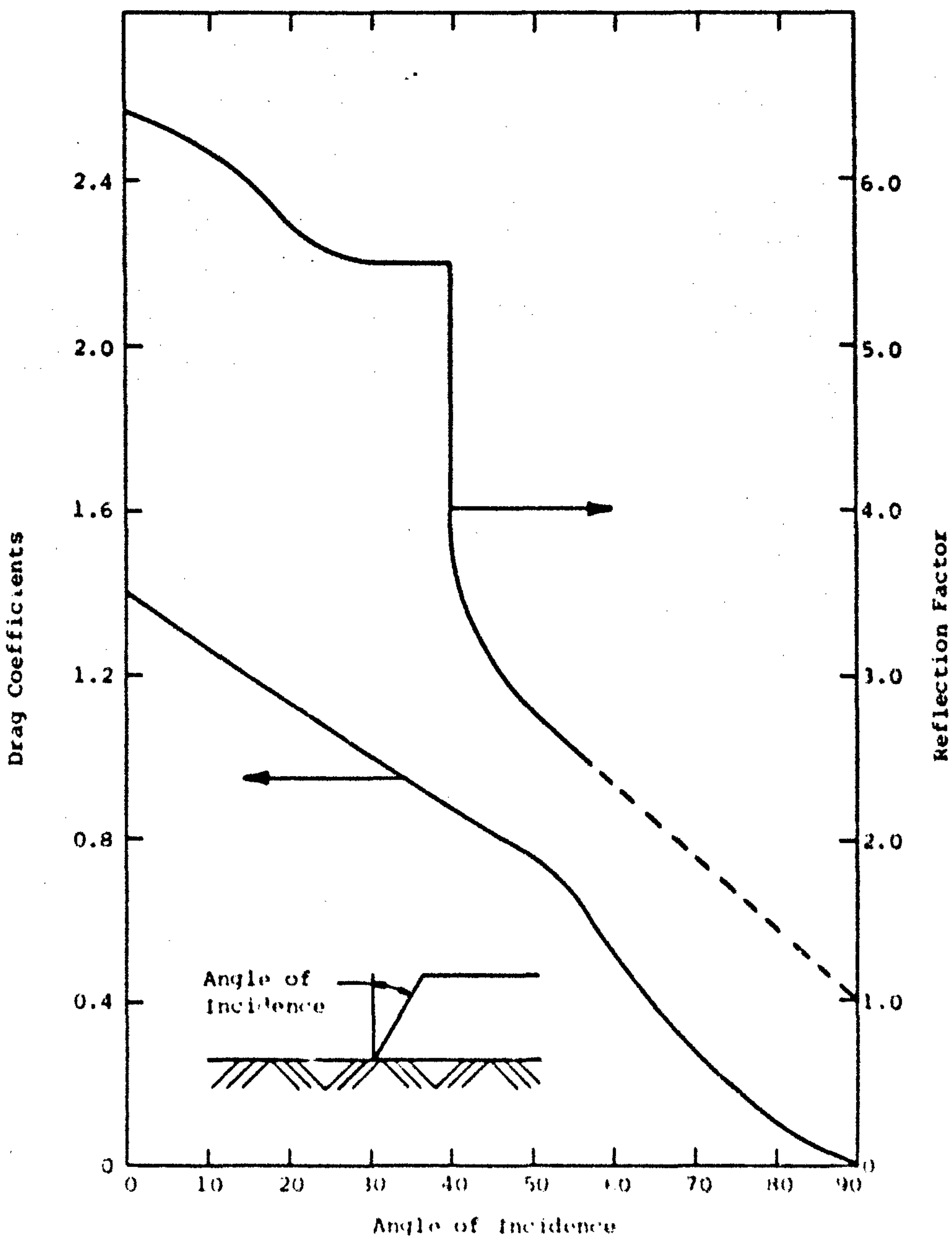


Figure 12. Variation of Airblast Parameters with Angle of Incidence

dynamic and reflected pressures were not part of the forcing function. Burial was assumed to be achieved if the structure was placed completely underground, or if gently sloped soil berms and cover were used with structures that are partially or completely above ground.

Initial hand calculations indicated that a circular shape for the shelter resulted in a rather severe penalty in the amount of material required when compared with other configurations that provided the same resistance. These results and the general shape of the minimum envelope of figure 11 suggested that half an ellipse with ties across the base would be the most desirable shelter configuration.

A NASTRAN dynamic analysis was performed to determine the required moment capacity of the arch cross section and to evaluate the effect of the aspect ratio on the maximum predicted moment in the arch. For all cases, the forcing function used in this analysis was an approximation to one that might be expected from a nuclear detonation. The spatial distribution was assumed to be sinusoidal with the peak value at the crown and zero at the shelter base as shown in figure 13. The temporal distribution was taken to be a triangular shape with the same impulse as an exponentially decaying pulse (zero rise time in both cases).

A rather coarse finite element model was used to represent the elliptical shaped arches with a floor segment

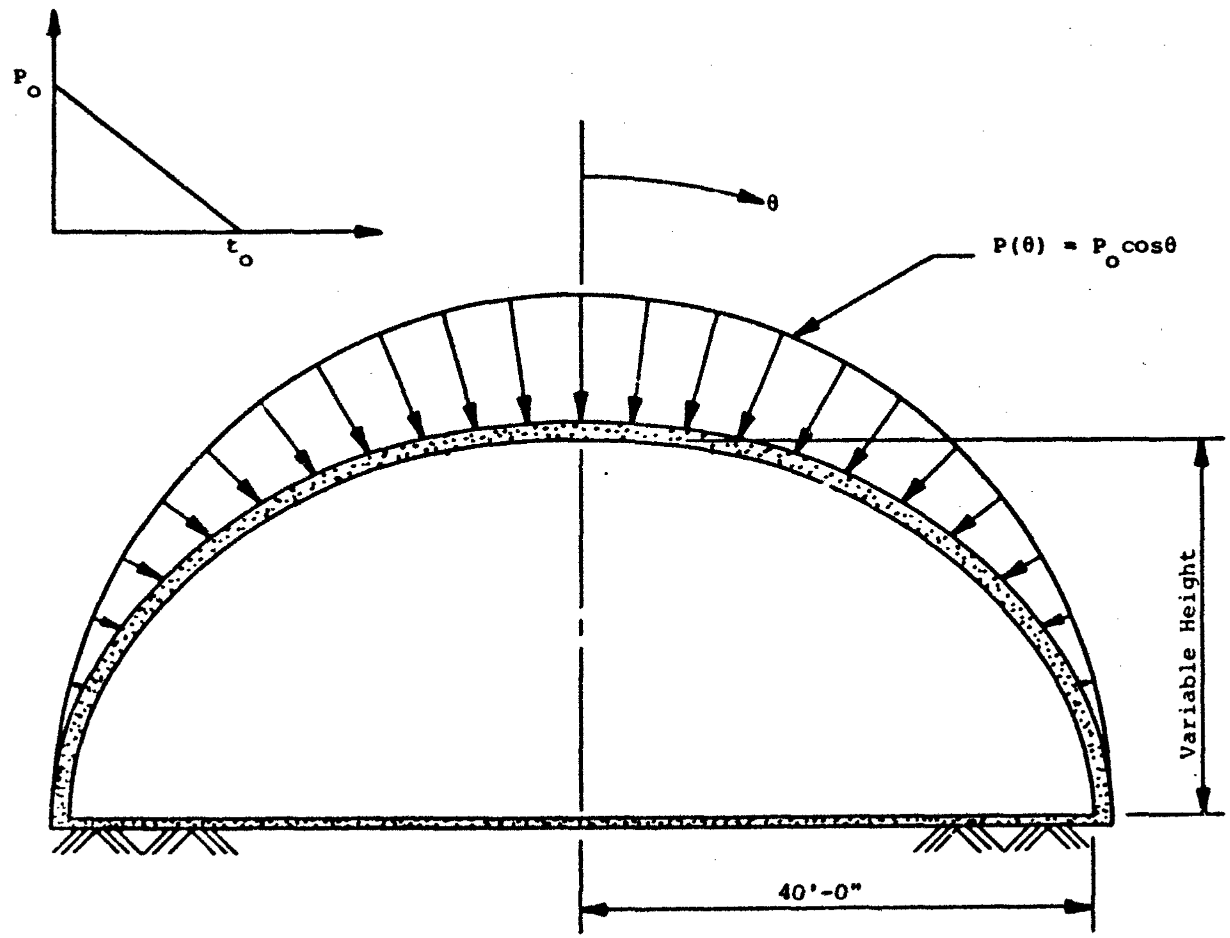


Figure 13. Forcing Function and Structural Configuration Used for NASTRAN Dynamic Analyses

that circumscribed the envelope of figure 11. A base width of 80 feet was used in all cases. The stiffness coefficient for the arch used in the analysis was one that corresponded to an 8-foot thick reinforced concrete section with 0.375 percent steel on each face. The floor was taken to be 3 feet thick nominally reinforced on each face. Since the predicted maximum moments based on the assumption of elastic behavior were not significantly affected by changes in stiffness, the task of selecting a suitable structural cross section reduced to that of meeting the ultimate moment requirement.

For the range of aspect ratios that accommodated the minimum opening envelope, the predicted maximum bending moment of 8×10^6 in-lb/in, which occurred at the crown and the spring line of the ellipse, did not depend on the arch shape to any great extent. Consequently, an elliptical configuration with a ratio of major to minor axis of 4:3 was chosen since this particular shape required the least amount of material to provide the amount of interior space required and presumably would be the least costly of the various shapes considered.

A design based on purely elastic assumptions is unduly conservative for an application such as this since it does not utilize the inelastic capacity of the structure. Furthermore, the thrust in a typical cross section is quite large so that the moment capacity of a reinforced concrete section can be considerably larger than that given by the ultimate moment equation for pure bending. Consequently, it was assumed that a cross section that provided 3×10^6 in-lb/in would be reasonable for preliminary design and evaluation.

Since a thickness of at least 6 feet of concrete is required to defeat other threats, a monolithic or composite section must contain at least this equivalent thickness of concrete. Other constraints include construction feasibility, minimum cost and the capability of withstanding the localized high impulse associated with contact detonation of conventional weapons. The use of more concrete does not appear to be a logical way of defeating this latter condition. Instead, a minimum depth of 6 feet of soil cover was chosen which provides an effective standoff distance from the shelter that permits dispersal of the impulse over a larger region.

(2) Closures

For closures that are subjected only to the incident overpressure, reasonable estimates for structural members were obtained with one-degree-of-freedom models. Closures that are exposed to reflected pressures, penetration, or breaching must withstand these additional effects as well. For each of the various concepts, the particular mix of threats that the closure must withstand was taken into account in determining a reasonable cross section. Resulting structural thicknesses vary from about the same as for buried shelters to significantly larger values depending upon the concept of interest.

6. GROUND SHOCK

The peak soil pressure is the same as that selected for the peak incident overpressure, i.e., 250 psi. This value, which occurs at the surface, is attenuated to something less by the procedures of reference 2 for the various depths of interest.

The peak vertical and horizontal free field displacements to be expected are 1.90 feet. Peak vertical and horizontal free field velocities and accelerations are 5.75 fps and 250 g, respectively. As was the case with soil pressure, these parameters are modified according to reference 2 depending upon the particular structural element and location being considered.

7. RADIATION EFFECTS

Considering the minimum structural thicknesses dictated by the penetration/breaching/airblast threat, the in-shelter gamma and neutron radiation levels will not be a threat to most types of equipment. For metal-to-metal closures, thermal and x-ray effects will have to be investigated on a case-by-case basis to ensure that exposed elements are not unduly ablated nor fused together. Since the shelter and closure structural concepts utilize continuous steel liners 0.25 inch or greater in thickness, these liners can also serve as EMP shields. Low carbon steel walls 0.25 inch thick would provide approximately 60 dB attenuation of EMP signal frequencies greater than 400 Hz. Reference 2 suggests a 60 dB attenuation of frequencies greater than 100 Hz as a general specification for surface bursts, but actual requirements should be based on an evaluation of the sensitivity of equipment within the shelter. Reinforcing steel within the shelter and closure can be placed so as to provide additional attenuation of the EMP signal. Penetrations of the steel liner, such as entranceways and utility or communication cables or ducts, must be properly designed to avoid degrading the effectiveness of the shield.

8. NAPALM

Similar to the cases of radiation and airblast effects, protection from napalm can be provided by ensuring that adequate seals are included at all exposed surfaces.

9. SUMMARY

Based on the preceding discussion, the airblast threat is quite severe and large structural members are required to defeat it. If the section incorporates 6 feet of concrete and is buried with at least 6 feet of cover, then the minimum requirements for defeating the breaching and penetration threats are also met.

For structural elements that are not buried or protected, even more massive structural sections will be required depending upon the configuration of interest due to reflected pressures and/or the large impulsive load.

All shelters have a steel lining as an integral part of the structure. With proper sealing and attention to design details, the concrete and steel that is required to withstand airblast/breaching/penetration will also provide adequate protection against radiation effects.

SECTION IV

SHELTER LAYOUT CONFIGURATIONS

1. GENERAL

The initial stages of this study included concepts in which the means of ingress and egress were intertwined with the aircraft shelters. However, the concepts that were chosen for more detailed evaluation had an interesting feature that was common to all. Each concept was symmetrical about a vertical plane with an opening to the shelters on each side of a platform or corridor.

The distinguishing feature of each concept became one associated primarily with closures and access to the shelters. Any one shelter layout could be used equally well with any of the closure systems. Consequently, shelter layouts are discussed in this section to indicate that each layout has certain advantages and disadvantages, but that shelter layout is not a significant factor to be considered in evaluating the final concepts.

The problem of selecting an optimum shelter layout configuration depends on several factors. Some of the more important are

- Ease of ingress or egress
- Availability of real estate
- Interference of one parked aircraft with movement of another
- Multiple purpose usage of structural components

The simplest configuration is probably a single shelter having one stationary endwall with a moveable closure at the other end. The existing TAB VEE shelters are a clear example of this type of construction. As overpressure levels increase, however, the strength of components must be increased accordingly, and special consideration must be given to reducing reflected and dynamic pressures on aboveground structures such as the TAB VEE shelters.

An aerodynamic obstruction, or spoiler, placed in front of an otherwise unprotected closure can greatly reduce the reflected pressure and impulse that the closure must withstand. In the development of layouts herein, the function of a spoiler is performed by locating the shelters such that the closures face each other. Hence, each shelter provides some protection to the opposite closure. This method of shelter placement was selected to avoid the cost of constructing separate spoilers and to utilize space that would otherwise be wasted. Consequently, all the shelter layout concepts shown herein have closures opposite to one another.

2. FOUR AIRCRAFT SHELTER LAYOUTS

The sketches illustrating layouts in this section show a covered corridor between shelters, but the layouts shown are equally compatible with the other shelter concepts chosen for evaluation. Figures 14 and 15 illustrate efficient layouts for four aircraft. Figure 14 demonstrates how two shelters

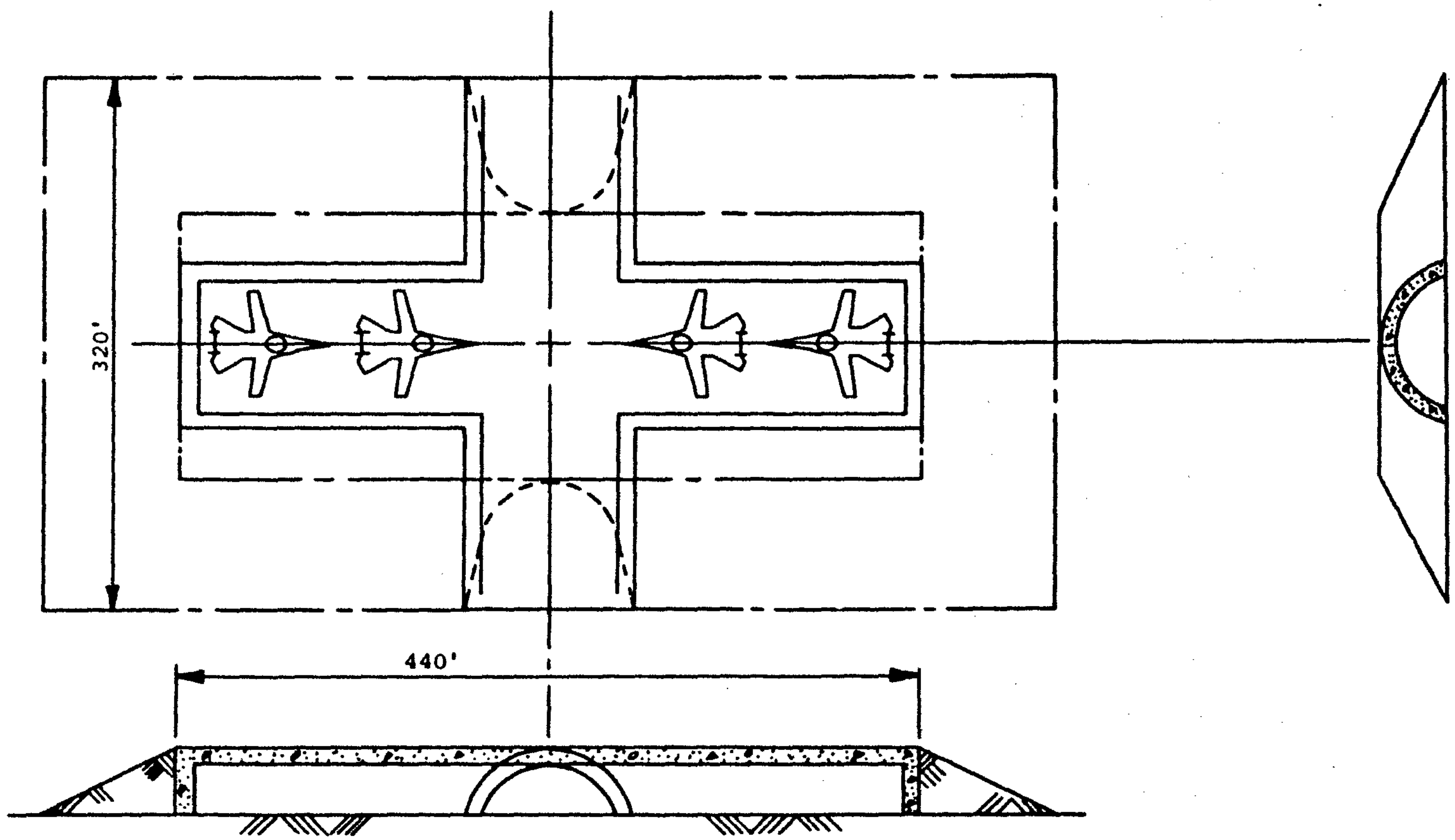


Figure 14. In-Line Shelters (4 Aircraft)

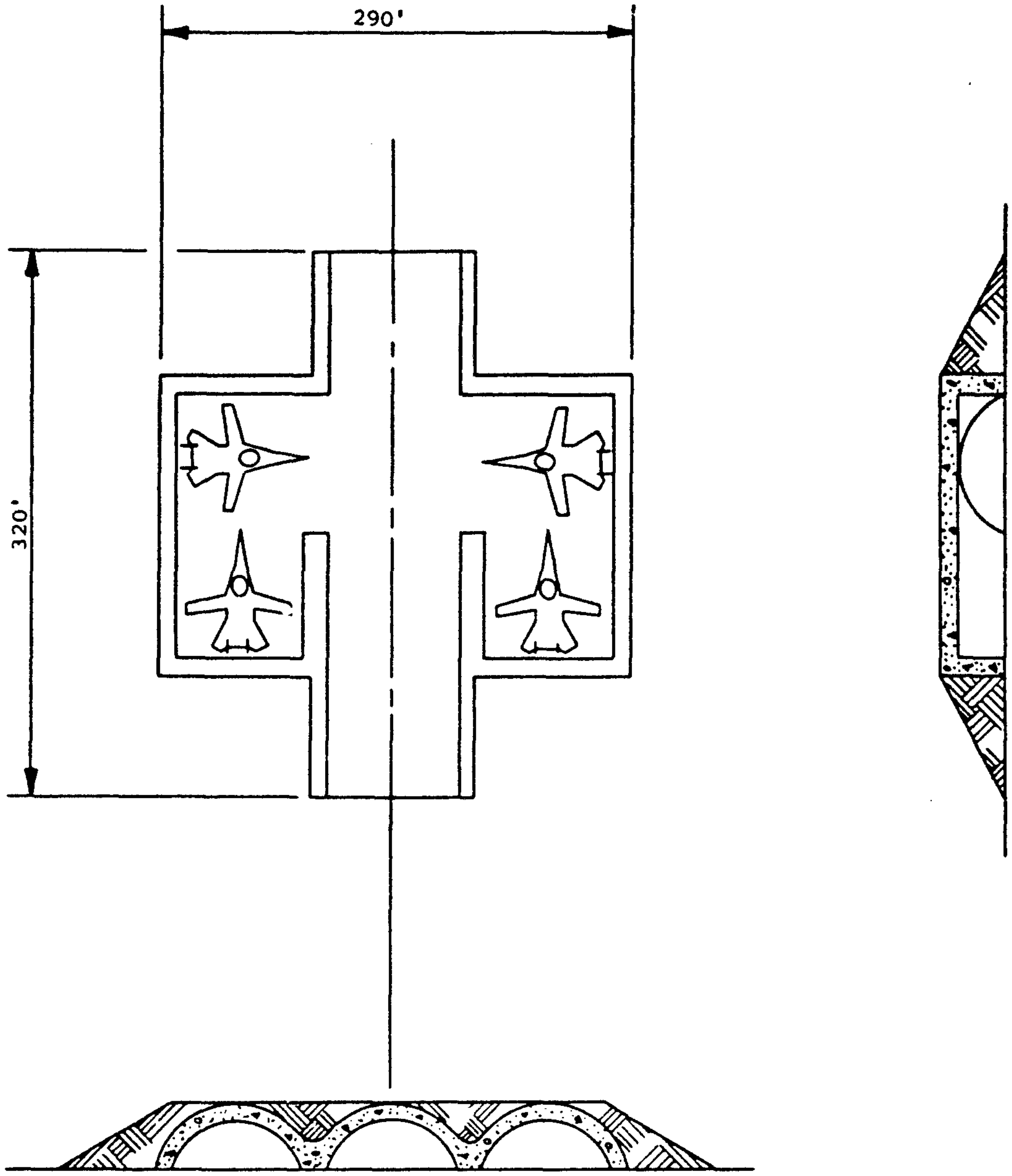


Figure 15. Side-By-Side Shelters (4 Aircraft)

placed with their centerlines in-line can be located, and figure 15 shows a concept with parallel centerlines.

Figure 16 illustrates a star shaped layout. This shape is efficient in terms of the amount of tunnel length required to house a given number of aircraft. In addition, each aircraft shelter could have an exhaust duct in the tunnel endwall allowing inside start-up and more rapid egress of aircraft under power. Further, aircraft in one arm of the shelter do not interfere with movement of aircraft from another. There is a large amount of empty space just inside the shelter entrance in front of the aircraft, but this may be useful for storage of AGE.

3. SHELTER LAYOUTS FOR MORE THAN FOUR AIRCRAFT

An efficient utilization of the parallel centerline layout is a shelter that contains six aircraft as shown in figure 17. The last aircraft into the shelter on each side must be the first out, but then either of the remaining two can be removed.

Each of the basic layout patterns outlined above can be easily extended to contain a larger number of aircraft. Examples are shown in figures 18 through 20. In addition to the vulnerability aspect, problems associated with tip-to-tail positioning and internal start-up capability become more formidable. However, such disadvantages must be weighed against the potential cost saving inherent in larger shelter facilities.

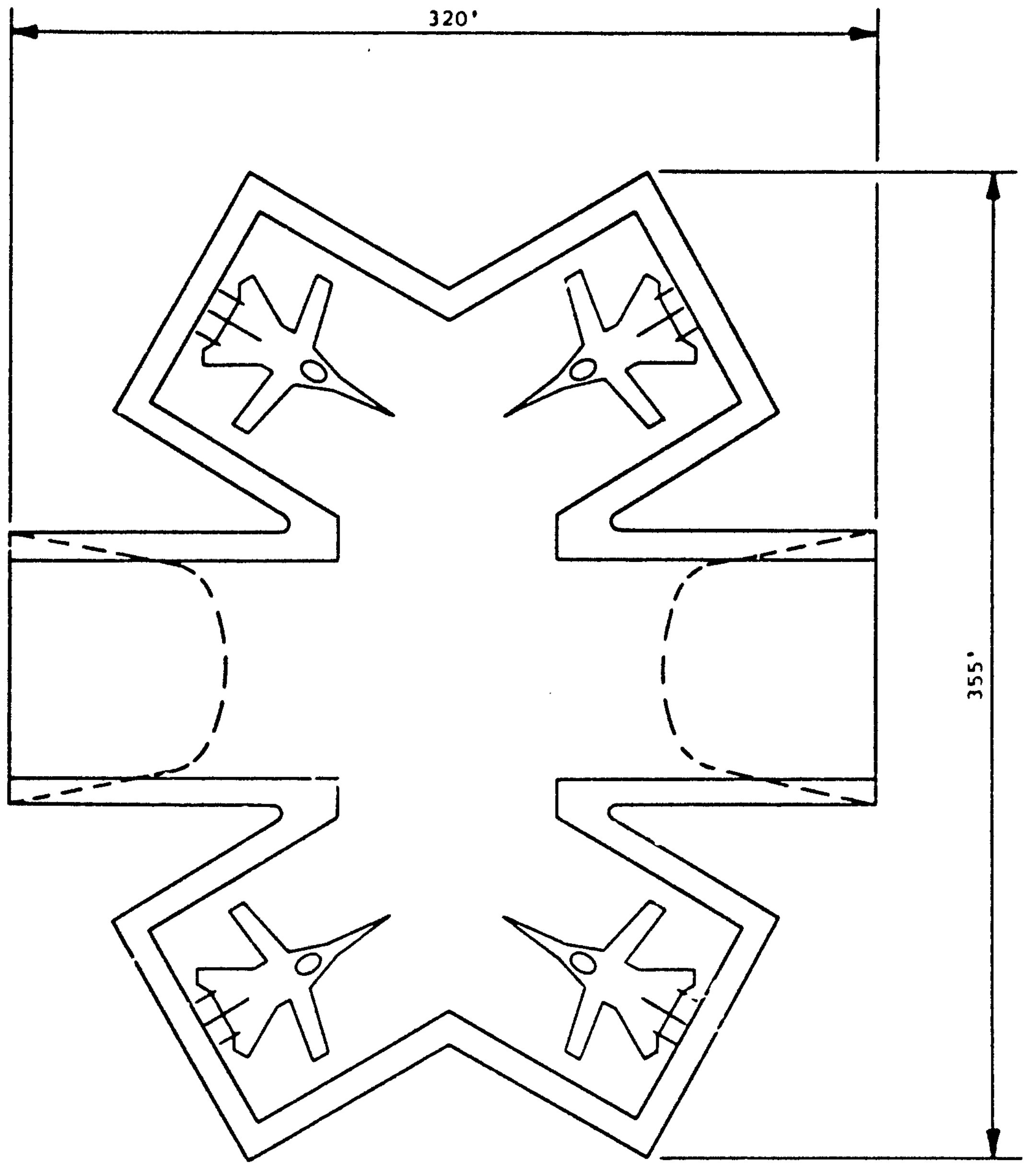


Figure 16. Star Pattern Plan Form (4 Aircraft)

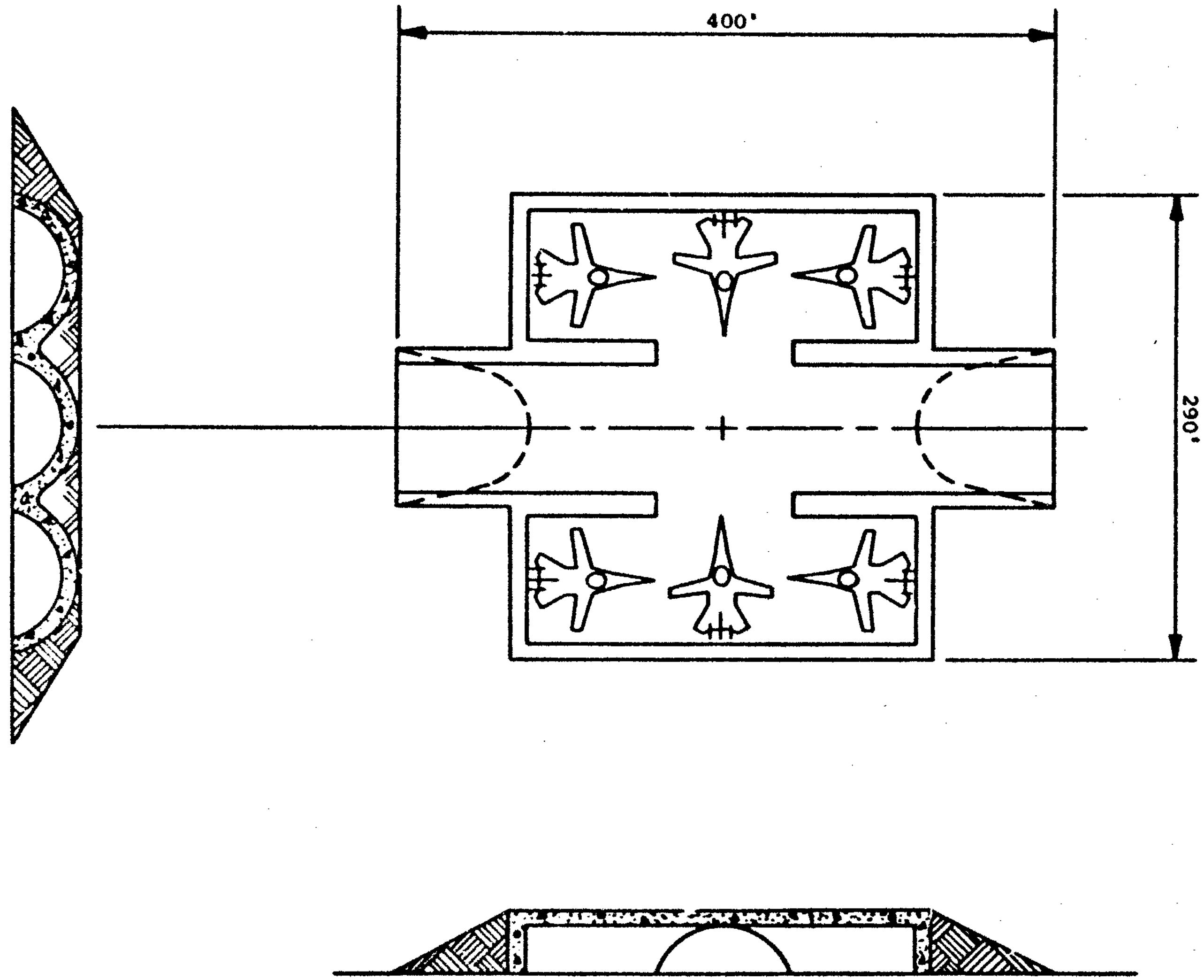


Figure 17. Side-By-Side Shelters (6 Aircraft)

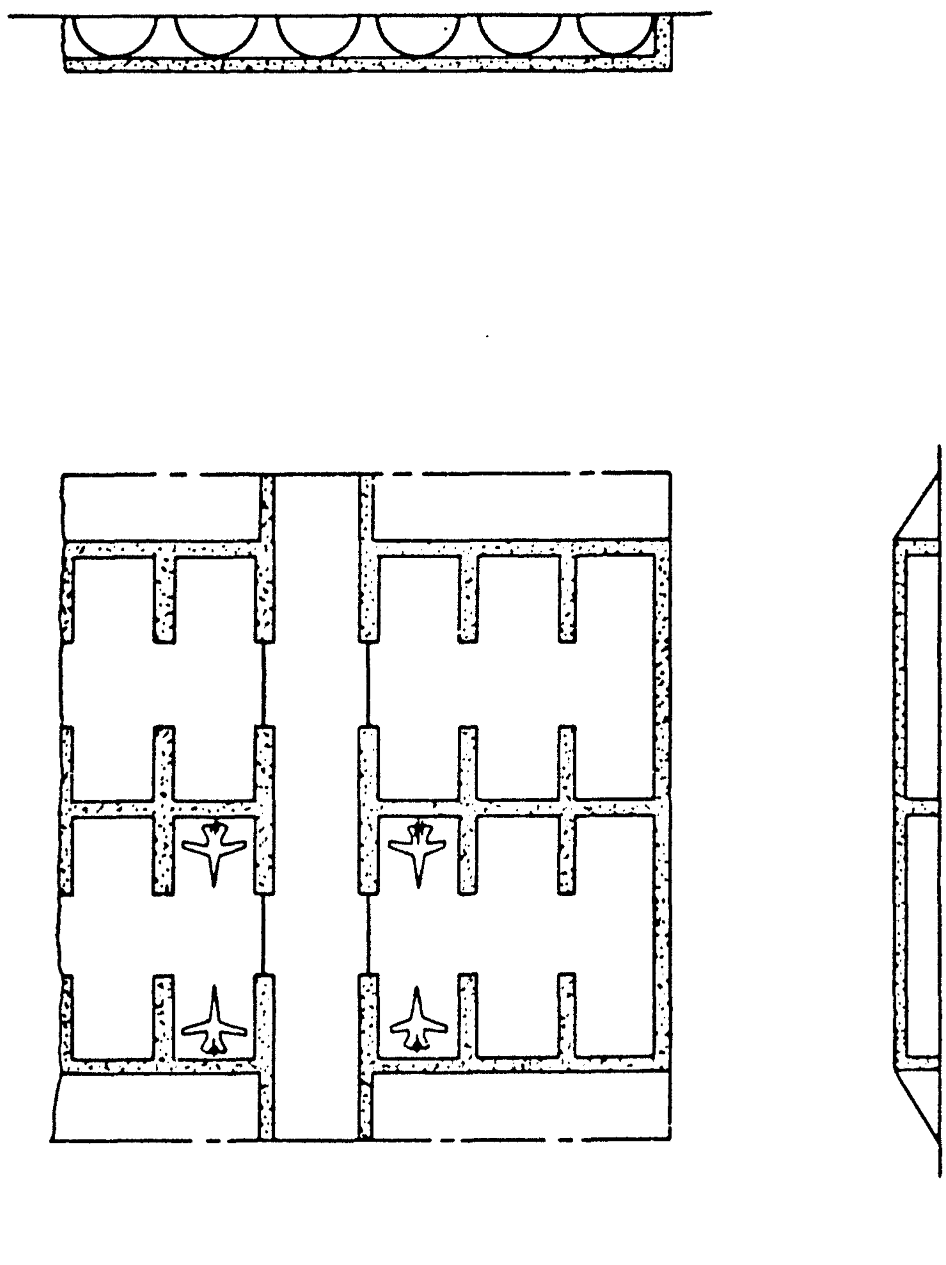


Figure 18. Side-By-Side Shelters Extended for Multiple Aircraft

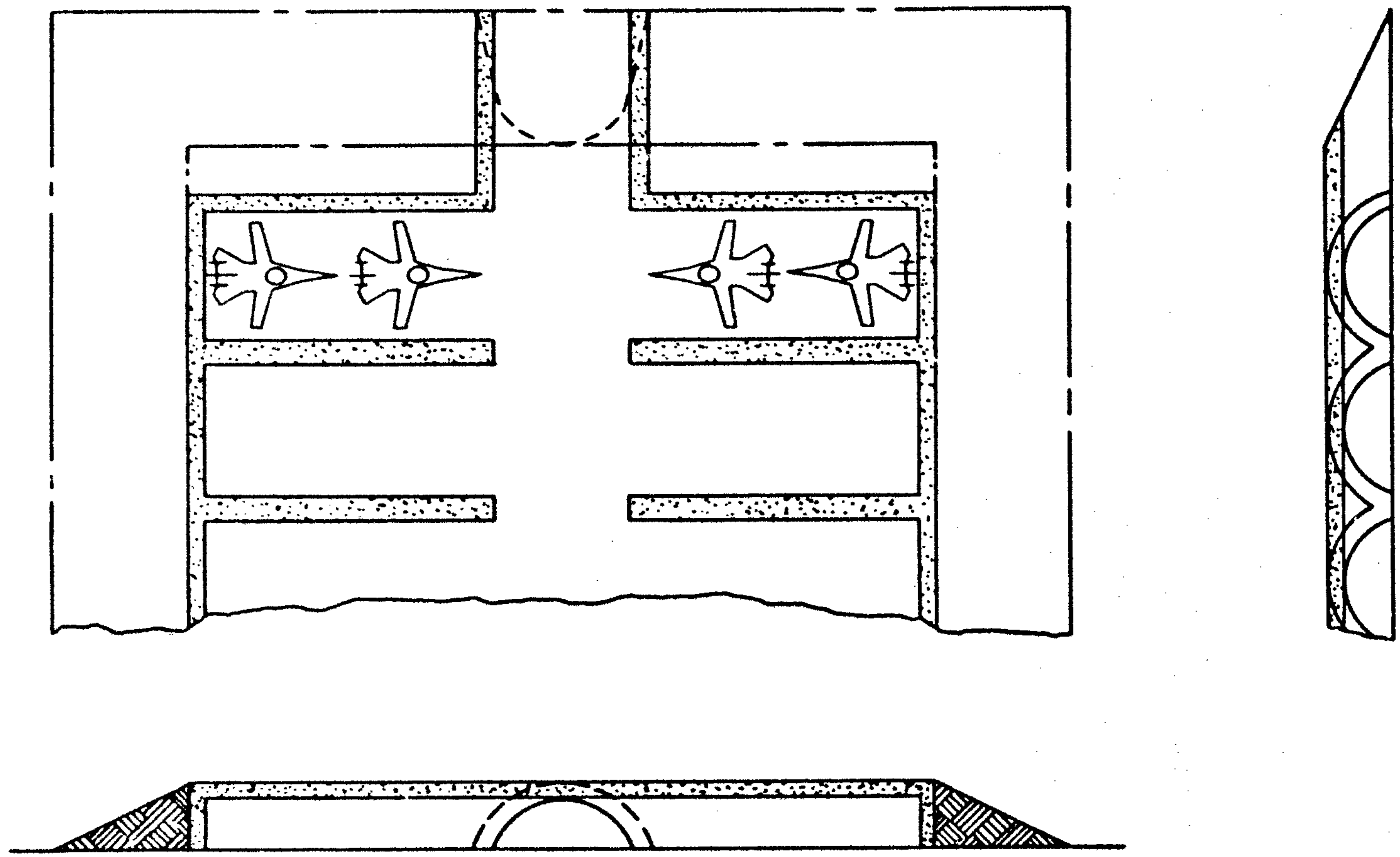


Figure 19. In-Line Shelters Extended for Multiple Aircraft

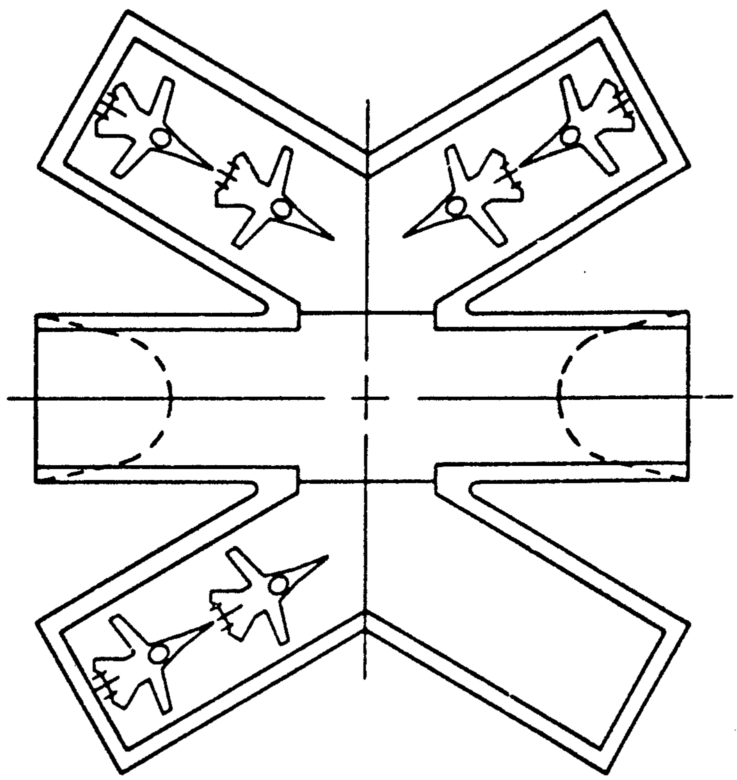


Figure 20. Star Pattern (8 Aircraft)

SECTION V

CONCEPTS SELECTED FOR EVALUATION

1. INTRODUCTION

Three basic concepts were selected for further evaluation from the numerous candidate concepts considered during the preliminary development phase. All three concepts make use of a reinforced concrete shelter that is buried. One shelter concept requires placement completely below ground. The other two concepts can be located either above or below ground or anywhere in between. Each concept is configured for multi-aircraft parking and utilizes the same shelter layout for a uniform basis of evaluation.

The concepts are named according to the manner in which access to the shelter complex is provided. The concepts are

- Elevator
- Open Corridor
- Covered Corridor

Each of these concepts is described in more detail in following paragraphs.

2. ELEVATOR CONCEPT

The elevator concept illustrated in figures 21, 22 and 23 consists of a totally buried shelter complex that is accessed by a hydraulic lift elevator. The elevator consists of a platform that raises or lowers aircraft and equipment

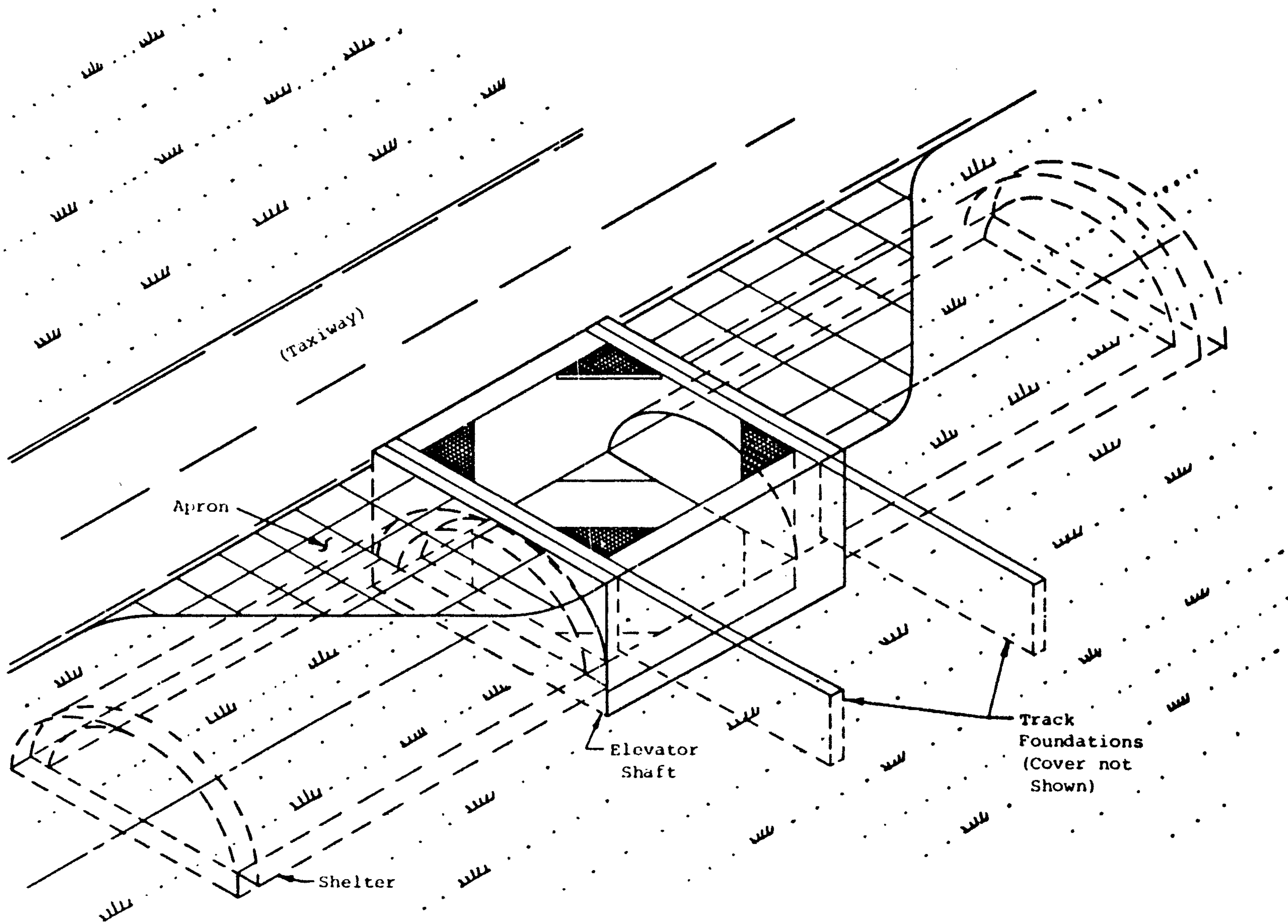


Figure 21. Elevator Concept - Isometric

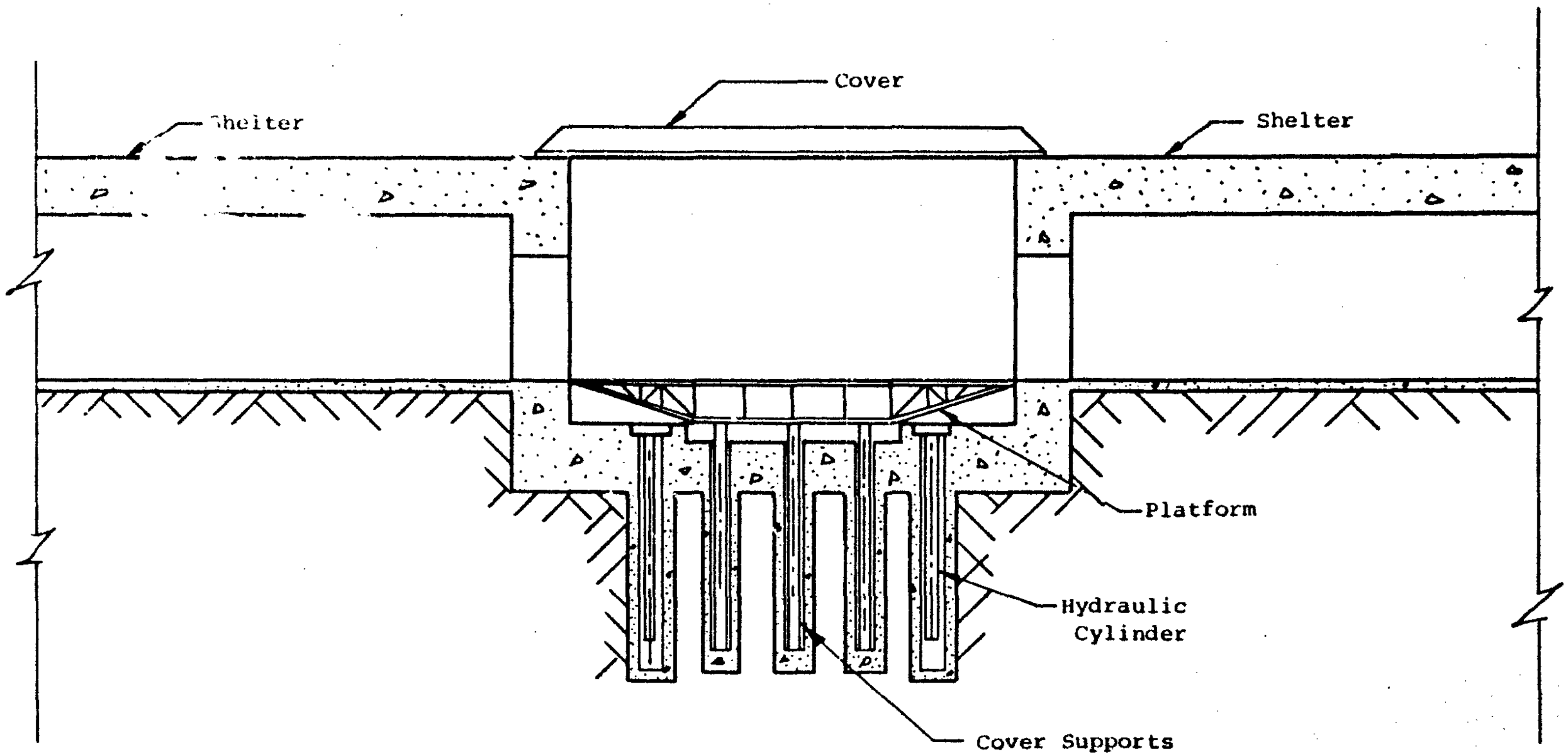


Figure 22. Elevator Concept - Sectional Side View

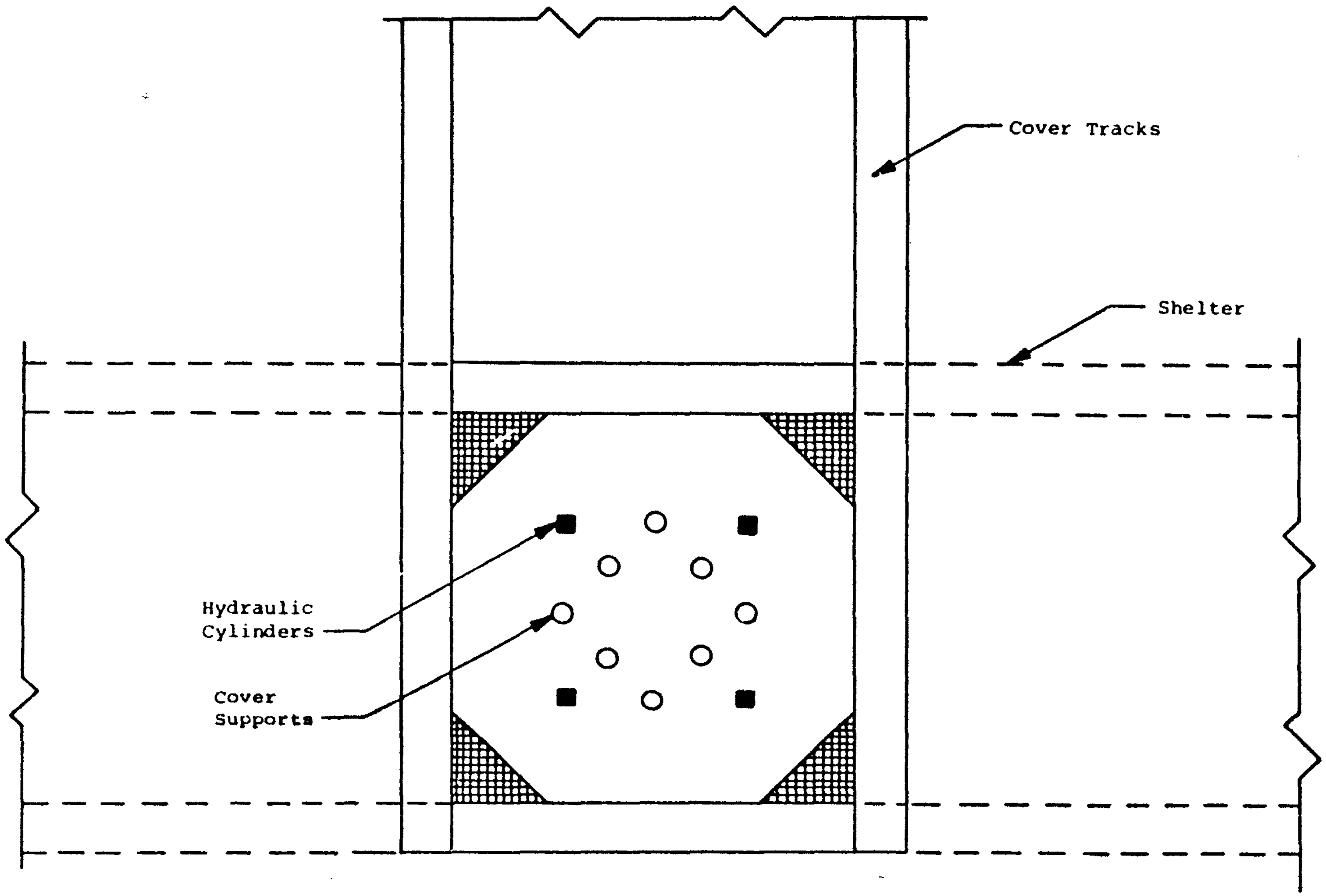


Figure 23. Elevator Concept - Plan View

between the apron and the shelter floor. The platform is basically 80 feet square with the corners cut on a diagonal to minimize platform weight as shown in figure 24. The 35-foot dimension between main structural trusses allows adequate clearance for wing and stabilizer tips when aircraft are positioned parallel to the primary load paths. The openings left by the diagonally cut corners on the elevator platform are covered with a permanent grating for safety. The apparent shape of the elevator shaft in figure 23 leads one to think that a round elevator platform may be the best shape to use. However, the intersection of the elevator shaft and shelter would be more complicated, support for part of the cover would be provided by concrete members that were cantilevered instead of in direct compression, and a round platform would have to be nearly 85 feet in diameter to provide adequate clearance for the F111.

The platform is raised and lowered by a hydraulic system containing four synchronized cylinders. Each cylinder has a bore of 14 inches and stroke of about 40 feet. The system operation pressure is 1000 psi. With the platform in the raised position, mechanical latches are engaged in the elevator shaft sidewalls to prevent tipping due to eccentric loading. A possible latch configuration is shown in figure 25.

Protection of the elevator shaft, platform, and shelter

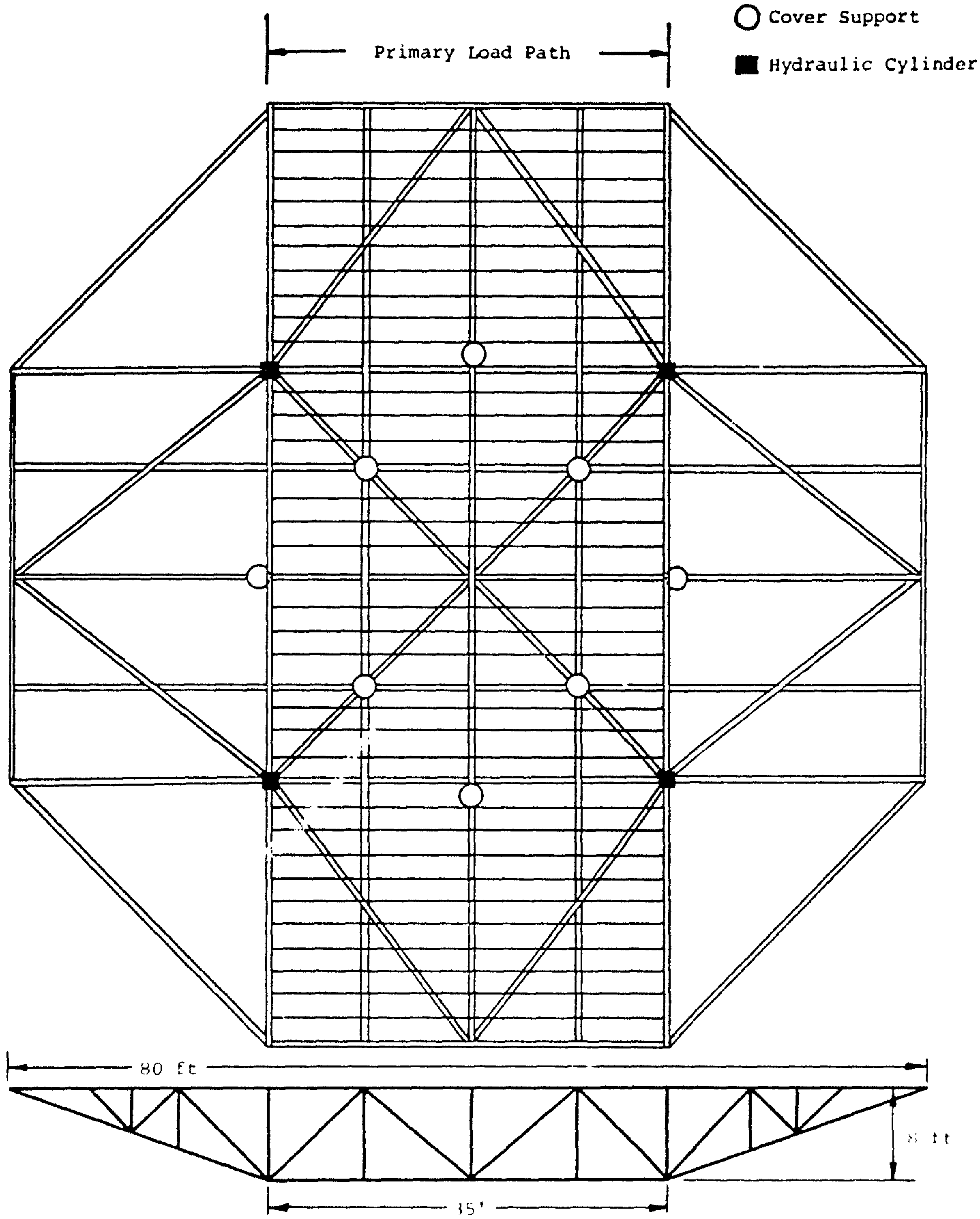


Figure 24. Elevator Platform

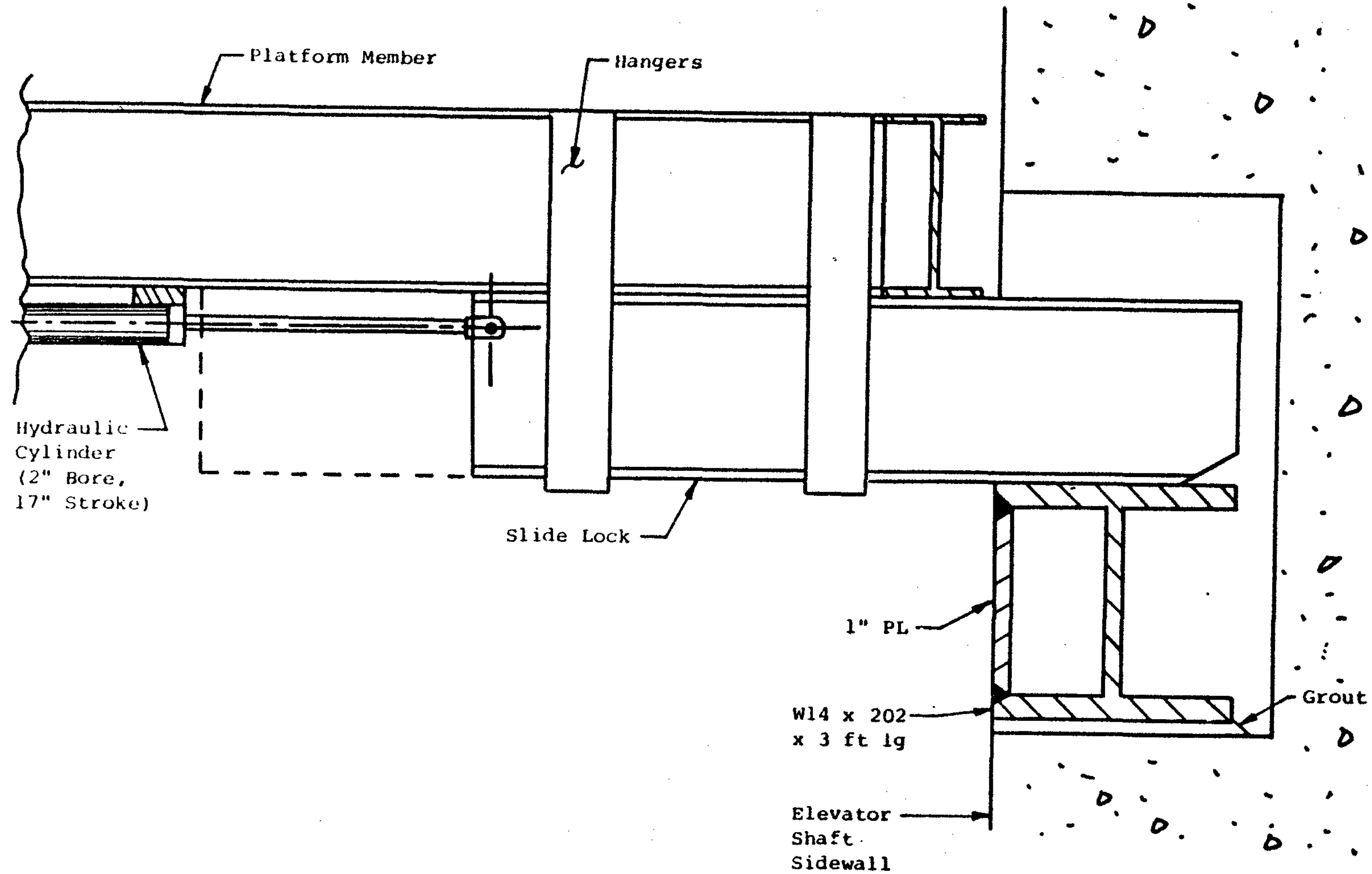
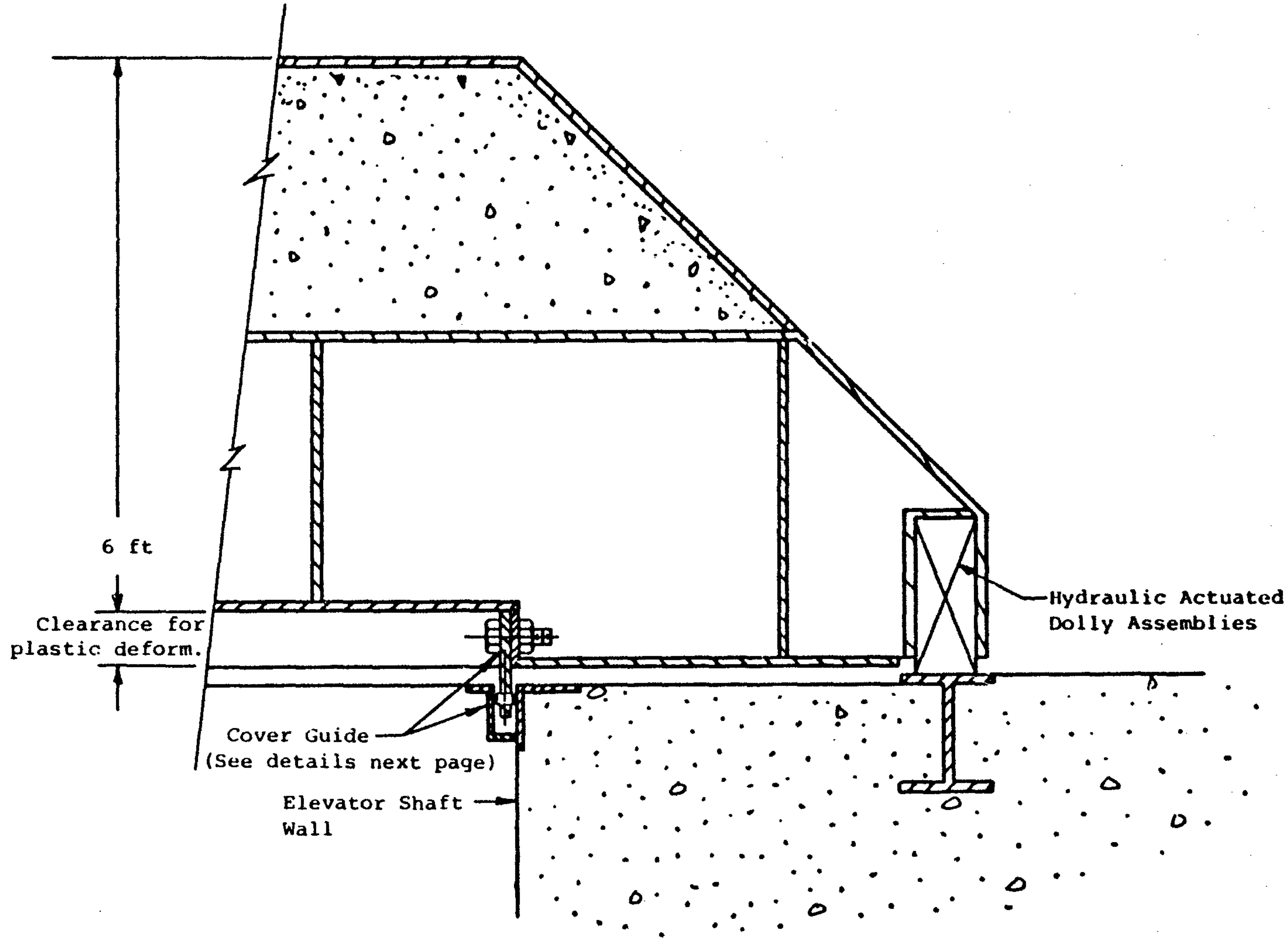


Figure 25. Platform Latch

contents is provided by a movable composite steel/concrete cover. The cross section of the cover shown in figure 26 consists of two 1-inch steel plates spaced 36 inches apart covered with 36 inches of concrete confined by a top steel plate. This thickness is marginally adequate to safeguard against the penetration threats that are anticipated. Reinforced concrete covers were considered. With a thickness of only 6 feet and 2 percent steel, a cover measuring 90 feet square would weigh 3750 tons. This weight is about twice the weight of the composite cover, would require that much more power to operate, but would cost just about the same as the composite cover.

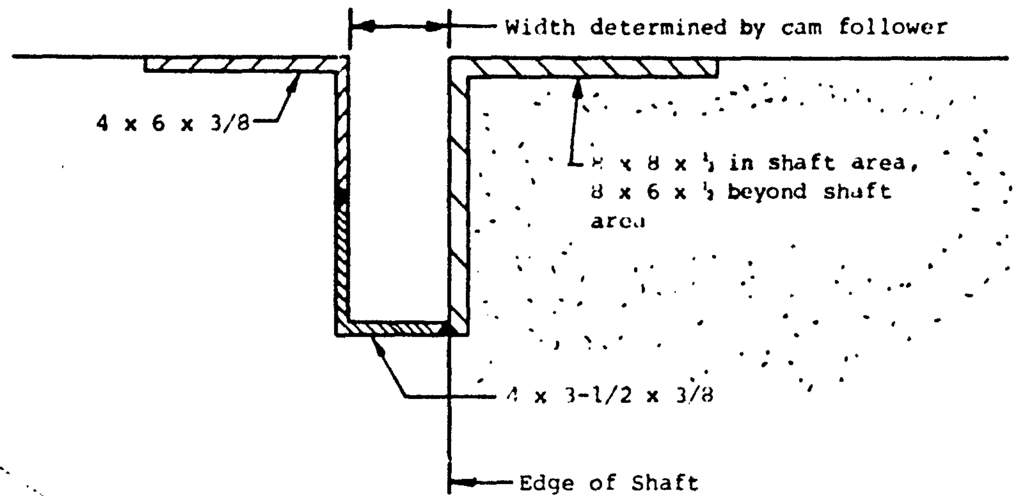
Lateral restraint of the cover is provided by lowering the cover so that it engages a steel edge exposed on the apron surface. The steel edge is 2 inches high and does present an obstruction to traffic. However, the edge of the exposed steel restraint and the mating region of the cover can be inclined in the normal traffic area so that the impediment to aircraft is minimal while continuing to provide some restraint.

The functions of raising and lowering the elevator shaft cover and providing a low friction device to roll the cover open and closed are accomplished by using the door jacking truck illustrated in figure 27. This unit consists of a heavy duty roller assembly and a specially designed hydraulic cylinder. The basic exterior structure is a plate steel box that

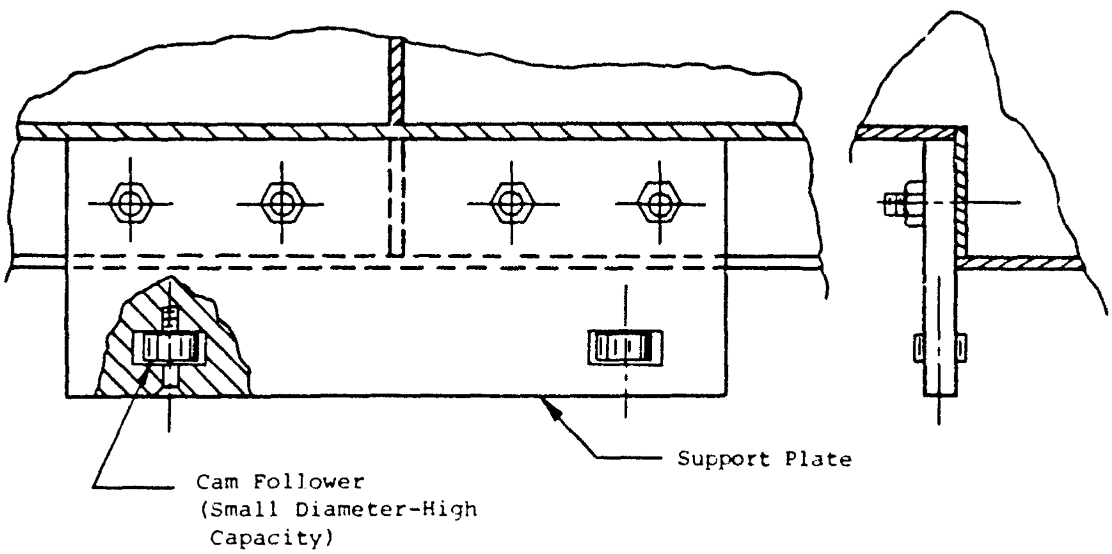


a. Composite Cover Edge Geometry

Figure 26. Elevator Shaft Cover



b. Grove Detail



c. Guide Detail

Figure 26. Elevator Shaft Cover (concluded)

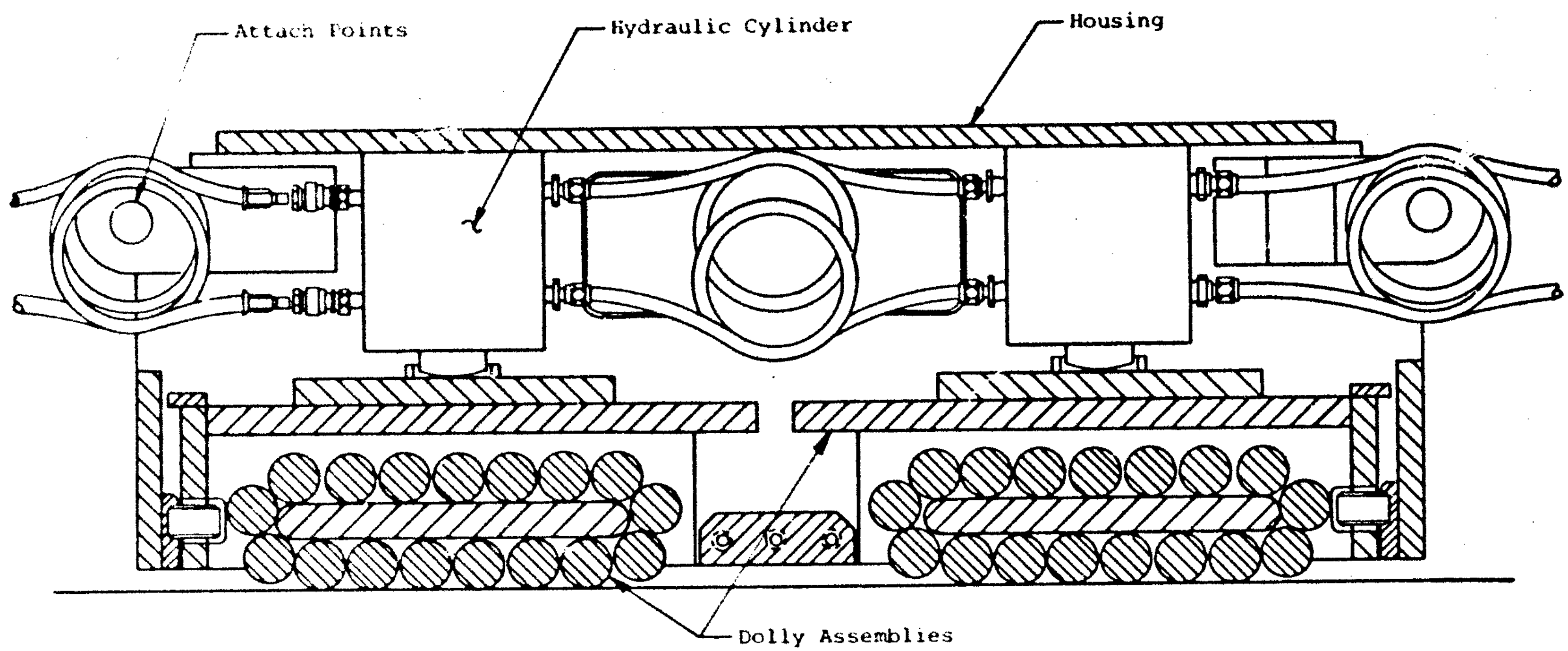


Figure 27. Door Jacking Truck

is rigidly attached to the cover. When the hydraulic cylinder is pressurized, the closure is pushed upward away from the dolly. The dolly rollers rest on a steel track embedded in the concrete apron. The unit illustrated is being used very successfully at Los Alamos Scientific Laboratory in the Clinton P. Anderson Meson Facility. Concrete shielding doors weighing almost 1000 tons are operated with relative ease. The overall geometry will be altered slightly to fit this application, but the principle of operation suits the needs perfectly.

Power to move the closure is provided by cables in the grooves parallel to the steel tracks shown in figure 26. Steel plates extending into the grooves also connect to the cable and provide the required guidance to keep the dollies on the tracks. Small diameter high capacity roller bearing wheels riding along the sidewalls of the groove or the elevator shaft prevent large sliding frictional forces from developing and prevent binding of the closure. If the location for the guide rollers shown in figure 26 proves to be inaccessible, the groove can be located along the outer edge of the embedded steel track and the roller guides connected to the exposed edge of the cover. The winch which provides the pulling power to move the cover is located below ground in a pit beyond the end of the cover travel. Two pits are required as well as two winches with synchronized pulls. Alternately, one winch with a split drum and additional pulleys to route the cables can be used to provide pulling power at the two attach points.

The elevator shaft opening measures 80 feet on a side and must be spanned by a protective cover. A plate spanning the full opening that can withstand the overpressure acting as a two way slab with simply supported edges is very massive. The length of the unsupported span for the cover can be decreased by adding supports near the center of the cover. However, the supports must be retractable to allow use of the elevator platform for its primary function, movement of aircraft and materials. Figures 22 and 23 illustrate how eight pipe columns can be added near the center of the cover to provide supports. Each support is a pipe column approximately 40 feet long with a 24-inch outside diameter and a 2.34-inch wall thickness. Large diameter pipe sections are required to provide resistance against buckling as well as axial yielding. When not in use, the pipe sections reside in steel lined circular concrete receptacles in the bottom of the elevator shaft. When the shelter is closed for maximum protection, the pipe columns are extracted from their receptacles by attaching them to the lower surface of the elevator platform with the mechanism shown in figure 28. The elevator then rises until it comes into contact with the underside of the cover. The lower ends of the pipe columns are supported by hydraulically sliding a one piece slider block under the column free ends (figure 29).

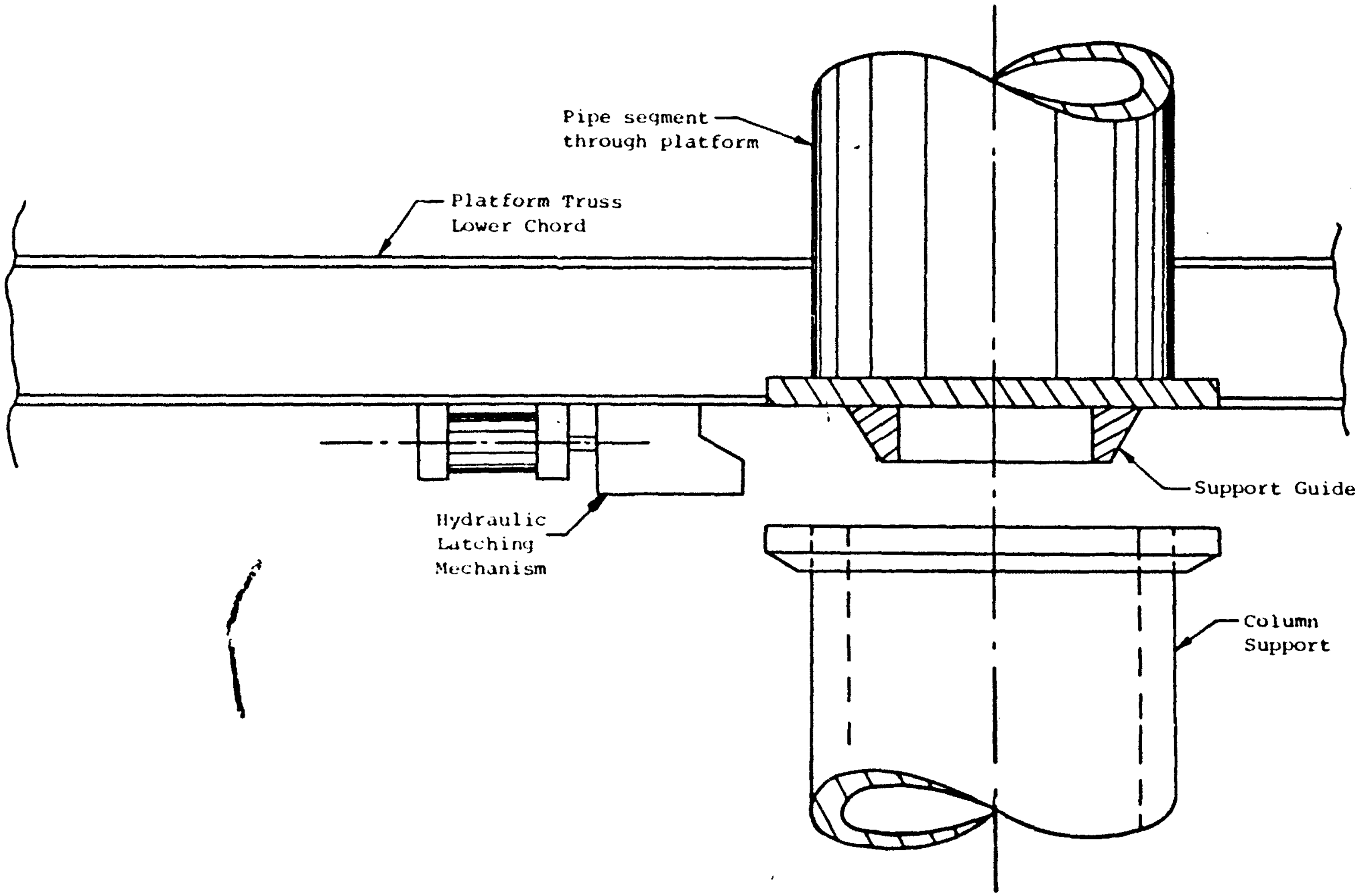
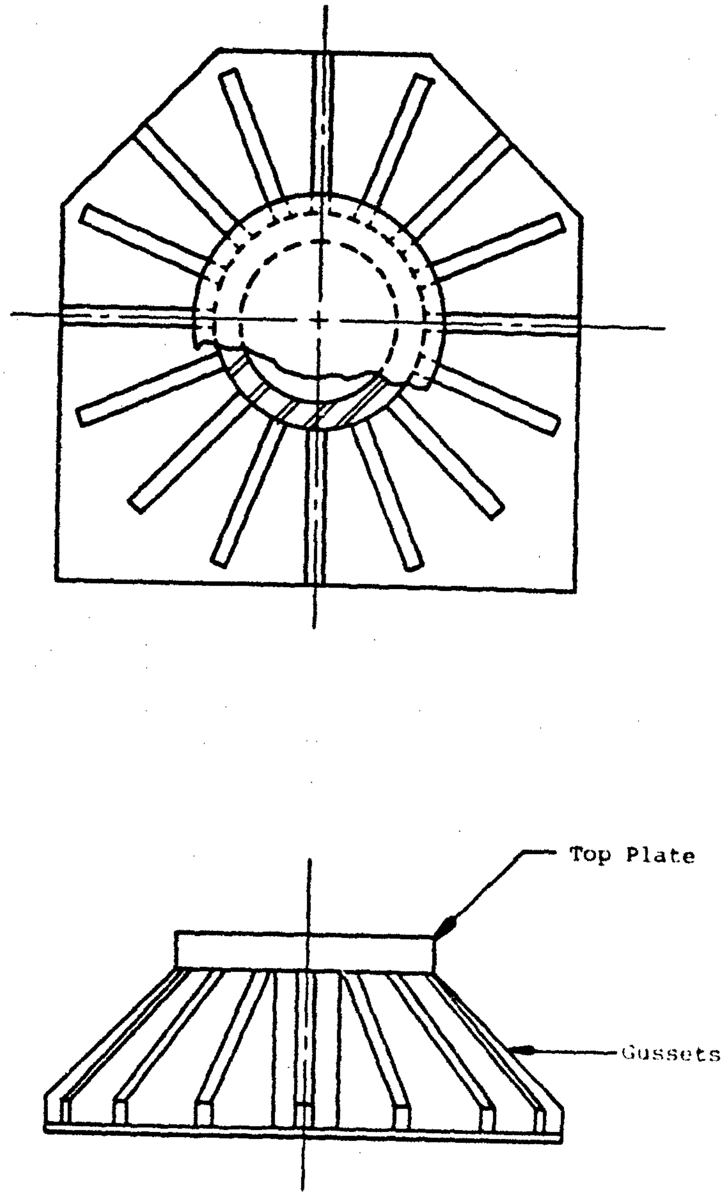
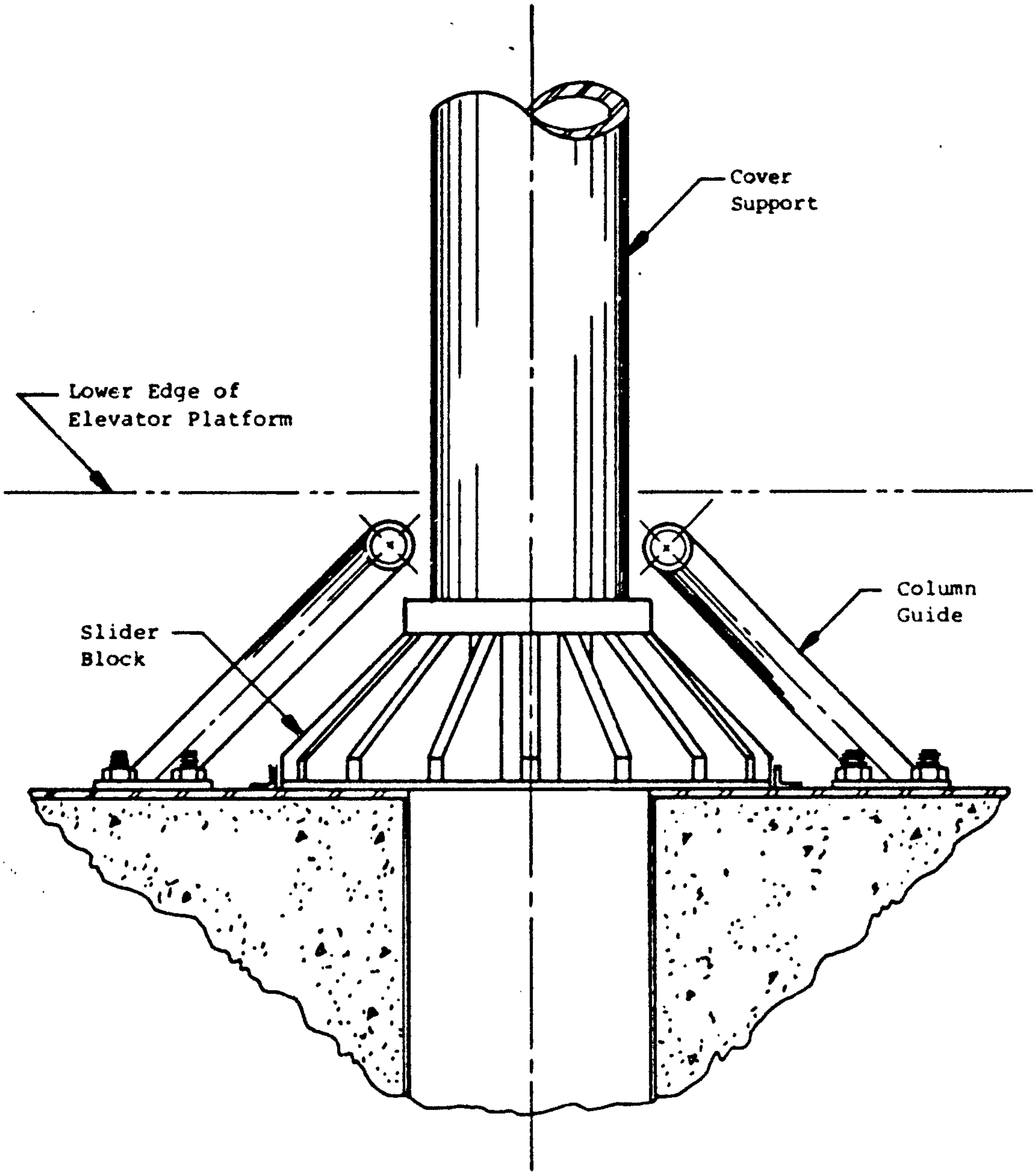


Figure 28. Upper Support Column - Platform Latch



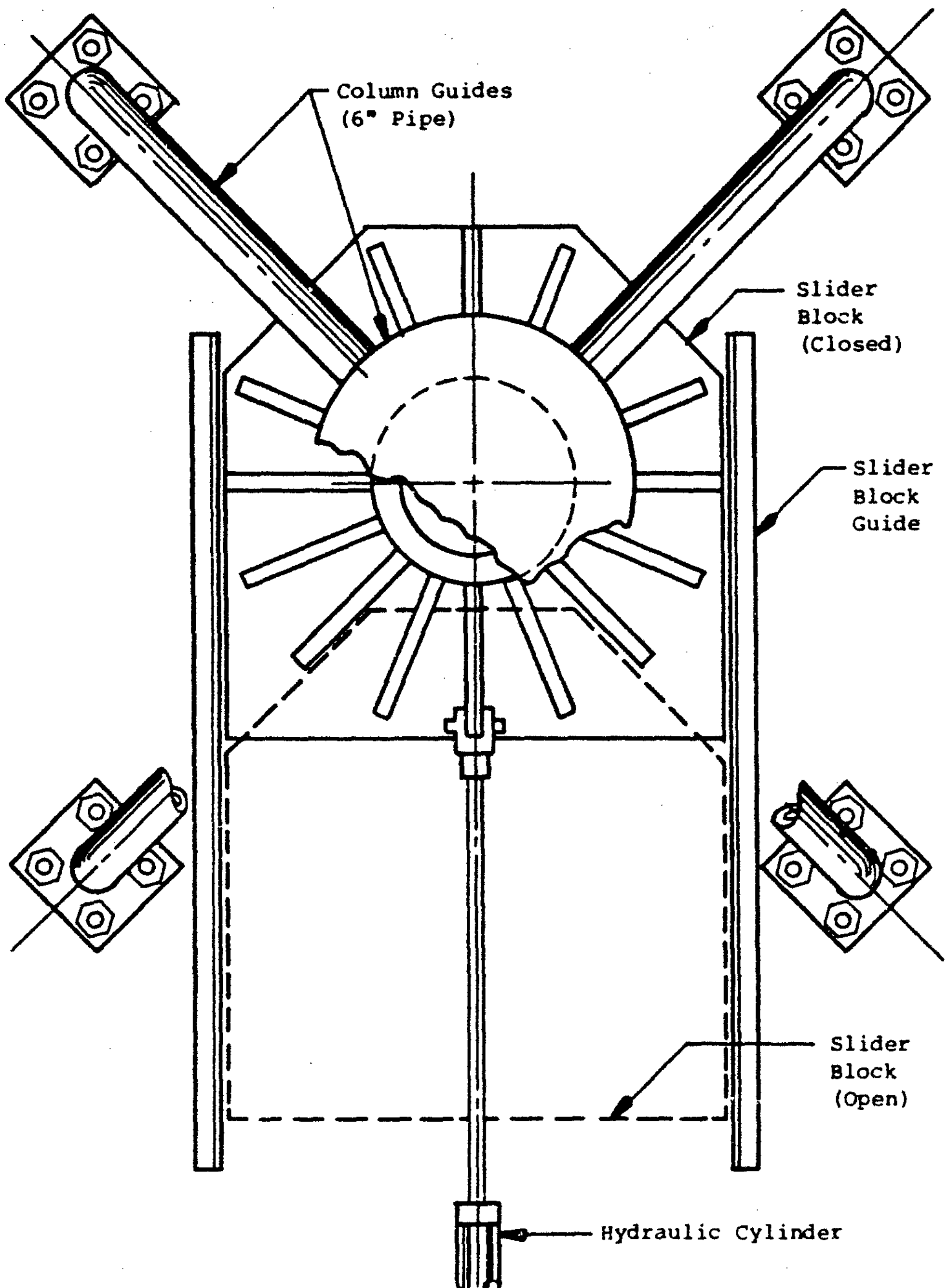
a. Slider Block

Figure 29. Column Support - Lower End



b. Side View

Figure 29. Column Support - Lower End (continued)



c. Plan View

Figure 29. Column Support - Lower End (concluded)

The slider block consists of two parallel plates top and bottom separated by a length of pipe that is surrounded by a number of radial gussets. The gussets are thick steel plates that provide the needed strength to distribute the large axial load in the column to the concrete surrounding the hole. With the slider blocks in position, the hydraulic cylinders used to raise and lower the platform are not subjected to blast loads which could otherwise damage the hydraulic system.

Support to the midspan region of the elevator shaft cover could also be provided by a structural member that extends from the cover to the bottom of the elevator shaft. The member can be structurally part of the cover as shown in figure 30 or be drawn out of the sidewall to cross the void before or after the cover is in place. An advantage of this type of center support is that it can also provide the required resistance against movement due to airblast loading on the sides of the cover. The largest disadvantage of such a support method is the pocket that must be constructed to contain the retracted support. The sidewalls of the pocket must be structurally able to withstand the 250 psi overpressure. Also, the pocket opening in the apron would have to be covered when the cover is closed.

Several alternate configurations were considered during the course of the study but each appeared to have features less desirable than the system described above. However, if

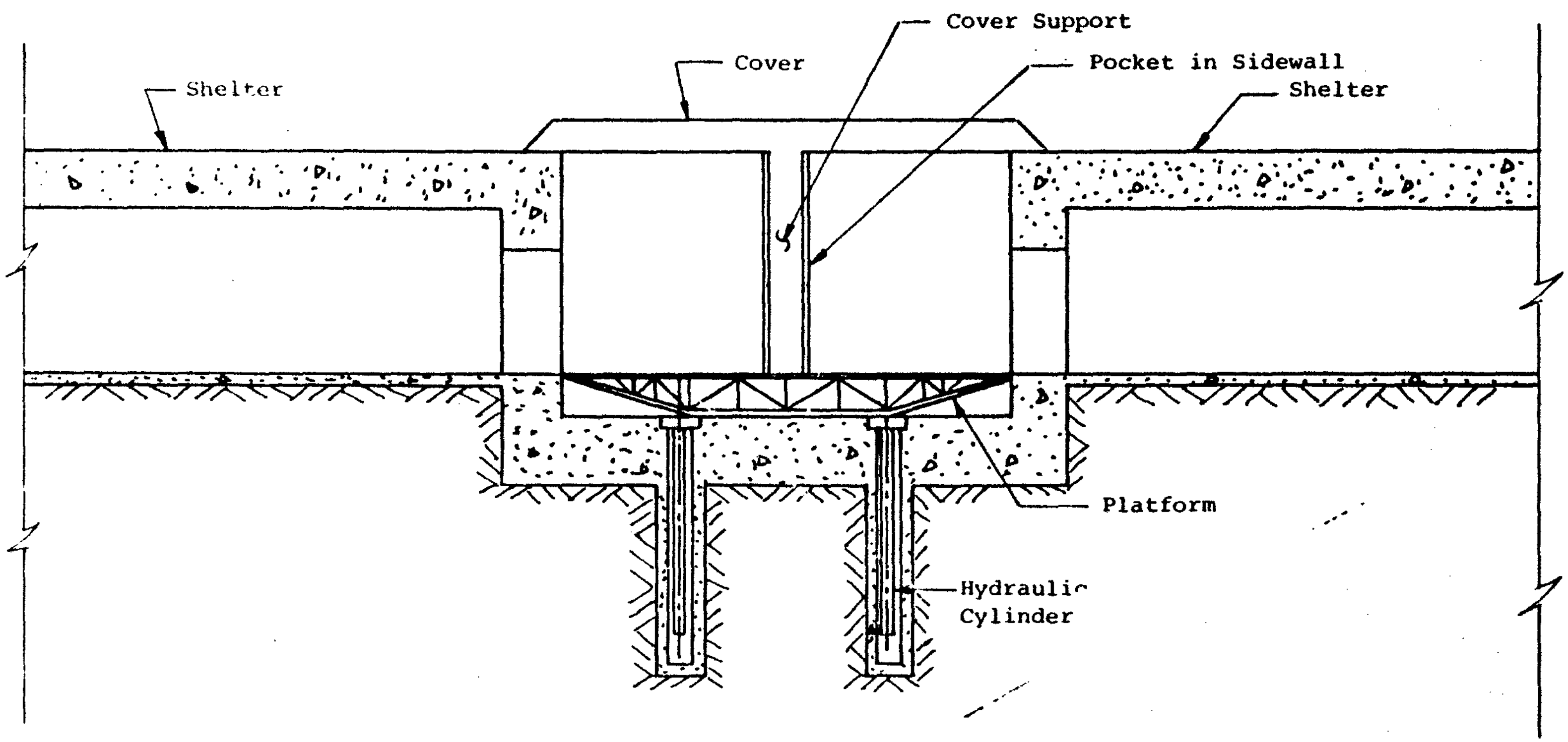


Figure 30. Elevator Concept - Alternate Cover Support

certain aspects of the threat spectrum are relaxed slightly, an alternate configuration could possibly be more appropriate.

One possible alternate elevator platform and cover concept consists of providing the two functions with a single structure. The platform can be used as before, but a significant increase in power will be required because of the large increase in dead weight. This can be partially offset by utilizing an accumulator in the hydraulic circuitry to accumulate energy when the platform is lowered.

For this alternate solution to be anywhere near practical, the penetration threat must be relaxed and the platform designed to withstand only the overpressure. The weight of the platform can be further reduced by decreasing span lengths. This would, however, require additional hydraulic cylinders. A reasonable solution appears to be one in which an equivalent steel thickness of 4 inches is used in the construction of the platform. The weight of such a platform measuring 80 feet square would be 525 tons. Allowing for a 100 ton platform load, the hydraulic system must then be capable of lifting 625 tons. Nine hydraulic cylinders with 14 inch diameter bores and an operating pressure of 1000 psi would be adequate.

Central support for the platform in the raised position could be provided by restraining the lower end of the hydraulic cylinder rods. For nine cylinders equally spaced in a 60-foot square pattern in the central region of the elevator shaft, the loading area attributed to each cylinder is 20 feet square. At 250 psi overpressure, the axial load applied to each cylinder rod is 14.4 million pounds. This load greatly exceeds the static buckling load of about 5.0 million pounds for a 14-inch diameter rod that is 40 feet long.

Another critical area involves providing support around the perimeter of the platform in the closed position. The vertical blast load which must be transferred from the cover to the shaft sidewalls amounts to at least 30,000 lb/in. In areas where there is a continuous sidewall in the elevator shaft, this may present no serious problem. In the region above the shelter openings, however, the problem could be severe. Above the shelter openings, the concrete section is only 10 to 18 feet deep. For an elevator platform that is 5 to 8 feet thick, the latching device required to support the lower surface of the platform would be located dangerously close to the lower edge of the concrete. Alternately, locating the latch too near the surface creates other problems in preventing shear failure in the platform above the latch.

A detailed design of the alternate elevator concept should result in a number of components significantly less than the number associated with the approach that was adopted. A separate cover assembly, all the jacking trucks, the steel track, and the additional power source for moving the cover could be deleted. The result would be a simpler system in terms of components, but not necessarily a significantly less expensive system. The penalty paid with this approach is an increase in power requirement, the number of hydraulic cylinders required to raise and lower the platform, and locking devices on the cylinders themselves.

3. OPEN CORRIDOR CONCEPT

The open corridor concept is named for the uncovered apron which exists between pairs of shelters. In figures 31, 32 and 33, the pairs of aircraft shelters are shown positioned to interrupt the incoming airblast and prevent the occurrence of full reflection. However, the applied pressure will still be significantly greater than the incident overpressure. Studies reported in references 4 and 5 indicate that aerodynamic shields similar to that provided by the open corridor configuration result in blast loads 35 to 78 percent lower than the loads on an unprotected vertical wall subjected to the same overpressure levels. Therefore, in the preliminary analyses performed for the conceptual study, a reflection factor equal to half the value for a fully reflected shock wave was used.

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4. Ferritto, J.M., Use of an Aerodynamic Obstruction to Shield a Blast Door - Results of an Experimental Model Test, Technical Report No. R739, Naval Civil Engineering Laboratory, Port Hueneme, Calif., October 1971.

5. Closures for Hardened Protective Hangars, AFSWC-TDR-62-77, Air Force Special Weapons Center, Kirtland Air Force Base, New Mexico, August 1962.

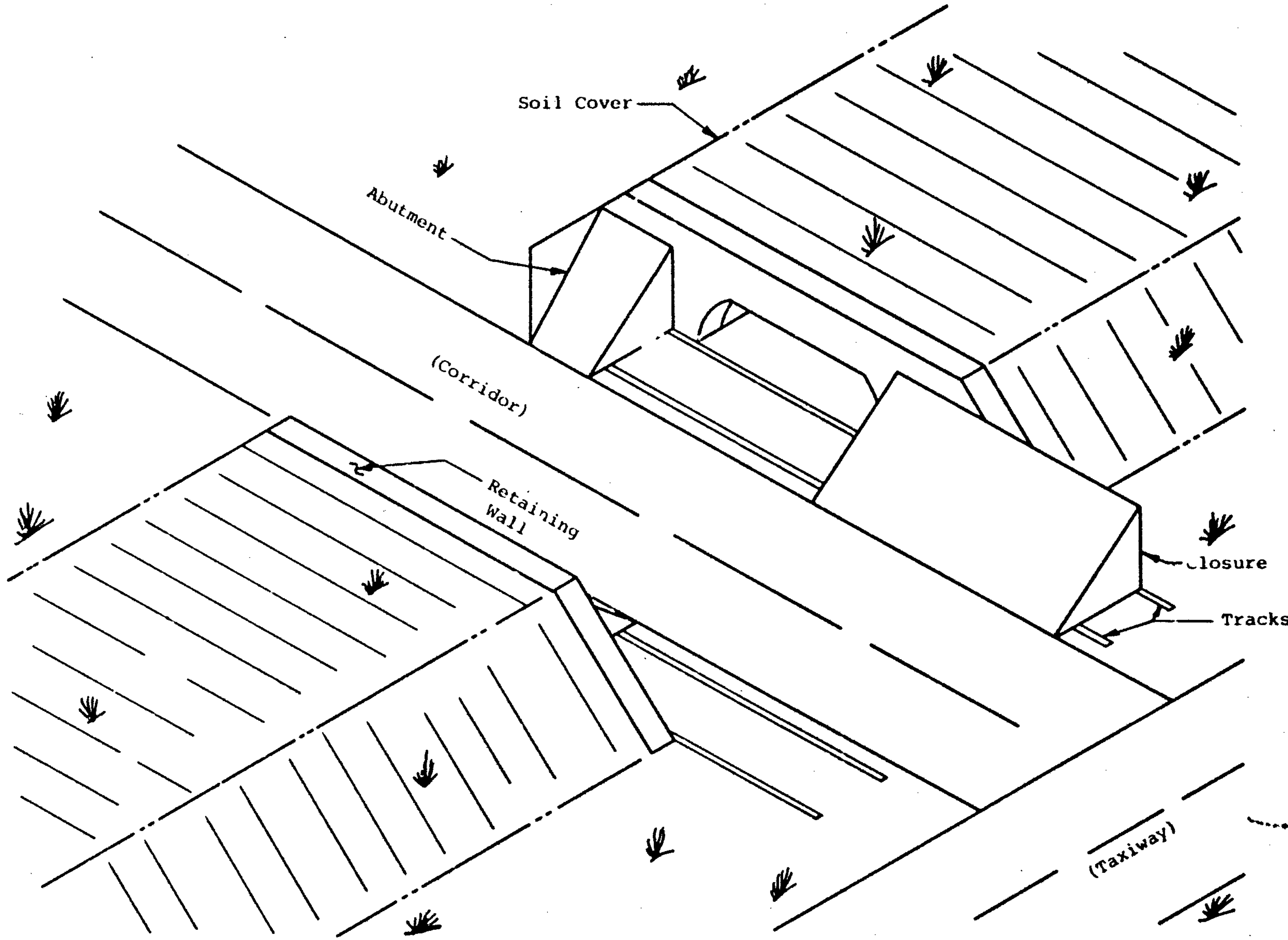


Figure 31. Open Corridor Concept - End Entry

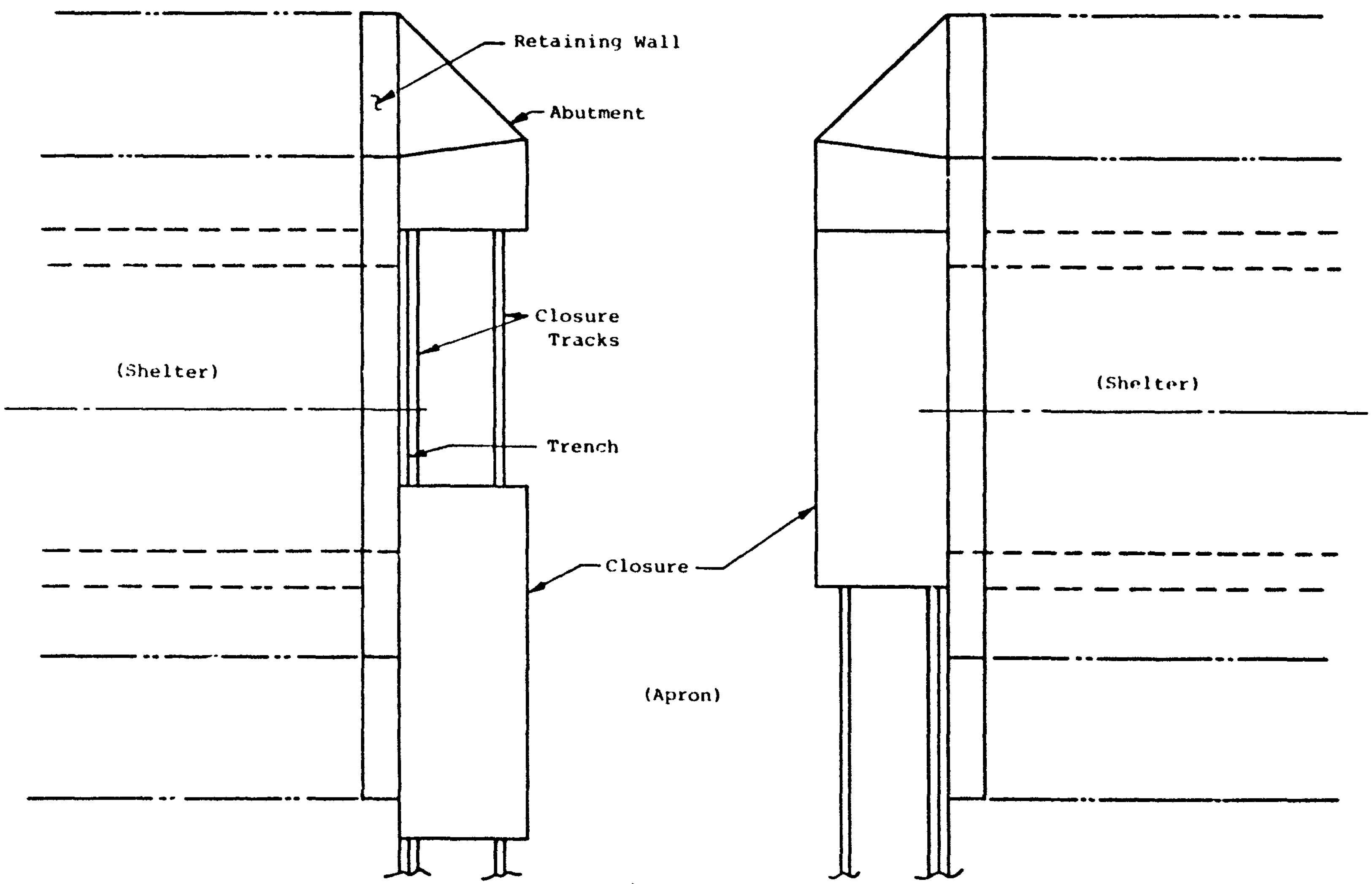
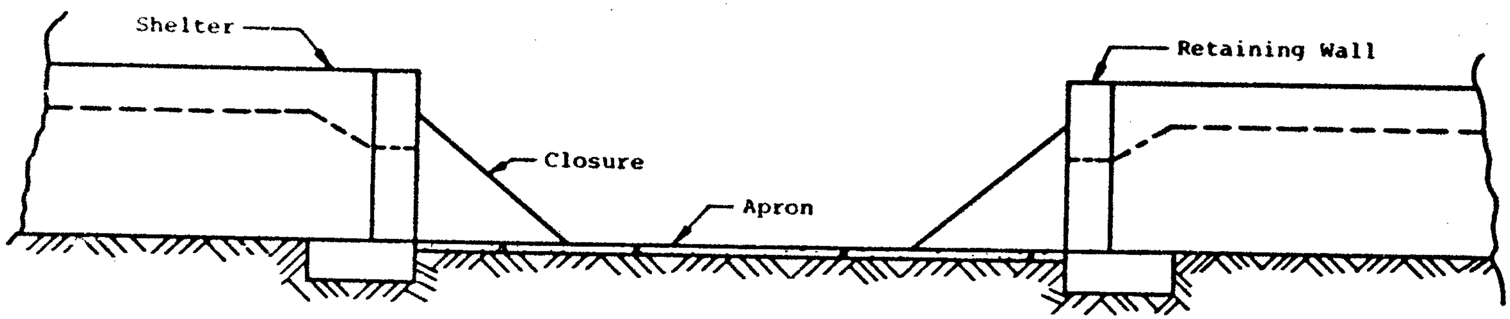


Figure 32. Open Corridor - End Entry, Plan View



Figrue 33. Open Corridor - End Entry, Side View

The threat presented to the closure by the weapons being considered requires that a rather massive door be employed. Such closures can be constructed relatively economically using concrete as the primary structural material. The penalty is that a large amount of power is required for movement. One possible closure configuration is shown in figure 34. Protection is provided by two reinforced concrete slabs which can be precast or poured in place. For a precast option, the 2-foot thick rear vertical slab and the 3-foot thick webs which support the front panel could be fabricated in segments and set in position. Next, the 4-foot thick panels forming the exterior slab would be placed on the sloping face and connected.

Restraint of the door is provided in an identical fashion for either method of fabrication. The top edge bears against the shelter but is free to rebound away from the shelter. The lower edge is restrained by a trench in the foundation.

Blast waves moving parallel to the corridor will tend to move the closure open. To prevent such an occurrence, a large abutment is provided. The abutment provides a bearing surface for the closure to prevent movement in the closing direction and shields the closure when the airblast is traveling in the opposite direction. For the latter case, the only forces acting on the closure are drag forces and the incident overpressure. The drag forces can be resisted by static friction alone.

Movement of the closure can be accomplished using standard

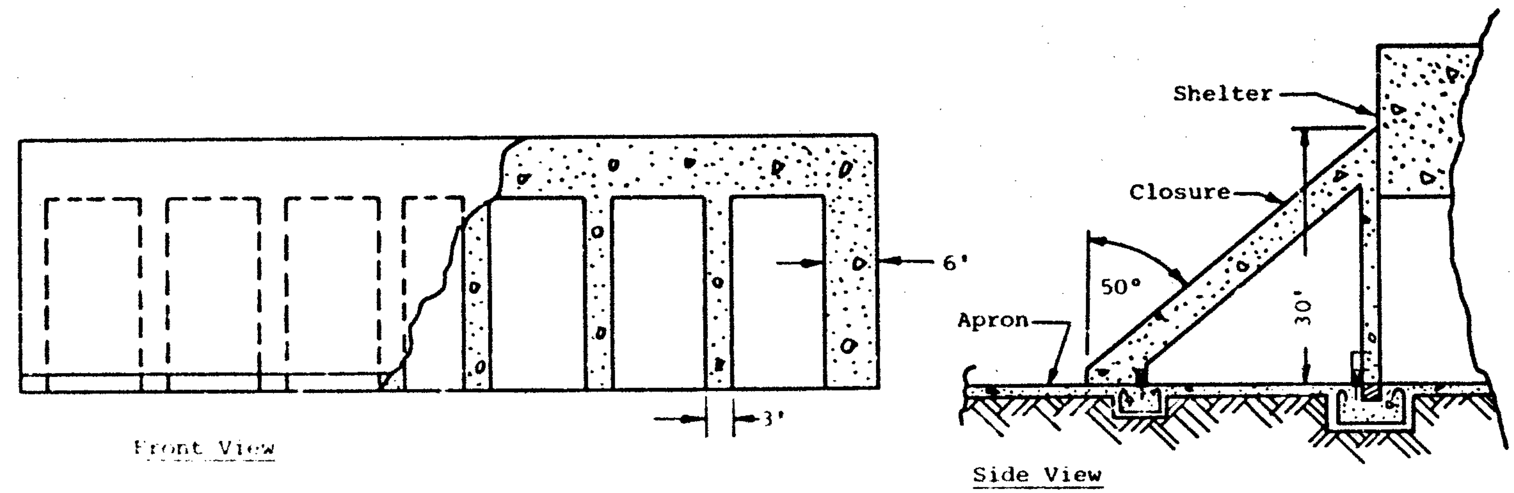


Figure 34. Reinforced Concrete Closure

items of hardware. The first task is to raise the closure off the ground, since it normally rests directly on the apron. The combination of hydraulic cylinders and dollies proposed for use with the elevator concept cover can also be used with this concept.

Power to move the closure would be provided by a cable and winch system operating below the apron surface. The closure will weigh in the neighborhood of 2200 tons and require about 130,000 pounds of pulling force for movement. To ensure adequate reserve cable strength, a 2.25-inch diameter wire rope is required in this application if a single cable is used. For a mechanical advantage of two, the cable size can be reduced to about 1.50-inch diameter. The pulling cable is attached to the portion of the closure extending into the trench. Unbalanced moments will cause the tongue to bind on the trench so that high capacity/low friction rollers will have to be installed along the sides of the tongue to eliminate sliding friction.

An alternate means for movement of the door which has not been fully explored involves the use of short stroke hydraulic cylinders. At 1000 psi operating pressure, a cylinder 13 inches in diameter can produce the 130-kip force required to move the closure. If the cylinder stroke is a few feet and the pushing points can be located along the track, a cylinder or pair of cylinders alternately can jack the closure open and closed by successively contacting the push points.

The basic orientation of the pair of shelters is either perpendicular to the corridor as shown in figure 31 or parallel to the corridor as shown in figures 35, 36 and 37. Either orientation allows ingress and egress from a shelter regardless of the closure position on the opposite shelter. Operationally, the concept with the shelter centerlines parallel to the corridor is preferable since movement of aircraft is not hindered as much by the presence of other aircraft. For example, the first aircraft in must be the last one out of a shelter for the configuration where the shelter centerlines are perpendicular to the corridor. There is an advantage to the configuration with the shelters perpendicular to the corridor in that the closure replaces one of the end walls. This advantage is offset to a certain extent by the cost of retaining walls along part of the corridor. The final layout selection would be based on desirable operational considerations as well as overall cost.

4. COVERED CORRIDOR CONCEPT

The covered corridor concept described in this paragraph utilizes the parallel shelter configuration. Other shelter layout configurations are described in section IV.

The covered corridor is the central arch shown in figure 38. The arches on either side of the corridor contain and protect the aircraft. Access to the shelters is provided by

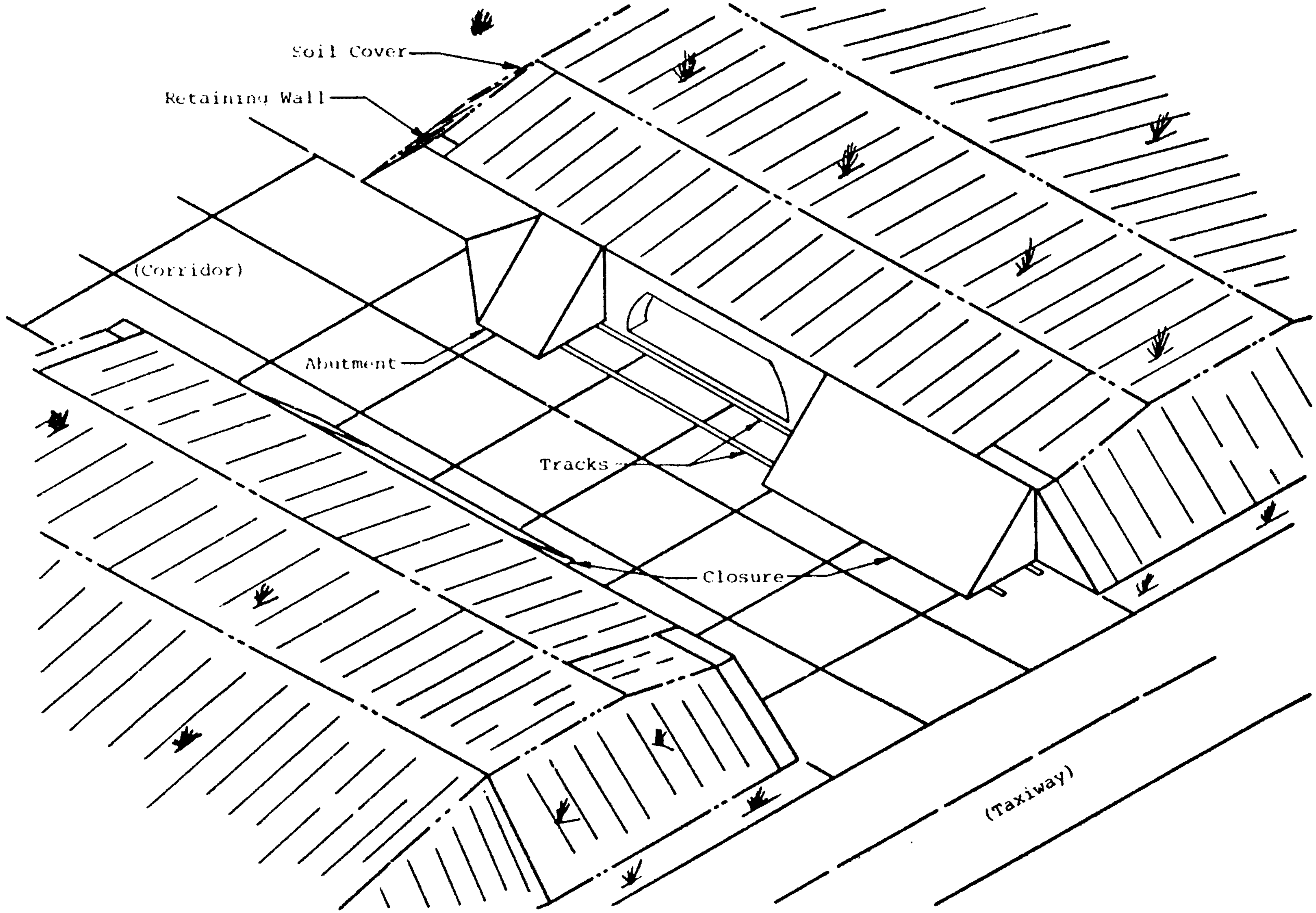


Figure 35. Open Corridor Concept - Side Entry

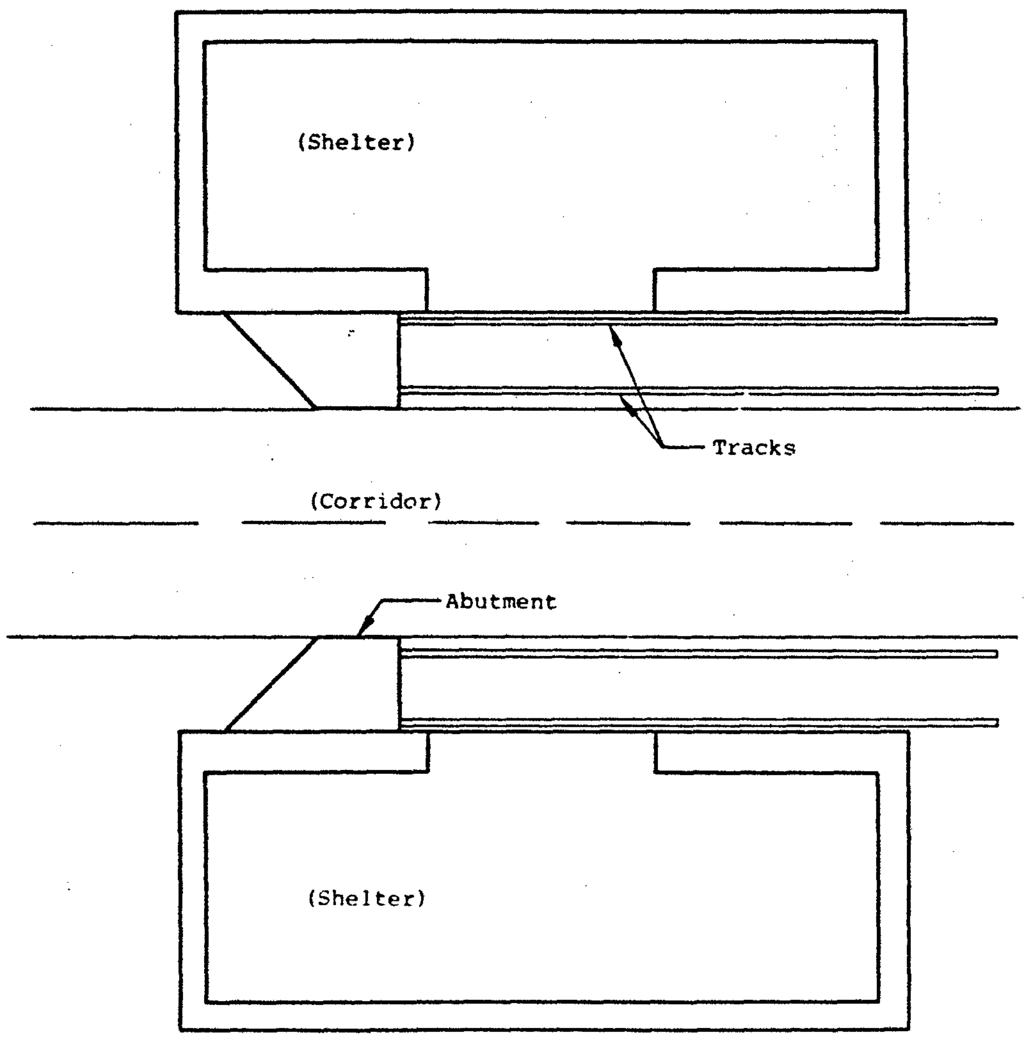


Figure 36. Open Corridor - Side Entry, Plan View

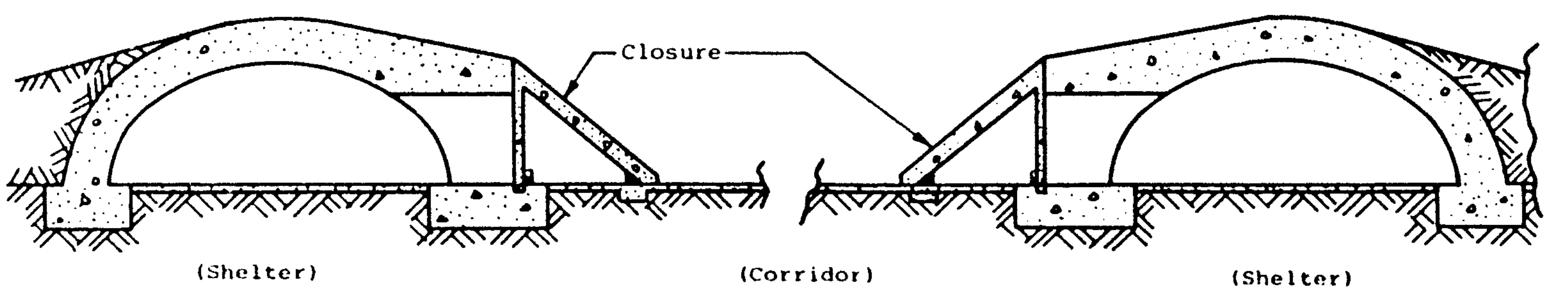


Figure 37. Open Corridor - Side Entry, Sectional Side View

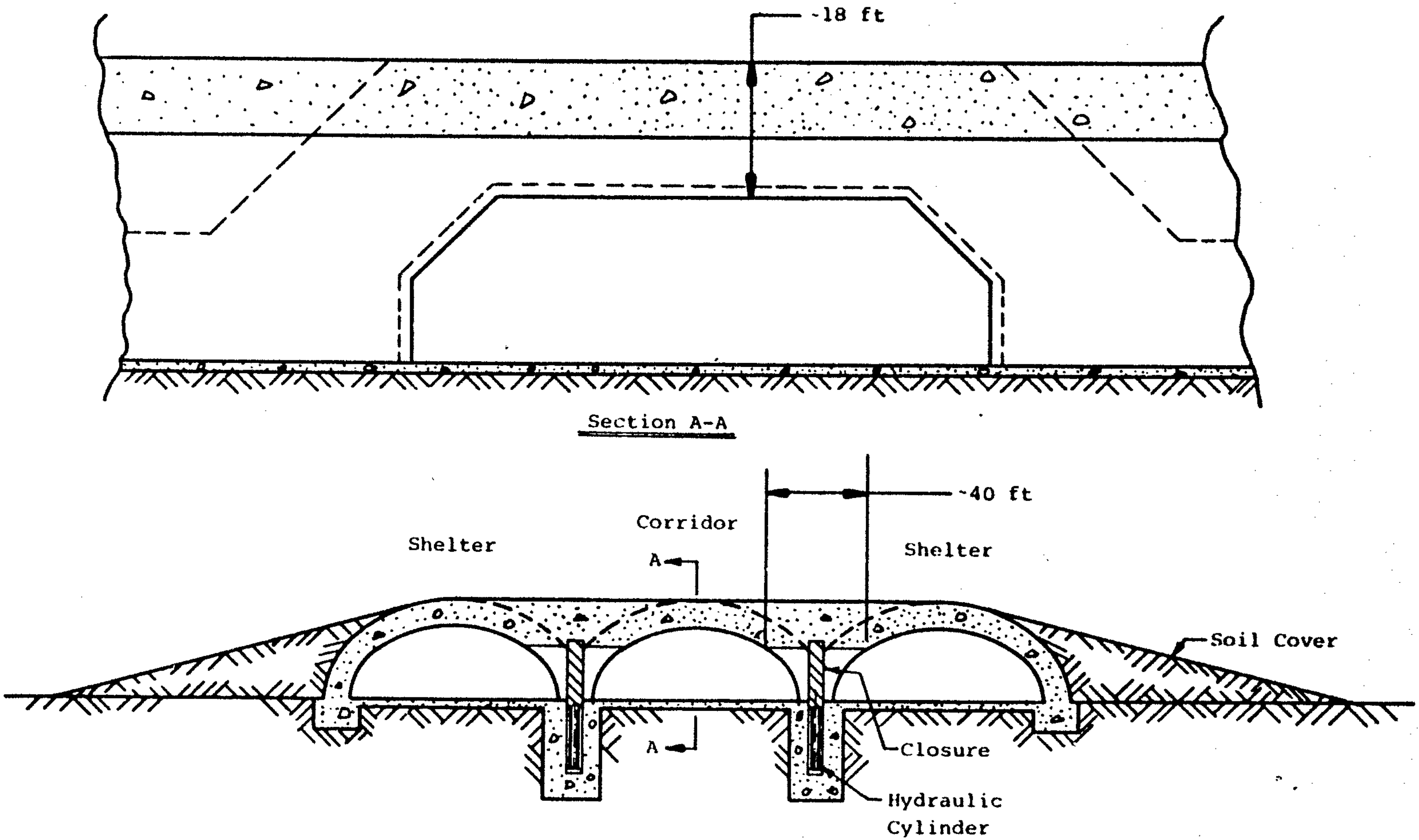


Figure 38. Covered Corridor Concept - Sectional Side View

openings through the sidewalls of the covered corridor as shown in section A-A of figure 38 and in the plan view of figure 39.

Such openings reduce the integrity of the shelter. The final design would provide means for transmitting the large load due to airblast on the exterior surface around the opening into the foundation. One possible approach is illustrated in figure 38 where a solid concrete section is shown above the opening. The width of concrete at the top of the opening would be on the order of 40 feet with a depth of 18 feet. The steel framing around the top of the closure can also be utilized as reinforcing steel. Consequently, it is believed that a detailed analysis will show that the concrete and steel section immediately above the opening will be more than adequate for transmitting overhead loads to the base.

The use of the covered corridor requires a closure that must be designed only for the anticipated peak overpressure. Reflection cannot occur, and penetrating weapons are resisted by the corridor cover.

The closure proposed as most convenient for this concept is a steel door that lifts vertically out of a pocket in the foundation as shown in figure 38. The geometry and construction of the steel closure is shown in figure 40. Movement of the steel closure is accomplished by the actuating mechanism illustrated in figure 41. Two hydraulically operated cylinders

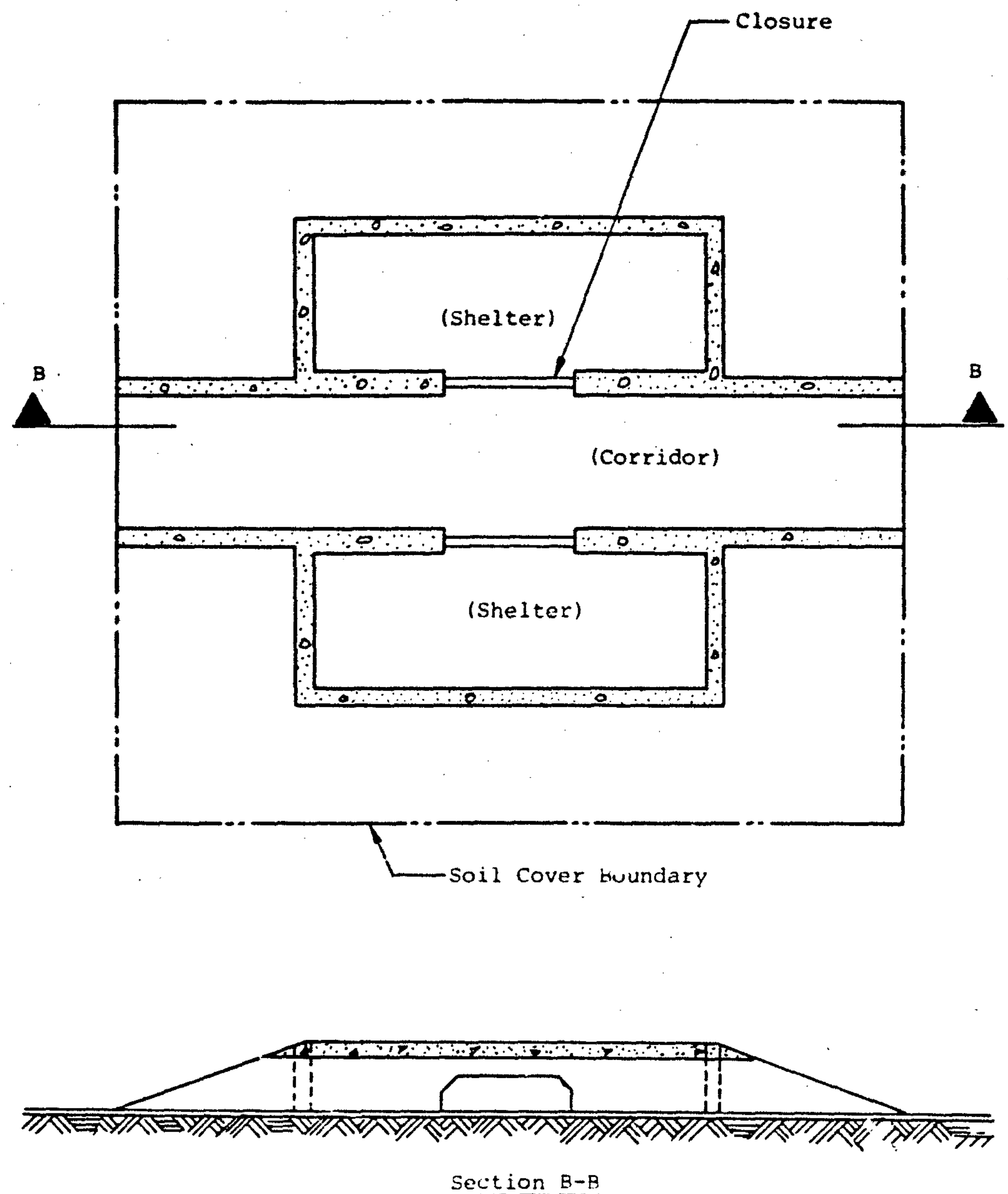
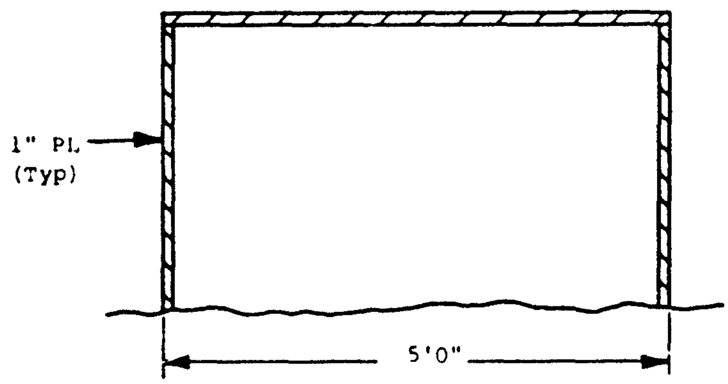


Figure 39. Covered Corridor Concept - Plan View



Section A-A

Typical Edge Detail

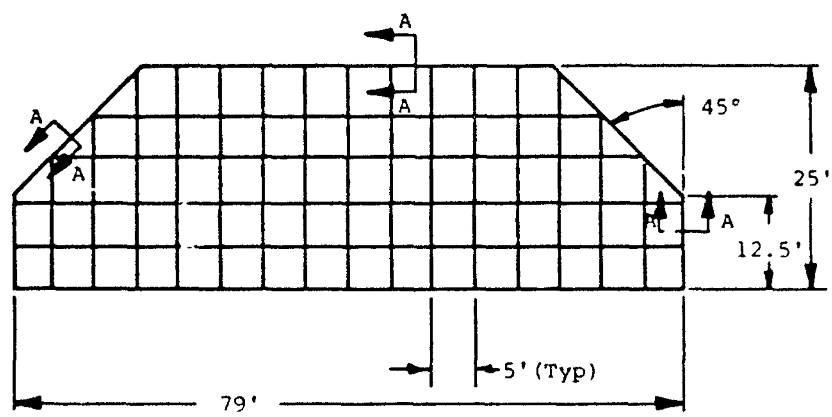


Figure 40. Closure Configuration and Internal Stiffeners

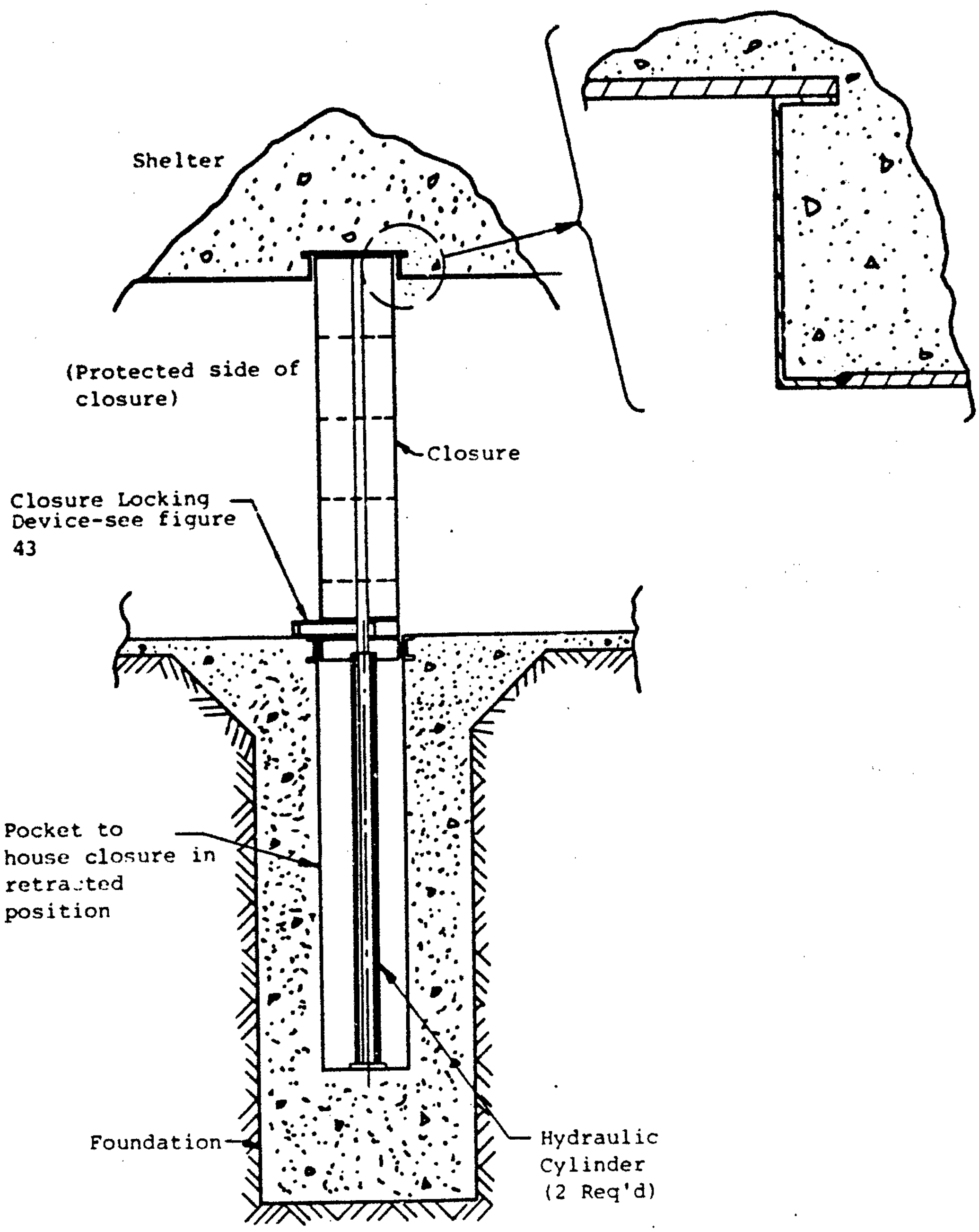


Figure 41. Closure Mechanism

within the closure are used to raise and lower the closure.

The closure is fully supported on all edges by steel lined reinforced concrete. The door design shown requires a minimal amount of steel because of two-way plate action. Fabrication is complicated, however, since the closure cannot be completely assembled outside of the shelter and set into place as a unit. The hydraulic system will have to be sized to lift only the closure, since airblast will not act against the cylinders. When open, the closure rests on stops so that the closure and not the cylinders will support vehicular traffic passing in and out of the shelter. Access to the hydraulic cylinders for maintenance and repair can be attained by retracting the cylinders into the door when it is closed and by providing access panels on the rear face of the door. Alternatively, providing access to the closure pit would allow maintenance and repair of cylinder components. When in the open position, the cylinders could merely be lifted up through the top of the door.

Pressurized hydraulic fluid is supplied to the cylinders by a conventional hydraulic pump and reservoir. For the size of closure presently being considered, a power source of approximately 100 horsepower is required to raise the closure in 3 minutes. By replacing the standard reservoir with a pressurized container (or accumulator), energy can be recovered during the closure opening and utilized during the closing phase. One such possible system is illustrated schematically in figure 42. Each 100 psi in the accumulator will reduce the power requirement for the hydraulic pump by

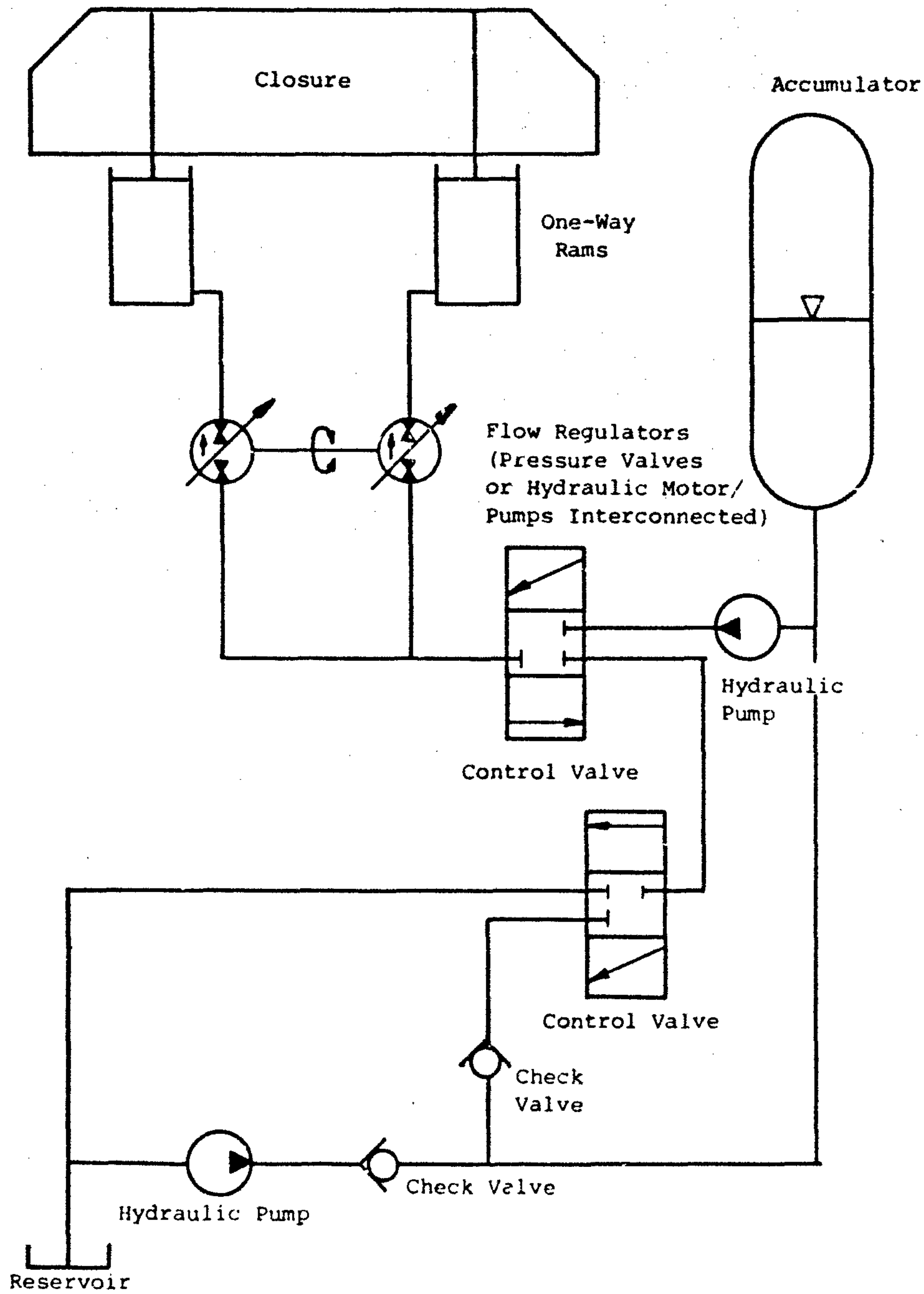


Figure 42. Hydraulic System Schematic

10 horsepower, if friction is neglected. Theoretically, by going to a minimum pressure of 1000+ psi, the system could conceivably store enough energy to raise the closure without additional power.

The simple lifting mechanism of two hydraulic cylinders can be augmented by a manual locking device such as shown in figure 43 that will hold the door in the fully closed position. In the case of loss of power after an attack, the door can be lowered manually.

The primary advantages of this closure system is that it utilizes all aspects of the covered corridor concept. No tracks are required, space within the corridor is not reduced by the door, and the pocket is automatically covered when the door is in the open position.

An alternate closure which has some appeal and which has been successfully used in the past is the drawbridge door shown in figure 44. A closure hinged along its lower edge and opening outward into the covered corridor as shown in figure 45 should be feasible. From its recessed position off the corridor, the closure would be lowered into a pit which would extend outward into the corridor a distance of about 15 feet from each sidewall. This leaves a solid apron width of about 50 feet between pits on opposite sides of the corridor which should be more than adequate for normal airplane traffic along the corridor. However, the pits would be exposed and

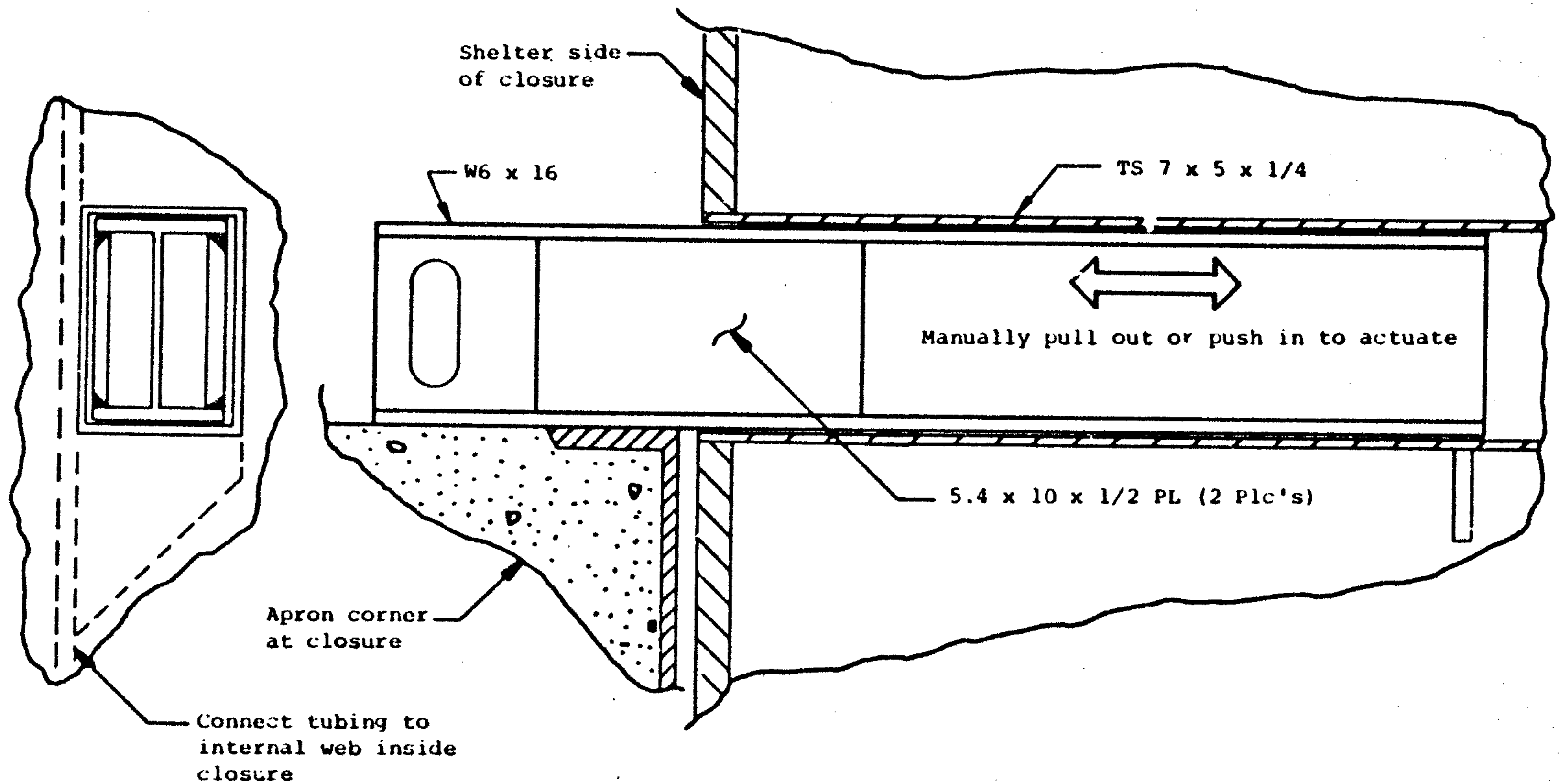


Figure 43. Closure Locking Device

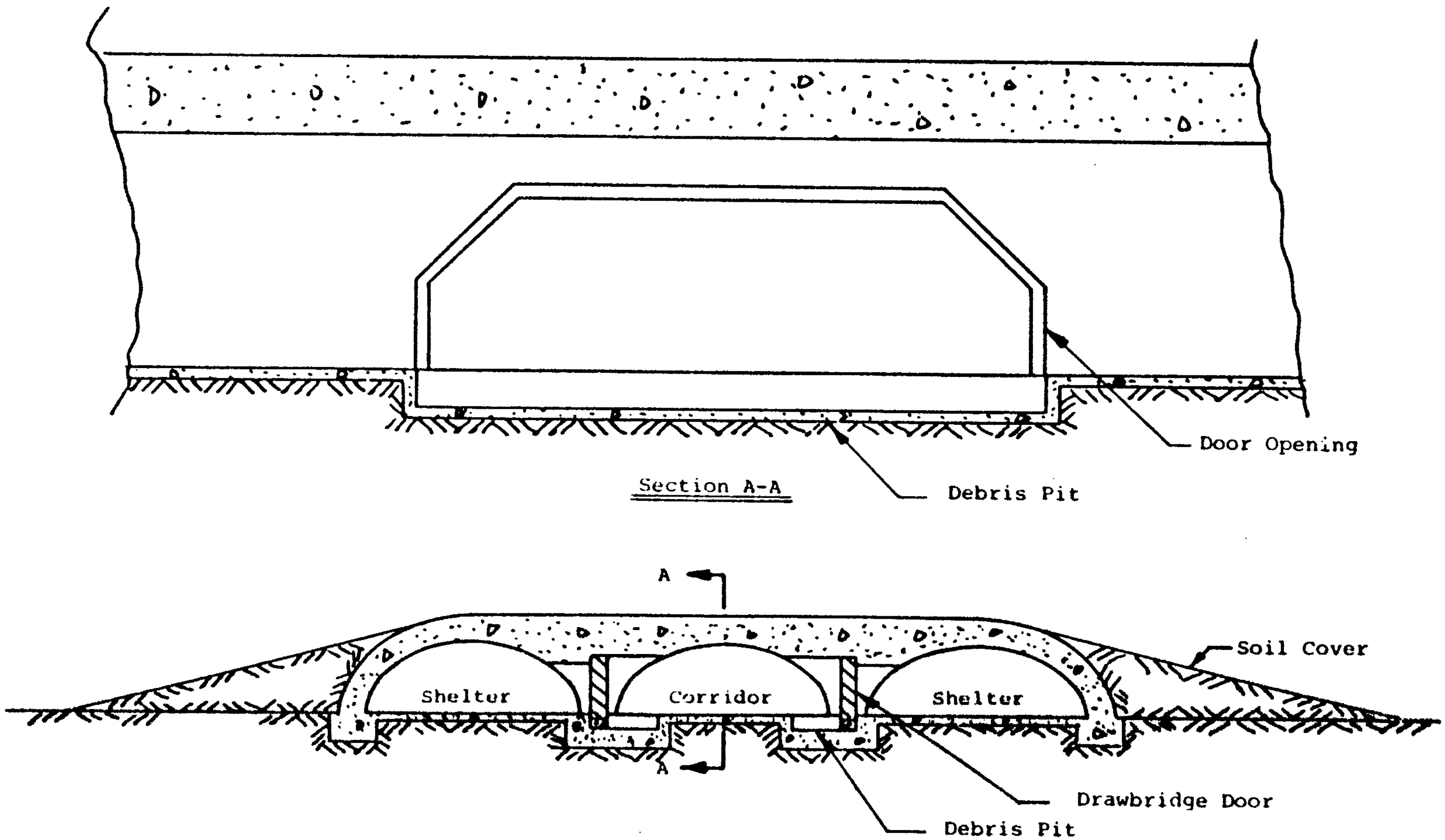


Figure 44. Alternate Covered Corridor Concept - Sectional Side View

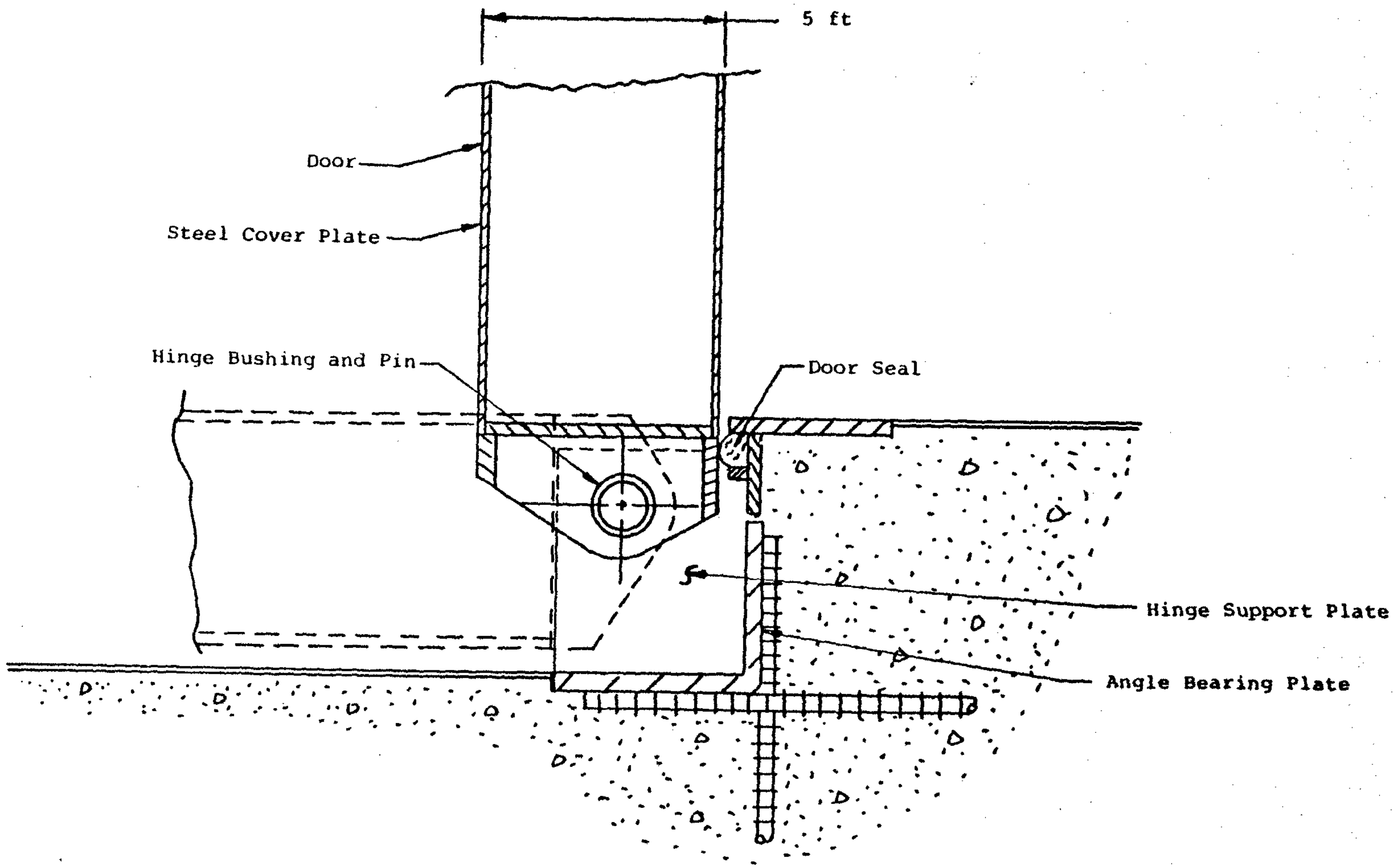


Figure 45. Door Hinge Detail

uncovered when the closure is raised and could present a minor hazard to traffic and personnel. An outward opening drawbridge door is considered preferable, since this direction provides a solid support to resist the primary blast forces. Rebound forces will have to be overcome by auxiliary latching devices. Hydraulic cylinders to open or close the door will provide the least amount of conflict with shelter usage if located on the unprotected side of the drawbridge door where they can push the door into the closed position. Protection of the hydraulic cylinders against fragment damage and napalm would be a problem, however.

Advantages of the drawbridge door concept relative to the vertical door are

- Closure does not operate out of a pocket.
- The amount of externally provided work required for closing is reduced.
- Opening without power is possible.
- Operation is not sensitive to structural tolerances.
- Closure can be fabricated in single or multiple sections.
- Protection level can be readily upgraded.
- Multiple section closure allows partial opening.

Its disadvantages include

- Cutout in apron is required.
- Hinges are non-redundant items.

- Elaborate system of rebound latches is required.
- Hydraulic cylinders on unprotected side of door require additional protection. If they are located on the protected side of the door, they interfere with usable space in both the open and closed positions.
- Maintenance is required on a large number of hinges and latches.

The drawbridge door was not chosen for final evaluation because the vertical lift door has many of the same advantages, and the number of critical operational components of the drawbridge exceeds that required with the vertical lift door.

SECTION VI

EVALUATION OF CONCEPTS

1. BASIS OF EVALUATION

A rating system was devised so that the concepts could be evaluated on as objective and quantitative a basis as possible. Certain aspects, such as cost and reliability, are clearly dominating factors. Consequently, a numerical weighting system has been employed in an effort to ensure that the most important factors are given appropriate consideration.

Certain conditions are implicit in the proposed rating scheme. It is assumed that all candidate shelter and closure concepts can defeat the weapon threat and that the opening and closing times of 3 minutes are met. In certain cases, these are rather severe conditions that could perhaps be relaxed. However, if these requirements were not imposed, the evaluation must then incorporate a vulnerability factor and the rating scheme becomes much more susceptible to subjective judgements. By rigorously imposing a required set of conditions, the effects are reflected in costs which are much more amenable to firm figures and rational means for assigning numerical ratings.

A maximum numerical rating of 200 was adopted for evaluation of the concepts. The rating for each of the factors used in the evaluation and a discussion of the items that are included in each factor follow.

Camouflage potential, emergency close, and repairability were assigned a numerical rating of 10. Each of these factors incorporate desirable, but not absolutely essential, aspects of a shelter complex. On the other hand, the ability to open a facility under emergency conditions is rather critical, and, accordingly, a numerical factor of 15 was assigned to this item.

Camouflage potential is interpreted to mean the capability of using tonedown, vegetation or inexpensive decoy structures that would hinder visual target acquisition from attacking aircraft. No attempt was made to make an evaluation of camouflage from infrared or other sophisticated detection devices.

There can be significant differences in the concepts based on the ability to operate the closures in an emergency (no power) situation. If a closure is light and is moved laterally on rollers, then hand winches and motorized vehicles can be utilized. On the other hand, heavy closures that must be raised or lowered can provide almost insurmountable problems with a loss in power supply. However, such a concept will usually have a very high numerical rating for either opening or closing, since the closure can be self-acting in one of these modes.

Repairability incorporates features such as ease of accessibility to damaged parts of a shelter complex, materials and equipment required to make repairs, and the complexity of a repair task. For example, a steel portion of a structure that could be repaired by simply welding a standard structural member into place would get a higher rating than one that required a component manufactured to close tolerances. Since the shelter portions of the concepts are essentially identical, the repairability factor will be largely based on the closures. An additional factor is the amount of sophisticated equipment that is needed for repair because such equipment may not be immediately available under post-attack conditions. Thus, the assigned total numerical rating of 15 for repairability has been separated into normal and post-attack situations.

A rating number of 20 was assigned to maintainability, which is important from an operations viewpoint and is an indicator of the simplicity of a shelter concept. The number and type of moving (rolling, sliding, rotating, etc.) parts essential to the operation of the facility are identified, evaluated and assigned a weight value commensurate with the frequency and difficulty of the maintenance, repair or replacement operation that would be required. The greater the probability or seriousness of any of these operations, the larger the weight number that is assigned. By multiplying the number of such components by the corresponding weight values and summing the results, a maintenance number, M_n, can be obtained for a concept. The lower the maintenance number, the more desirable the concept.

Reliability must be considered for both normal operating and post-attack conditions. Consequently, a higher rating number of 30 is used. Normal operating reliability is important in that an emergency situation can arise with practically no forewarning; and, if the reliability of a system is poor, the higher probability of down time of a complex makes that shelter system more vulnerable. High reliability under post-attack conditions implies that needed repairs can be performed or alternate systems invoked to quickly bring a shelter complex back to an operational status.

Reliability under normal conditions is rated in a manner very similar to that for maintainability. Each moving component is identified and a weight value assigned to it based on the probability of malfunction with larger weight values associated with the higher probabilities. A multiplication of the number of components by the weight values and a sum of the results will yield a reliability number, R_n. The most reliable operational concept will have the lowest value for R_n.

It is unrealistic to assume that an aircraft shelter system will always be operational immediately after an attack. Even though the system will defeat a given weapon threat, there will likely be at least a debris problem. Although the probability is very low, there is the possibility of a direct hit on an operational component such as a track, for example. Although the contents of the shelter may not be damaged, some time will be required to make the necessary repairs. Since this time is a critical factor, a time of repair is used as a means for evaluating the post-attack reliability of a shelter system. For each concept, conceivable situations within the specified threat range are postulated. For each situation, an estimate is made for the time required to repair the damage. If machinery or parts are required from a centralized facility, this will be reflected rather drastically in the repair time. A similar situation holds if parts must be removed, replaced, repaired or manufactured. The maximum estimated repair time, T_{max}, for a variety of situations is recorded for each concept. The concept with the lowest such time will be considered the best insofar as post-attack reliability is concerned. A maximum rating number of 15 out of the total of 30 for reliability will be assigned to the parameter T_{max}.

The dominant effect of cost is reflected in the assigned rating value of 100. However, the fact that this accounts for only one half the total possible rating allows the operational considerations discussed above to be reflected in the overall rating for each concept. Although detailed cost estimates are impossible from conceptual drawings, it is believed that previous experience with similar uncertainties provides a basis for cost estimates that should be within ± 25 percent of actual costs. Furthermore, the use of the same unit costs for each concept and the assignment of a rating by means of comparison tends to minimize the effect of approximations used in the cost estimates. A shelter complex that holds four aircraft will be used as the standard unit for estimating costs, with six- and eight-unit costs also provided to indicate the added potential cost benefit that could be achieved by parking a larger number of aircraft in a single shelter complex.

Additional costs such as contractor overhead, taxes and profit have not been included in the preliminary estimates. Such costs are typically proportional to the base cost and, consequently, would have very little effect on the final rating of a concept.

The maximum cost factor rating is assigned to the concept with the lowest cost. The rating for other concepts is reduced by the ratio of the lowest cost to the cost of each other concept. A similar procedure is followed for evaluating maintainability and reliability. For the remaining factors, rating numbers are assigned based primarily on engineering judgement.

A summary of the evaluation scheme is shown in table 5.

2. EVALUATION OF ELEVATOR CONCEPT

a. Camouflage Potenetial

The only visual aboveground features associated with

Table 5

SUMMARY OF EVALUATION RATING FACTORS

Factor	Basis	Maximum Rating Number
Camouflage Potential	Judgement	10
Emergency Open	Judgement	15
Emergency Close	Judgement	10
Repairability	Judgement	
Normal Conditions		7
Post-Attack Conditions		8
Maintainability	Lowest M_n	20
Reliability		
Normal Operation	Lowest R_n	15
Post-Attack Operation	Lowest T_{max}	15
Cost	Lowest Cost	100
Maximum Possible Rating		200

the elevator concept are the cover and tracks. With the use of tonedown and placement of the shelter so that no obvious clues are provided by the taxiway layout, the camouflage potential is considered quite good. Numerical Rating : 10

b. Emergency Open

The rather massive closure associated with this concept provides a difficult situation when the standard power source is not available. The use of hand winches or motorized vehicles does not appear feasible, so an adequate emergency power source will be required. Numerical Rating : 5

c. Emergency Close

The same situation exists for emergency closing as for emergency opening. Numerical Rating : 5

d. Repairability

(1) Normal Conditions

Winches, cables and electric motors for driving the winches are all easily accessible for visual inspection, repair and replacement. However, some difficulty would be associated with the hydraulic rams which are located lower than the shelter floor. Special portals would be necessary for providing access to some of the hydraulic fittings and end caps of the rams. The hydraulic power supply would be a standard unit in a convenient location. The steel cylinders that help support the cover and locking devices are all readily accessible and repairable.

The primary drawback of this concept from the point of view of repairability is the large number and variety of parts that could conceivably need repair or replacement. Thus, it is believed that a low rating is warranted. Numerical Rating : 2

(2) Post-Attack Conditions

The cover and elevator platform are highly redundant structural elements so that, in most instances, immediate repairs will not be necessary. Welding in alternate structural elements will generally suffice for these components. The cover tracks will require some care to ensure a smooth surface, but standard grouting and structural steel replacement should be adequate here as well. Numerical Rating : 6

e. Maintainability

Track dollies, cables, winches, pulleys and all hydraulic components must be periodically inspected, replaced or serviced as required.

Part	Number Parts	Weight	Part Maintenance Number
Cover			
Jacking Truck	30	2	60
Hydraulic Pump & Motor (Jacking Trucks)	1	5	5
Winch	2	7	14
Winch Motor	2	5	10

Part	Number Parts	Weight	Part Maintenance Number
Cable	400 LF	0.01	4
Pulley	6	1	6
Track	340 LF	0.01	3
Winch Control Unit	1	1	1
Platform			
Elevator Latch	8	1	8
Hydraulic Pump & Motor	4	4	16
Hydraulic Ram	4	2	8
Hydraulic Hose	1000 LF	0.01	10
Cover Support Column & Latch	8	2	16
Hydraulic Synchronizer	1	1	1
		Concept Maintenance Number:	162

f. Reliability

(1) Normal Conditions

There are a large number of components associated with the operation of the elevator shelter concept. With respect to some components, a certain degree of redundancy makes the system more reliable, whereas other components are absolutely essential to normal operation. The weighting numbers were assigned according to how the loss of such a component would affect normal operations and the probability of such an event occurring.

Possible Failure	Number of Parts	Weight	Part Reliability Number
Cover			
Loss of Jacking Truck	30	0.5	15
Hydraulic Pump or Motor Failure (Jacking Truck)	1	5	5
Winch Failure	2	20	40
Winch Motor Failure	2	10	20
Cable Break	400 LF	0.005	2
Pulley Failure	6	1	6
Track Failure	340 LF	0.005	2
Winch Control Unit Outage	1	4	4
Platform			
Elevator Latch	8	0.5	4
Hydraulic Pump or Motor Failure	4	15	60
Hydraulic Ram Failure	4	5	20
Hydraulic Hose Failure	1000 LF	0.005	5
Cover Support Column or Latch Failure	8	0.5	4
Hydraulic Synchronizer Failure	1	4	4
		Concept Reliability Number:	191

(2) Post-Attack Conditions

Almost all components are protected within the shelter itself and, consequently, very few parts are vulnerable. The more likely post-attack situations are summarized below.

Event	Estimated Time of Repair (hours)
Direct Hit on Track	2
Direct Hit on Cable Conduit	4
Direct Hit on Edge of Cover	0.5
Permanent Deformation of Cover (Requiring some removal of material)	0.5
Debris on Track	0.5
Maximum Time of Repair:	4 hours

g. Cost

(1) General

The natural choice of a layout for the elevator concept is the in-line configuration for four aircraft shown in figure 14. Simple extensions of the shelters can be made to provide room for six or eight aircraft. Although other layouts may be more advantageous operationally, the difference in estimated cost at this stage of design should be minimal.

(2) Excavation and Backfill

It is assumed that a scraper would be used for excavation and that approximately twice the amount of soil must be removed over that of the shelter volume to provide reasonable slopes for the scraper. Excavation is assumed to be to a depth of 45 feet.

Item	Cost
Excavate Shelter and Elevator Region (150 x 40 x 15) x 2 = 180,000 yd^3 @ \$0.75	\$ 135,000
Backfill First 2 Yards Adjacent to Shelter (180 x 2 x 15) = 5400 yd^3 @ \$5.00	27,000
Stockpile, Fill and Compact Half Original Volume 90,000 yd^3 @ \$0.95	86,000
Transport Excess Soil 1/2 Mile Away 85,000 yd^3 @ \$0.075	6,000
Base Cost	\$ 254,000
Contingency @ 20%	51,000
Cost of Excavation and Backfill:	\$ 305,000

(3) Shelters and End Walls

There are two end walls, and the required shelter length is 80 feet per aircraft.

Item	Cost
Shelter : 320 feet @ \$10,000/ft	\$ 3,200,000
End Wall : 2 @ \$170,000	340,000
Cost of Shelters and End Walls:	\$ 3,540,000

(4) Cover

Item	Cost
Composite Cover (Steel & Concrete), 900 yd^3 @ \$50 + 1,175,000 lb @ \$1.00	\$ 1,220,000
Jacking Truck, 30 ea @ \$2000	60,000
Hydraulic Power Unit	5,000
Foundation and Track, 2 ea x 170 ft @ \$500/ft	170,000
Winch and Motor (75 ton @ 30 fpm), 2 @ \$60,000	120,000
Cost of Cover:	\$ 1,575,000

(5) Elevator

Item	Cost
Structural Platform, 370,000 lb @ $1.00	$ 370,000
Platform Latches, 8 @ $1000	8,000
Hydraulic Power Unit, 4 @ $6,000	24,000
Hydraulic Cylinders (14 in x 40 ft stroke), 4 @ 32,000	128,000
Cover Supports, 8 @ $18,000	144,000
Cylinder Synchronizer, 4 @ $5,000	20,000
Shaft Sidewall-no opening, 2 @ $250,000	500,000
Shaft Sidewall-opening, 2 @ $160,000	320,000
Foundation for Sidewalls, Cylinders, Supports (est)	500,000
Cost of Elevator:	$2,014,000

(6) Total Cost for 4 Aircraft

Item	Cost
Excavation and Backfill	$ 305,000
Shelters and End Walls	3,540,000
Cover	1,575,000
Elevator	2,014,000
Total Cost, 4 Aircraft:	$7,434,000

(7) Total Costs for Six and Eight Aircraft

Additional shelter space for aircraft does not affect the cost of the cover nor the elevator. For six and eight aircraft, an additional 160 feet and 320 feet, respectively, of shelter space must be provided. For four aircraft, an equivalent length of 2 x 150 yards or 900 feet had to be excavated and a corresponding amount of backfill placed. Thus, a reasonable estimate of the additional cost for excavation and backfill is to increase the four-unit cost by the factors 160/900 and 320/900 for the six- and eight-unit shelters respectively. The shelters themselves cost an additional $10,000/ft.

6-Unit Concept

Item	Cost
Excavation and Backfill	$ 359,000
Shelters and End Walls	5,140,000
Cover	1,575,000
Elevator	2,014,000
Total Cost, 6 Aircraft:	$9,088,000

8-Unit Concept

Item	Cost
Excavation and Backfill	$ 413,000
Shelters and End Walls	6,740,000
Cover	1,575,000
Elevator	2,014,000
Total Cost, 8 Aircraft:	$10,742,000

3. EVALUATION OF OPEN CORRIDOR CONCEPT

a. Camouflage Potential

The shelters themselves can be covered with soil and vegetation so that the camouflage potential of a portion of a complex based on this concept is quite good. However, the open corridor and the closures are large and exposed, so that even with tonedown, there would still be a strong likelihood of visual detection. The camouflage potential of this concept as a whole can be considered fair. Numerical Rating : 6

b. Emergency Open

The difficulties associated with opening these closures are considered approximately equal to those of the elevator concept. Numerical Rating : 5

c. Emergency Close

The same situation exists for closing under emergency conditions as for opening. Numerical Rating : 5

d. Repairability

(1) Normal Conditions

Under normal operating conditions, repair and replacement of components of the jacking truck, hydraulic system and the winch and cable system would be expected on a regular basis. There is some redundancy with regard to the jacking trucks, but all of the other components are essential items. Accessibility is fair. Numerical Rating : 4

(2) Post-Attack Conditions

The concrete door that is proposed for this concept is exposed to direct hits; and, consequently, rather thick concrete panels may have to be repaired. However, the door would be operational without such panels; therefore,. this is not a critical factor other than the associated debris problems in an emergency situation. Any damage to the track would have to be repaired immediately. Other components are designed to be protected from the threat spectrum. Numerical Rating : 6

e. Maintainability

The jacking truck units, cable, winch, motor and track must be checked periodically and serviced or replaced.

Part	Number Parts	Weight	Part Maintenance Number
Jacking Truck	30	2	60
Hydraulic Pump & Motor (Jacking Trucks)	1	5	5
Winch	2	7	14
Winch Motor	2	5	10
Cable	400 LF	0.01	4
Pulley	6	1	6
Track	340 LF	0.01	3
Winch Control Unit	1	1	1
Hydraulic Hose	160 LF	0.01	2

Maintenance Number for One Closure: 105

Concept Maintenance Number : 210

f. Reliability

(1) Normal Conditions

Under normal conditions, the winch system would be the component most vulnerable to breakdown. There is also the possibility that the hydraulic system used for raising the closure prior to opening or closing might fail.

Possible Failure	Number of Parts	Weight	Part Reliability Number
Loss of Jacking Truck	30	0.5	15
Hydraulic Pump or Motor Failure (Jacking Truck)	1	5	5
Winch Failure	2	20	40
Winch Motor Failure	2	10	20
Cable Break	400 LF	0.005	2
Pulley Failure	6	1	6
Track Failure	340 LF	0.005	2
Winch Control Unit Failure	1	4	4
Hydraulic Hose Failure	160 LF	0.05	8
		Reliability Number for One Closure:	102
		Concept Reliability Number :	204

(2) Post-Attack Conditions

As with the elevator concept, tracks are exposed and a cable conduit may be damaged. However, the other components are protected within the shelter or by the closure.

Event	Estimated Time of Repair (hours)
Direct Hit on Track	2
Direct Hit on Cable Conduit	4
Direct Hit on Edge of Closure	0.5
Debris on Track	0.5
Maximum Time of Repair:	4 hours

g. Cost

(1) General

The same shelter layout utilized for the elevator concept cost estimate is used for the open corridor concept. The same general procedure for estimating extensions to six- and eight-unit complexes is also adopted.

Equal cut and fill requirements will be approximated by arbitrarily assuming an excavation to a depth 24 feet below the surface. This will provide an excess of material for gently sloping berms and cover over the shelters.

(2) Excavation and Backfill

Assume that a scraper is used for excavation and that approximately twice the minimum amount of soil must be removed to provide reasonable slopes for the scraper and taxiways. The length of excavation for each side shelter is taken as 60 yards, the width as 40 yards, and the corridor dimensions between the shelter faces as 40 yards by 50 yards. Close-in backfill is placed around three sides (170 yards) to the full height of the shelters.

Item	Cost
Excavate Shelter and Taxiway Region (170 x 40 x 8) x 2 = 109,000 yd^3 @ \$0.75	\$ 82,000
Stockpile, 109,000 yd^3 @ \$0.10	11,000
Backfill First 2 Yards Adjacent to Structures (170 x 15 x 2) x 2 = 10,200 yd^3 @ \$5.00	51,000
Construct Berm and Compact Remaining Soil, 99,000 yd^3 @ \$0.85	84,000
Base Cost	\$ 228,000
Contingency @ 20%	46,000
Cost of Excavation and Backfill:	\$ 274,000

(3) Shelters, End Walls and Retaining Walls

The shelters and end walls away from the corridor are identical to those used in the elevator concept. However, with this concept, retaining walls must be constructed adjacent to the corridor to hold the berms in place. For estimating purposes, each concrete retaining wall is assumed to be triangular in shape, 3 feet thick, 21 feet high next to the shelter, and have a base length of 105 feet (5:1 span to rise ratio of the berm).

Item	Cost
Shelter, 320 feet @ $10,000/ft	$3,200,000
End Wall, 2 @ $170,000	340,000
Retaining Wall, 4 @ $21,000	84,000
Cost of Shelters, End Walls and Retaining Walls:	$3,624,000

(4) Closures

With this concept, the closures are composed of reinforced concrete elements. The use of concrete provides a relatively low cost door, but the large weight requires significant expenditures for components of the moving mechanism. Only the major components are included for the purpose of this estimate. Track costs include excavation and backfill and are based on more detailed analyses performed on a previous study. Winch and motor costs were obtained from commercial sources.

Item	Cost
Concrete, 1370 yd^3 @ $50	$ 69,000
Steel, 132,000 lb @ $1.00	132,000
Jacking Truck, 30 @ $2000	60,000
Foundation and Track, 160 ft @ $1000	160,000
160 ft @ $500	80,000
Winch and Motor, 2 @ $60,000	120,000
Hydraulic Pump and Hose	5,000
Abutment (est)	5,000
Seal Between Door and Shelter, 110 ft @ $50	6,000
Cost per Closure :	$ 637,000
Cost for Two Closures:	$ 1,274,000

(5) Total Cost for 4 Aircraft

Item	Cost
Excavation and Backfill	$ 274,000
Shelters, End Walls and Retaining Walls	3,624,000
Closures	1,274,000
Total Cost, 4 Aircraft:	$ 5,172,000

(6) Total Costs for Six and Eight Aircraft

Additional aircraft do not affect the costs of end walls, retaining walls or closures. For six and eight aircraft, an additional 160 feet and 320 feet, respectively, of shelter space must be provided. For four aircraft, an equivalent length of 2 x 160 yards or 960 feet had to be excavated with a corresponding amount of backfill. Thus, an estimate for the added cost of two or four more aircraft can be obtained by using the add on factors 160/960 and 320/960, respectively. The increase in shelter cost is computed at $10,000/ft.

6-Unit Concept

Item	Cost
Excavation and Backfill	$ 320,000
Shelters, End Walls and Retaining Walls	5,224,000
Closures	1,274,000
Total Cost, 6 Aircraft:	$ 6,818,000

8-Unit Concept

Item	Cost
Excavation and Backfill	$ 365,000
Shelters, End Walls and Retaining Walls	6,824,000
Closures	1,274,000
Total Cost, 8 Aircraft:	$ 8,463,000

4. EVALUATION OF COVERED CORRIDOR CONCEPT

a. Camouflage Potential

Both the shelters and the corridor can be covered with soil and vegetation so that the only distinguishing features would be the taxiway and the ends of the corridor. The camouflage potential is considered to be good. Numerical Rating : 8

b. Emergency Open

The proposed closure is raised vertically with hydraulic rams; and, in the event of a power failure, the closure can be opened by utilizing the pressurized accumulator to withdraw the locking devices and then to bleed the hydraulic fluid in the rams into a storage tank. The emergency opening power is provided by the weight of the door, so the opening capability is considered good. Numerical Rating : 13

c. Emergency Close

With this concept, the situation for providing power to close the door is considerably different than for the other two concepts. Power must be provided to raise the door. One possibility is the use of high pressure inert gas cylinders which could be stored underground and which would provide the necessary lifting force. Otherwise, an emergency electrical power source will be necessary. Thus, the emergency closing capability is considered to be no better than the other two concepts. Numerical Rating : 5

d. Repairability

(1) Normal Conditions

Under normal operating conditions, such items as hydraulic seals, lines, pumps and guide rollers may have to be repaired or replaced. Since there are a small number of such components and they are directly accessible within the shelter or by the removal of panels on the inside surface of the door, repairability is considered good. Numerical Rating : 7

(2) Post-Attack Conditions

The primary weapon threat to the closure is that of airblast. In addition, some projectiles may reach the door so that certain sections may have to be replaced. Since the door is constructed of steel and the tolerances are not close, repair of the door itself should pose no significant problem. The hydraulic components are off-the-shelf items and readily accessible for quick replacement of parts. Consequently, the repairability of this concept under emergency conditions is also considered good. Numerical Rating : 7

e. Maintainability

The primary unit is the hydraulic system which must be checked periodically for deterioration and seal leaks.

Part	Number of Parts	Weight	Part Maintenance Number
Hydraulic Pump and Motor	2	4	8
Hydraulic Ram	2	2	4
Hydraulic Hose	100 LF	0.01	1
Control Unit	1	1	1
Guide Roller	8	0.1	1

Maintenance Number for One Closure: 15

Concept Maintenance Number : 30

f. Reliability

(1) Normal Conditions

The hydraulic system is essential to the operation of the closure system proposed for this concept. No redundancy in the number of hydraulic rams has been provided.

Possible Failure	Number of Parts	Weight	Part Reliability Number
Hydraulic Pump or Motor Failure	2	15	30
Hydraulic Ram Failure	2	5	10
Hydraulic Hose Failure	100 LF	0.05	5
Control Unit Failure	1	4	4
Guide Roller Failure	8	0.1	1
		Reliability Number for One Closure:	50
		Concept Reliability Number :	100

(2) Post Attack Conditions

The essential aspect of the covered apron concept is that there are no elements exposed to weapon effects other than airblast. There may be some permanent deformation of closure structural elements, but allowance for this would be provided in the design. The major item concerns a direct hit on the edge of the corridor which would provide a large debris problem.

Event	Estimated Time of Repair (hours)
Direct Hit on End of Corridor	2
Penetration or Cutting of Hydraulic Hose	1
Maximum Time of Repair:	2 hours

g. Cost

(1) General

The shelter layout utilized for the cost estimate is the configuration used for the previous concepts to provide a consistent basis for comparison. The same procedure is used for estimating the cost of a six- or eight-aircraft shelter complex based on the cost of a four-aircraft complex.

Excavation to a depth of 24 feet is assumed to provide an adequate amount of material to provide a balance of cut and fill. The soil overburden will be gently sloped and will accommodate any excessive fill that may result.

(2) Excavation and Backfill

The amount of excavation required for the covered corridor concept will be almost identical to that for the open corridor scheme. There will obviously be some differences between the two concepts as to where the backfill is placed. The cost differential, however, is considered to be negligible, particularly with a balanced cut and fill design. The estimated excavation and backfill quantities and costs are, therefore, taken the same as for the open corridor concept.

Item	Cost
Excavate Shelter and Taxiway Region (170 x 40 x 8) x 2 = 109,000 yd^3 @ \$0.75	\$ 82,000
Stockpile, 109,000 yd^3 @ \$0.10	11,000
Backfill First 2 Yards Adjacent to Structures (170 x 15 x 2) x 2 = 10,200 yd^3 @ \$5.00	51,000
Construct Berm and Compact Remaining Soil, 99,000 yd^3 @ \$0.85	84,000
Base Cost	\$ 228,000
Contingency @ 20%	46,000
Cost of Excavation and Backfill:	\$ 274,000

(3) Shelters, End Walls and Corridor

The shelters and end walls are identical to those used in the previous two concepts. In this concept, there is a corridor between the shelters that has a cross section identical to the shelters. Therefore, the cost of this concept must include the additional length of shelter required to construct the covered corridor. The length of the corridor is assumed to be 150 feet.

Item	Cost
Shelter, 320 ft @ $10,000/ft	$ 3,200,000
End Wall, 2 @ $170,000	340,000
Corridor, 150 ft @ $10,000/ft	1,500,000
Cost of Shelters, End Walls and Corridor:	$ 5,040,000

(4) Closures

The closure chosen as most suitable for this concept is the vertically operated, spaced steel planar door which retracts into a pocket in the shelter floor. The minimum steel thickness in the closure is 2.0 inches. The closure is sized to accommodate only the incident overpressure, since significant reflections cannot occur. Fragmentation weapons present a minor threat to the closure if an extremely accurate or lucky delivery places a weapon at the corridor entrance. However, fragments from a weapon detonation at this position can strike the closure only at oblique angles. Consequently, it is not necessary that the closures provide the full zero obliquity fragment penetration protection thickness. Current manufacturer price lists were utilized for costs of standard hardware items.

Item	Cost
Steel Door, 295,000 lb @ $1.00	$ 295,000
Steel Frame to Contain Door in Closed Position and Reinforcing for Shell Intersection Region, 120 ft @ $400	48,000
Additional Concrete for Shell Intersection Region, 960 yd^3 @ $50	48,000
Hydraulic Cylinder, 2 @ $21,500	43,000
Hydraulic Power Unit, 2 @ $6,000	12,000
Cylinder Synchronizer, 2 @ $5,000	10,000
Pit under Closure	92,000
Cost per Closure :	$ 548,000
Cost for Two Closures:	$ 1,096,000

(5) Total Cost for 4 Aircraft

Item	Cost
Excavation and Backfill	$ 274,000
Shelters, End Walls and Corridor	5,040,000
Closures	1,096,000
Total Cost, 4 Aircraft:	$ 6,410,000

(6) Total Cost for Six and Eight Aircraft

The addition of more aircraft does not affect the cost of the corridor, end walls or closures. In the standard shelter, the equivalent length of excavation is 960 feet. The approximate cost for the excavation and backfill operations can, therefore, be approximated by increasing the previous amounts by an amount of 160/960 and 320/960 for the six- and eight-aircraft facilities. Additional length of shelter costs $10,000 per foot.

6-Unit Concept

Item	Cost
Excavation and Backfill	$ 320,000
Shelters, End Walls and Corridor	6,640,000
Closures	1,096,000
Total Cost, 6 Aircraft:	$ 8,056,000

8-Unit Concept

Item	Cost
Excavation and Backfill	$ 365,000
Shelters, End Walls and Corridor	8,240,000
Closures	1,096,000
Total Cost, 8 Aircraft:	$ 9,701,000

5. COST SUMMARY

One method of illustrating the estimated costs for each concept is shown in table 6. The shelter and end wall costs are the same for all concepts. The excavation and backfill costs do vary from concept to concept, but this variation is rather inconsequential when compared with the total cost for any system. Thus, an average figure has been used in this table to illustrate the relative significance of the components in arriving at the total estimated cost. The resulting

Table 6

COMPARISON OF CONCEPT COMPONENT COSTS

Features Common to All Concepts

Component \ Shelter Capacity	4 Aircraft ($000)	6 Aircraft ($000)	8 Aircraft ($000)
Excavation and Backfill	300	350	400
Shelter and End Walls	3,540	4,740	6,740

Features Unique to Each Concept

Concept	Closure System	Cost ($000)
Elevator	Elevator and Cover	3,500
Open Corridor	Doors and Retaining Walls	1,350
Covered Corridor	Doors and Corridor	2,600

figures emphasize that the fundamental cost variable is that associated with the closure system.

The total estimated costs for each concept for four, six, and eight aircraft shelter facilities are given in table 7 together with the shelter cost per aircraft in each case. These figures show that the open corridor concept is the least expensive, while the elevator concept is the most expensive.

6. CONCEPT EVALUATION SUMMARY

The proposed evaluation system requires that certain factor ratings be determined for all concepts before relative rating values can be assigned individual concepts. The concept with the lowest evaluation number for these factors receives the highest rating. Ratings of the remaining concepts on these factors are scaled down according to the ratios of the evaluation numbers. Ratings for these factors are computed as shown in table 8. A summary of all rating numbers and the total for each concept are given in table 9.

7. CONCLUSIONS AND RECOMMENDATIONS

Based on the rating scheme that was adopted, the covered corridor concept is clearly superior. In addition, the following conclusions can be drawn from this study.

- The elevator concept is the costliest of the three concepts and would offer the most problems with regard to reliability and maintainability. There are no obvious advantages to the concept to offset these rather serious detractions.

Table 7

CONCEPT COST SUMMARY

Shelter Capacity / Concept	4 Aircraft		6 Aircraft		8 Aircraft	
	Total Cost ($000)	Cost/ Aircraft ($000)	Total Cost ($000)	Cost/ Aircraft ($000)	Total Cost ($000)	Cost/ Aircraft ($000)
Elevator	7,434	1,859	9,088	1,515	10,742	1,343
Open Corridor	5,172	1,293	6,818	1,136	8,463	1,058
Covered Corridor	6,410	1,603	8,056	1,343	9,701	1,213

Table 8

COMPUTATION OF RATING NUMBERS FOR CERTAIN FACTORS

Factor	Concept	Evaluation Number	Rating
Maintainability	Elevator	162	4
	Open Corridor	210	3
	Covered Corridor	30	20
Reliability			
(a) Normal Conditions	Elevator	191	8
	Open Corridor	204	7
	Covered Corridor	100	15
(b) Post-Attack Conditions	Elevator	4 hrs	8
	Open Corridor	4 hrs	8
	Covered Corridor	2 hrs	15
Cost (4 Unit)	Elevator	$7,434,000	70
	Open Corridor	$5,172,000	100
	Covered Corridor	$6,410,000	81

Table 9

SUMMARY OF RATINGS FOR EACH CONCEPT

Evaluation Basis	Elevator Concept	Open Corridor Concept	Covered Corridor Concept	Maximum Possible Rating
Camouflage Potential	10	6	8	10
Emergency Open	5	5	13	15
Emergency Close	5	5	5	10
Repairability				
Normal Conditions	2	4	7	7
Post Attack	6	6	7	8
Maintainability	4	3	20	20
Reliability				
Normal Conditions	8	7	15	15
Post Attack	8	8	15	15
Cost	70	100	81	100
TOTAL	118	144	171	200

- The lowest cost concept is the one with the open corridor. However, disadvantages to this concept are the exposed closures and the uncertainties associated with the reflected pressure on the closures.

- Although the covered corridor concept is not the least expensive, the configuration provides several desirable features. The door experiences the incident overpressure only and, with the minimum opening profile, the door can be relatively light. This, in turn, yields low power requirements and good characteristics with regard to reliability and maintainability. Good radiation shielding is provided except for neutrons; a more detailed investigation of this threat is required.

- Costs associated with excavation and berming are relatively minor. Consequently, the decision concerning the placement of the structure with respect to ground level should be based primarily on operational considerations.

- The overwhelming portion of the total cost of a complex is that attributed to the shelter. The combination of severe dynamic loads and large spans requires very strong sections, and the required amounts of concrete and steel are directly reflected in the shelter cost.

It was recommended that the covered corridor concept be adopted for more detailed preliminary design and analysis. Furthermore, attention during the study should be placed on minimizing the material and erection costs of the shelter itself, since this offers the greatest opportunity for making significant reductions in the estimated cost.

SECTION VII

FEATURES OF SELECTED CONCEPT

1. GENERAL DESCRIPTION

The covered corridor concept with a star pattern layout configuration housing four aircraft was selected as the most promising concept for further development. The general relationship of the various components of the concept without soil cover is illustrated in figure 46. As may be seen, access to the shelter areas is via a covered corridor which is open at both ends. Short entry corridors at the midpoint of the main corridor each lead to two aircraft shelter bays. The basic type of construction is reinforced concrete with an inside steel spall liner. Personnel accessways are provided between the main corridor and the shelter areas.

A recessed vertically operated blast closure seals off the aircraft shelter areas from the open main corridor (figure 47). The entire complex is covered by earth to a minimum depth of 6 feet at the crown of either corridor or shelter and which extends to grade with a maximum slope of 1 on 5.

Although the star layout is the most feasible configuration for incorporation of inside start-up capability, it was decided that such capability would not be included in the conceptual development of the aircraft shelter bays in this study. The additional complexities and expense were considered

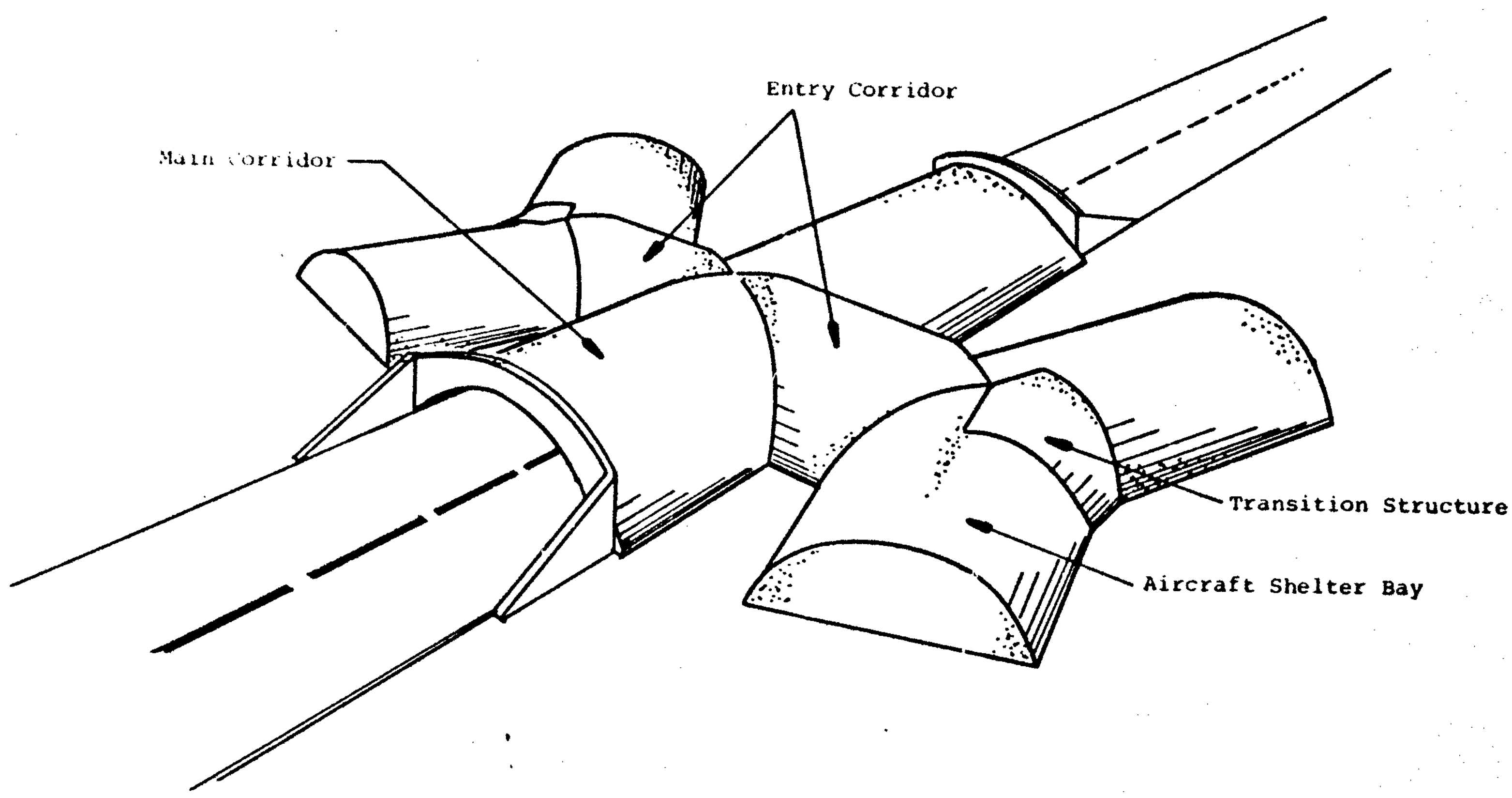


Figure 46. Covered Corridor Concept Components

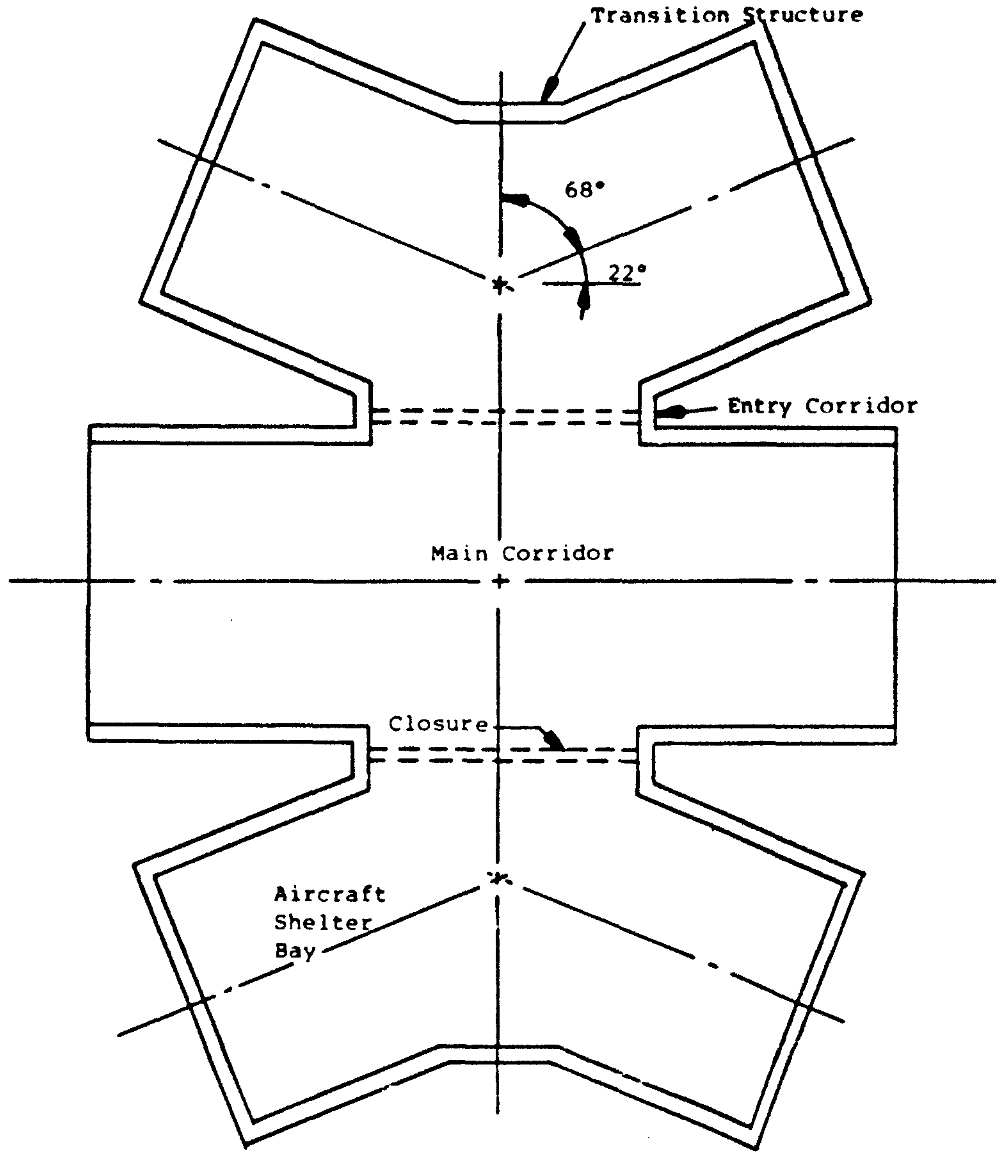


Figure 47. Covered Corridor Concept Foundation Plan

too great in proportion to the benefits gained, particularly in view of the fact that the aircraft can be started in the open main corridor where they would still have a considerable degree of protection.

2. CORRIDORS

The full length of the main corridor at the crown is approximately 238 feet. Retaining walls are provided at the open ends of the corridor to maintain the soil overburden in the required configuration.

A typical main corridor cross section is shown in figure 48. The section is comprised of a cylindrical roof segment and straight skirt sections. The arched roof segment subtends a central angle of 106.26 degrees and has an inside radius of 50 feet. This curved portion of the section transitions into the straight skirts at an angle of 56.13 degrees from the vertical, which corresponds to a height of 10 feet from the reference surface to the inside edge of the section. The inclined skirts make an angle of 56.13 degrees with the horizontal and are at right angles with the base of the footings. Tie beams extend between the footings as shown in figure 48.

The rise from the reference surface, which corresponds to the top outer surface of the footings as shown in figure 48, to the inside of the arch crown is 30 feet. Provisions have been made for an apron slab 12 inches thick which is isolated from the tie beams and shelter walls by 1 foot of backfill. The

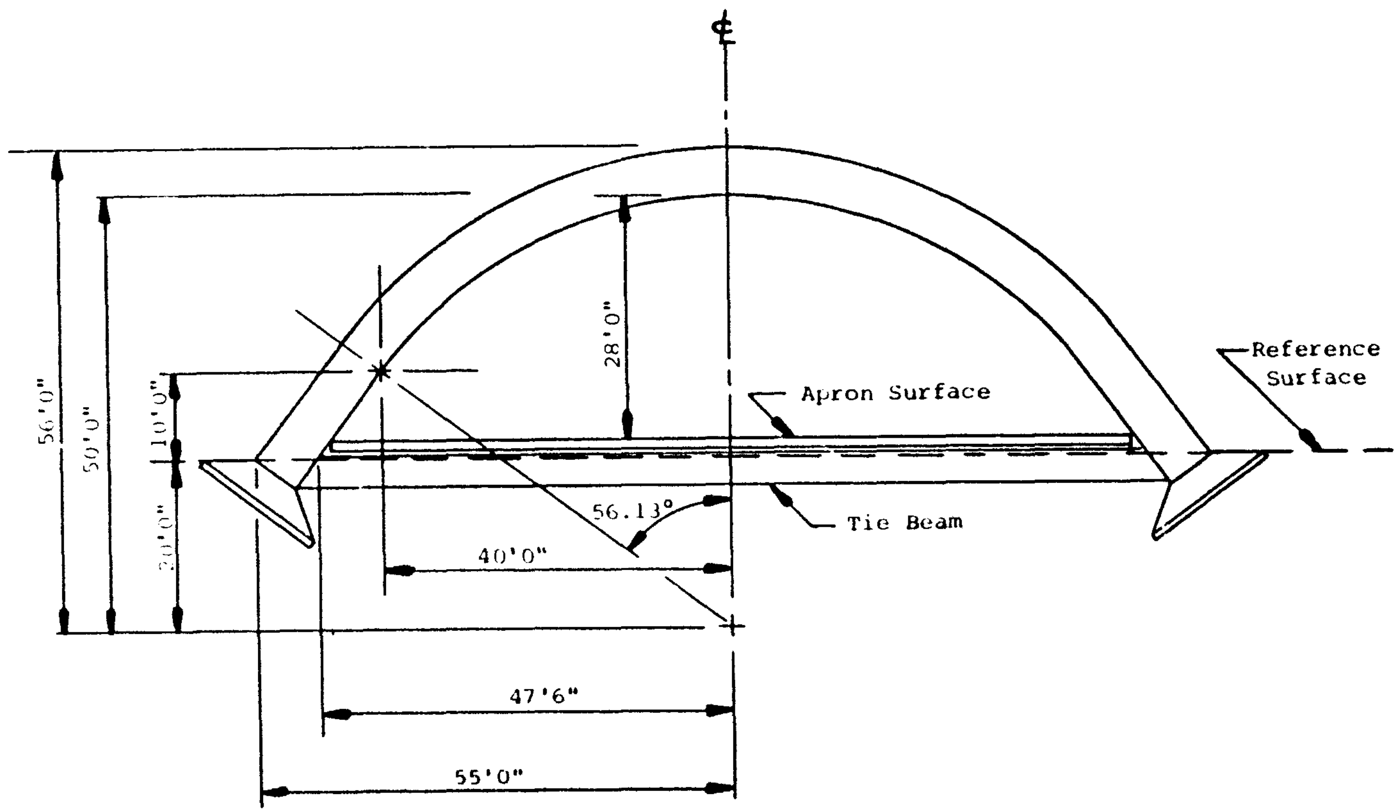


Figure 48. Typical Corridor Cross Section

resulting clear height from the apron surface to the crown is 28 feet.

The entry corridors intersect the main corridor at right angles and have the same typical cross section as the main corridor. The entry corridors contain bulkheads to restrain the closure in its closed position. The general features of these bulkheads are shown in figure 49.

3. AIRCRAFT SHELTER BAYS

The centerlines of the aircraft shelter bays make an angle of 22 degrees with the centerline of the main corridor and 68 degrees with the centerline of the entry corridors. Each bay is nominally 80 feet long (inside) and has the same cross section as the corridors. The transition structure (figure 46) at the end of the entry corridors and the front of the aircraft shelter bays is formed by straight lines connecting the inner and outer surfaces of the opposing bays.

Even though provisions for operating the aircraft engines inside the shelters have not been included in this study, it seems certain that various types of equipment will have to be operated inside the sheltered area. Such equipment could include different types of AGE such as described in appendix A and transport vehicles both for aircraft and materials. Air quality resulting from operating aircraft and AGE inside a TAB VEE shelter with and without ventilation is reported in reference 1. While the shelters considered herein are a great

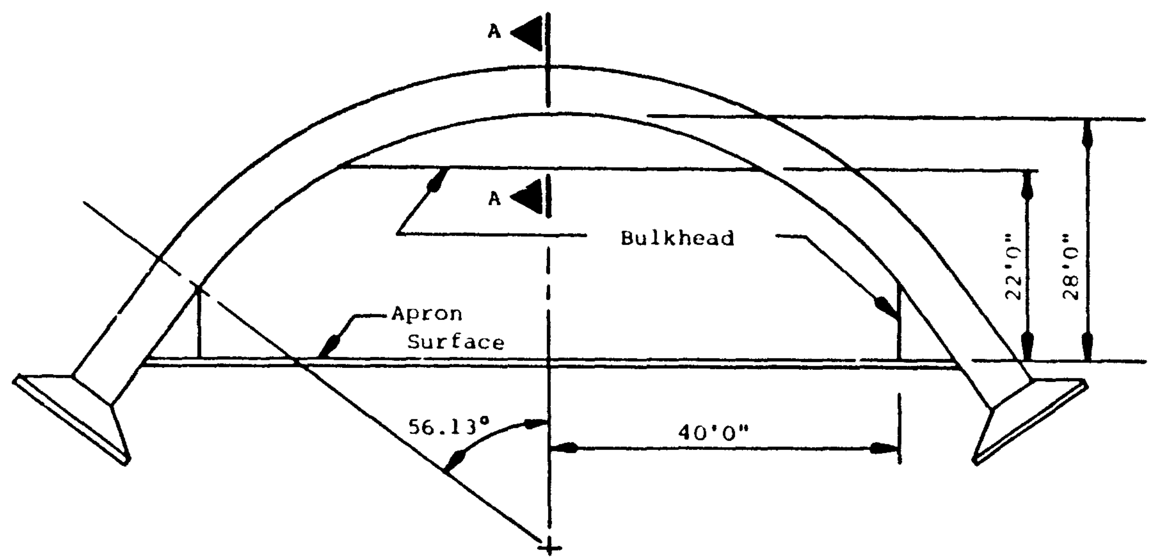
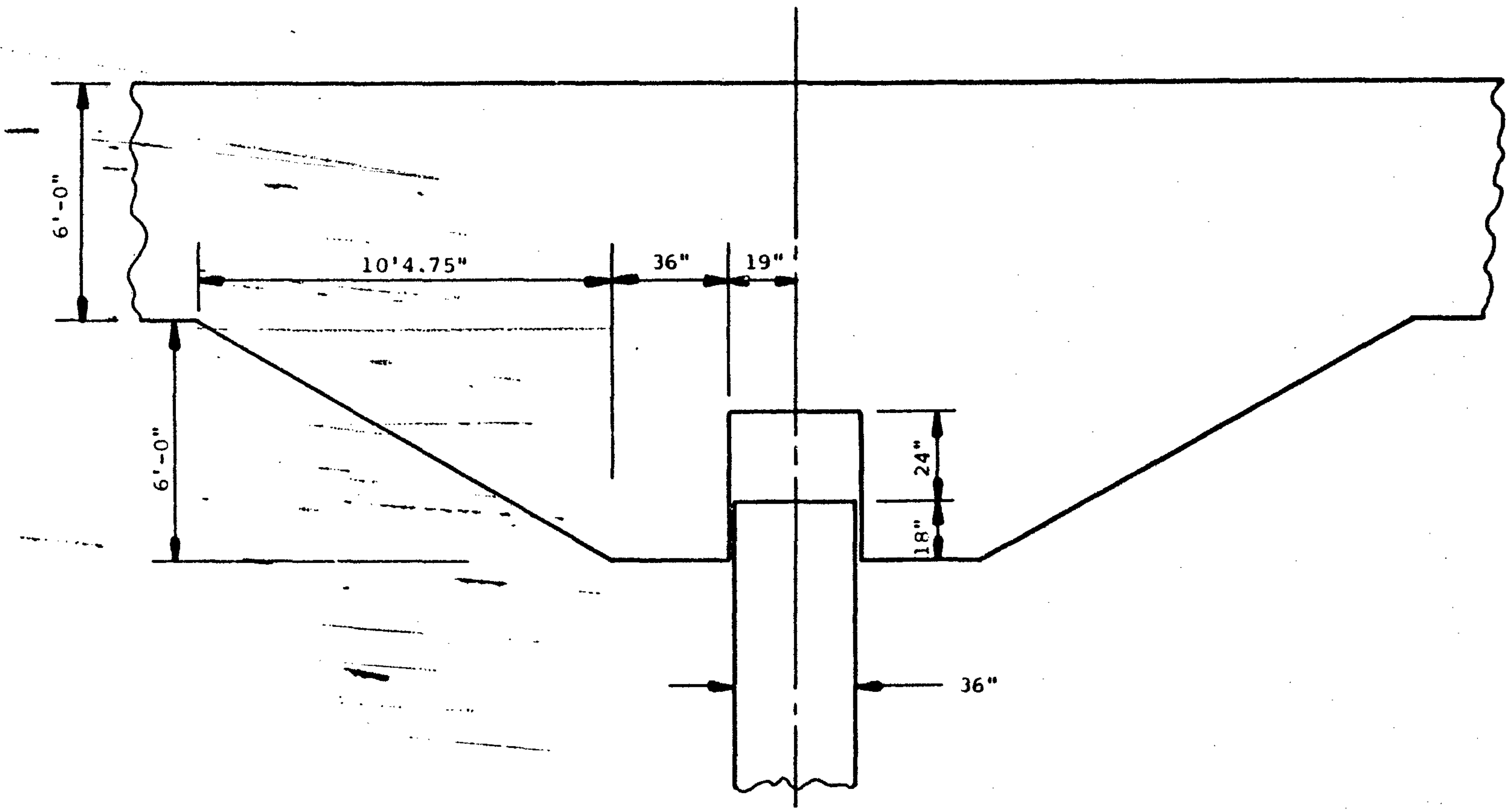


Figure 49. Entry Corridor Bulkheads



Section A-A

Figure 49. Entry Corridor Bulkheads (concluded)

deal larger than a TAB VEE shelter, it appears certain that at least some minimum amount of ventilation will be required. For the purposes of this study and pending more accurate determination of the requirements, it has been assumed that such ventilation as may be necessary can be provided during final engineering design.

4. CLOSURE SYSTEM

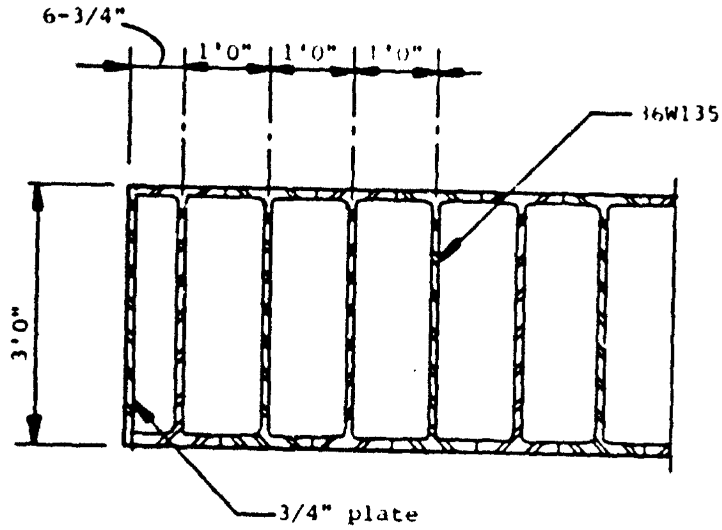
a. Closure

The structural concept selected for the closure differs somewhat from that described in section V. The system of internal stiffeners shown in figure 40 would require extensive and difficult welding operations to fabricate the internal grid and attach the outer cover plates. In addition, stiffener spacing would have to be kept relatively small to prevent plate deflections due to airblast loading from becoming large and causing premature buckling of the closure. Alternatively, the cover plates subjected to airblast loading might be thickened to reduce deflections but at a significant increase in weight and cost. These and other considerations led to the decision to use standard structural wide-flange shapes welded flange-to-flange to form the closure. The webs of the wide-flange shapes are oriented vertically to span the smaller dimension of the closure opening. A steel cover plate is provided around all edges of the closure. The wide-flange sections used in the closure were selected so that the door response would be essentially elastic under the predicted airblast loading.

The overall dimensions of the closure and a typical cross section are shown in figure 50. Closure dimensions were chosen to provide 18 inches of bearing around the perimeter of the closure. The weight of the closure, excluding the lifting mechanism, is about 150 tons.

b. Movement Mechanism

The shafts of the hydraulic cylinders used to raise and lower the closure are located within the closure structure and attached to it near its top edge as shown in figure 51. The cylinders are located in the pit structure into which the closure is lowered. The cylinder assemblies can be removed only through the top of the closure. Support collar assemblies can be unbolted from the closure permitting access to the seals at the top of each cylinder or removal of the entire cylinder. The closure pit provides access to the bottom of the cylinders and removable closure plates are not required. Since the length of the cylinder assembly is greater than the distance between the shelter floor and the overhead arch structure, access holes must be provided in the arch to permit withdrawal or placement of the cylinders from the ground surface. These holes must be covered with an adequate closure to prevent infiltration of surcharge materials and seal against airblast or airblast-induced ground shock. A corrugated metal pipe extending to the overburden surface could be placed over these holes



Section A

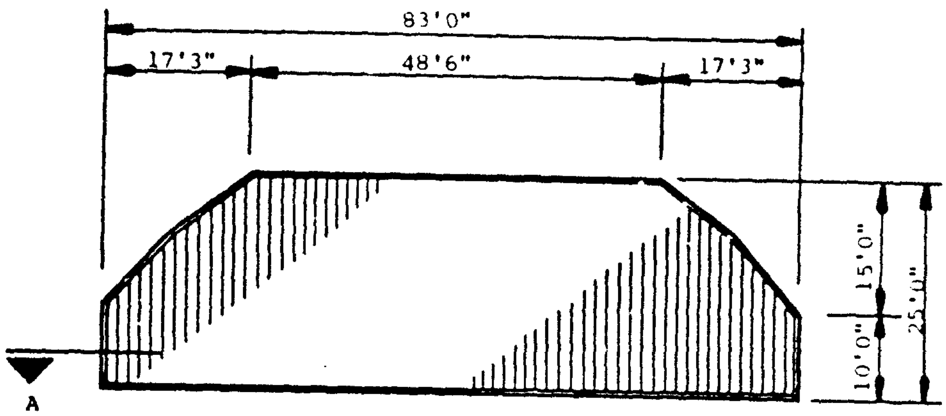


Figure 50. Overall Closure Dimensions

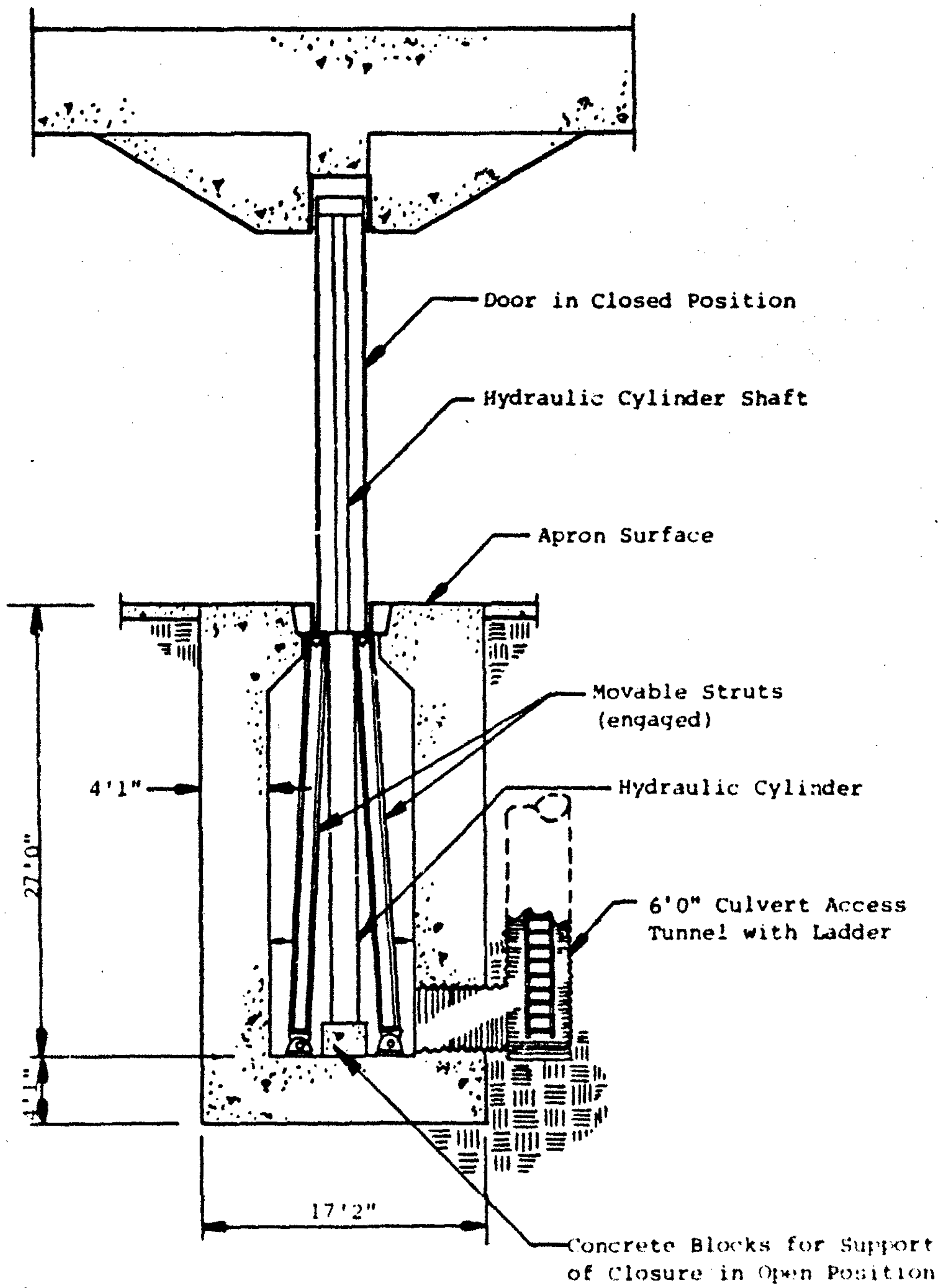


Figure 51. Closure in Closed Position

to facilitate cylinder removal. The pipes should be backfilled with a clean granular material to increase their resistance to airblast and ground shock loads. Once in position, it should not be necessary to remove the hydraulic cylinders. Normal maintenance functions can be performed through access provided at top and bottom of the cylinders.

The hydraulic system for the closure is essentially as described in section V, except that the power requirements are estimated to be 140 to 150 horsepower instead of the 100 horsepower estimated initially. System pressure should not exceed 2000 psi under any anticipated loading conditions. The closure is provided with a movable strut latching system (figure 51) which carries all vertical dead and dynamic loads when the closure is in the closed and latched position. This approach avoids the imposition of large dynamic loads on the hydraulic system.

The closure pit provides space for the closure in the lowered position and houses the hydraulic cylinders and latching struts. It provides maintenance access to the lower ends of the cylinders. Access to the closure pit is by means of a corrugated steel pipe tunnel from the interior of the shelter as shown in figure 51. The closure pit and arch structure are tied together to minimize relative motions between the two elements. This monolithic construction of framing elements around the closure should help avoid impact on the closure under blast loads.

c. Closure Operation

The operation of the closure system is generally similar to that presented in section V. Two 12-inch diameter, single-acting hydraulic cylinders raise and lower the closure. The closure is guided by reinforced concrete structural elements which also provide the required closure support during the airblast loading and rebound phases. When in the open position, the top edge of the door is flush with the floor of the shelter. Because the upper corners of the closure are truncated, there will be an open area in the shelter floor when the closure is in the open position. This opening would be covered with a removable grating or fenced to avoid accidents.

In the open position (figure 52), the bottom edge of the closure rests on concrete supports cast in the floor of the closure pit. These supports are provided to prevent the weight of the closure being applied to the unpressurized hydraulic cylinders.

In the closed position, the closure is supported by a system of latching struts extending from the floor of the closure pit to the bottom of the closure (figure 51). These struts, which pivot about their bases on the floor of the closure pit, are actuated by hydraulic cylinders attached to the walls of the closure pit. When in the latched position, the struts carry all vertical static and dynamic loads.

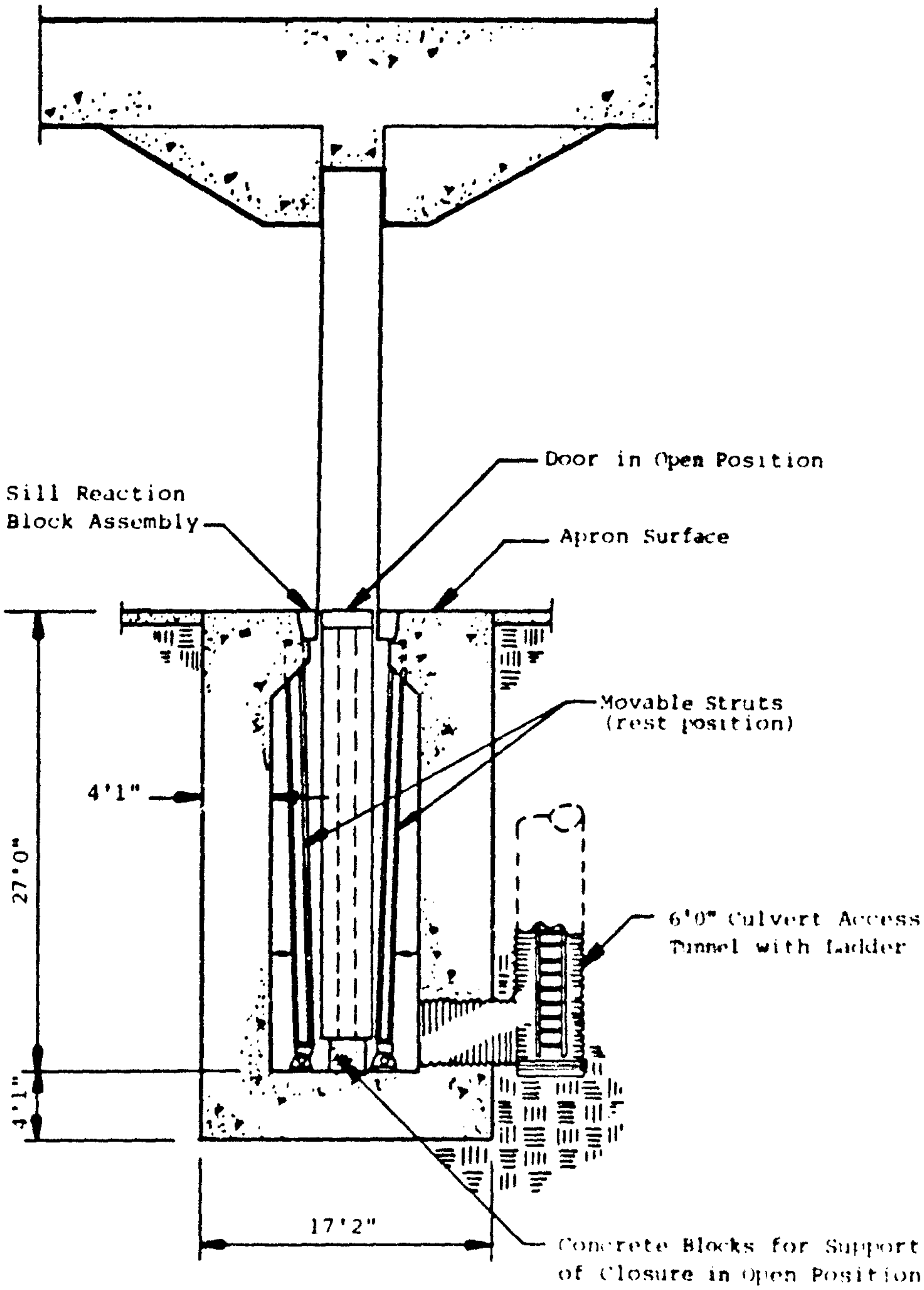


Figure 52. Closure in Open Position

In the unlatched position, the struts are withdrawn into recesses in the haunched upper portion of the closure pit walls to permit lowering of the closure. Latching struts were chosen over the locking devices shown in figures 41 and 43 because of the possibility of permanent deformations of the latter under dynamic load. If such deformations should occur, it might be very difficult to withdraw the locking devices from the closure.

One-inch clearance is provided between each face of the closure and the sill reaction block assemblies. Although the closure is designed to remain elastic under predicted airblast loads, higher pressures might cause permanent deformations and binding of the closure. In this event, the sill reaction block assemblies can be removed to increase the clearances at each face of the closure to 6 inches.

Hydraulic system controls should be placed in a protected enclosure in the access corridor on both sides of the closure so that it can be opened from either side. Emergency, no-power opening of the closure could be readily accomplished by controlled release of fluid from the cylinders, if the latching struts are not in position beneath the closure. If the latching struts are in position, the closure must be raised approximately 1 inch before the struts can be withdrawn. The cylinders actuating the latching struts are small (2-inch diameter) and operate at low pressures (less than 500 psi). A small accumulator system could be used to provide the necessary pressure fluid for emergency no-power operation of the latching struts. Cylinder pressures on the order of 1500 psi are required to lift the closure, but, if the cylinders are kept filled with fluid, only small quantities of fluid would be required to raise the closure 1 inch and relieve loading on the struts. Small accumulators could easily satisfy this requirement.

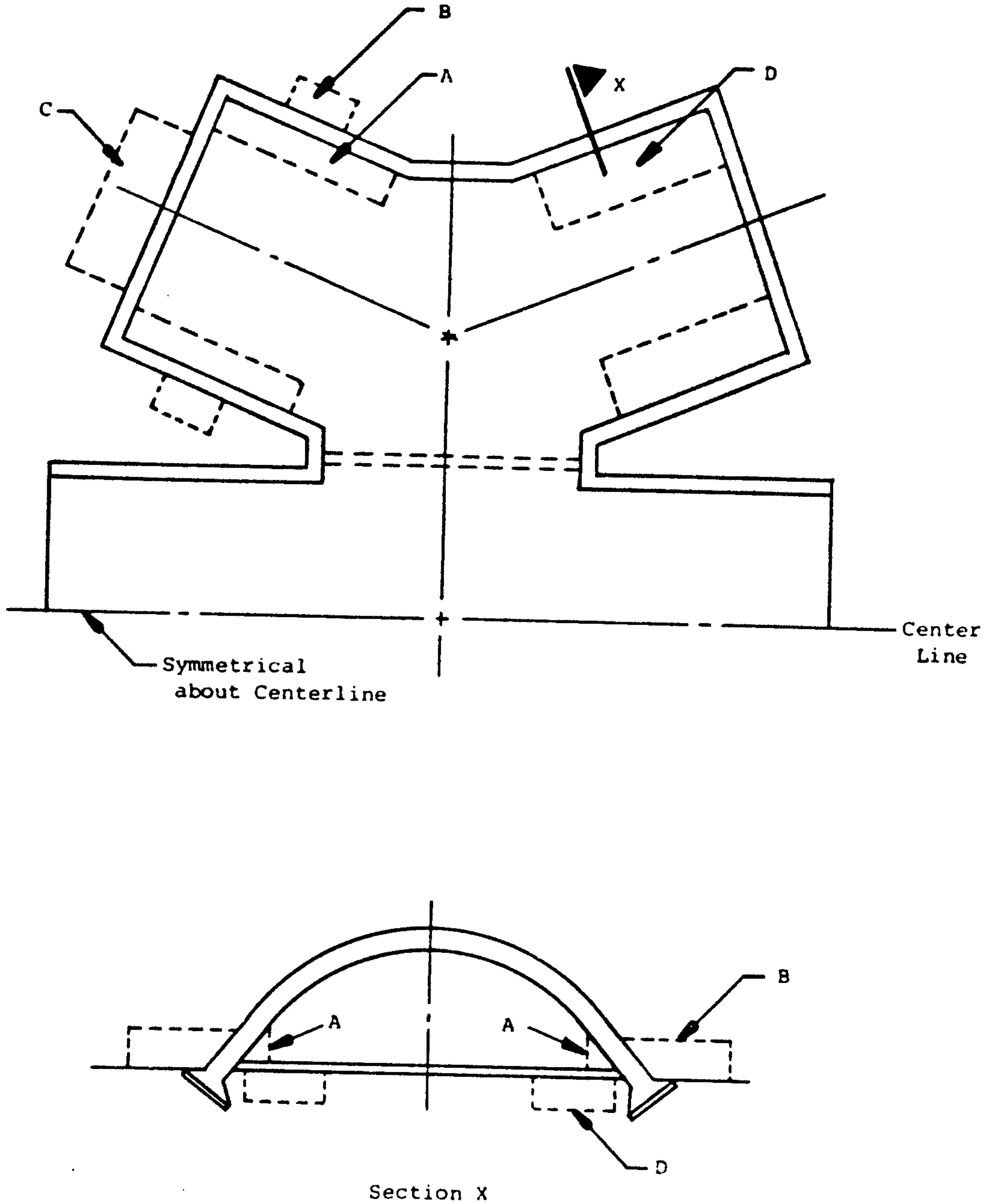
Although accumulators could also provide a capability for emergency, no-power closing of the closure, the higher pressures and larger volumes of fluid required would make these devices costly. A more practical solution would be to provide emergency or back-up electrical power to operate the hydraulic system. Alternatively, diesel engine driven hydraulic pumps could be used as the prime or secondary energy source. A protected plug-in receptacle in the access corridor could supply emergency electrical power to the hydraulic system from portable generators.

A curtain-type lightweight steel overhead door is suggested for normal day-to-day operations and weather protection. It would be much cheaper to operate and would prevent unnecessary wear of major closure system components. If such a door were installed, it should be positioned on the protected inner side of the closure to prevent its becoming debris which might block egress from the shelter after an attack.

5. AUXILIARY STORAGE AREAS

In any facility as large and complex as considered herein, there are several regions where auxiliary storage areas can be located. Some of the more accessible and desirable locations are discussed below. Although the requirement for storage areas, e.g., roughly 400 square feet for AGE, is readily apparent, firm operational criteria for such space were not available during the course of this study. Storage areas are, therefore, treated conceptually in general terms rather than developed in detail and included in the preliminary design.

The most readily available space for an auxiliary storage area for small items is along the sidewalls of the shelters as illustrated in figure 53, storage area A. Previous examination of aircraft clearance requirements has shown that 70 feet clear is adequate for a drive-through area, and 76 feet is required for a turning aircraft (figures 10 and 11). The clear span dimension of the shelter is determined from the geometry of the section at the level of the apron surface. This dimension turns out to be about 92 feet. Therefore, there will be up to 11 feet of space available on each side of the shelter in the area where aircraft are stored and not expected to negotiate turns. This space can be blocked off from the central portion of the shelter by a light duty divider or left as open space using lines painted on the apron surface to indicate storage limits for equipment. Note, however, that the shelter wall is not vertical where it meets



A. Along shelter sidewalls
B. Outside shelter sidewalls
C. Outside shelter endwall
D. Below apron floor

Figure 53. Potential Locations of Auxiliary Storage Areas

the apron surface but slopes inward with a rise of four units for a run of three units. This sloping wall will greatly reduce the usefulness of the outer portion of this storage since only very small items can be placed there; accessibility will also be limited.

Storage areas can also be located outside the main sheltered area by constructing auxiliary structures. The floor surface in these locations can be at the same level as the apron surface. An opening on the order of 7 feet square will allow access to the storage area by the largest AGE and also by other light duty vehicles. A likely location for this type of storage area is on either side of the main sheltered areas. The berm cover depth is sufficient to provide a large degree of protection. Additionally, if located between a shelter and the main corridor, the protection provided by these structures will be substantial. Additional access could be provided from the covered corridor side. A drawback is presented by the sloping lower portion of the shelter, since this must be penetrated to create an entryway. The sloped wall could present problems in constructing a suitable closure and, also, the loss of material in the opening could reduce the strength of the shelter section to an unacceptable level.

Alternatively, the storage areas can be positioned next to the rear wall or bulkhead (designated C in figure 53) where the strength requirements of the shelter structure are not as severe and the wall is vertical. Adequate depth of cover can also be readily provided at these locations.

Storage locations indicated by D in figure 53 are located below the apron floor inside the shelter footings. The width of the storage area will be limited by the width of apron required in the center of the shelter. Storage areas would not be located under the main traffic path of the aircraft. The widest landing gear is 17.92 feet. If the region below the center 40 feet of the shelter is preserved as the traffic lane for aircraft, there will be about (92 - 40)/2 or 26 feet available along each side of the shelter for storage areas below the apron. The length can be varied up to the point where the storage areas are as long as the shelter segment. Depth can be as much as desired, but the vertical walls should be proportioned for the same loads as the shelter closure pit. Access to these areas would have to be by ladder, stairway, elevator, ramp, or hoist.

Areas designated A in figure 53 are clearly preferable in terms of economy. The areas D afford the most protection, but also have the most awkward access. Areas C appear to have the advantage of simpler construction and ease of access in the event it was determined that additional storage space outside the shelters proper was required. Areas B are almost equal to the C areas in terms of feasibility, except for the complications arising from penetrating the sloping shelter walls and the potentially severe effects of loss of arch wall strength due to the penetrations necessary for access to these storage areas.

6. SHELTER CONSTRUCTION

With the exception of the closure and shelter elements framing the closure, there are no unusual restraints on construction operations. The size and weight of the closure and the way it is framed by shelter elements indicate a preferred construction sequence for these items. It is recommended that the closure pits be constructed first. The next operation should be the placement of the closures in the pits. After the closures are in position, the construction of overhead shelter elements can proceed without further restraints. Construction of these overhead elements before the closure is in place will greatly complicate construction operations. A completely assembled closure cannot be set in place once the reinforced concrete elements surrounding it are completed.

Although partial fabrication of the closure might be accomplished off-site, its size and weight will probably require final assembly on-site. If access holes are provided in the shelter roof, the hydraulic cylinders for operating the closure can be placed during any construction phase.

SECTION VIII

DESIGN OF SELECTED CONCEPT

1. INTRODUCTION

Preliminary design drawings of the covered corridor concept are contained in appendix B. Major components of the concept were typically designed independently of each other. This section presents a brief summary of the design criteria and the simplified design calculations. More refined analyses accomplished to verify the adequacy of the design are described in section IX.

2. DESIGN CRITERIA

a. Operational Requirements

All shelter dimensions were determined on the basis of providing adequate clearance for the following aircraft.

- A-7, all versions
- A-10
- F-4, all versions
- F-15
- F-100
- F-101
- F-105
- F-111 with fully extended wings

At least 1 meter horizontal and 0.5 meter vertical clearance from the top of all parts of the aircraft is required. It is not necessary, however, to provide space for turn-around of the aircraft inside the shelter.

The straight line taxiing clearance requirements are summarized in figure 10. The turning aircraft clearance requirements are summarized in figure 11. A maximum slope of 2.5 percent is specified for all ramps upon which aircraft will be towed or taxied.

Aircraft start-up capability within the main shelters is not a requirement.

The mechanical system for operating the closure shall be capable of completing an opening or closing cycle within a period of 3 minutes.

b. Protection Requirements

The weapon threat spectrum which the shelter must survive is specified in a classified attachment to the statement of work for contract number F29601-75-C-0128. The following paragraphs briefly summarize the most severe aspects of the environment resulting from the specified weapon threat.

(1) Cratering and Crater Ejecta

The depth of cratering and ejecta debris may be as great as 1 foot and could, therefore, affect aircraft taxiing. Provisions should be made to remove this debris from taxiways and the access corridor.

(2) Penetration and Breaching

Six feet of reinforced concrete is required to prevent penetration or breaching by the specified weapon threats.

(3) Airblast Loading

Airblast parameters of interest for structural loading are primarily those for weapon no. 4. A peak overpressure of 250 psi was specified and the corresponding peak dynamic pressure is 430 psi. The effective durations of each type of pressure were determined in accordance with procedures outlined in reference 2. Variations of peak reflected pressure and drag coefficient with angle of incidence are as shown in figure 12.

(4) Ground Shock

Peak vertical soil stresses are equal to the incident overpressure, i.e., 250 psi, and are assumed to attenuate with depth in accordance with the procedures outlined in reference 2. Peak vertical and horizontal displacements are predicted to be 1.90 feet. Peak vertical and horizontal free field velocities and accelerations are predicted to be 5.75 fps and 250 g, respectively. These parameters are also modified with depth according to procedures described in reference 2.

(5) Nuclear Radiation

The free field radiation environment is estimated not to exceed the following levels

Initial Gamma	- 5.62×10^5 r
Neutrons	- 3.1×10^{15} n/cm^2
X-radiation	- 612 cal/cm^2
Thermal	- 2192 cal/cm^2
EMP	- 50,000 volts/meter

c. Material Properties

All structural steel shapes and plate in the closure are of ASTM A572 high strength low-alloy structural steel. The latching struts are Grade 50 and all other elements are Grade 60. The tie beams in the foundations are ASTM A36 steel.

All structural concrete should have a minimum 28-day compressive strength of 4000 psi. With one exception, Grade 40 reinforcing steel is specified for all reinforced concrete members. Grade 60 reinforcing steel is specified for longitudinal reinforcing in some shelter foundations.

Wherever appropriate a 10 percent increase in yield strength is assumed for structural steel elements subjected to dynamic loads. Concrete elements are assumed to have a 20 percent increase in yield strength when subjected to dynamic loads.

Although a static bearing capacity of 0.5 tsf was initially specified for design of arch foundations, the large dead loads made it necessary to increase this to 3 tsf and specify a select subgrade material. In the dynamic analysis of arch response the horizontal and vertical stiffnesses (moduli) of the foundation materials were assumed to be 200 pci. The ratio of horizontal to vertical soil pressures was assumed to be 0.5 (K_o = 0.5) for determining horizontal loads on the closure pit walls.

With some exceptions as to allowable stresses under dynamic loads, all structural elements are designed in accordance with provisions of the American Institute of Steel Construction Specifications for the Design, Fabrication and Erection of Structural Steel for Buildings, The American Concrete

Institute Building Code Requirements for Reinforced Concrete (ACI 318-71) and American Society for Testing and Materials (ASTM) specifications.

d. Ventilation Requirements

Although the design of a ventilation system for the shelters is not a part of this study, some criteria important to the design of a system should be noted. The required capacity can be determined from consideration of the types of equipment and numbers of personnel occupying the shelter. This phase of the design can be accomplished using the standardized criteria and procedures used for any ventilation system. Additional requirements include the protection of exhaust and intake ducts from airblast and airblast induced ground shock. Openings must be designed to prevent the infiltration of radioactive particles and airblast or degradation of the radiation shielding effectiveness of the structure. The shock tolerance of mechanical equipment must be evaluated to determine the need for shock isolation of components. Structure penetrations must be designed so as to accommodate relative motions at the soil-structure interface. Protection from chemical/biological warfare agents must also be incorporated into the ventilation system design.

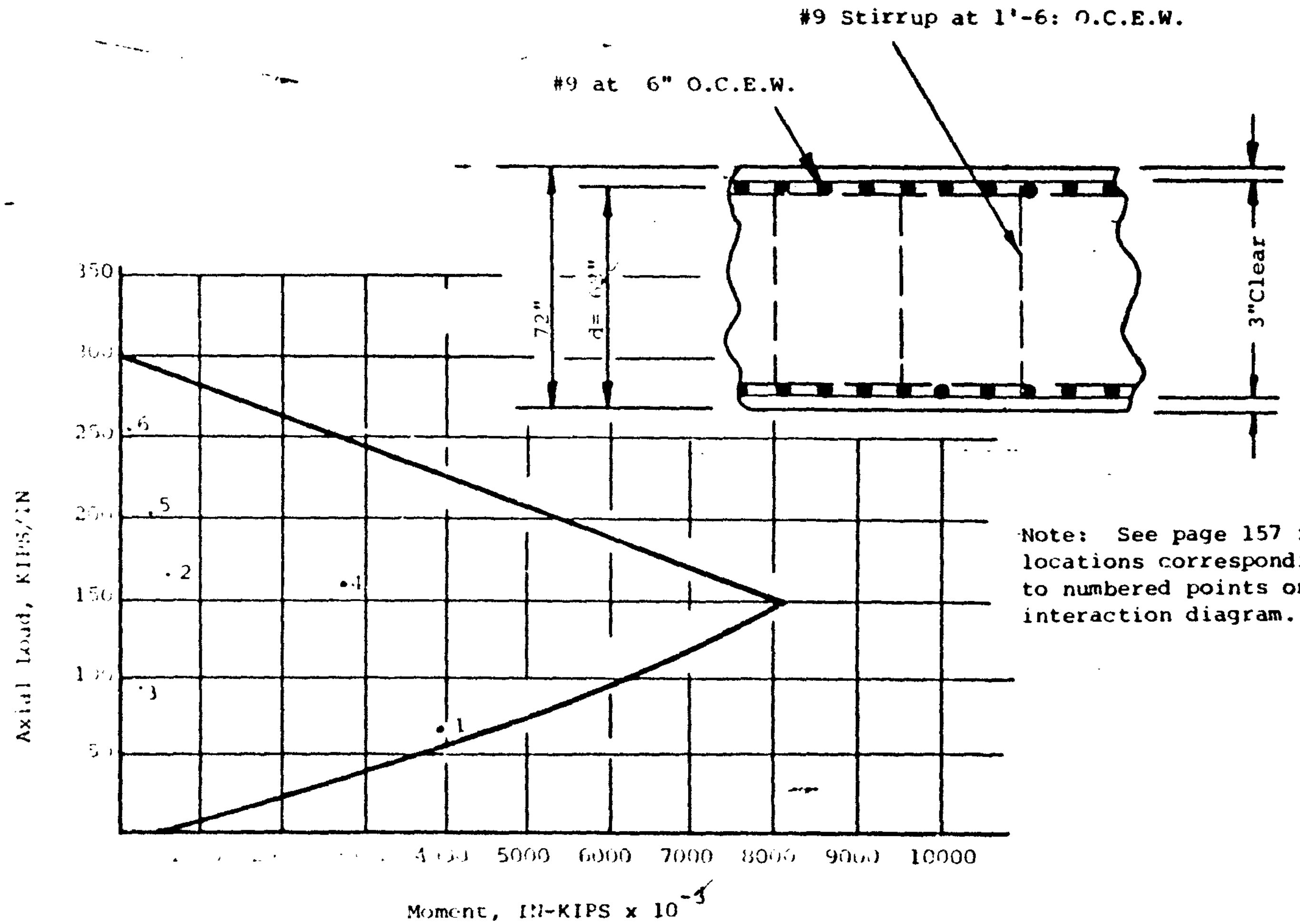
e. Mechanical/Hydraulic Systems

The hydraulic systems required for operation of the closure should be designed using standard criteria and procedures. In order to minimize system costs and maintenance, it is desirable that the system be designed to operate at pressures less than 2000 psi.

3. MAIN CORRIDOR/SHELTER CROSS SECTION

A trial section was selected on the basis of NASTRAN analyses of a unit strip of the cross section. Early preliminary studies had indicated that an elliptical shape would be more efficient from a materials viewpoint than a circular section. These studies utilized an applied roof pressure that varied sinusoidally from the crown. As the investigations continued, it became clear that the predominant factor determining the cost and structural efficiency of the section was the eccentricity and angle that the resultant forces on the roof made with the middle surface of the roof. A reduction of the eccentricity utilized the concrete more efficiently, and a reduction of the angle greatly reduced the stirrup requirements. These findings led to selection of the shape shown in figure 48 rather than the elliptical shape. A cross section through the arch shell is shown in figure 54. The outermost layers of #9 bars run in the circumferential direction. An interaction diagram for the section is also shown in figure 54 and was developed using procedures described in reference 2.

The two dimensional model of the shelter used for the NASTRAN analysis represents a transverse strip 1 inch wide normal to the centerline of the entry corridor. The model consisted of 33 nodes and 32 bar elements. An elastic bar element representing the tie-beam connected the two base nodes of the model. A spring constant of 5000 lb/in was assumed for



Note: See page 157 for locations corresponding to numbered points on interaction diagram.

Figure 54. Interaction Diagram for Arch Cross Section

that element as an approximation of the flexural stiffness of the shelter cross section.

The soil at the base of the structure was modeled with nonlinear springs. The springs approximated the soil resistance in directions parallel and normal to the base of the foundation. The soil resistance parallel to the base of the foundation was assumed to be a constant 90 psi and acted in a direction opposite to the motion of the foundation. The soil resistance normal to the base of the foundation was represented by a linear spring with a stiffness of 200 psi/in. This spring force acted normal to the base of the foundation.

The two-dimensional model was dynamically loaded by an airblast pressure wave traveling across the structure from one springing line to the other. The airblast pressure was approximated by a triangular pulse having a peak overpressure of 250 psi and a duration appropriate for the weapon yields considered. The loads induced by the airblast pressure were applied vertically to the structure.

Results of the NASTRAN analysis of the two-dimensional model revealed that the maximum bending moment at the crown was 3,890,000 in-lb/in. This was a positive bending moment producing compression in the outer surface of the arch shell. At the time of maximum bending, the thrust at the crown was a compressive force of 67,300 lb/in. The maximum thrust occurring at the crown during the positive phase was a compressive force of 160,000 lb/in. The maximum total vertical deflection occurred at the crown and was equal to 9.36 inches.

At the top of the upstream foundation, the maximum bending moment was 588,100 in-lb/in. Occurring simultaneously was a compressive force of 166,330 lb/in. The maximum thrust occurring at this point during the loading phase was a compressive force of 205,200 lb/in. The maximum vertical deflection of the upstream springing line was 8.7 inches. At the downstream foundation, the maximum bending moment was 246,510 in-lb/in. Occurring simultaneously was a compressive thrust of 95,230 lb/in. The maximum thrust at this point was 254,000 lb/in and the maximum vertical deflection was 8.36 inches.

The maximum force occurring in the tie-beam was 6,090 pounds of tension.

The most severe combinations of bending moments and thrust occurring at the crown and springing lines are plotted on figure 54. Point 1 represents the combination of maximum bending moment at the crown and the simultaneous thrust. Points 2 and 3 represent similar combinations for the upstream and downstream springing lines, respectively. Point 4 represents the combination of maximum thrust at the crown and the simultaneous bending moment. Points 5 and 6 represent similar combinations for the upstream and downstream springing lines. All points fall within the limits of the interaction diagram for the trial arch section.

4. SHELTER/CORRIDOR FOUNDATION

The foundation that supports the shelter wall was designed in general accord with the procedures described in reference 6. The angle of internal friction in the foundation soil was assumed to be zero degrees. The foundation subgrade was not required to withstand the maximum thrust occurring in the arch shell during passage of the blast wave, because its short duration makes its use in any simplified foundation design questionable. If used with any reasonable value of static soil bearing capacity, extremely large foundations would result. The foundation width was, therefore, determined on the basis of estimated dead loads. A NASTRAN analysis determined that the vertical dead load at the top of the foundation was 4800 lb/in. Adding 1000 lb/in as the estimated weight of the foundation resulted in a total vertical dead load on the soil of 5800 lb/in. Other foundation structural details were based upon an equivalent static vertical load of 250 psi. This 250 psi vertical load resulted in a foundation load of 150,000 lb/in.

As noted earlier, the large dead load on the foundations made it necessary to assume that a select compacted backfill with a bearing capacity higher than the initially specified value of 1000 psf would be provided. Using a 16-foot wide footing inclined at 36.87 degrees (see figure 55) would require a soil bearing capacity of

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6. Site Hardening for Aerodynamic Weapon Systems, Volume II - Design Compendium, WADD TR 60-219, Aeronautical Systems Center, Wright-Patterson AFB, Ohio, August 1970. (U)

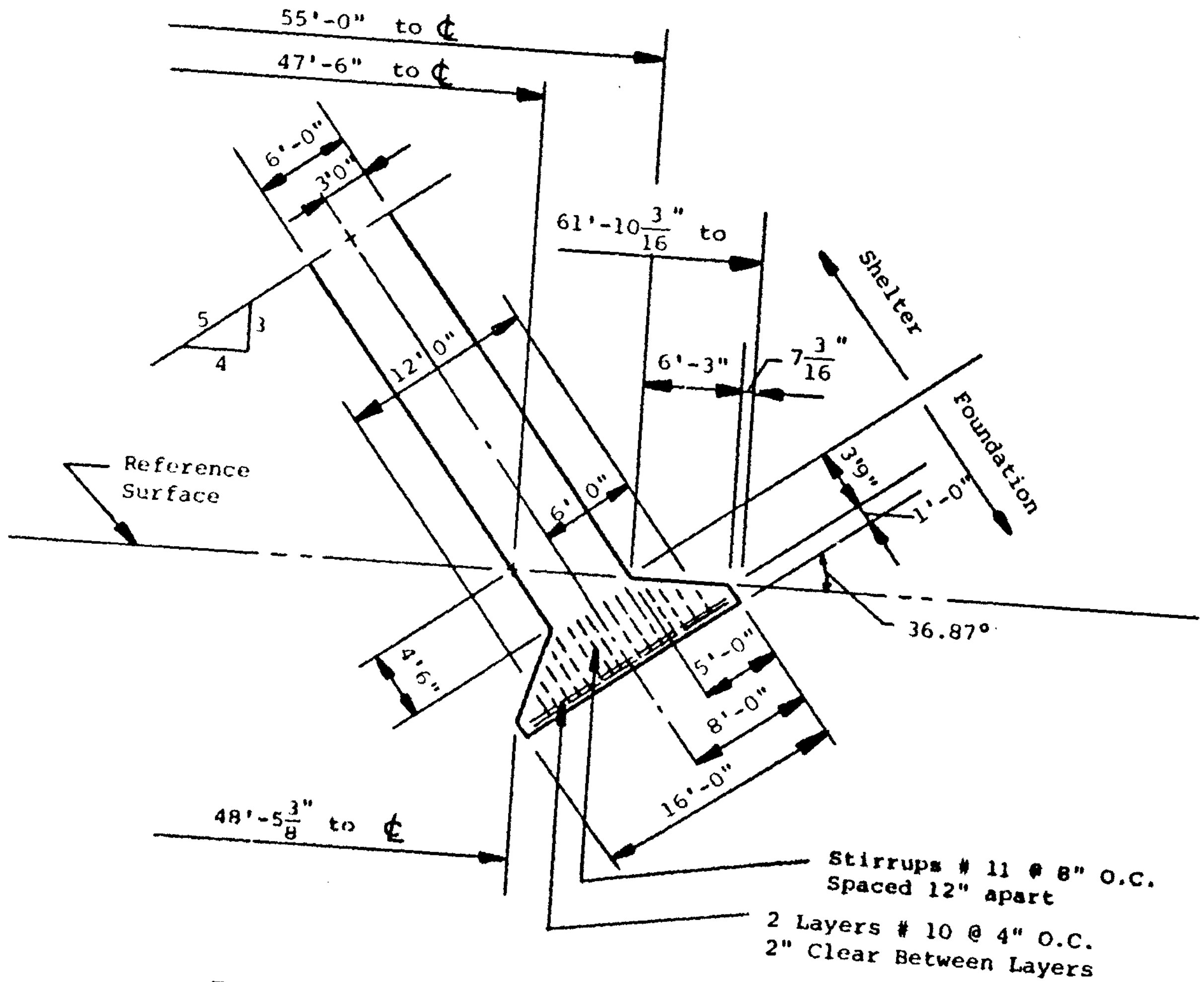


Figure 55. Foundation Cross Section

$$f_{soil} = \frac{5800(12)}{16\cos 36.87°}$$

$$= 5437 \text{ psf}$$

Thus a subgrade with a static bearing capacity of 6000 psf is satisfactory.

The transverse reinforcing details were based upon foundation loads resulting from the 250 psi vertical airblast loading. The longitudinal steel details are dependent on the results of the NASTRAN analysis of the quarter section of the shelter complex. The assumed foundation loading of 150,000 lb/in from airblast is equivalent to a pressure of about 780 psi on the base of the foundation. The moment at a section through the base of the overhang of the foundation is

$$M = \frac{w\ell^2}{2}$$

$$= \frac{780(60)^2}{2}$$

$$= 1{,}404{,}000 \text{ in-lb/in}$$

Using an effective depth of 52.37 inches and a reinforcing steel ratio of 0.0125, the resisting moment per inch is

$$M_r = 0.9pf_{dy}d^2$$

$$= 0.9(0.0125)(44{,}000)(52.37)^2$$

$$= 1{,}358{,}000 \text{ in-lb/in}$$

Two layers of #9 bars at 3 inches on center would provide a steel ratio of 0.01274.

The revised moment capacity is

$$M_r = 0.9(0.01274)(44{,}000)(52.37)^2$$

$$= 1{,}384{,}000 \text{ in-lb/in}$$

This is close enough for a preliminary design. Use two layers of #9 bars 3 inches on center with 3 inches clear to the first layer and 1 inch clear between layers.

Next check the shear at a distance d/2 from the base of the overhang. At this distance the cantilevered length is

$$\ell = 60 - \frac{52.37}{2}$$

$$= 33.8 \text{ in}$$

The total shear at this location is

$$V = w\ell = 780(33.8)$$

$$= 26{,}360 \text{ lb/in}$$

The depth of the section at this point is

$$d = \frac{33.8(45)}{60} + 7.4 = 32.72 \text{ inches}$$

Then the shearing stress in the concrete is

$$v = \frac{V}{d} = \frac{26{,}360 \text{ lb/in}}{32.72 \text{ in}}$$

$$= 805.6 \text{ psi}$$

The moment at the section under investigation is

$$M = \frac{w\ell^2}{2} = \frac{780(33.8)^2}{2}$$

$$= 445{,}500 \text{ in-lb/in}$$

The allowable shear stress in the concrete at this section will be

$$v_c = 1.9\sqrt{f'_{dc}} + 2500p\,\frac{Vd}{M}$$

$$= 132 \text{ psi} + 2500(0.01274)(32.72)\left(\frac{26,360}{445,550}\right)$$

$$= 164 \text{ psi}$$

Note that Vd/M cannot exceed 1.0 in the equation for allowable shear stress in the concrete.

The excess shear must be carried by stirrups. The area required is given by

$$A_v = (v_u - v_c)\,\frac{bs}{f_y}$$

Using

b = 12 inches   s = 8 inches

$$A_v = (805.6 - 164)\,\frac{(8)(12)}{44000}$$

$$= 1.4 \text{ in}^2$$

Use #11 bars (A_s = 1.56 in^2) 8 inches on center spaced 12 inches apart.

The longitudinal steel requirements are determined in section IX which presents the results of the NASTRAN analysis of a quarter-section of the shelter complex.

5. TIE BEAMS

The arch shown in figure 48 was initially analyzed with a flat footing tied near the reference surface. The soil under the footing was assumed to provide only a vertical reaction. The tie beam forces resulting from this analysis would have required steel members with minimum yield strengths of 100,000 psi and weights of 370 lb/ft spaced on 10 foot centers. These tie beams alone would have cost approximately $1,600,000 per shelter complex. The stiffness represented by these tie beams would have allowed relative horizontal displacements of 4.20 inches between the footings. The dynamic response of the shelter with the flat footing was a complex mixing of two independent modes of vibration. One was due to stretching of the tie beams, and the other was a result of the shelter being pushed vertically into the soil. The variation of moment and thrust in the roof section of the arch was such that very large moments were occurring without the benefit of thrust loads. Very large shear forces were also acting on some sections.

The results of the initial analysis influenced the decision to use an inclined base footing. The inclined base of the footing produced a horizontal component of soil reaction which is a function of the vertical component. The magnitude of the overpressure acting on the shelter causes very large vertical and horizontal reactions. The required tie beam forces were reduced as the horizontal component of soil reaction increased.

The NASTRAN analysis of the unit strip of the shelter cross section utilized the inclined footing and a tie beam spring constant of 5000 lb/in/in. The maximum relative horizontal displacement between the footings for these conditions was 1.22 inches. The maximum stress in the tie beams was lowered to 28,650 psi, and the use of mild steel sections became practical. The required area of tie beams with a 10 foot spacing and 102.5 foot length is

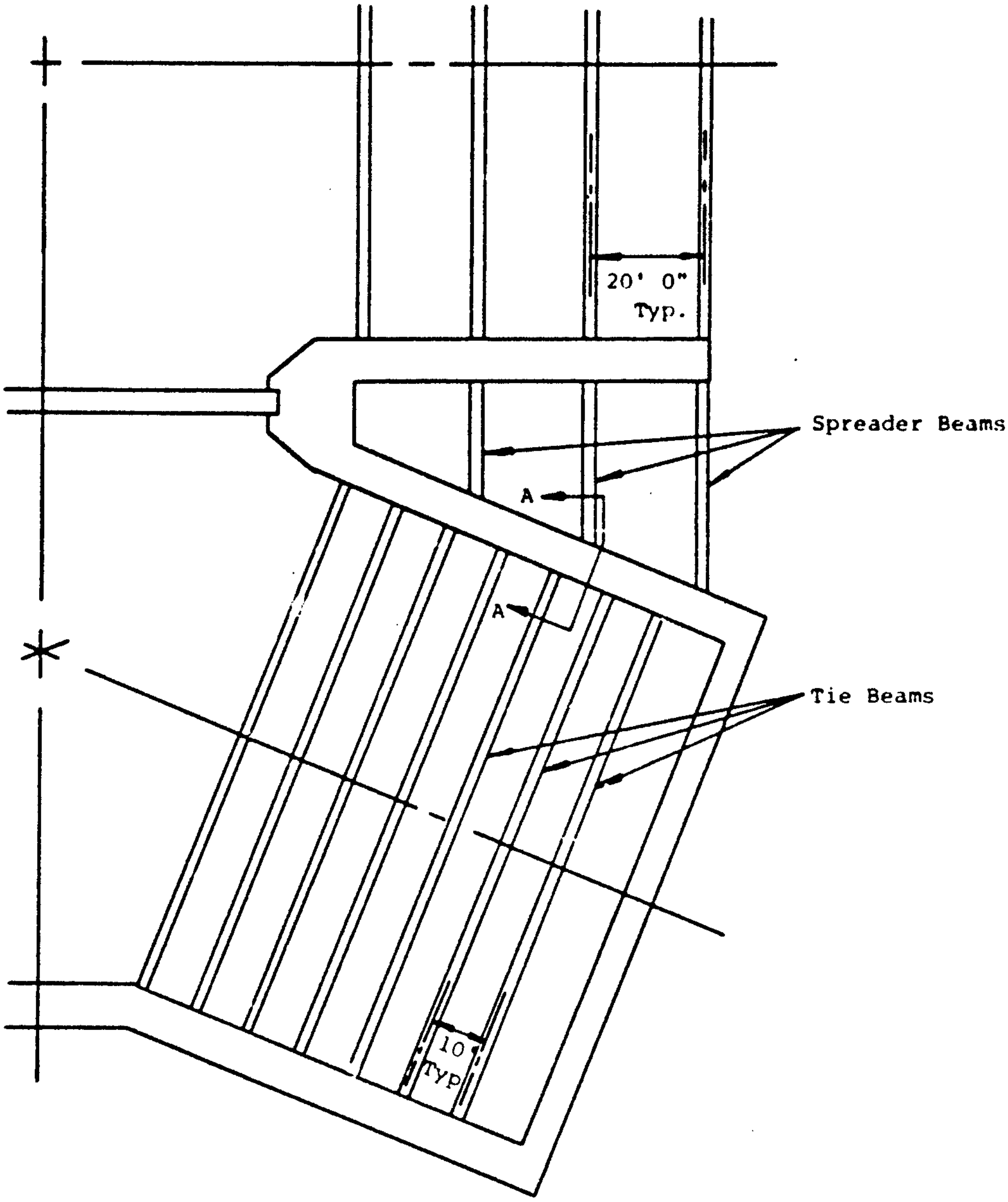
$$A = \frac{k\ell b}{E}$$

$$= \frac{5000(1230)(120)}{29{,}000{,}000}$$

$$= 25.45 \text{ in}^2$$

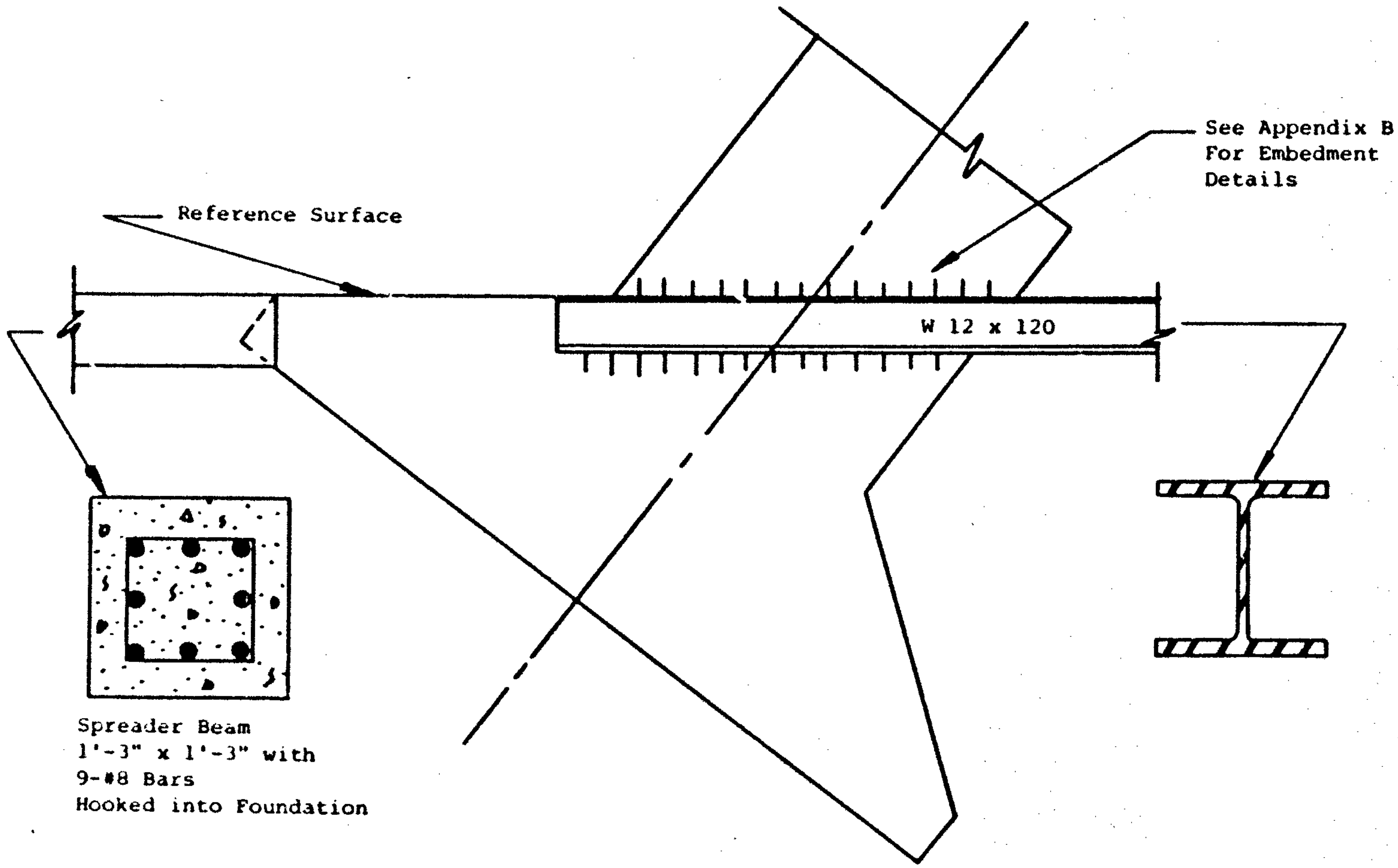
A W12 x 120 section with a cross section area of 35.3 in^2 was selected for the tie beams. With the inclined footing and this tie beam section, the stress resultants in the roof are within acceptable limits The shear forces acting on the arch walls were small enough so that only minimal shear reinforcing was required.

Six tie beams with a spacing of 10 feet are used in the shelter bays. Four tie beams with a spacing of 20 feet are used in the main corridor. Reinforced concrete spreader beams placed between the corridor and shelter foundations provide the remaining required capacity for the main corridor. These beams were designed to provide the same ultimate force capacity as the steel section, however, they will receive loads as a result of relative displacement between the corridor and shelter foundations. The spreader beams are also spaced 20 feet apart. Beam locations and typical cross sections are shown in figure 56.



a. Plan View

Figure 56. Tie Beam and Spreader Beam Details



b. Typical Cross Sections

Figure 56. Tie Beam and Spreader Beam Details (concluded)

6. SHELTER ENDWALLS

The shelter endwalls were designed as one-way vertical slabs. Because of the haunch in the endwall foundation cross section (see appendix B), the maximum span of the wall was taken as 30 feet. The top edge of the endwall is made monolithic with the shelter wall, and the bottom edge is braced by diagonal columns placed below the tie beams. A 1 inch wide strip of the endwall was analyzed as a propped cantilever (see figure 57). From reference 2, the natural period of the propped cantilever is

$$T_n = 2\pi\sqrt{\frac{K_{LM}M_t}{k}}$$

Assuming that the response of the endwall is more elastic than plastic, use a load-mass factor of

$$K_{LM} = 0.78$$

The mass of the strip is

$$M_t = \frac{(1)(72)(360)(150)}{(1728)(386)}$$

$$= 5.83$$

and the effective spring constant is

$$k = 160\,\frac{EI}{L^3}$$

For 4000 psi concrete

$$E = 3.6 \times 10^6 \text{ psi}$$

The moment of inertia of a compressed concrete section with an effective depth of 68 inches, in which cracking is inhibited, is

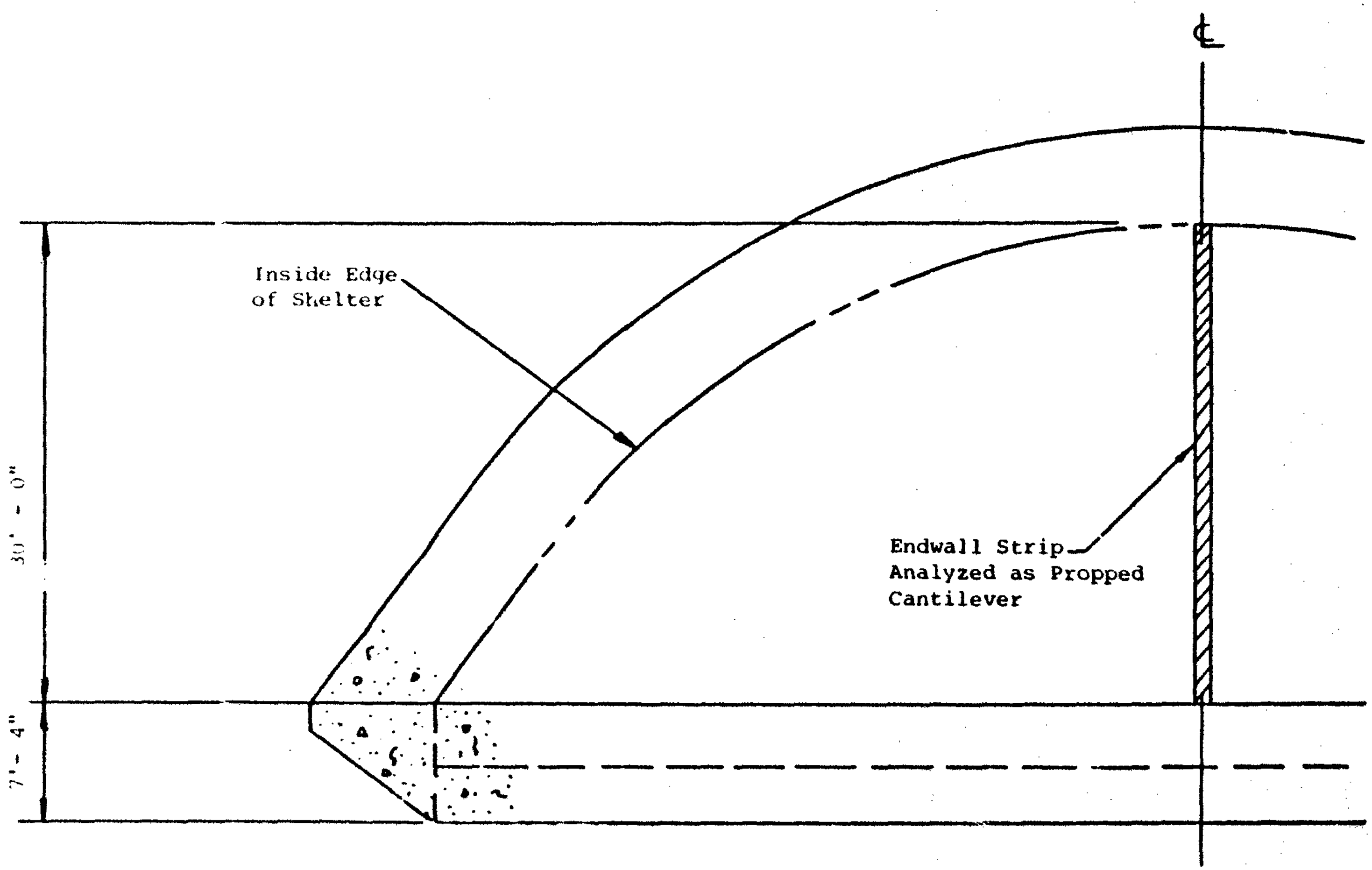


Figure 57. Endwall Vertical Strip

$$I = \frac{(1)(68)^3}{12}$$

$$= 26{,}202 \text{ in}^4/\text{in}$$

and the period of vibration is

$$T_n = 6.28\sqrt{\frac{0.78(5.83)(360)^3}{160(3.6 \times 10^6)(26202)}}$$

$$= 0.0235 \text{ sec}$$

For the most severe weapon threat

$$t_d = 0.1287$$

It follows that

$$t_d/T_n = 5.48$$

For elastic response, reference 2 gives the ratio of maximum required resistance to peak load as

$$\frac{R_m}{F_1} \text{ (req'd)} = 2.0$$

Using a horizontal to vertical soil stress ratio of 0.5 and a vertical pressure of 250 psi results in

$$R_m\text{(req'd)} = 250 \text{ psi}$$

In order to determine the maximum resistance provided, an account was made of the stresses acting vertically in the endwall. The endwall top surfaces are loaded by 250 psi vertical soil pressure. The endwall also acts as a support for a portion of the shelter roof. In the NASTRAN analysis of the quarter section of the shelter no load was applied to the endwall. Thus, the calculated compressive stresses in the endwall, in the plane of the endwall, are those due to the blast pressure loading on the roof. Figure 58 is a depiction of these compressive stresses at two times in the loading history. In the area of interest the compressive stresses are at least 300 psi. The maximum moment capacity of the vertical endwall strip can be obtained from the interaction diagram of figure 54. The thrust load is 39,600 lb/in for a 72-inch deep strip loaded with 550 psi compressive stress. The maximum moment capacity is

$$M_p = 2.9 \times 10^6 \text{ in-lb/in}$$

Reference 2 gives the maximum resistance of a propped cantilever with equal moment capacities at its fixed end and midspan as

$$R_m(\text{provided}) = \frac{12M_p}{L^2}$$

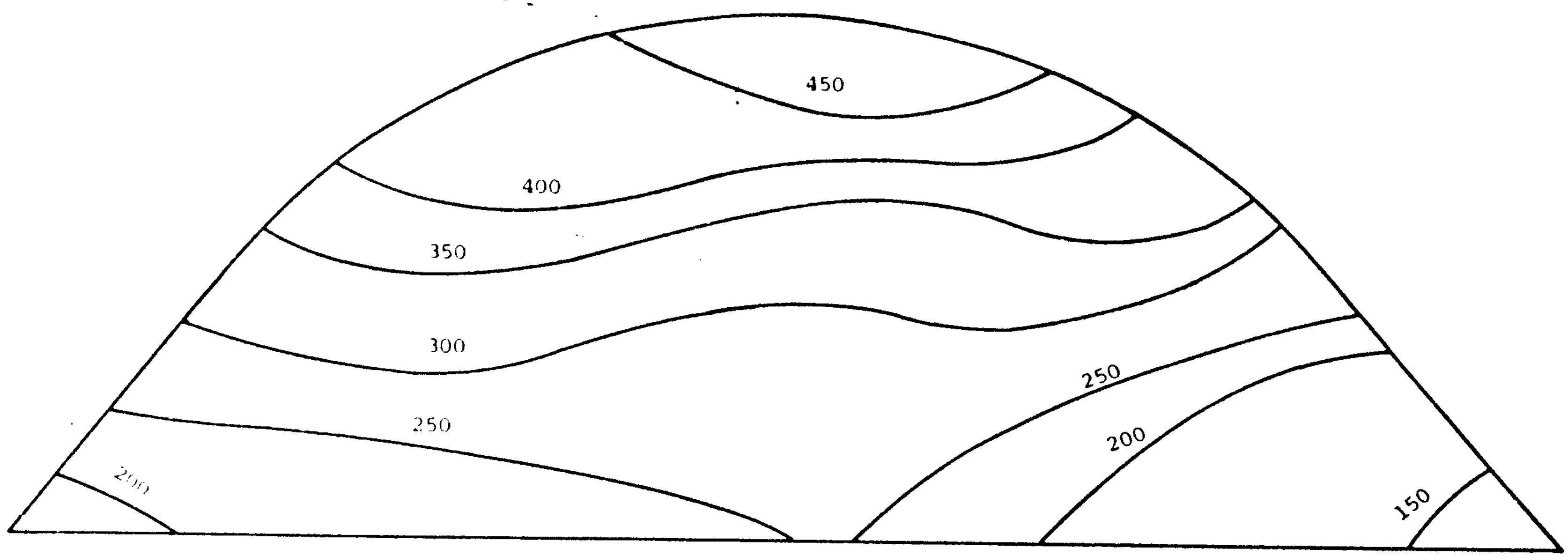
Thus

$$R_m(\text{provided}) = \frac{12(2.9 \times 10^6)}{(360)^2}$$

$$= 268.5 \text{ psi}$$

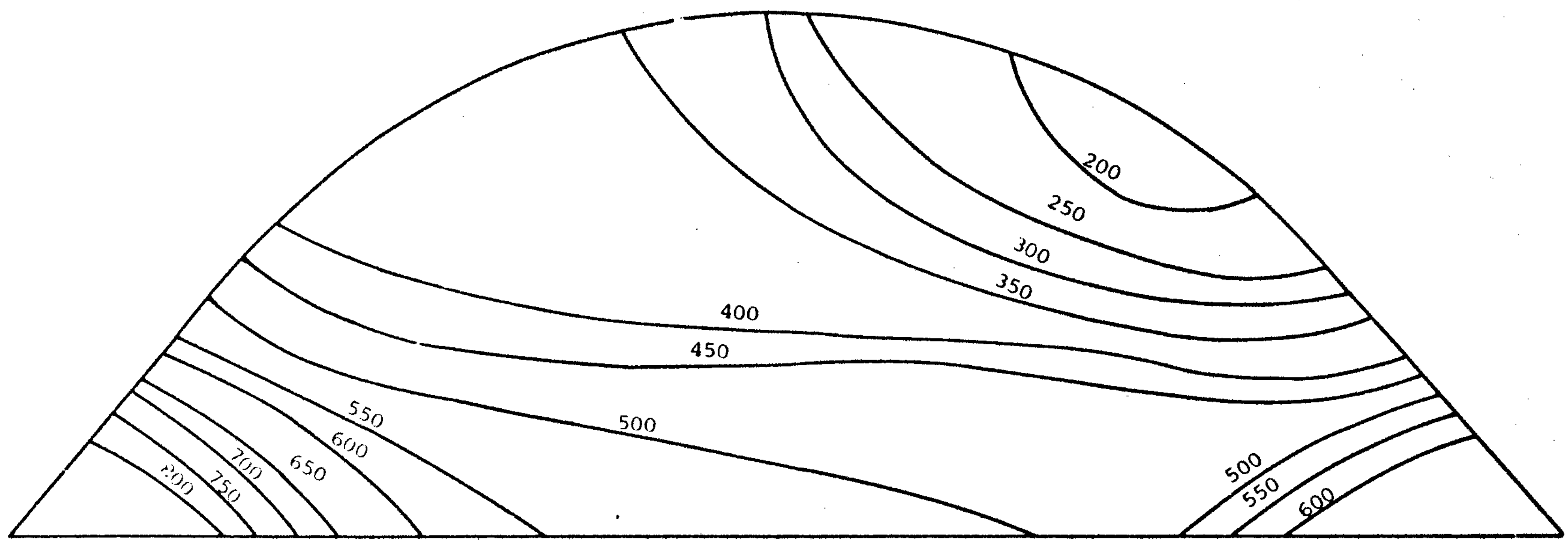
Since the resistance provided is greater than the required resistance the endwall design is adequate.

Use #9 bars at a 6-inch spacing for vertical reinforcing. Assuming that the endwall will behave in a fashion similar to that of a plate with an aspect ratio of 2 to 1, it is reasonable to double this spacing for the horizontal steel, i.e., #9 bars at a 12-inch spacing.



a. Endwall Stresses at t = 0.020 sec.

Figure 58. Compressive Stresses in the Wall



b. Endwall Stresses at t = 0.060 sec.

Figure 58. Compressive Stresses in the Wall (concluded)

Shear reinforcing requirements were also based on the assumption that the response is primarily elastic. From reference 2, the reaction at the top edge of the wall is

$$V_1 = 0.43R_mL + 0.19F_1L$$

$$= 0.43(250)(360) + 0.19(125)(360)$$

$$= 47,250 \text{ lbs/in}$$

The reaction at the bottom edge is

$$V_2 = 0.26R_mL + 0.12F_1L$$

$$= 31,950 \text{ lbs/in}$$

The equivalent distributed load is then

$$w = \frac{47,250 + 31,950}{360}$$

$$= 220 \text{ psi}$$

The shear forces at a distance d from the supports are

$$V_1 = 47,250 - 220(68)$$

$$= 32,290 \text{ lbs/in}$$

$$V_2 = 31,950 - 220(68)$$

$$= 16,990 \text{ lbs/in}$$

The corresponding shear stresses

$$v_{1_u} = 475 \text{ psi}$$

$$v_{2_u} = 250 \text{ psi}$$

Neglecting the effect of compressive stresses and any gain from deep beam action, the shear stress apportioned to the concrete was taken as

$$v_c = 2\sqrt{f'_{dc}}$$

$$= 139 \text{ psi, say } 140 \text{ psi}$$

Near the top edge of the wall the required area of shear reinforcing is

$$A_{v_1} = (v_{1_u} - v_c) \frac{b_s}{f_{dy}}$$

Using a 6-inch by 12-inch spacing of the reinforcing

$$A_{v_1} = \frac{(475 - 140)(6)(12)}{44{,}000}$$

$$= 0.55 \text{ in}^2$$

Place a #7 bar at each intersection of the horizontal and vertical reinforcing.

Using a 12-inch vertical and 18-inch horizontal spacing near the lower edge the required area of reinforcing is

$$A_{v_2} = \frac{110(18)(12)}{44{,}000}$$

$$= 0.54 \text{ in}^2$$

Use #7 at a 12-inch vertical and 18-inch horizontal spacing. The diagonal braces were designed for a lower edge reaction load of 31,950 lbs/in and a brace spacing of 162.88 inches. The required axial load capacity is

$$P = 31{,}950(162.88) \sec 45°$$

$$= 7{,}359{,}364 \text{ lbs}$$

Use a section 36 inches wide and 44 inches deep with twenty #11 bars. Its axial capacity will be

$$P_u = 0.85f'_{dc}(A_g - A_s) + A_s f_{dy}$$

$$= 0.85(4800)(36)(44) - 31.20 + 31.20(44{,}000)$$

$$= 7{,}708{,}224 \text{ lbs}$$

7. CLOSURE

a. Closure Structure

The closure opening is 80 feet wide by 22 feet high. The closure is 83 feet wide by 25 feet high with the two upper corners truncated to fit within the shelter arch. The overall dimensions of the closure provide a bearing surface 18 inches wide along all supported edges. A beam element spanning the 22-foot vertical dimension of the access corridor opening was taken as a reasonable representation of the total closure structure. Although there will be local variations because of its location in the access corridor, the exterior surface of the closure was assumed to be subjected to a maximum airblast load equal to the peak overpressure, 250 psi. The transit time of the shock front across the 80-foot wide closure opening is taken as the width divided by the shock front velocity,

$$t_r = L/U$$

$$= \frac{80}{4400} = 0.018 \text{ sec}$$

This is assumed to be the rise time of the total load on the closure to its peak value. Assuming a Dynamic Load Factor (DLF) equal to 1.1, and that the entire closure is subjected to 250 psi, the maximum moment in the simply supported beam element is

$$M = \frac{wL^2}{8} = \frac{1.1(250)(22 \times 12)^2}{8}$$

$$= 2,395,800 \text{ in-lb/in}$$

Assuming a 10 percent increase in the static yield strength of the steel due to dynamic loading,

$$f_{dy} = 1.1f_y = 1.1(60{,}000)$$

$$= 66{,}000 \text{ psi}$$

A W36 x 135 has a section modulus, S, of 440 in^3 and an elastic moment capacity of

$$M = f_{dy}S = (66{,}000)(440)$$

$$= 29{,}040{,}000 \text{ in-lb}$$

If these members are placed 12 inches on center, the above translates into

$$M = \frac{29{,}040{,}000}{12}$$

$$= 2{,}420{,}000 \text{ in-lb/in}$$

and the section provides the required bending resistance.

Other properties of the W36 x 135 are as follows.

Depth,	d	$= 35.55$ in
Web thickness,	t_w	$= 0.598$ in
Weight/inch		$= 11.25$ lb/in
Mass/inch,	ρ	$= 0.0291$ lb-sec^2/in^2
Moment of inertia,	I	$= 7820$ in^4

From reference 2, the fundamental frequency of vibration of the beam element is

$$\omega_N = \frac{9.87}{L^2}\sqrt{\frac{EI}{\rho}} = \frac{9.87}{(22 \times 12)^2}\sqrt{\frac{(29 \times 10^6)(7820)}{0.0291}}$$

$$= 395 \text{ rad/sec}$$

and the period of vibration is

$$T_N = \frac{2\pi}{\omega_N} = \frac{6.28}{395}$$

$$= 0.016 \text{ sec}$$

For $t_r/T_N = 1.125$ and a ductility ratio, μ, equal to 1, figure 9-11 of reference 2 gives $R_m/F_o = 1.1$. Thus, the initial assumption of a DLF of 1.1 is satisfactory.

The end reaction at each end of the vertical span is

$$R = \frac{wL}{2} = \frac{1.1(12)(250)(22 \times 12)}{2}$$

$$= 435{,}600 \text{ lb for 12-inch beam spacing}$$

The shear stress in the web of the W36 x 135 is

$$v = \frac{R}{dt_w} = \frac{435{,}600}{35.55(0.598)}$$

$$= 20{,}490 \text{ psi}$$

Reference 2 suggests an allowable shear stress of

$$v_{allow} = 0.55f_{dy} = 0.55(66{,}000)$$

$$= 36{,}300 \text{ psi}$$

and the section is satisfactory for shear capacity.

The W36 x 135 section should also be checked for web crippling. An 18-inch bearing is provided at each support, and the AISC Specifications suggest that in order to avoid web crippling at end reactions,

$$\frac{R}{t_w(N + k)} \leq 0.75f_{dy} \text{ psi}$$

where R is the reaction, N is the length of the bearing surface, and k is the distance from the outer face of the flange to the web toe of fillet for the shape of interest. For the calculated end reaction and W36 x 135 properties,

$$\frac{435{,}600}{0.598(18 + 1.69)} \leq 0.75(66{,}000) \text{ psi}$$

$$36{,}995 \text{ psi} < 49{,}500 \text{ psi}$$

and the section is safe against web crippling at the supports.

The flanges of adjacent sections should be connected by full length, full depth welds to provide a maximum degree of continuity in the transverse direction. At the locations of the two hydraulic cylinders, the beam spacing will have to be increased to 24 inches in order to provide space for the cylinders within the closure structure when it is in the open position. See appendix B for details. The increased beam spacing at these two locations should not significantly degrade the closure's resistance to airblast.

b. Hydraulic Cylinders

The weight of the closure is approximately 135 lb/ft^2 of surface area. The surface area is about

$$A_1 = (83 \times 25) - (17.37 \times 15)$$
$$= 1815 \text{ ft}^2$$

and the weight of the structural shapes is

$$W_1 = 135(1815)$$
$$= 245{,}025 \text{ lbs}$$

The length of edge plates is

$$L = 48.25 + 83 + 2(10) + 2(23)$$

$$= 197.25 \text{ ft}$$

For 0.75-inch plate, the added weight is

$$W_2 = \frac{(197.25 \times 12)(36)(0.75)(490)}{1728}$$

$$= 18,120 \text{ lbs}$$

The total weight of the door, less the cylinder shafts, is

$$W_T = W_1 + W_2 = 245,025 + 18,120$$

$$= 263,145 \text{ lbs}$$

or 131,573 lbs/cylinder. A design load of 200,000 lbs/cylinder was assumed, and Miller Fluid Power Corporation Bulletin No. 4908-474 was used to obtain all cylinder dimensions and capacities.

It is desirable to keep operating pressures below 2000 psi to minimize leakage and to avoid use of special purpose fittings. A 12-inch diameter cylinder could be operated at a pressure of

$$p_{op} = \frac{P}{A} = \frac{200,000}{3.14(6)^2}$$

$$= 1770 \text{ psi}$$

The cylinder stroke must be the sum of the opening height + 18-inch bearing at edges + 3-inch reserve stroke. This gives a total required stroke of 285 inches. The overall length of the cylinder is the sum of the stroke and the thickness of the cylinder end plates. For the Miller Model H66-B 12-inch diameter hydraulic cylinder, the thickness of the end plates and seals is about 20 inches. The overall cylinder length is, therefore, 285 + 20 = 305 inches.

As shown in figure 59, the shaft collar assembly at the top of the cylinder is 15.25 inches deep. Adding another 3.75 inches to provide for grouting of the cylinder base plate and clearance between the top of the cylinder and the bottom of the shaft collar assembly when the closure is in the open position, the required closure pit depth is

$$305 + 15.25 + 3.75 = 324 \text{ inches} = 27 \text{ feet}$$

The required cylinder shaft diameter depends on the applied load and the effective length of the shaft. Since the closure is guided throughout its travel, it can be assumed that the upper end is a pin connection prevented from translating in any lateral direction. Anchor bolts in the base plate of the cylinder would fix it against lateral translation and provide some unknown degree of fixity against rotation. Assuming the worst condition of no resistance to rotation at the cylinder base, the effective length of the cylinder shaft assembly is equal to the sum of the cylinder length and stroke, i.e.,

$$L = 305 + 285 \approx 590, \text{ say } 600 \text{ in}$$

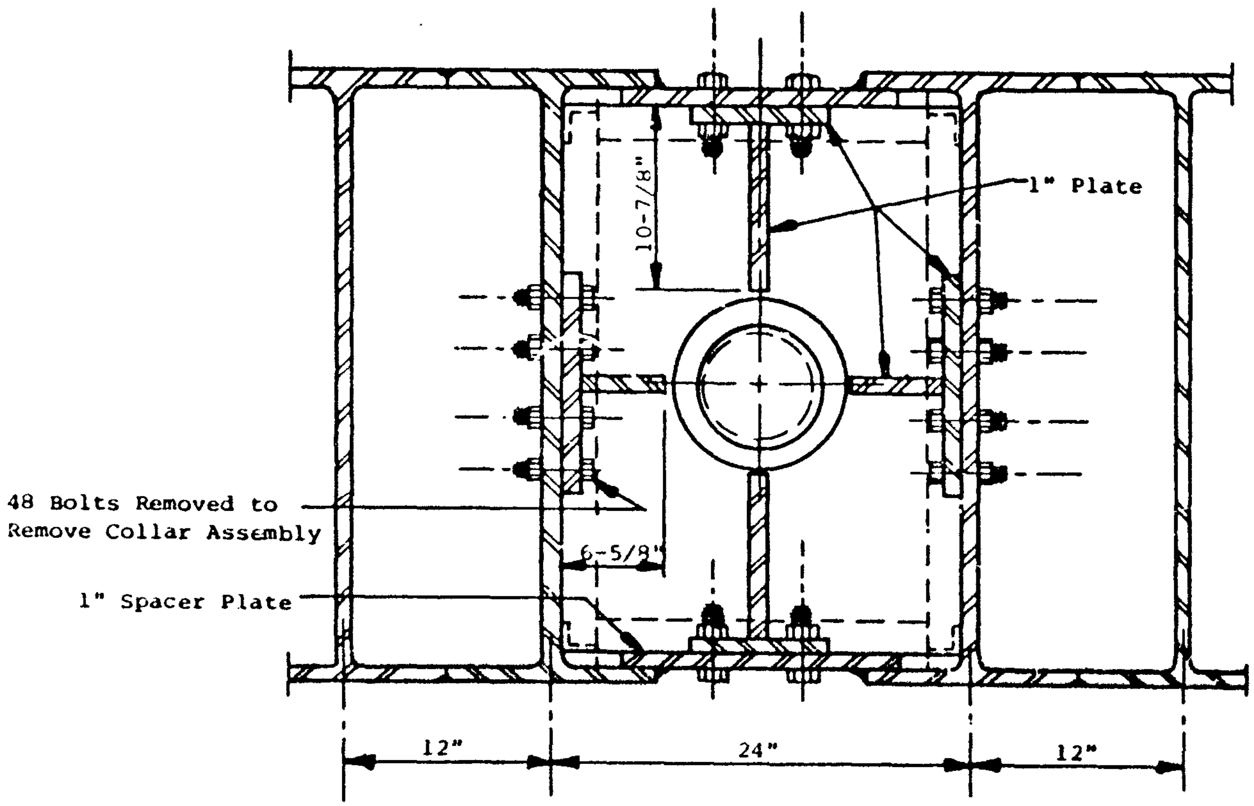
If a 10-inch diameter shaft is used, the radius of gyration is

$$r = \frac{d}{4} = \frac{10}{4}$$

$$= 2.5 \text{ in}$$

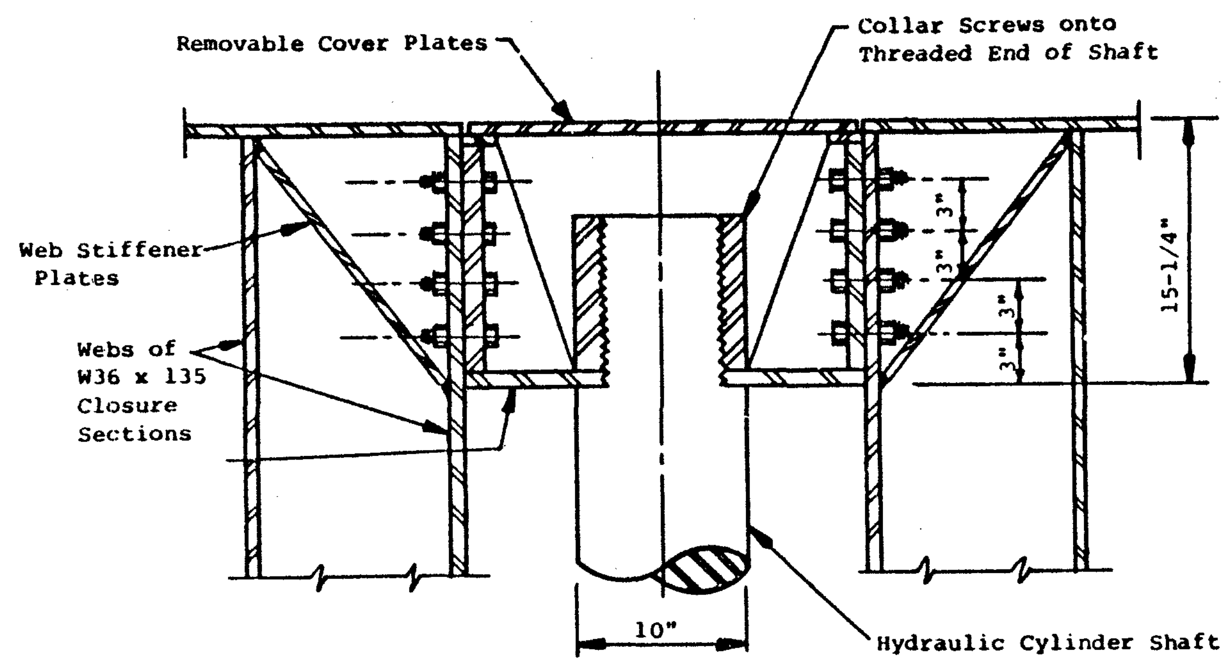
Based upon this radius of gyration, the slenderness ratio is

$$\frac{KL}{r} = \frac{1.0(600)}{2.5} = 240$$



a. Top View of Shaft Collar Assembly with Cover Plate Removed

Figure 59. Shaft Collar Assembly



b. Section Through Shaft Collar Assembly

Figure 59. Shaft Collar Assembly (concluded)

Using Euler's equation for column buckling, the allowable load is

$$P = \frac{\pi^2 EA}{\left(\frac{KL}{r}\right)^2} = \frac{(3.14)^2(29 \times 10^6)(3.14)(5)^2}{(240)^2}$$

$$= 390{,}000 \text{ lbs}$$

The 10-inch shaft gives a factor of safety of almost 2 for the assumed conditions. Depending on the actual fixity of the cylinder base, a smaller shaft diameter could be used.

The shaft collar assembly is fitted between webs and flanges of two W36 x 135 sections placed 24 inches apart. The collar assembly is bolted in place with forty-eight 7/8-inch diameter high strength bolts. The bottom plate and stiffeners are fabricated from 1-inch plate. A 7-inch hole is cut in the center of the bottom plate to permit insertion of the 7-inch diameter threaded end of the cylinder shaft. A threaded collar is screwed onto the threaded shaft after it is inserted through the collar bottom plate and attaches the cylinder shaft to the closure near its top edge. Attachment of the cylinder shaft to the closure near the top, rather than the bottom, edge of the closure was considered to provide a more stable arrangement. Providing space within the closure for the cylinder in the lowered position avoids the need for placing the cylinder below the floor of the closure pit.

The moment resistance of each 1-inch stiffener of the collar assembly is approximately

$$M = \frac{0.6f_y bh^2}{2} = \frac{0.6(60{,}000)(1)(10.5)^2}{2}$$

$$= 1{,}984{,}500 \text{ in-lb}$$

The maximum possible eccentricity of a load on any stiffener is about 11 inches, so the load capacity of each stiffener is in excess of 180,000 lbs. The load capacity of the collar assembly is also determined by details of the connections to the flanges and webs of the W36 x 135 sections.

Assuming an equal distribution of design load to all forty-eight bolts, the shearing stress in each bolt is

$$f_v = \frac{200{,}000}{48(0.6013)}$$

$$= 6{,}930 \text{ psi}$$

which is less than the 15,000 psi allowable. In accordance with AISC specifications, the allowable tensile stress on these bolts is

$$f_t = 50{,}000 - 1.6f_v = 50{,}000 - 1.6(6930)$$

$$= 38{,}912 \text{ psi}$$

Assuming the neutral axis of the bolts is at their centroid, their total moment of inertia is

$$\Sigma Ad^2 = 0.6013(24)(4.5^2 + 1.5^2)$$

$$= 324.7 \text{ in}^4$$

Using an average lever arm of 9.6 inches for eccentricity of the 200,000 pound load, the maximum tensile stress on a bolt is

$$f_t = \frac{M_c}{I} = \frac{200{,}000(9.6)(4.5)}{324.7}$$

$$= 26{,}609 \text{ psi} < 38{,}912 \text{ psi allowable}$$

A stiffener plate is welded between the web the collar assembly is bolted to and the web of the adjacent W36 x 135 to provide increased resistance to web buckling. The base plate of the cylinder is 22.25 inches square and the bearing stress on the concrete is

$$\frac{200{,}000}{(22.25)^2} = 404 \text{ psi}$$

The ACI code allowable bearing stress is

$$0.595f_c' = 0.595(4000)$$

$$= 2380 \text{ psi} > 404 \text{ actual}$$

c. Latching Struts

The centerline of the base of the latching struts is placed 27 inches off of the centerline of the pit and closure. When in the latched position, the centerline of the strut is positioned underneath the center of gravity of one-half of the W36 x 135. This requires that the top of the W14 x 127 strut move inward 14.16 inches from the vertical position. Allowing 6 inches for the height of the pin connection at the base of the strut and 2 inches for a bearing plate on the bottom of the closure, the vertical distance between the bottom of the closure and the center of rotation of the strut is

$$(27 \times 12) - 6 - 2 - 18 = 298 \text{ inches}$$

The angle of rotation from the vertical is

$$\theta = \tan^{-1} \frac{14.16}{298}$$

$$= 2.72°$$

If the end of the strut is cut off at this angle and its web is stiffened, the effect of load eccentricity can be minimized. The centerline length of the strut from the top of the bearing plate to the center of the pin at the base is

$$L = \sqrt{298^2 + 14.16^2}$$

$$= 298.3 \text{ in}$$

A stop fabricated from 0.5 inch plate is placed at each location where the strut will bear against the bottom of the closure. This stop serves to positively position the strut under the closure.

Three pairs of W14 x 127 structural sections were somewhat arbitrarily selected for the latching struts. If made from Grade 50 A572 steel, the static load capacity of each strut for a 28 foot long member is 637,000 lbs. The load capacity is controlled by the slenderness ratio of the strut, and there was no advantage to using Grade 60 steel. The total capacity of the six struts is approximately

$$6(637{,}000) = 3{,}822{,}000 \text{ lbs}$$

or a load factor of 9.5 times the weight of the closure.

If a cylindrical pin is used in double shear at the base of the strut, the required cross sectional area is

$$A_P = \frac{637{,}000}{0.55f_y(2)} = \frac{637{,}000}{0.55(66{,}000)(2)}$$

$$= 8.77 \text{ in}^2$$

The pin diameter should be

$$d = \sqrt{\frac{4(8.77)}{3.14}}$$

$$= 3.34\text{, say } 4.0 \text{ in}$$

For f_y = 60,000 psi, the bearing area on the pin and pin hinge must be at least

$$A = \frac{P}{0.9f_y} = \frac{637{,}000}{0.9(60{,}000)}$$

$$= 11.80 \text{ in}^2$$

For a 4.0-inch diameter pin, the length of bearing must be at least

$$L = \frac{11.80}{4.0}$$

$$= 2.95 \text{ in}$$

Use a 4-inch diameter pin with a 3-inch wide hinge bearing plate attached to the strut base. Two 1.5 inch wide hinge bearing plates are used on the base portion of the hinge attached to the closure pit floor.

Using ACI criteria for allowable bearing stresses in the concrete, the required area of the base plate is

$$A_B = \frac{637{,}000}{0.595(4000)}$$

$$= 268 \text{ in}^2 = 16.4\text{-inch square plate}$$

Using a 17 inch square plate, the bearing pressure would be

$$\frac{637{,}000}{(17)^2} = 2{,}204 \text{ psi}$$

The maximum moment in the base plate will occur at a section through the junction of the base plate and the vertical 1.5-inch hinge plates (see figure 60). It is equal to

$$M = \frac{wL^2}{2} = \frac{2,204(5.375)^2}{2}$$

$$= 31,837 \text{ in-lb/in}$$

Use a 1-inch thick base plate with two 1 inch thick triangular stiffeners running from the centerline of the hinge pins to the edge of the base plate.

Next compute the moment of inertia of the stiffened base plate about its centroid. The centroid is located at

$$\bar{x} = \frac{(17)(1)(0.5) + 2(5)(1)(3.5)}{17(1) + 2(5)(1)}$$

$$= 1.61 \text{ inches}$$

The moment of inertia about this axis is

$$I = \frac{2(1)(5)^3}{12} + 2(5)(1)(1.89)^2 + \frac{17(1)^3}{12} + 1(17)(1.11)^2$$

$$= 78.92 \text{ in}^4$$

The total moment acting on this section is

$$M = 17(31,837)$$

$$= 541,200 \text{ in-lb}$$

And the maximum bending stress is

$$f_b = \frac{M_c}{I} = \frac{541,200(4.39)}{78.92}$$

$$= 30,100 \text{ psi} < 36,000 \text{ psi allowable}$$

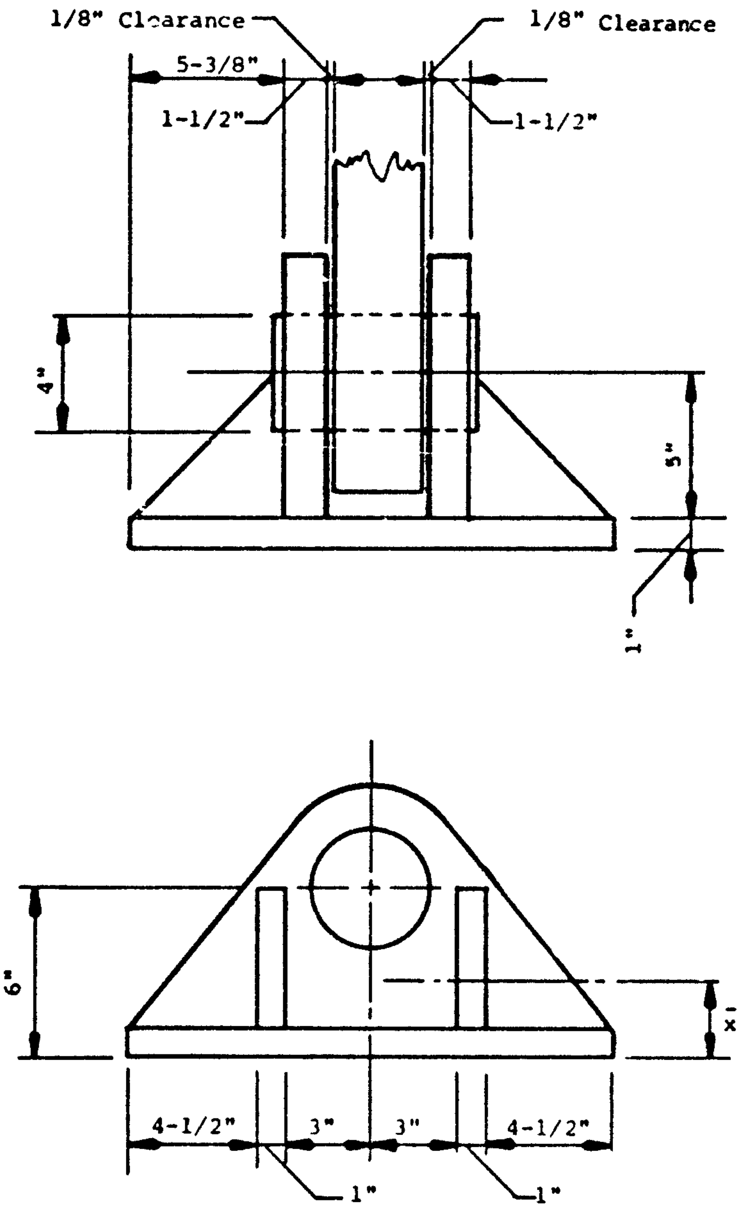


Figure 60. Hinge Mechanism for Latching Struts

The latching struts are actuated by hydraulic cylinders positioned between the struts and sidewalls of the closure pit. They are placed at 7 feet above the closure pit floor.

From the latched position to the fully recessed position the latch rotates through an angle of 3.58°. The required stroke of the actuating cylinders is

$$[(7 \times 12) - 6]\tan 3.58°$$

$$= 4.88 \text{ in}$$

The overall length of the cylinder assembly, including end eyes and clevises, is 14 inches. The space between the strut and wall when the strut is fully recessed is 18.51 inches, so there is sufficient space for the cylinder assemblies.

When fully latched the center of gravity of the strut has moved inward from the vertical position a distance

$$d = \frac{298.3}{2} \tan 2.72°$$

$$= 7.08 \text{ in}$$

The total weight of the strut is approximately 3500 pounds, so the moment due to displacement of the center of gravity is

$$M = 3500(7.08)$$

$$= 24{,}780 \text{ in-lb}$$

The pulling force required of the actuating cylinders is

$$P = \frac{24{,}780}{(72 - 6)}$$

$$= 375 \text{ lbs}$$

A 2-inch diameter Model J Miller Fluid Power cylinder, with a 0.625-inch diameter shaft and operated a 500 psi, provides a withdrawal force of 1417 pounds. Its maximum thrust is 1571 pounds. The effective length of the cylinder assembly is about 23 inches. For this effective length, a 0.625 inch shaft is limited by buckling considerations to 1800 pounds of thrust. Since the maximum capacity of the cylinder is only 1571 pounds, the shaft is safe against buckling.

The maximum reaction against the latch stop on the bottom of the closure is

$$R = \frac{1571(72 - 6)}{298.3}$$

$$= 348 \text{ lb}$$

This reaction is well within the capabilities of the proposed latch stops.

Use two actuating cylinders on each of the struts to provide a redundant capability for strut movement. If higher vertical acceleration forces are predicted, either larger struts or a greater number will be required.

The width of the W14 x 127 strut is 14.69 inches, and the spacing of the W36 x 135 closure sections is 12 inches. The struts should be positioned so that each strut bears on two W36 x 135 webs plus 14.69 inches of flange. When the centroid of the W14 x 127 is in line with the centroid of one half of the W36 x 135, the inner edge of the W14 x 127 is 12.25 inches inside of the outer edge of the W36 x 135. The total area in bearing on the W36 x 135 is

$$A_b = 14.69t_f + 2(t_w)(12.25 - t_f)$$

$$= 14.69(0.794) + 2(0.598)(12.25 - 0.794)$$

$$= 25.37 \text{ in}^2$$

The total bearing capacity is

$$P = F_aA_b = 0.9(60,000)(25.37)$$

$$= 1,369,980 \text{ lbs vs a strut capacity of } 637,000 \text{ lbs}$$

8. CLOSURE BULKHEADS

Bulkheads have been provided as shown in figure 49 to transmit airblast reactions from the closure into the shell roof and side walls. A continuous bulkhead section was selected to provide the required resistance. With this section, the closure reactions will be carried both in shear and flexural response.

The maximum closure reaction on the top bulkhead is taken as 36,300 lb/in based on the closure structure design. With a continuous bulkhead configuration as shown in figure 49, the critical section for shear will probably occur at the same elevation as the top edge of the closure. Subtracting 2 inches for concrete cover, the effective depth of the bulkhead section at this elevation is

$$d = (36 + 18\cot 30°) - 2$$

$$= 65.2 \text{ in}$$

According to reference 2, the maximum allowable shear stress in a deep member with proper amounts of shear reinforcing is

$$v_u = 8\sqrt{f'_{cd}} = 8\sqrt{4800}$$
$$= 554.3 \text{ psi}$$

and the maximum possible shear capacity of the bulkhead section with shear reinforcing is

$$V_u = bdv_u = 1(65.2)(554.3)$$
$$= 36{,}140 \text{ lb/in vs } 36{,}300 \text{ lb/in computed reaction}$$

This is considered close enough in view of the conservatism in the criteria for maximum allowable shear stress. If properly reinforced, the bulkhead section has adequate depth for shearing stresses.

Assuming that the closure reaction is spread out over its 18-inch wide bearing on the bulkhead, the moment in the bulkhead at the top of the closure is

$$M = \frac{wL^2}{2} = \frac{36{,}300(18)^2}{18(2)} = 326{,}700 \text{ in-lb/in}$$

At this location, the required reinforcing steel ratio is

$$p = \frac{M}{0.9f_{dy}bd^2} = \frac{326{,}700}{0.9(44{,}000)(1)(65.2)^2}$$
$$= 0.0019$$

This is less than the minimum steel ratio recommended for shrinkage and temperature reinforcing in slabs. The ACI code also recommends that the minimum reinforcing of flexural members be taken equal to either

$$p_{min} = \frac{200}{f_{dy}} = \frac{200}{44{,}000}$$
$$= 0.0045$$

or 1.33 times that determined by an analysis. Since the latter criteria results in a lower steel ratio, use

$$p = 1.33(0.0019)$$
$$= 0.0025$$

The required steel area is

$$A_s = 0.0025bd = 0.0025(1)(65.2)$$
$$= 0.163 \text{ in}^2/\text{in}$$

Use #11 bars 9.5 inches on center.

Next check the flexural steel requirements at a section through the top of the slot. The moment is

$$M = 36,300\left(24 + \frac{18}{2}\right)$$
$$= 1,197,900 \text{ in-lb/in}$$

The effective depth of the section is

$$d = 36 + 42\cot 30° - 2$$
$$= 106.75 \text{ in}$$

The required steel ratio is

$$p = \frac{M}{0.9f_{dy}bd^2} = \frac{1,197,900}{0.9(44,000)(1)(106.75)^2}$$
$$= 0.00265$$

The total shear on this section will be the same as at the lower section checked above. Since the area is larger, the shearing stress will be lower, and the vertical shear reinforcing discussed below will not be stressed as severely. It will, therefore, contribute some moment capacity. For this reason, the multiplier 1.33 is not applied to the computed steel ratio. The #11 bars at 9.5 inches on center provide

$$p = \frac{1.56}{9.5(106.75)} = 0.00154$$

So, the steel area has to be increased.

$$A_s = 0.00265(1)(106.75)$$
$$= 0.2829 \text{ in}^2/\text{in}$$

Use #11 bars at 5.5 inches on center.

The required development length for a #11 bar is

$$\ell_d = \frac{0.04A_b f_{dy}}{\sqrt{f'_{dc}}} = \frac{0.04(1.56)(44{,}000)}{\sqrt{4800}}$$
$$= 39.6 \text{ in}$$

Cut off every other bar at the beginning of the sloped portion of the bulkhead. Extend every other bar along the bottom surface of the sloped bulkhead portion into the arch shell.

The #11 bar cannot be bent to a radius of less than 5.6 inches. If smaller bars are desired, a closer spacing or multiple layers must be used. If multiple layers are used, the overall depth of the section must be increased to maintain the same effective depth.

At the lower section of the bulkhead (top of closure), the steel ratio is

$$p = \frac{0.2829}{1(65.2)}$$
$$= 0.0043$$

The ultimate shear strength of the concrete alone, as given by equation 8-38 of reference 2, is

$$V_{uc} = bd(3.5 - 2.5M/Vd)(1.9\sqrt{f'_{cd}} + 2500pdV/M)$$

The quantity

$$(3.5 - 2.5M/Vd) =$$
$$[3.5 - (2.5)(326{,}700)/(36{,}300)(65.2)]$$
$$= 3.15, \text{ but cannot exceed } 2.5$$

Therefore,

$$V_{uc} = (1)(65.2)(2.5)(1.9\sqrt{4800} + 2500 \times 0.0043 \times 65.2 \times 0.11)$$
$$= 34{,}023 \text{ lb/in}$$

Excess shear which must be carried by shear reinforcing is

$$V_{us} = 36{,}300 - 34{,}023$$
$$= 2277 \text{ lb/in}$$

The shear capacity of an orthogonal system of shear reinforcing, as given by equation 8-39 of reference 2, is

$$V_{us} = f_{dy}d\left[\frac{A_V}{12s}\left(1 + \frac{L}{d}\right) + \frac{A_{VH}}{12s_H}\left(11 - \frac{L}{d}\right)\right]$$

Taking equal spacing in the vertical and horizontal directions and equal areas of reinforcing steel in the two directions,

$$s = s_H$$

$$A_V = A_{VH}$$

With

$$\frac{L}{d} = \frac{18}{65.2}$$
$$= 0.28$$

$$V_{us} = 44{,}000\left[65.2\right]\left[\frac{A_V}{12s}(1 + 0.28) + \frac{A_V}{12s}(11 - 0.28)\right]$$

$$= 2{,}868{,}000\ \frac{A_V}{s} \text{ /in of bulkhead}$$

Placing a grid of vertical and horizontal bars 18 inches on center each way every 22 inches in the bulkhead would require a cross sectional area of

$$A_v = \frac{22V_{us}s}{2,868,800} = \frac{22(2277)(18)}{2,868,800}$$

$$= 0.314 \text{ in}^2$$

Use #5 reinforcing bars.

The vertical bars should be extended into the arch shell. The horizontal bars can be stopped at the first row above the top of the slot in the bulkhead.

Minimum reinforcing should be provided in the transverse direction of the bulkhead, i.e., parallel to the closure surface.

For a maximum spacing of 17 inches on center vertically and horizontally, the required area of steel in the transverse direction is

$$A_{min} = 0.002A_c = 0.002(17)^2$$

$$= 0.578 \text{ in}^2$$

Use #7 at 17 inches maximum spacing. Cut off these bars after extending them into arch structure a sufficient distance to develop the strength of the bars.

9. CLOSURE PIT

a. Pit Walls

The design of the closure pit walls is dependent on the characteristics of the loads they must resist. These loads are, in turn, a function of the airblast loading on the ground surface and the properties of the soil adjacent to the pit walls. For some overpressure regions and geological conditions, ground shock transmitted from the vicinity of the detonation may also be a factor. For the case of saturated soils with low shear strengths, the horizontal component of airblast induced loads will be approximately equal to the vertical component, and there may be little or no attenuation of peak stresses over the depths of interest. For well drained soils with significant shear strength, the horizontal component may be as small as 0.25 to 0.33 times the vertical component, and significant attenuation of peak stresses may occur. The following analyses are based upon an assumed ratio of horizontal to vertical loads of 0.5 ($K_o = 0.5$). This ratio would be characteristic of drained soils of medium to stiff consistency with unconfined compressive strengths of 0.5 to 2.0 tons per square foot.

The length of the hydraulic cylinders and the dimensions of the cylinder shaft attachment collar in the closure require that the closure pit be 27 feet deep from the access corridor floor to the floor of the pit. This depth will allow the top surface of the closure to be flush with the access corridor slab when the closure is in the open position.

A cross section through the closure pit is shown in figure 51. The haunched upper 5.5 feet of the pit walls reduces the spacing between the walls from 9 feet to 4 feet. For design purposes, the clear span of the pit wall is taken to be 21.5 feet, the distance from the bottom of the haunch to the pit floor.

The haunched upper end of the closure pit wall can deflect only 2 inches before it impacts upon the closure. Although the haunch has a large cross section, its torsional stiffness is probably not sufficient to consider the upper edge of the wall fixed. The following analysis treats the wall as a propped cantilever beam. It is also assumed that some slight inelastic deformation of the walls is acceptable and a ductility ratio, μ, of 2 is assumed in the dynamic analysis.

Neglecting any attenuation of peak soil stresses with depth and assuming that the airblast induced ground shock is predominant, the maximum load on the wall is

$$w = DLF\ K_o\ P_{so}$$

Assuming a DLF of 1.0 and taking $K_o = 0.5$ and $P_{so} = 250$ psi,

$$w = 1.0(0.5)(250)$$
$$= 125 \text{ psi}$$

There are many combinations of reinforcing steel percentages and concrete strength and thicknesses which will satisfy this requirement. After several trials, a section was selected which provides the required resistance. Depending on labor, steel and concrete costs, other sections might be more economical. The wall thickness chosen has the following properties.

Overall thickness	t	= 49 in
Effective depth	d	= 44 in
Steel ratio	p	= 0.01 (two layers, both faces)
Concrete dynamic unconfined compressive strength	f'_{dc}	= 4800 psi
Steel dynamic yield strength	f'_{dy}	= 44,000 psi
Unit weight of reinforced concrete	w_c	= 144 lb/ft^3
Young's modulus, steel	E	= 29,000,000 psi
Young's modulus, concrete	E_c	= 3,600,000 psi

The moment of inertia of a unit strip of the wall is taken as

$$I_c = \frac{bd^3}{2}(5.5p + 0.083) = \frac{1(44)^3}{2}[(5.5)(0.01) + 0.083]$$

$$= 5878 \text{ in}^4/\text{in}$$

The mass of concrete per square inch of wall is

$$\rho = \frac{144(49)(1)^2}{1728(386)}$$

$$= 0.01058 \text{ lb-sec}^2/\text{in}/\text{in}^2$$

Treating the wall as a beam strip, its natural frequency of vibration is

$$\omega_N = \frac{15.4}{L^2}\sqrt{\frac{E_c I_c}{\rho}} = \frac{15.4}{(21.5 \times 12)^2}\sqrt{\frac{(3.6 \times 10^6)(5878)}{0.01058}}$$

$$= 327 \text{ rad/sec}$$

and its period is

$$T_N = \frac{2\pi}{\omega_N} = \frac{6.28}{327}$$

$$= 0.019 \text{ sec}$$

As in the case of the closure, a rise time to peak load on the full width wall of 0.018 sec is assumed. Then,

$$\frac{t_r}{T_N} = \frac{0.018}{0.019} = 0.95$$

For this ratio of load rise time to period and a ductility ratio of 2, figure 9-9 of reference 2 gives R_m/F_o = 1.02. Thus, the wall must resist a load of

$$w = 1.02(0.5)(250)$$

$$= 127.5 \text{ psi}$$

Figure 8-18 of reference 2 gives the resistance of the wall in flexure as

$$p_f = 7.2(f_{dy})\left(p_c + \frac{p_e}{2}\right)\left(\frac{d}{L}\right)^2$$

For the pit wall, the tension steel ratio at the bottom edge, p_e, is equal to the tension steel ratio at mid-span, p_c. Substituting $p_e = p_c = 0.01$ in the above equation,

$$p_f = 7.2(44{,}000)\left(0.01 + \frac{0.01}{2}\right)\left(\frac{44}{21.5 \times 12}\right)^2$$

$$= 138 \text{ psi vs. } 127.5 \text{ psi required}$$

So the section is satisfactory in bending.

The critical sections for shear are at a distance d from each support. The shear at the bottom edge is

$$V = \left[\frac{5}{8}\ wL - wd\right] = \left[\frac{5}{8}\ (127.5)(21.5 \times 12) = 127.5(44)\right]$$

$$= 14{,}950 \text{ lb/in}$$

The moment at this point is

$$M = \frac{3}{8}\ wLx - \frac{wx^2}{2} = \frac{3}{8}\ (127.5)(21.5 \times 12)(214) - 127.5\ \frac{(214)^2}{2}$$

$$= 279{,}670 \text{ in-lb/in}$$

and

$$\frac{Vd}{M} = \frac{14{,}950(44)}{279{,}670}$$

$$= 2.35, \text{ but cannot be taken greater than } 1.$$

The shear capacity of the concrete is given by equation 8-32 of reference 2.

$$V_{uc} = d[1.9\sqrt{f'_{dc}} + 2500pdV/M] = 44[1.9\sqrt{4800} + 2500(0.01)(1)]$$

$$= 6892 \text{ lb/in}$$

The actual shear exceeds the capacity of the section, so shear reinforcing must be added. The excess shear is

$$14{,}950 - 6892 = 8058 \text{ lb/in}$$

If a 14-inch spacing of stirrups horizontally and vertically is used, the required area of steel is

$$A_v = \frac{Vs^2}{f_{dy}d} = \frac{8058(14)^2}{44{,}000(44)}$$

$$= 0.816 \text{ in}^2$$

The shearing force at a distance d from the top edge is

$$V = \frac{3}{8}\ wL - wd = \frac{3}{8}\ (127.5)(21.5 \times 12) - 127.5(44)$$

$$= 6726 \text{ lb/in}$$

The moment at this point is

$$M = \frac{3}{8} wLx - \frac{wx^2}{2} = \frac{3}{8} (127.5)(21.5 \times 12)(44) - \frac{127.5(44)^2}{2}$$

$$= 419{,}348 \text{ in-lb/in}$$

$$\frac{Vd}{M} = \frac{6726(44)}{419{,}348}$$

$$= 0.71$$

The shear capacity of the concrete is

$$V_{uc} = 44[1.9\sqrt{4800} + 2500(0.01)(0.71)]$$

$$= 6572 \text{ lb/in vs } 6726 \text{ lb/in actual}$$

The excess shear at the top end of the wall is

$$6726 - 6572 = 154 \text{ lb/in}$$

The excess shear is so small that the need for reinforcing is questionable; however, some shear reinforcing will be provided. Where shear reinforcing is required, the ACI code recommends a maximum spacing of 0.5d. If a 28-inch spacing vertically and horizontally is used, the required reinforcing area is

$$A_v = \frac{Vs^2}{f_{dy}d} = \frac{154(28)^2}{44{,}000(44)}$$

$$= 0.06 \text{ in}^2$$

However, the ACI also recommends a minimum amount of shear reinforcing given by

$$A_v = \frac{50s^2}{f_{dy}}$$

For the 28-inch spacing, this criterion would require

$$A_v = \frac{50(28)^2}{44,000}$$

$$= 0.89 \text{ in}^2$$

At the bottom edge of the wall, the shear drops to the shear capacity of the concrete at a distance

$$\frac{14950 - 6892}{127.5} + 44$$

$$= 107.2 \text{ inches}$$

above the closure pit floor. A 14-inch spacing will require eight rows of stirrups.

One percent tension reinforcing in the vertical direction requires a reinforcing steel area of

$$A_s = 0.01(44)(14)$$

$$= 6.16 \text{ in}^2$$

every 14 inches of wall. Two layers of #11 bars at 7 inches on center provide 6.24 in^2 every 14 inches. Two layers are placed in both the inner and outer surfaces of the wall. The clearance between the two layers should be 1 inch. The distance from the outer surface of the concrete to the centroid of reinforcing steel in the opposite surface of wall should be 44 inches. One layer of #11 bars in the outer surface of the wall is cut off at midheight.

Eight rows of #8 stirrups at 14 inches vertical and horizontal spacing are placed in the bottom portion of the wall for shear reinforcing. Number 8 stirrups are placed at a 28-inch vertical and horizontal spacing over the rest of the wall height. Although the latter does not quite meet the ACI criteria, it is considered adequate in view of the low shear stresses over the upper portion of the wall.

Minimum reinforcing is provided in the horizontal direction. It is taken equal to

$$A_s = 0.0025A_g = 0.0025(14)(49)$$
$$= 1.72 \text{ in}^2$$

if a 14-inch spacing is used. Place #3 bars at 14-inch spacing in each face of the wall. The endwalls and foundation slab were arbitrarily made the same as the side walls.

The vertical blast loads transmitted to the end walls by the overhead arch structure and vertical loads transmitted to all walls through frictional forces at the soil-wall interface require a more detailed analysis than is possible here. Since the shelter floor is not subjected to air-blast loading, the pit wall facing the main shelter is probably subjected to a much lower stress than the wall facing the access corridor. Excluding the weight of the overhead arch and soil cover, the static load on the soil beneath the pit is less than 2 tsf.

b. Sill Reaction Block Assembly

A removable sill reaction block assembly is installed at the top edge of the closure pit on each side of the closure. A cross section through the assembly is shown in figure 61. In the event of permanent deformations of the

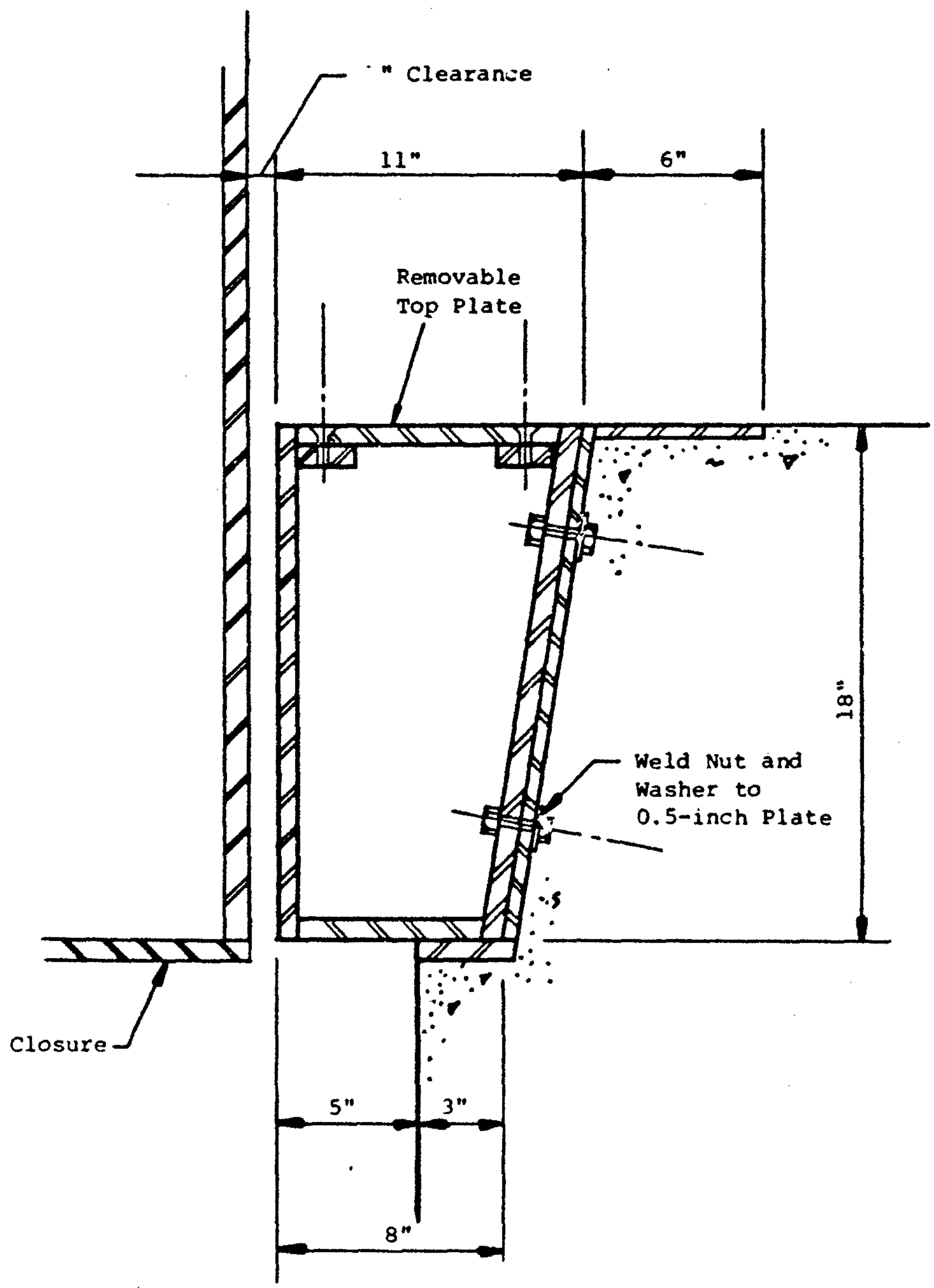


Figure 61. Sill Reaction Block Assembly

closure, these assemblies can be removed to provide increased clearance for closure operation. The top plate is removable to allow access to the hold down bolts.

The reaction of the closure upon the assembly is 36,300 lb/in. If stiffeners are placed so as to coincide with the webs of the W36 x 135 members in the closure, a bearing area of

$$A_b = t_w(18 + k) = 0.598(18 + 1.688)$$

$$= 11.78 \text{ in}^2/\text{ft}$$

is provided. The load capacity would be

$$P = 0.75f_{dy}A_b = 0.75(66,000)(11.78)$$

$$= 583,110 \text{ lbs/ft}$$

The applied load is

$$12(36,300)$$

$$= 435,600 \text{ lbs/ft}$$

Use 0.625 inch thick stiffeners 12 inches on center. Use 0.75 inch thick plate for front, back, top and bottom plates of assembly.

The top plate will be subjected to wheel loads from taxiing aircraft. The maximum tire pressure of any aircraft considered is 290 psi. Applying this as a uniform load to the top plate, the maximum moment in a plate with fixed edges is

$$M = 0.0581wa^2 = 0.0581(290)(11)^2$$

$$= 2039 \text{ in-lb/in}$$

And the maximum stress is

$$r = \frac{Mc}{I} = \frac{2039(0.375)}{1/12(1)(0.75)^3} = 21{,}750 \text{ psi}$$

The allowable is $0.6F_y = 36{,}000$ psi.

The sloping rear face of the assembly raises the possibility that a horizontally applied load might cause the assembly to move up and out of position. The coefficient of friction for steel on steel depends on the type of steel and the condition of the surfaces between the elements. It can vary from less than 0.1 to 0.74. The slope of the rear face is

$$\theta = \tan^{-1} 3/18$$
$$= 9.5°$$

The component of the closure reaction parallel to this face is

$$36{,}300 \sin 9.5° = 5968 \text{ lbs}$$

The normal component is

$$36{,}300 \cos 9.5° = 35{,}802 \text{ lbs}$$

Using a coefficient of friction of 0.1, the resultant sliding force is

$$5968 - 0.1(35{,}802) = 2388 \text{ lb/in}$$
$$= 28{,}653 \text{ lb/ft}$$

Use two 0.75-inch diameter A490-X High Strength Bolts every 12 inches. These bolts provide a shear capacity of only 28,280 lb/ft but should be adequate in view of the low coefficient of friction assumed. They are also required to hold the assembly in place in the event of rubbing contact between the closure and assembly while raising or lowering the closure. It is recommended that the sill reaction block assembly be made up in 10-foot sections to facilitate installation or removal.

10. RADIATION SHIELDING

As currently envisioned, the closure is the weakest portion of the shelter insofar as radiation shielding is concerned. The 6 feet of concrete plus more than 6 feet of earth cover will attenuate the initial and residual gamma and neutron radiation inside the shelter to very low levels. Similarly the X-radiation and thermal radiation pose no threat to the earth covered shell.

The 0.25-inch thick steel spall and liner plate inside the shelter portions of the structure would provide 100 dB attenuation of EMP frequencies greater than 1 kHz and about 50 dB attenuation of signals between 200 and 1000 Hz. Additional attenuation can be obtained by welding reinforcing steel at cross over points to form a continuous grid. However, care must be taken to avoid degrading the strength of the bars.

A similar treatment of reinforcing steel in the shelter floor slab is required. Since the floor slab does not have a spall plate, it may be necessary to include additional EMP shielding. The additional shielding material can be steel plate, steel or copper wire mesh, or copper sheathing. Overhead portions of the shelter, the shelter floor and the steel closure should form a continuous shield around sensitive items of equipment. The closure provides a minimum thickness of 1.6 inches of steel and should provide adequate EMP shielding at the shelter entrance.

The required EMP attenuation depends on the type of equipment to be protected. Many other construction details such as cable penetrations, doors, etc., must also be designed to avoid degrading the overall effectiveness of the shield. Reference 2 contains many valuable suggestions.

The 1.6 inches of steel in the closure provides only minor attenuation of gamma and neutron shielding, and it may be necessary to add shielding material to the closure. Water, concrete, lead, steel and other materials all offer advantages in attenuating certain forms of radiation. All will greatly increase the weight of the closure, and the closure mechanism will have to be modified to handle the increased load. Before any shielding material is added to the closure, a more detailed analysis of the incident radiation levels and required protection factors should be made.

If personnel are to be housed in the shelter during an attack, the amount of shielding required depends on their length of stay in the shelter. Considering all the problems related to greatly increasing the weight of the closure, the most economical approach would be to provide a personnel shelter within the shelter. Using this approach, very high levels of protection could be provided smaller areas at lower cost.

It is doubtful that the current door design could be modified to incorporate sufficient shielding materials without increasing its overall thickness.

If equipment and personnel are to be leaving and reentering the shelter after an attack, some type of decontamination facilities must be provided to prevent the carrying of radioactive debris into the shelter. The many operational considerations inherent in operating in radioactive (or CW/BW contaminated) environments outside the sealed shelter itself could well be the subject of an entire separate study.

SECTION IX

ANALYSIS OF DESIGN

1. SHELTER COMPLEX

A NASTRAN analysis was performed for a quarter section of the shelter complex. A relatively coarse model was prepared in order to keep the cost of the computer analysis within limits. The strip model used for the analysis presented in section VIII was a two dimensional model for which longitudinal forces and strains were zero. Three dimensional effects were included in the quarter section model. In addition, forces and moments in the areas of shell discontinuities could be examined. The model was first input to the CDC 6600 version of NASTRAN, but for reasons which could not be determined, files on the storage tapes were unretrievable by NASTRAN upon attempted restarts. The original data deck was then resubmitted to the CDC 7600 using a newer version of NASTRAN. Because of the speed of the new machine, tape storage was not required, and the analysis was completed with a reasonable amount of computer time.

Attempts were made to model frictional forces opposing motion of the footings by applying forces in a direction opposite to that of the calculated velocity at the footing node point. Although the analyses were performed, the output could not be retrieved for undetermined reasons. A decision was then made to resist lateral displacements of the footing with passive soil pressures and to model this resistance with non-linear springs. This resulted in a slightly poorer mathematical model, but usable data output was obtained.

A quarter section was used to represent the complete structure. In order to utilize symmetric boundary conditions, the mathematical model was loaded with a vertical pressure that acted on the full section and decayed at every point with time. The model geometry is depicted in figure 62.

The model geometry required 258 nodes, 241 bending elements and 241 membrane elements. The basic coordinate system had the x-axis along the centerline of the entry corridor, the z-axis along the main corridor and the y-axis vertical. The origin of the basic system was at the center of the shelter complex. The coordinates for the node points were generated by an auxiliary program. This was done in order to utilize the repetitious pattern that existed for the cross section. The coordinate systems utilized in order to generate the node coordinates are shown in figure 63. Note that the origins of the two auxiliary coordinate systems lie in the x-z plane of the basic coordinate system. One auxiliary coordinate system defined the entry corridor geometry; the other the main shelter geometry.

For ease of input the structure was defined in four zones. The first zone was the main corridor, the second zone was the entry corridor, the third zone was the shelter and the fourth zone was the transition roof section between shelters. Symmetry

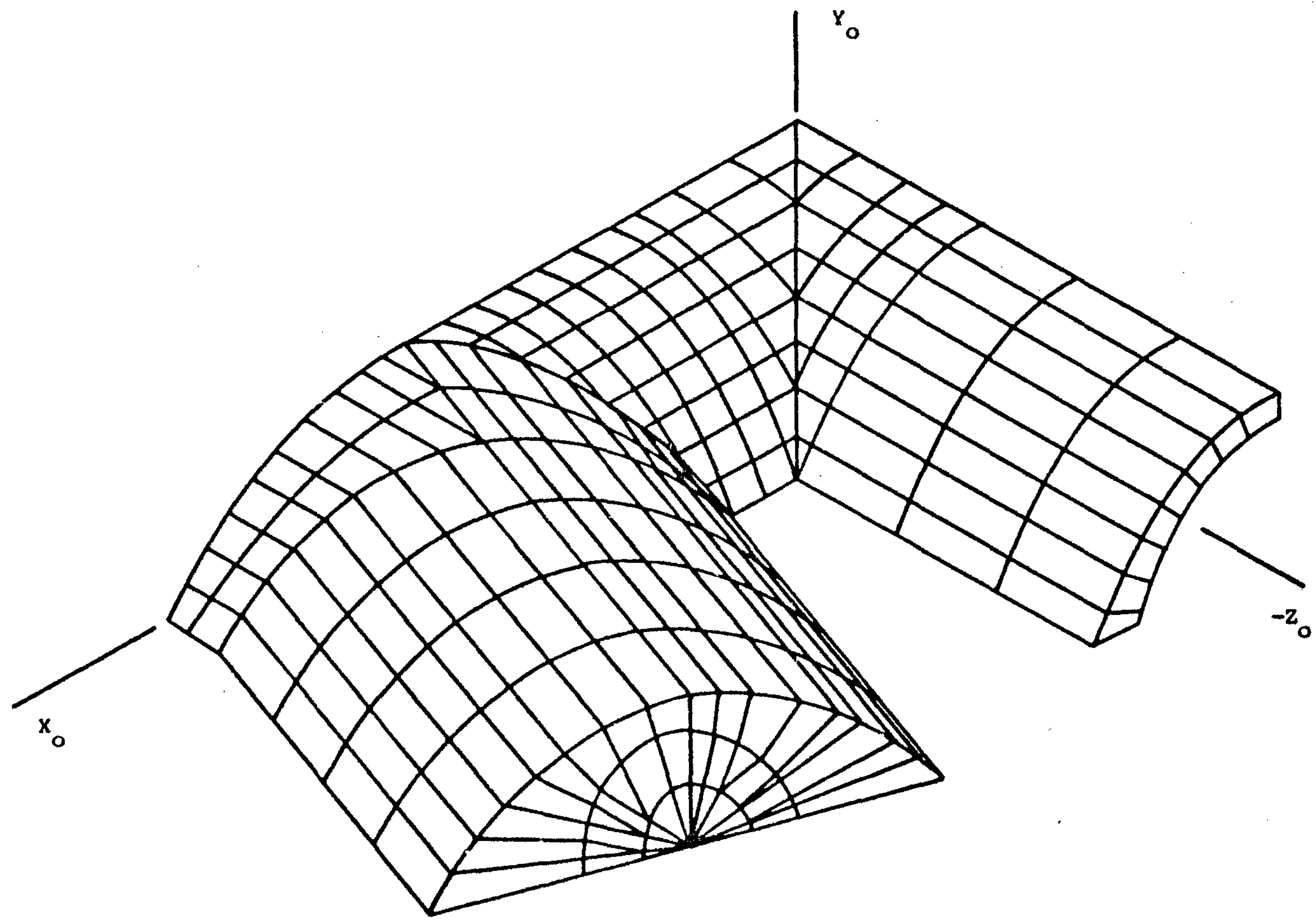


Figure 62. Quarter Section Model of Shelter Complex

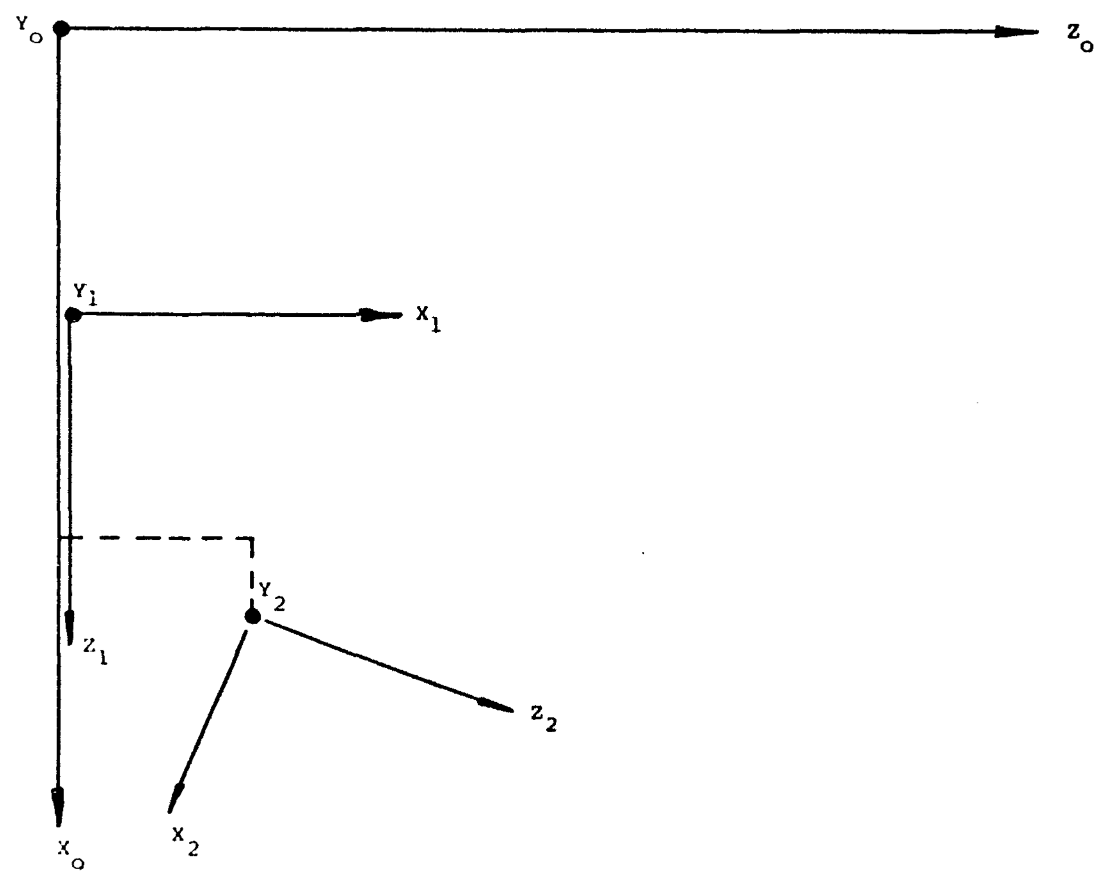
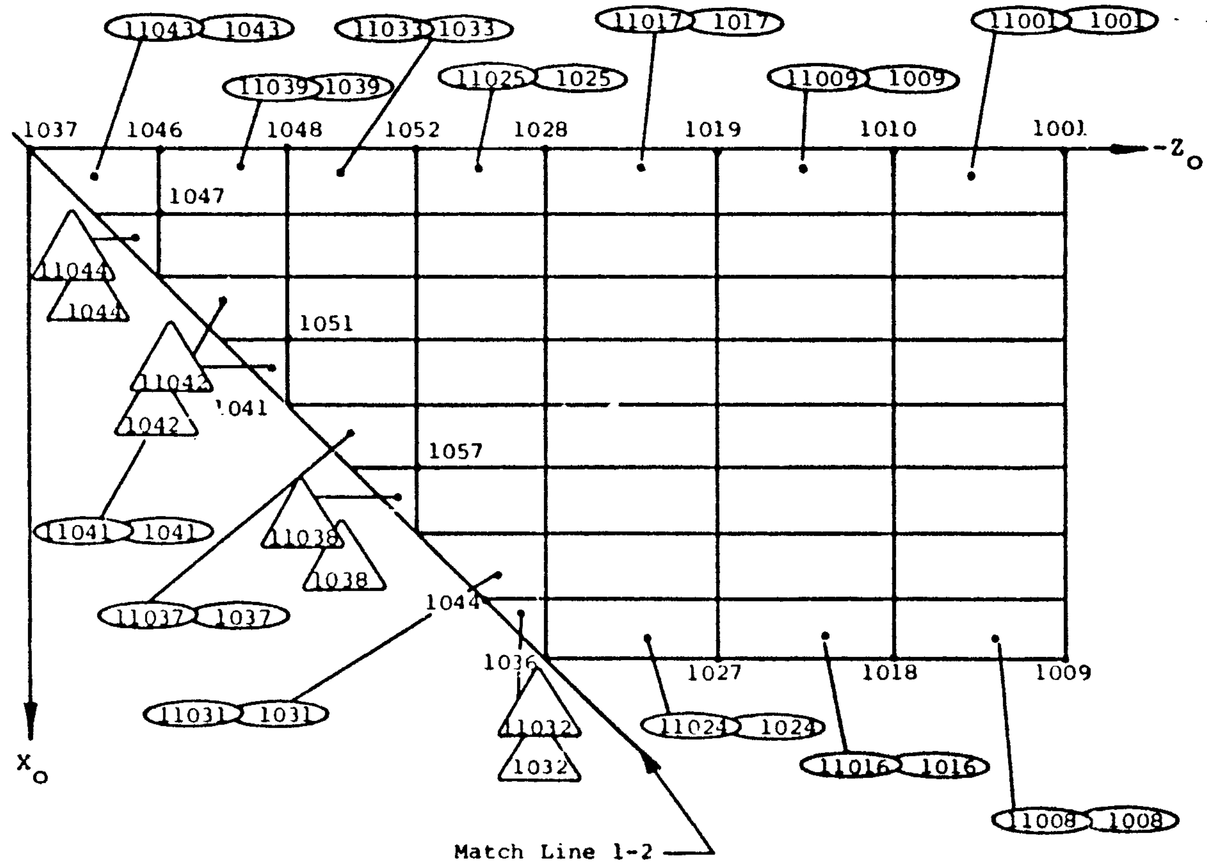


Figure 63. Coordinate Systems for Model of Quarter Section of Shelter Complex

conditions were enforced along the global y-z plane and along the y-x plane. Figure 64 presents the node and element geometry for each of the four zones in turn. The nodes and bending elements are numbered in thousands by zone. The membrane elements are numbered in ten-thousands. The range of thousands and ten-thousands correspond to the zone in which the element is located. The matching lines referred to in the figures indicate the zones that intersect at that line. For ease of display the surfaces shown in figure 64 are developed surfaces of the shelter complex. Numbers have been omitted for any nodes or elements for which linear interpolation can be used.

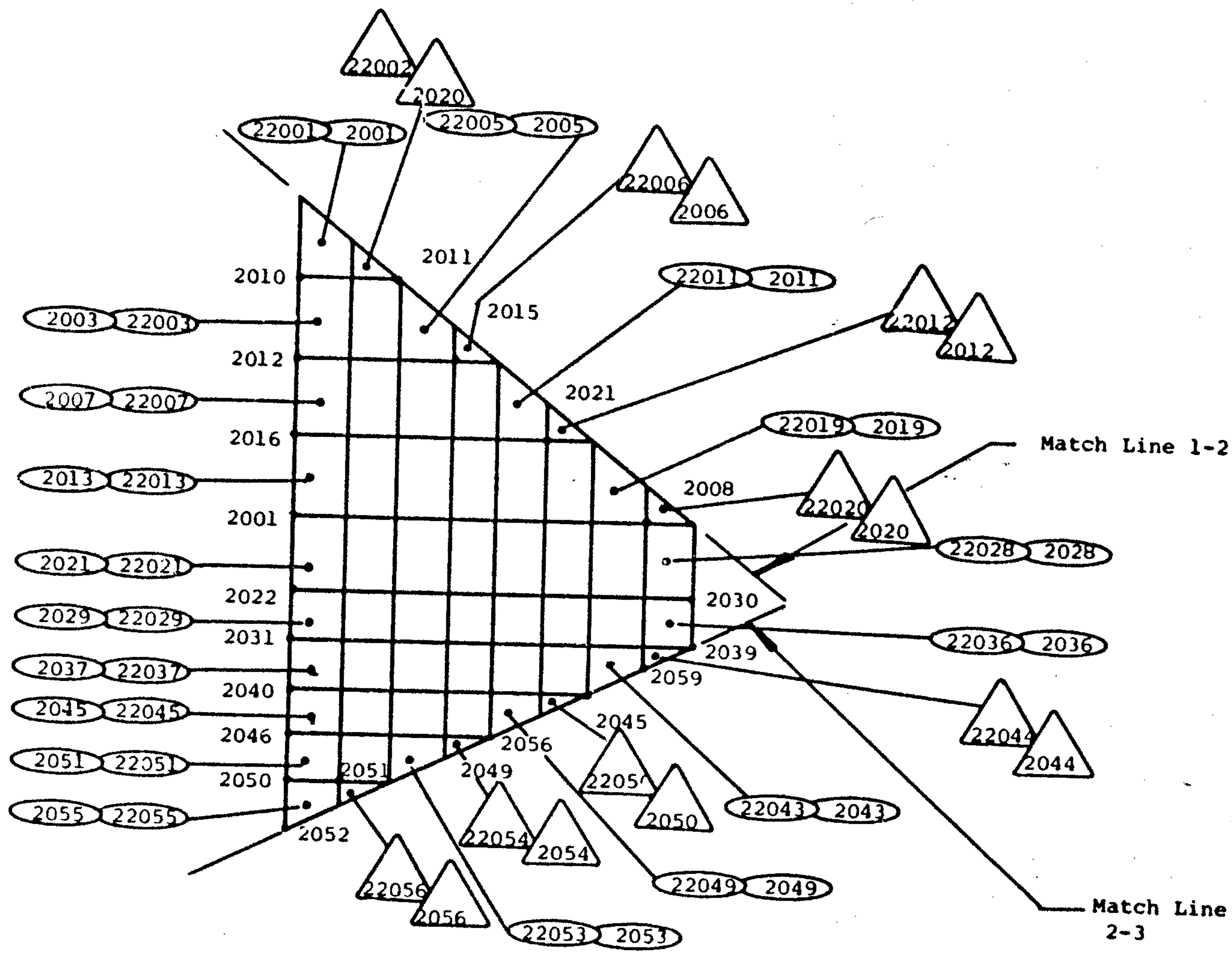
The bottom nodes were supported by nonlinear springs that represented the combination of soil pressures resulting from foundation motions normal and perpendicular to the base of the foundation.

There were substantial differences between the results of the unit strip model discussed previously and the results from the quarter section analysis. The reasons for the differences were shelter geometry effects and barrel shell behavior of the shelter bay and main corridor. The main corridor and shelter exhibited bending about a horizontal axis normal to their centerlines. Figure 65 shows displacements along the two foundation lines and along the crown of the shelter. The two end points of the three lines are connected by dashed lines in order to depict the bending behavior. Figure 66 is



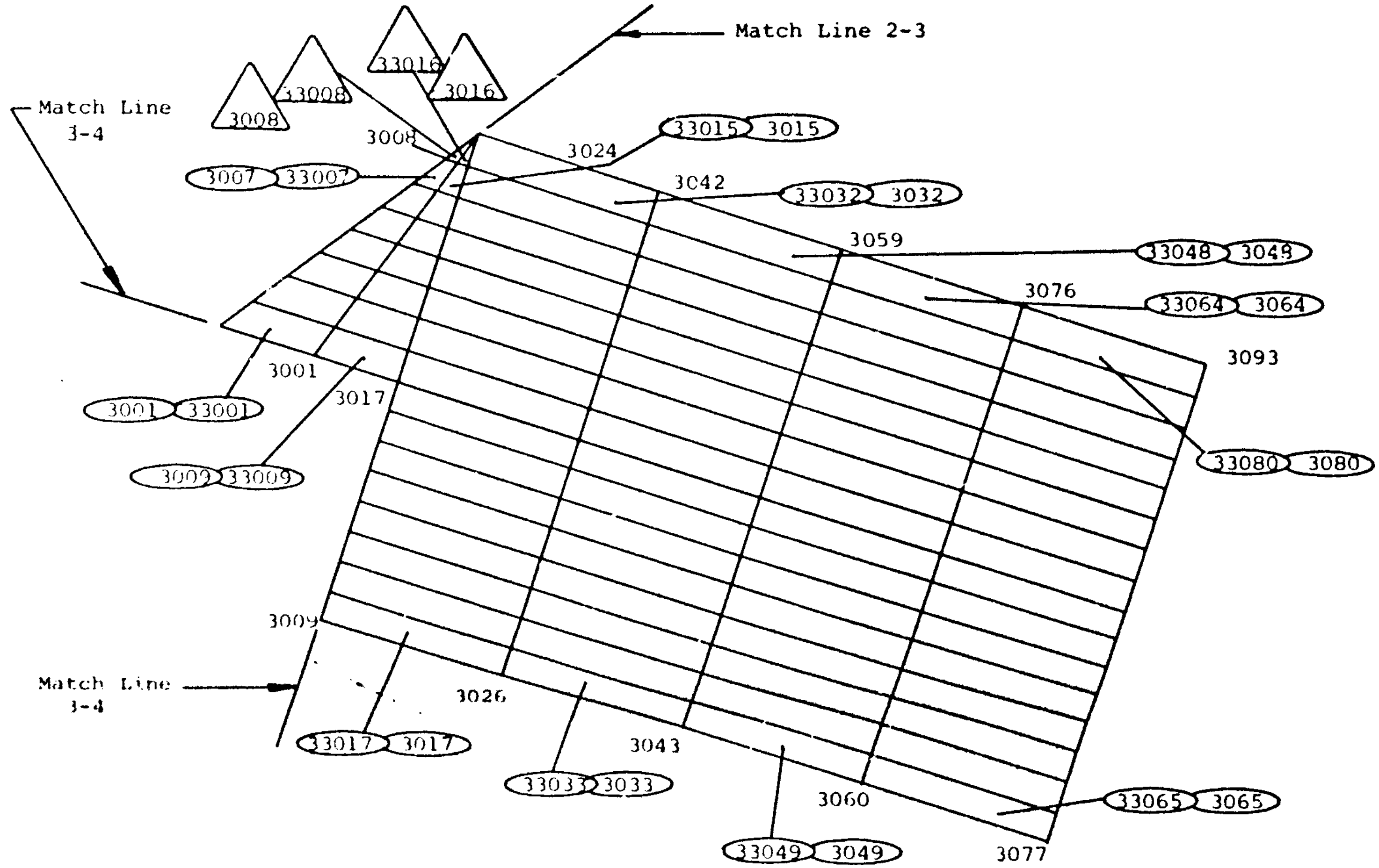
a. Main Corridor

Figure 64. Quarter Section NASTRAN Model



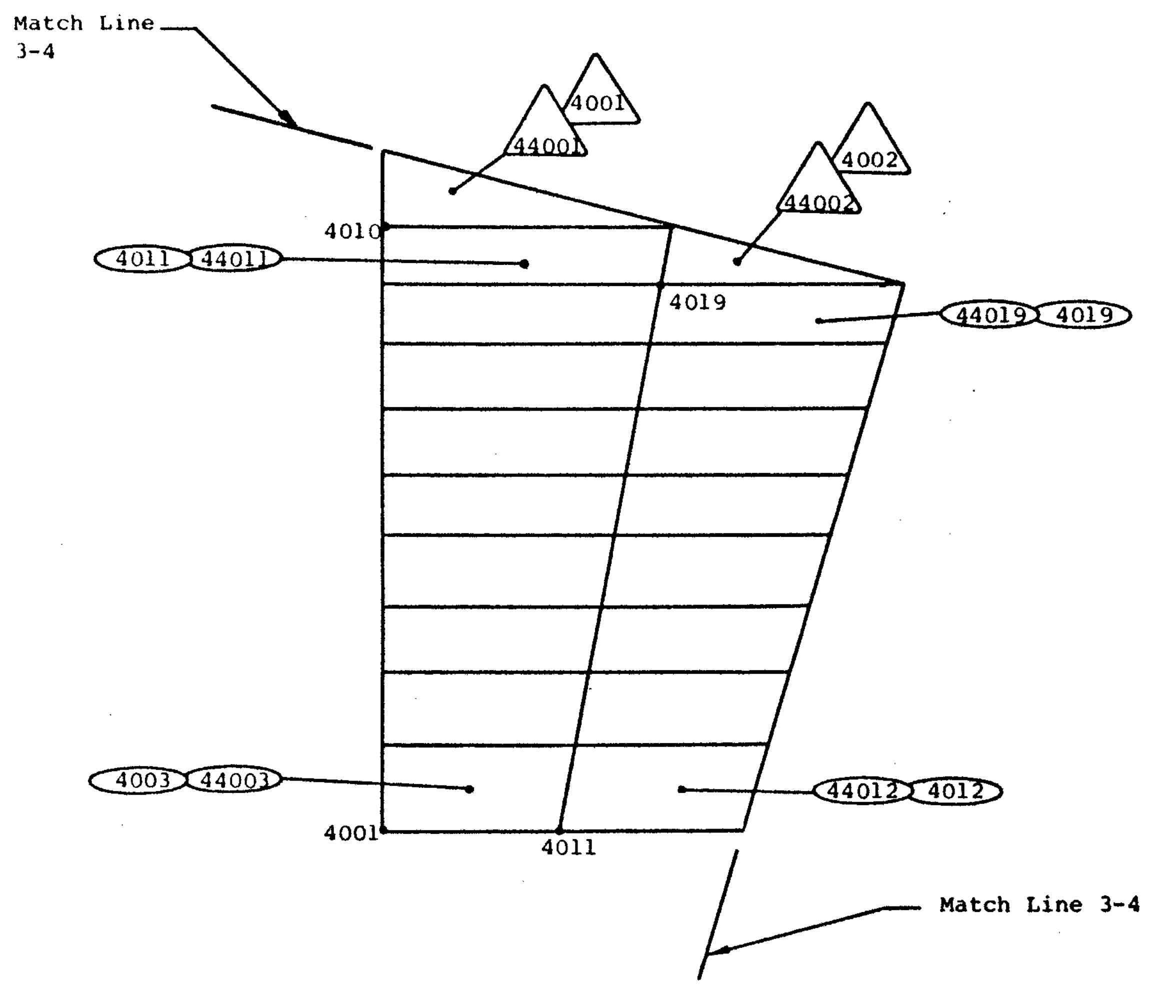
b. Entry Corridor

Figure 64. Quarter-Section NASTRAN Model (continued)



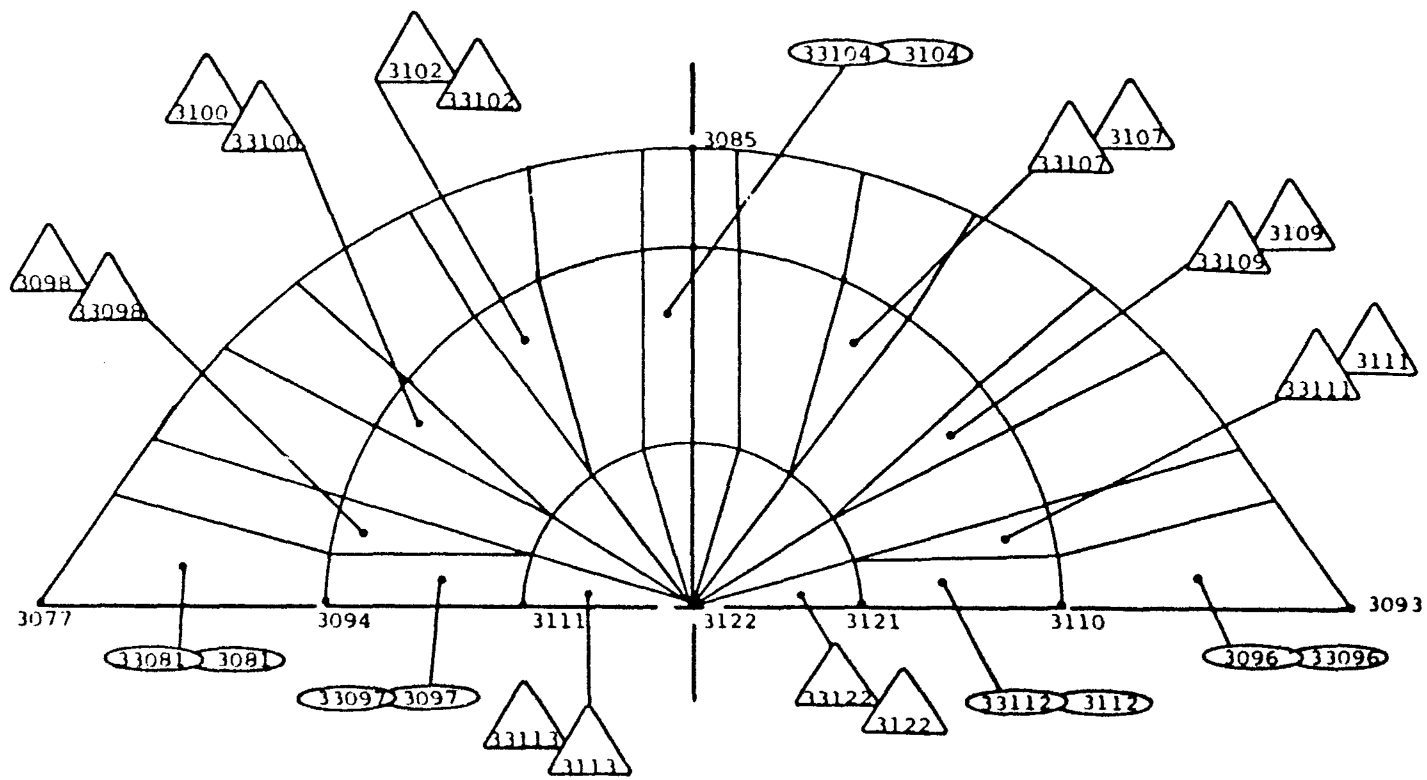
c. Shelter

Figure 64. Quarter Section NASTRAN Model (continued)



d. Transition Roof

Figure 64. Quarter Section NASTRAN Model (continued)



e. Endwall

Figure 64. Quarter Section NASTRAN Model (concluded)

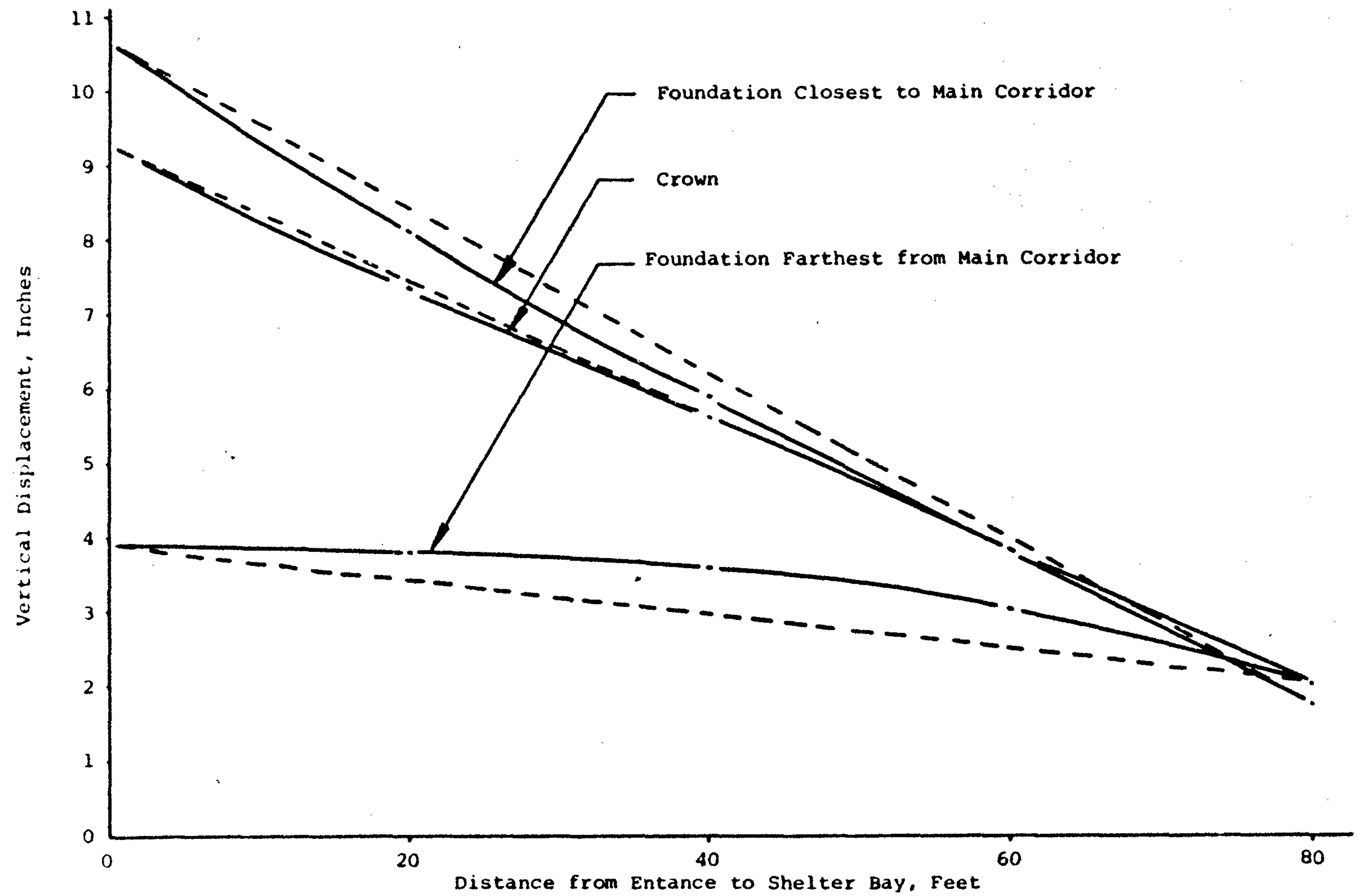


Figure 65. Displacement Variation in Shelter Bay

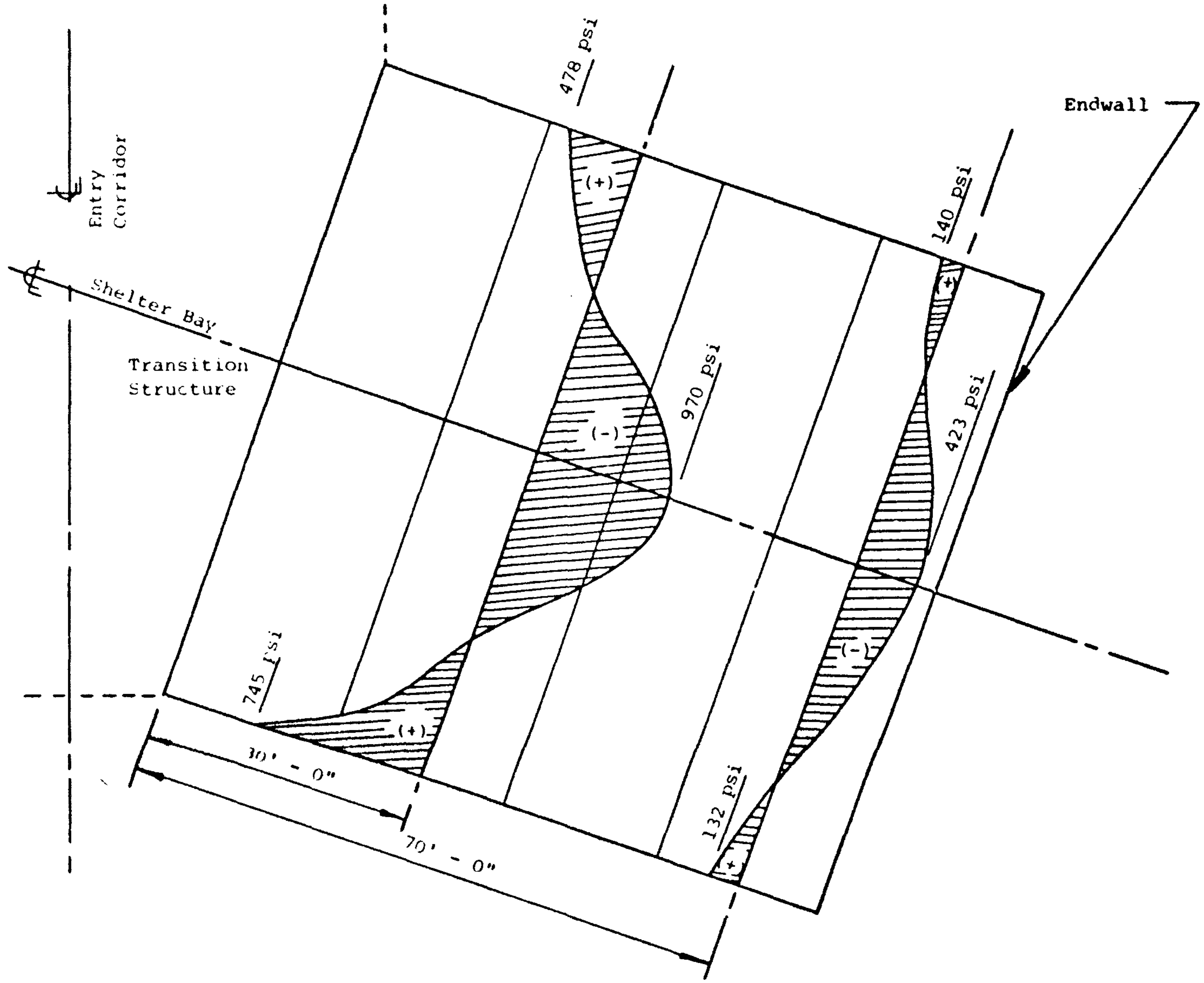


Figure 66. Stress Variation Across Shelter Roof

a plot of the longitudinal stress variation around the circumference of the shelter roof at two different lines. These plots clearly show the bending behavior. As a result of the longitudinal bending of the shelter and main corridor, the circumferential bending forces were reduced significantly from those of the unit strip model. However, a reduction in section thickness was not possible because of the thickness required to meet other weapon threats. The circumferential reinforcing steel ratio was already at a very small value, and further reductions were considered inadvisable. The only advantage that could be gained from a better definition of forces in the shell was to reduce the longitudinal reinforcing ratio from #9 bars at 6 inches on center to #9 bars at 12 inches on center. This reduction in the longitudinal reinforcing ratio does not impair the roof's load carrying ability since the longitudinal tensile stresses can be safely carried by the remaining steel. The reinforcing steel layout is shown in figure 67.

The behavior of the entry corridor in the quarter section model was also markedly different; especially near lines of intersection with the m. in corridor and shelter arches. Bending moments along the crown were high, and thrust loads in the wall near the springing line were well in excess of the ultimate load capacity of the section.

Strengthening ribs were added at the lines of intersection of the entry corridor with the main corridor and shelter

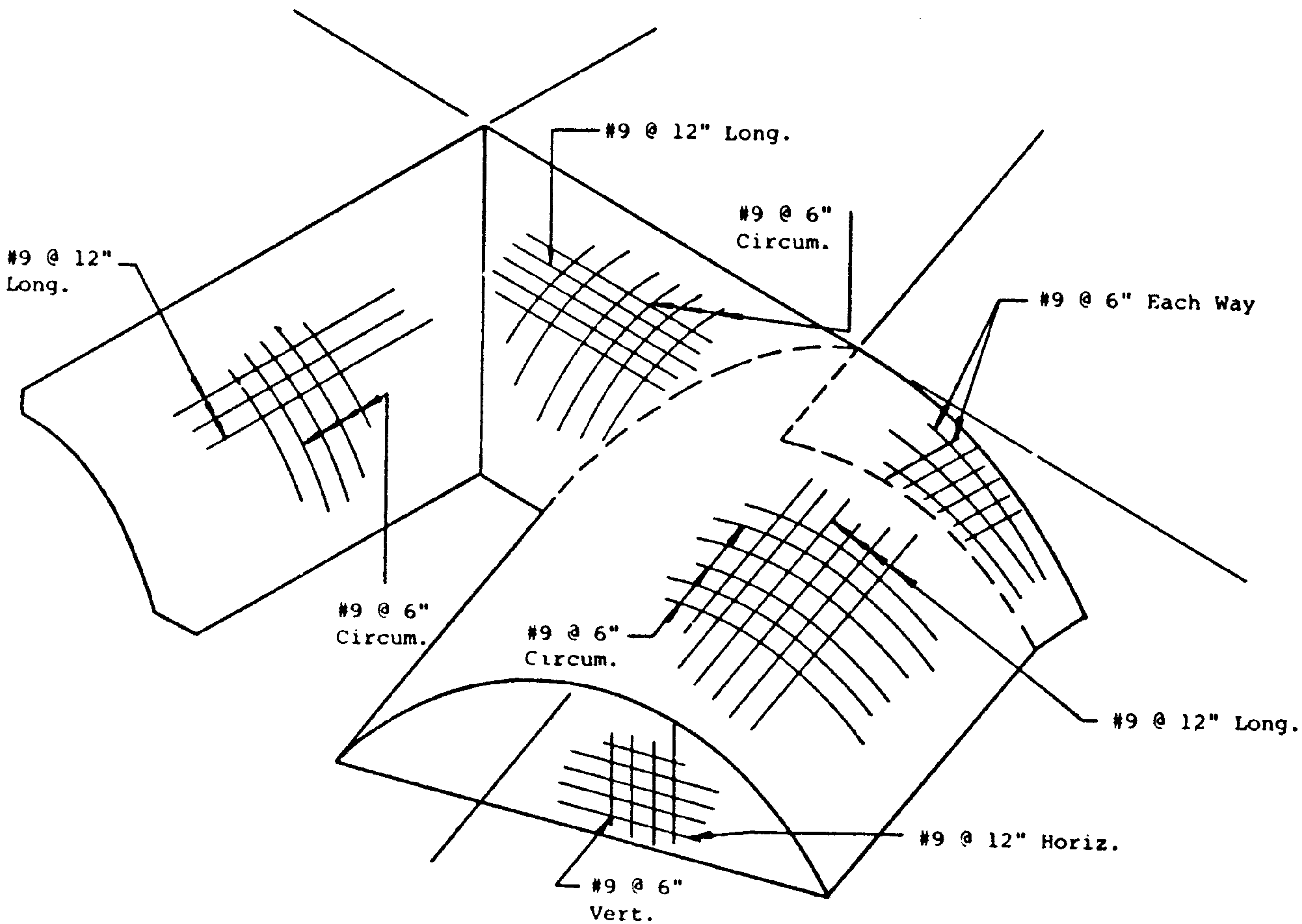


Figure 67. Reinforcing Details in Shelter Roof

arches, and the structure was reanalyzed. The initial rib dimensions were 6 feet wide by 10 feet deep. In order to ease reinforcing steel placement, the depth of the ribs was subsequently increased to 10.33 feet. The location of the centerline of the inner and outer surfaces of the rib were established as the intersection of two cylindrical surfaces at radii of 49.67 feet and 60 feet, respectively. The width of the rib was constructed at right angles to these reference lines. The ribs were reinforced with 0.5 percent positive reinforcement and 0.25 percent negative reinforcement. A cross section normal to the face of the rib is shown in figure 68.

Table 10 provides a comparison of several parameters that show the effect of the strengthening ribs. The displacement, moments and compressive forces in the shell adjacent to the rib were all reduced by the addition of the rib. Figure 69 shows the variation of moment and thrust loads around the arc of the rib at the time of maximum moment at the crown of the rib. The maximum moment in the rib is 1.716×10^9 in-lb, and the maximum thrust is 3.859×10^7 lbs. The thrust load is compressive throughout the length of the rib with a very large magnitude near the springing line. The bending moment is positive (compression on outer face) near the crown and negative near the springing line. Figure 70 shows the moments and thrusts from figure 69 plotted on an interaction curve for

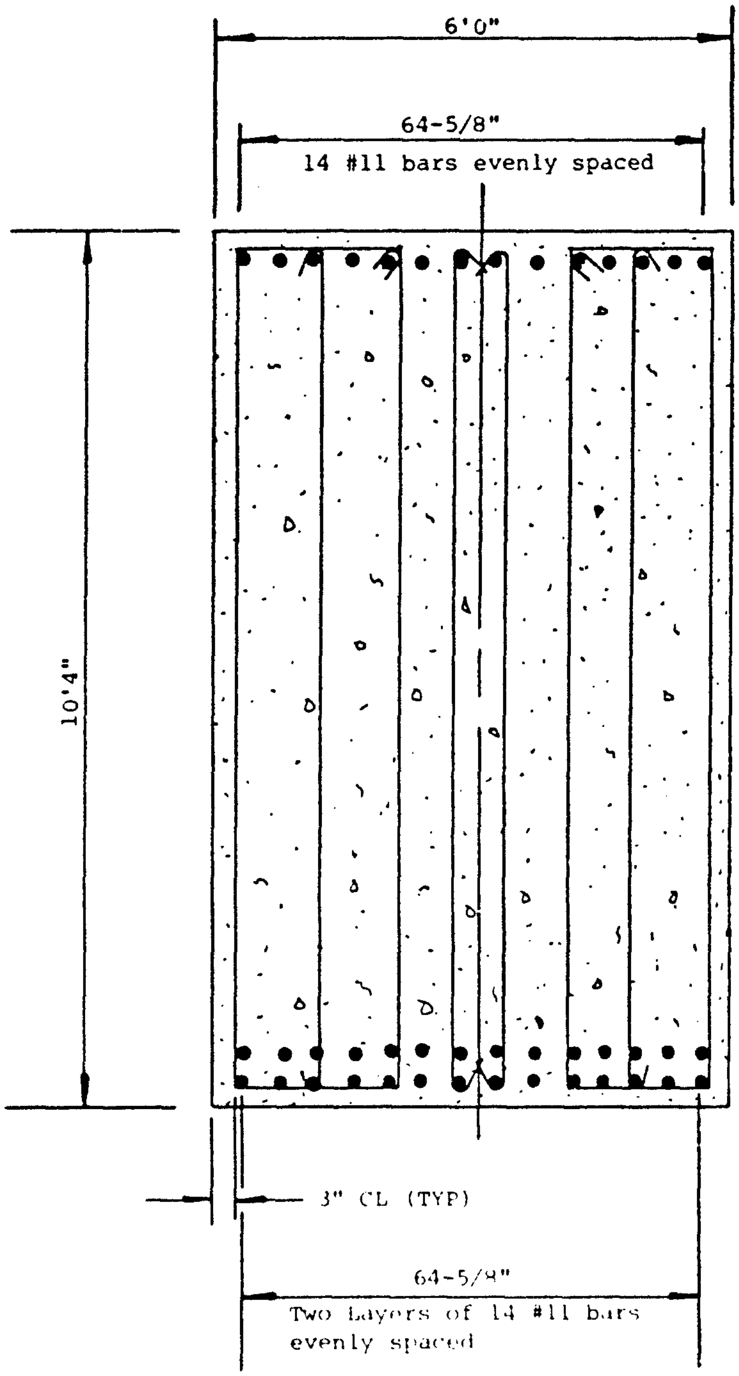


Figure 68. Rib Cross Section

Table 10

EFFECT OF STRENGTHENING RIB ON DISPLACEMENTS, MOMENTS AND THRUSTS

Item	Without Stiffening Ribs	With Stiffening Ribs
Maximum Displacement at Center of Shelter	22.81 in	14.35 in
Maximum Moment (M_y) in Element 2003	9.09×10^6 in-lb/in	7.10×10^6 in-lb/in
Maximum Thrust (T_y) in Element 22003	90,864 lb/in	48,744 lb/in
Maximum Principal Thrust in Element 22012	545,976 lb/in	383,976 lb/in
Maximum Principal Thrust in Element 22044	560,440 lb/in	392,976 lb/in

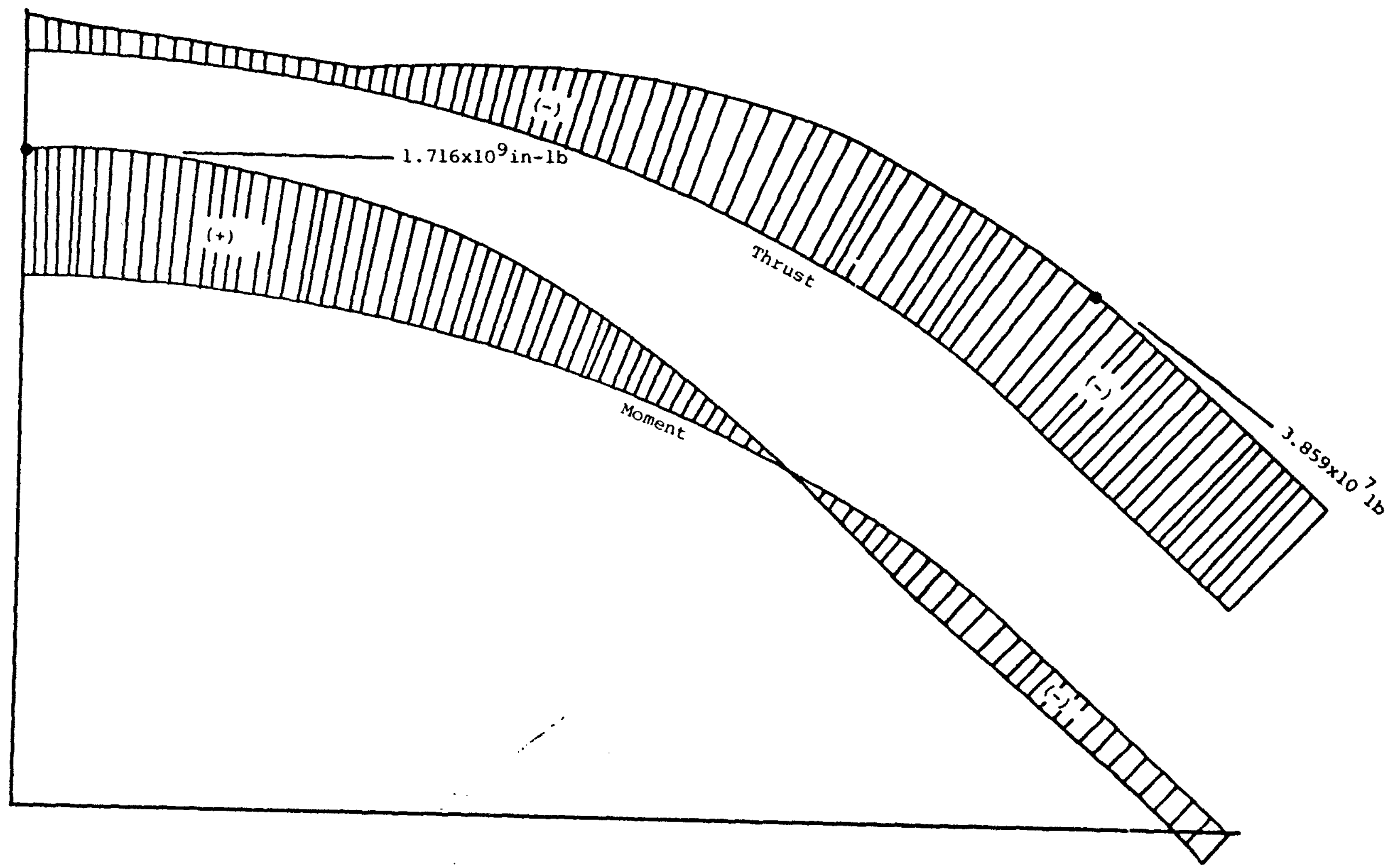


Figure 69. Moment and Thrust Variation in Rib

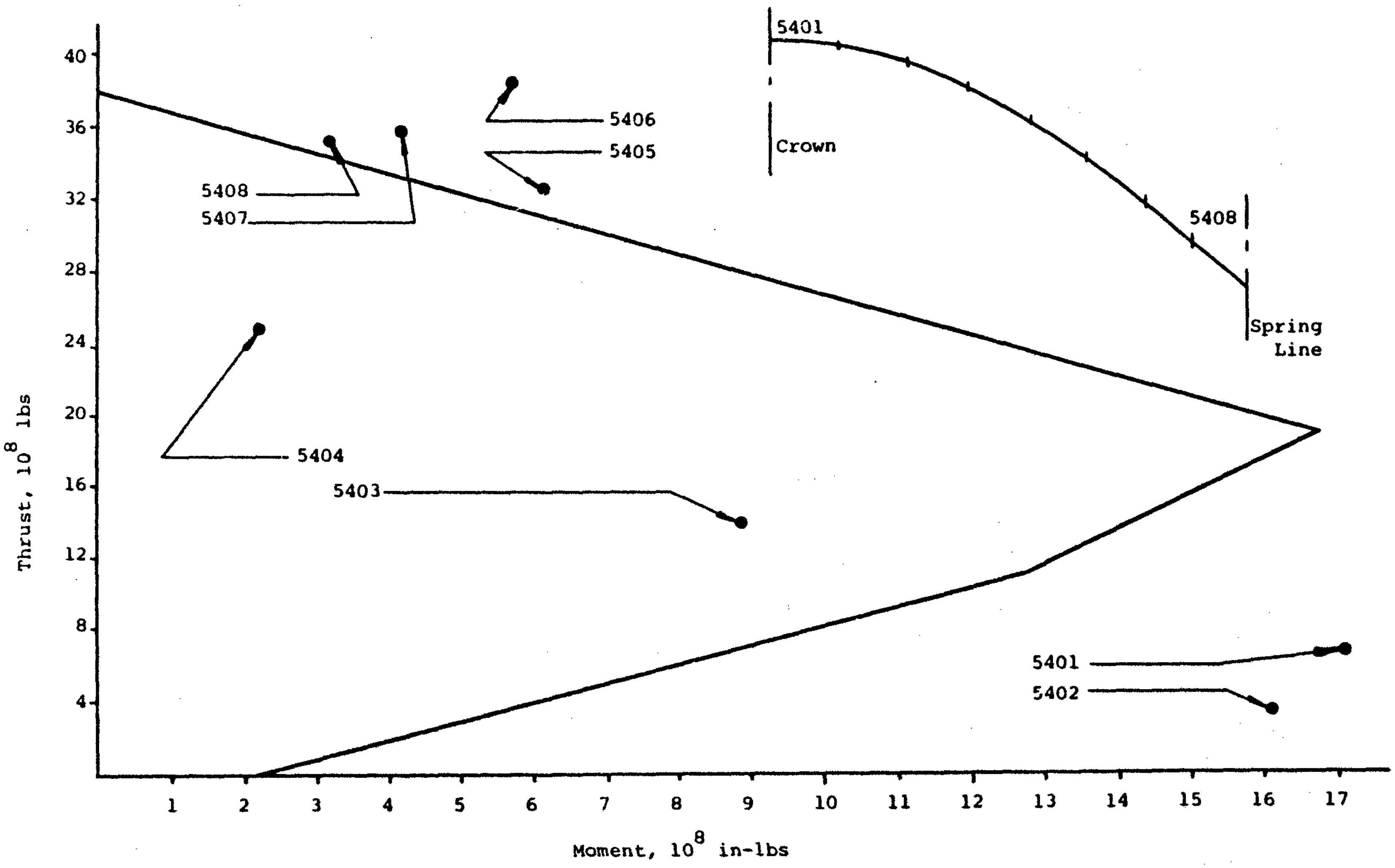
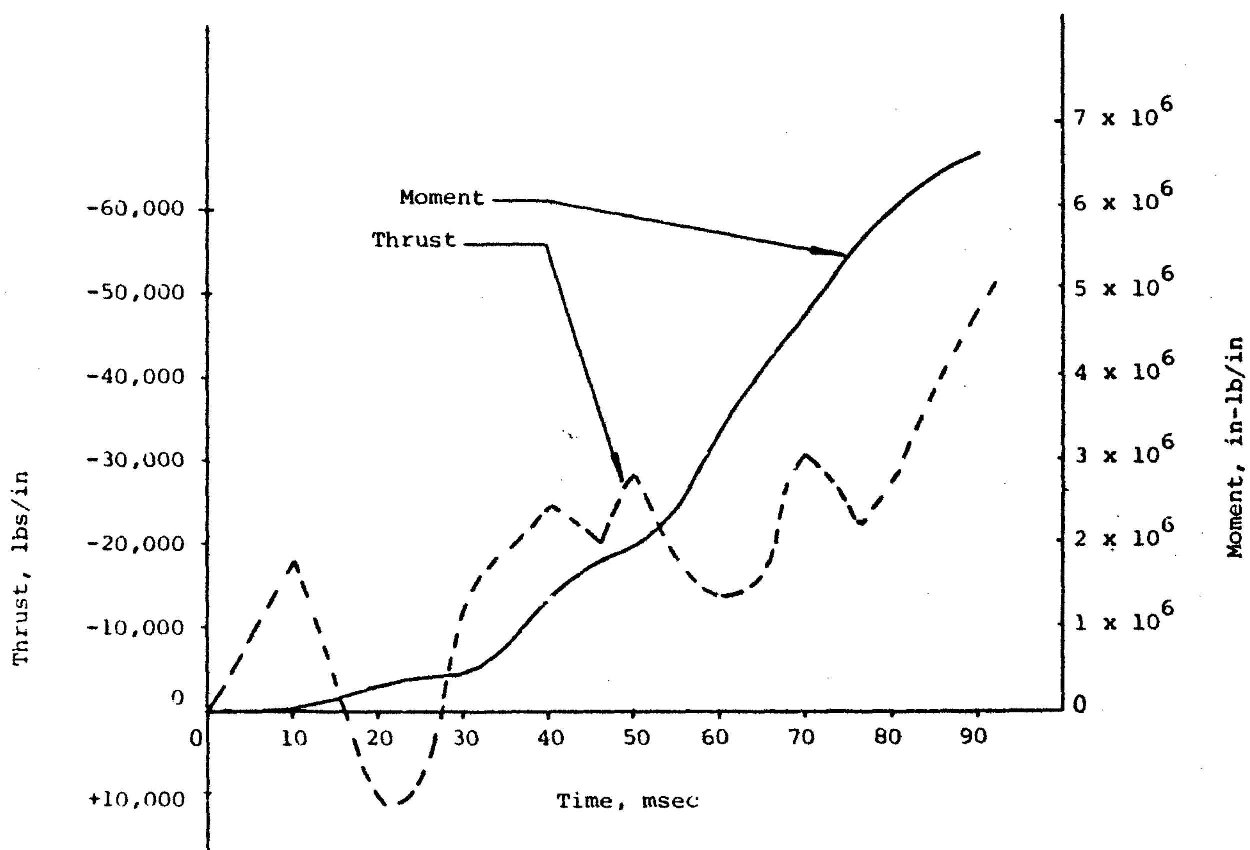


Figure 70. Moment-Thrust Points for Strengthening Rib

the rib section used in the analysis. It can be seen that two points indicate failure in flexure (nearest the crown), two points fall within the limits of the interaction diagram, and the remaining points indicate failure in compression. Figure 71 shows the variation of moment and thrust in Element 2003 with time. This is the most severely loaded shell element near the center of the shelter. Figure 71a shows moment and thrust variation as a function of time, and figure 71b plots combined moment and thrust at selected times on an interaction curve for the shell cross section. Figure 71 indicates that the shell in this area will yield in flexure at about 50 msec.

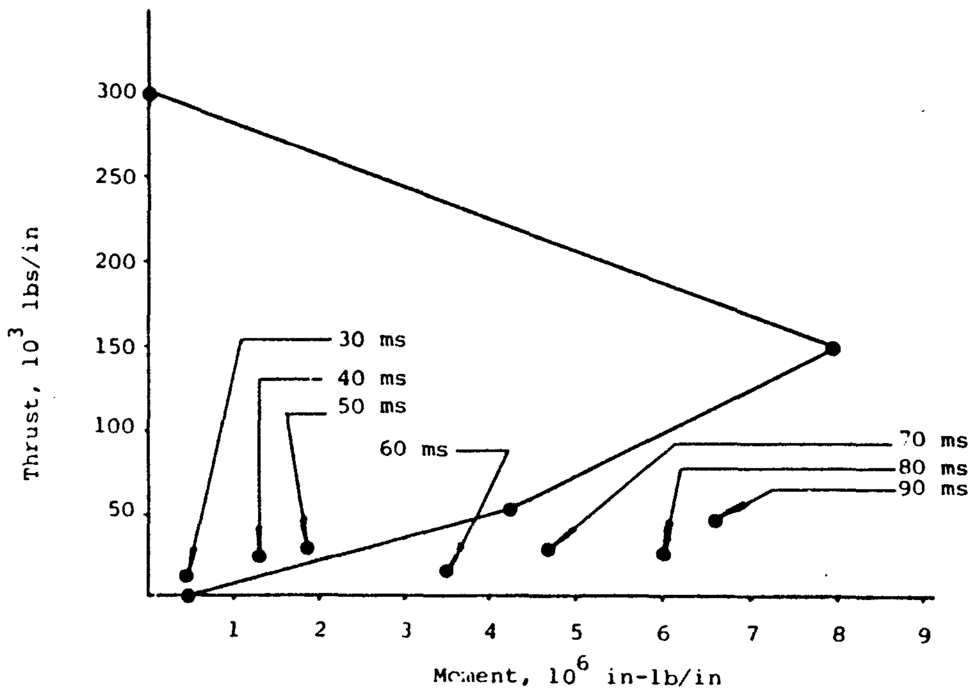
Figure 72 shows the location of combined moment-thrust points for several elements in the entry corridor. It can be seen that yielding will probably occur along the centerline of the entry corridor. Because the NASTRAN analysis is a linear elastic analysis, the amount of inelastic response which occurs is difficult to estimate. Twenty-four inches of clearance is provided between the closure and the bulkhead under the entry corridor.

The displacement-time history of several points in the shelter complex is presented in figure 73. The points shown in figure 73a lie at the crown and points shown in figure 73b are located along the foundation. Figure 74 shows the displacements at 70 msec as a function of the distance along



a. Variation of Moment and Thrust with Time

Figure 71. Moment-Thrust Relations in Element 2003



b. Interaction diagram

Figure 71. Moment-Thrust Relations in Element 2003 (concluded)

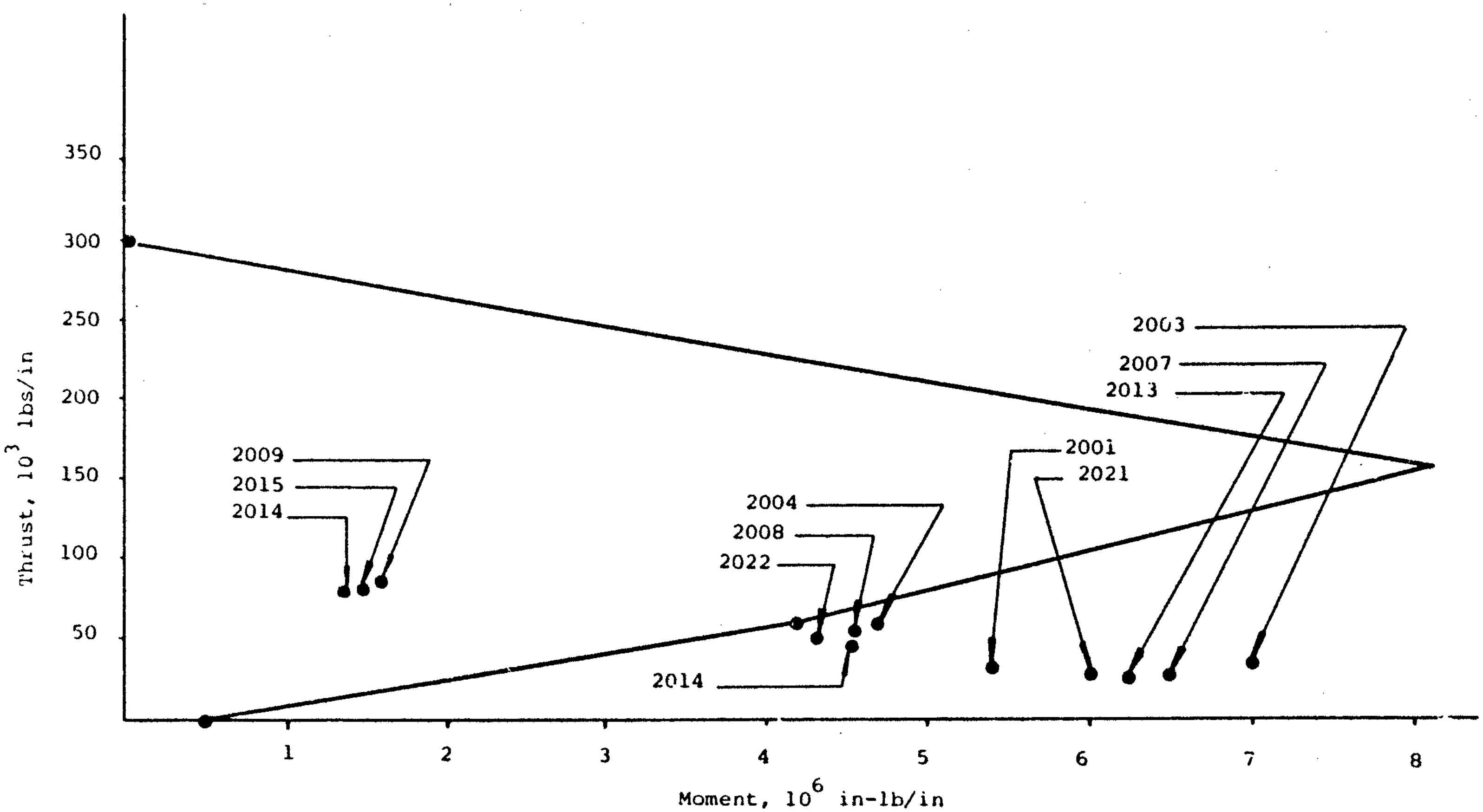
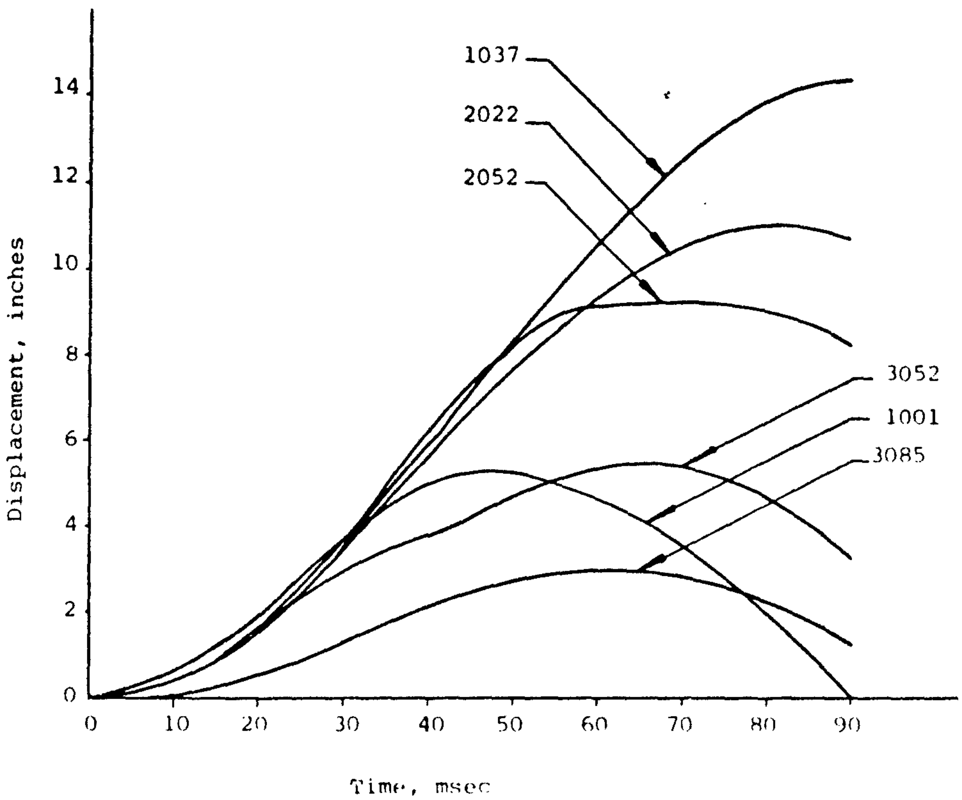
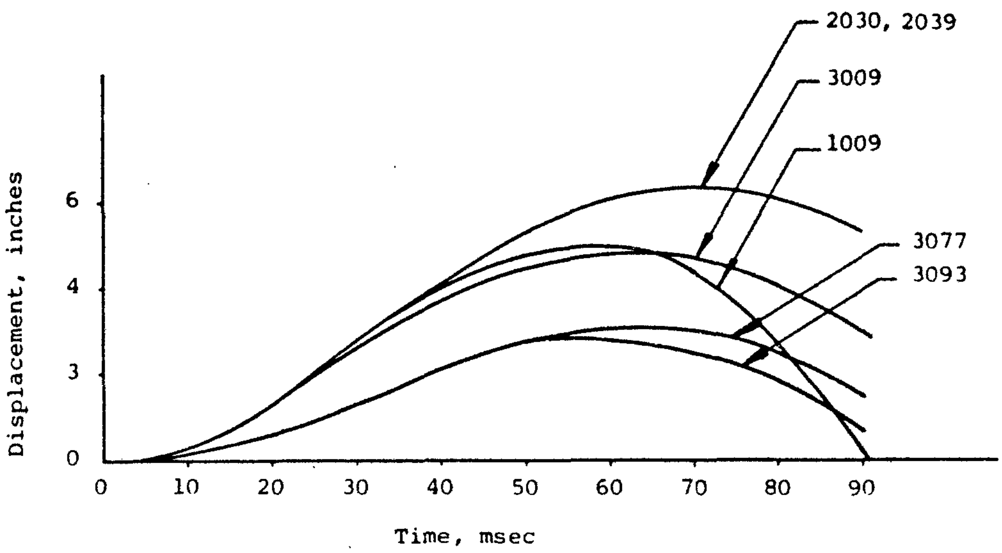


Figure 72. Combined Moment and Thrust at Selected Points in Entry Corridor



a. Points at Crown in Shelter Complex

Figure 73. Displacement Time History



b. Points Along Foundation in Shelter Complex

Figure 73. Displacement Time History (concluded)

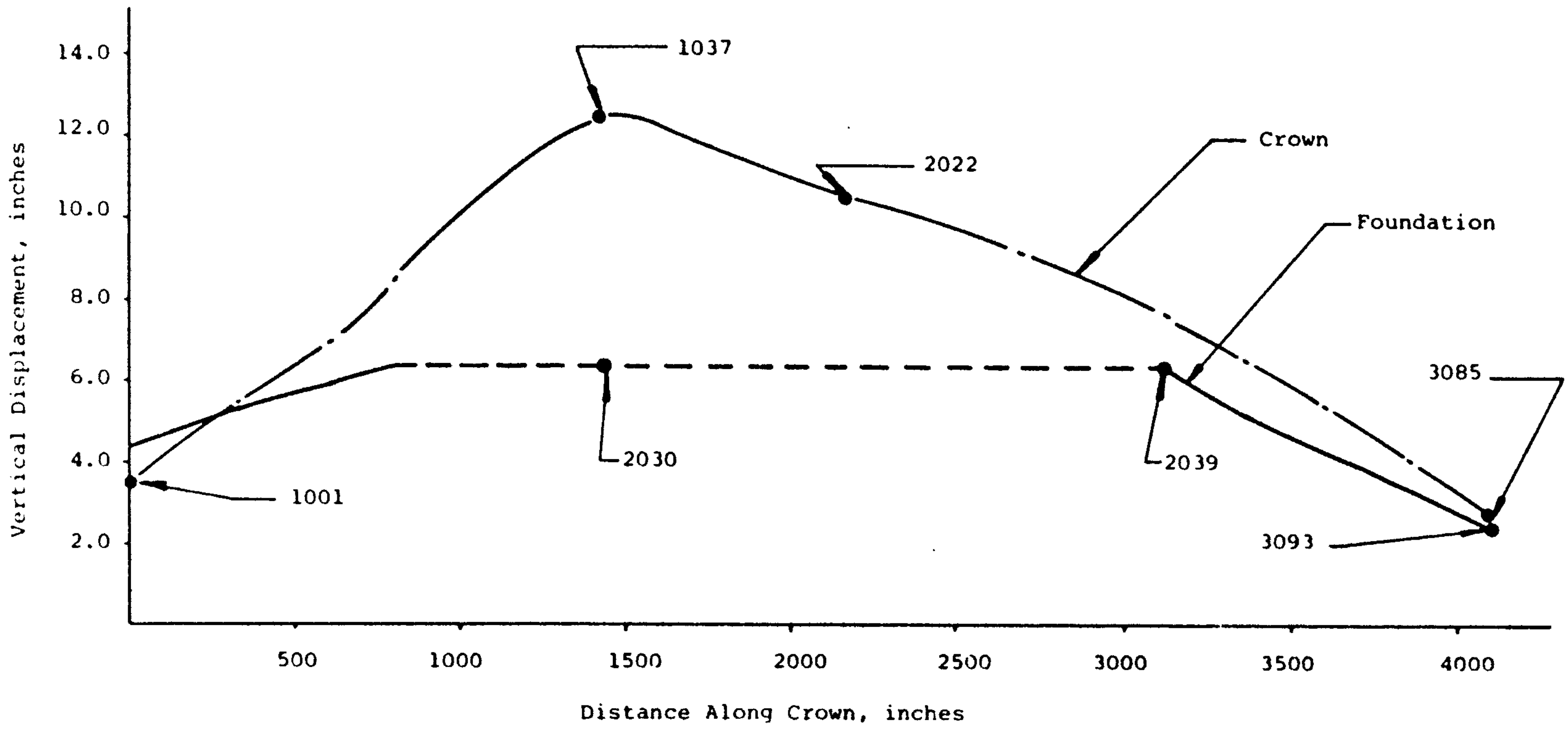


Figure 74. Crown and Foundation Displacements at 70 msec

the crown from the entrance of the main corridor. Points 3085 and 3093 occur at the rear of the shelter bay. At 70 msec the maximum relative vertical displacement in the foundation occurs between points 3093 and 2039. It is equal to 3.9 inches. The maximum absolute vertical displacement occurs at the center of the shelter at node 1037, and it is equal to 14.35 inches.

The NASTRAN analysis still indicates compressive failure in the intersection area of the shell near the bottom of the ribs and the bottom portion of the ribs. Several possible solutions to this problem should be investigated. One choice is to increase the rib depth and width. Another would be to use encased steel sections in both the rib and shell near the rib. The foundation area might also be increased near the springing line of the ribs. Since the shelter and the main corridor sections comprise a major portion of the volume of the entire shelter complex, and these sections remain elastic, no further study was done on the quarter section NASTRAN model. It is recognized that the intersection area near the ribs and the rib section below the points of tangency are critical areas for the shelter loading considered. No further attempt was made to reduce the forces in these critical areas.

The largest principal moment in the shelter foundation was 986.87×10^6 in-lbs. This was an elastic moment that resulted from the forced displacement of a section 192 inches deep with an uncracked moment of inertia of 17,484,768 in^4. Since the first-crack moment (tension failure in concrete) is only 94.7×10^6 in-lbs, the indicated elastic moment is too high for design purposes.

In the absence of a more detailed analysis of the foundation-soil-structure interaction problem, it was decided to design the foundation for a longitudinal bending moment equal to two and one-half times the cracking moment. It is recognized that the ratio of the moment indicated by NASTRAN to the moment capacity provided is approximately 4.2, however, it is believed that a more appropriate analysis would indicate lower moments. The design moment is

$$M = 2.5(94.7 \times 10^6)$$
$$= 236.75$$

Use 14 #11 bars, Grade 60 steel, placed at d = 176 inches

$$M_u = A_s f_y (d - \frac{a}{2})$$

$$a = \frac{P_s f_y d}{0.85 f'_c}$$

$$= \frac{21.84(66000)(176)}{(8244)0.85(4800)}$$

$$= 7.54$$

$$M_u = 21.84(66000)(176 - 3.77)$$

$$= 248.26 \times 10^6 \text{ in-lbs}$$

2. CLOSURE

A model of the beam section chosen for the closure was also subjected to a dynamic analysis using the computer program NASTRAN. The model represented a 12 inch wide vertical strip from the closure (see figure 75). Since the response of the beam element is symmetrical about the midspan of the closure, only the lower half of the strip was modeled. The span of the model was taken equal to one-half of the maximum vertical clear span of the closure. Instead of modeling a single W36 x 135 structural shape, the model represented those portions of two adjacent W36 x 135 shapes between the centerlines of their webs. The front and rear faces were membrane elements 0.794 inch thick representing the beam flanges. The left and right faces were membrane elements 0.299 inch thick representing one-half of each web. The 0.75-inch cover plate on the lower end was also represented using membrane elements. The geometry of the finite element model is illustrated in figure 76. The actual nodal and plate element numbers are not important to the discussion of the analysis and are not presented.

Boundary conditions were applied to the nodes along the upper end of the strip model to represent the plane of symmetry at midspan. The nodes on this plane were restrained to prevent upward or downward motion (motion in the plane of the closure). These nodes were also restrained against rotations in planes parallel to the plane of the side faces. The

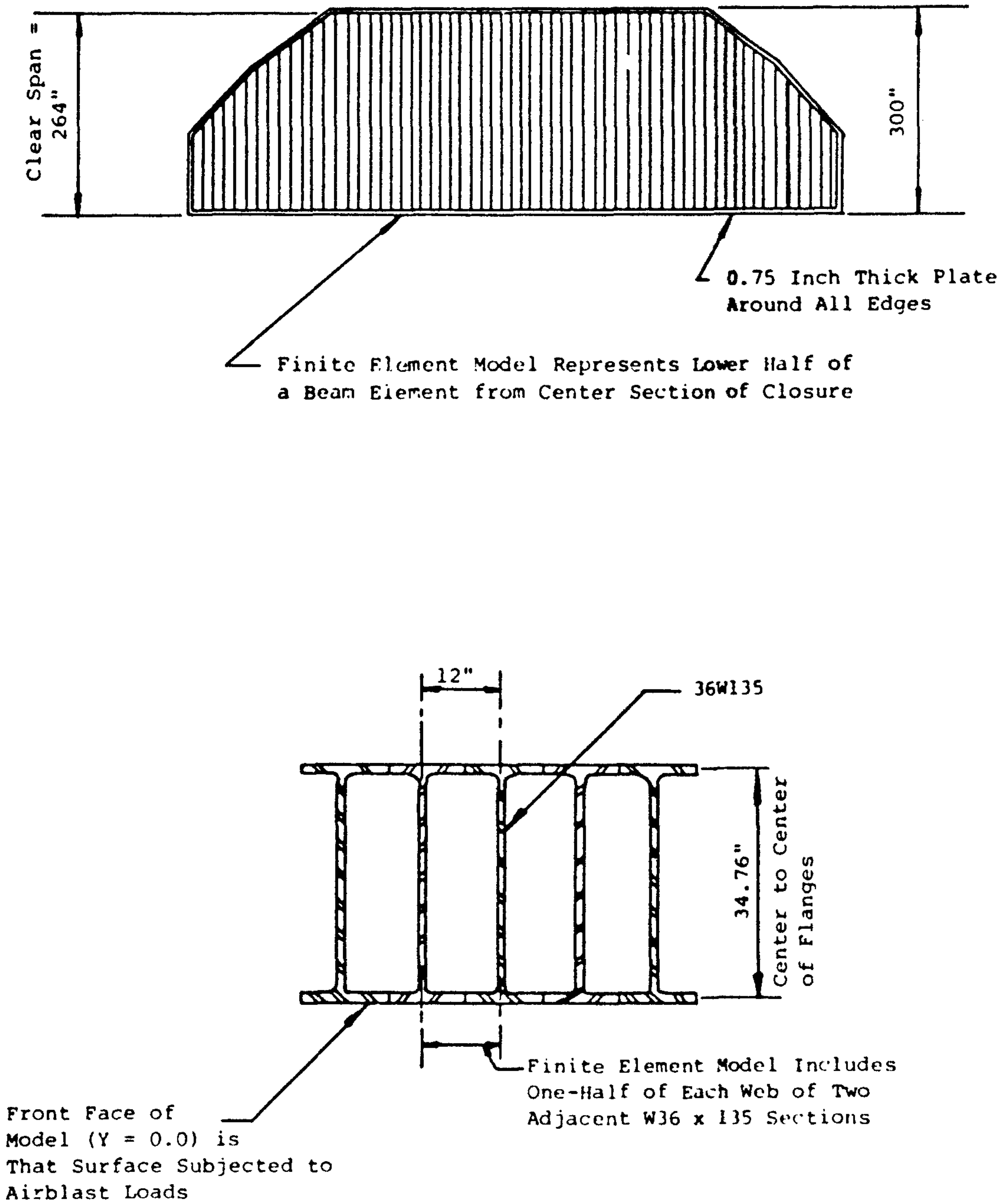
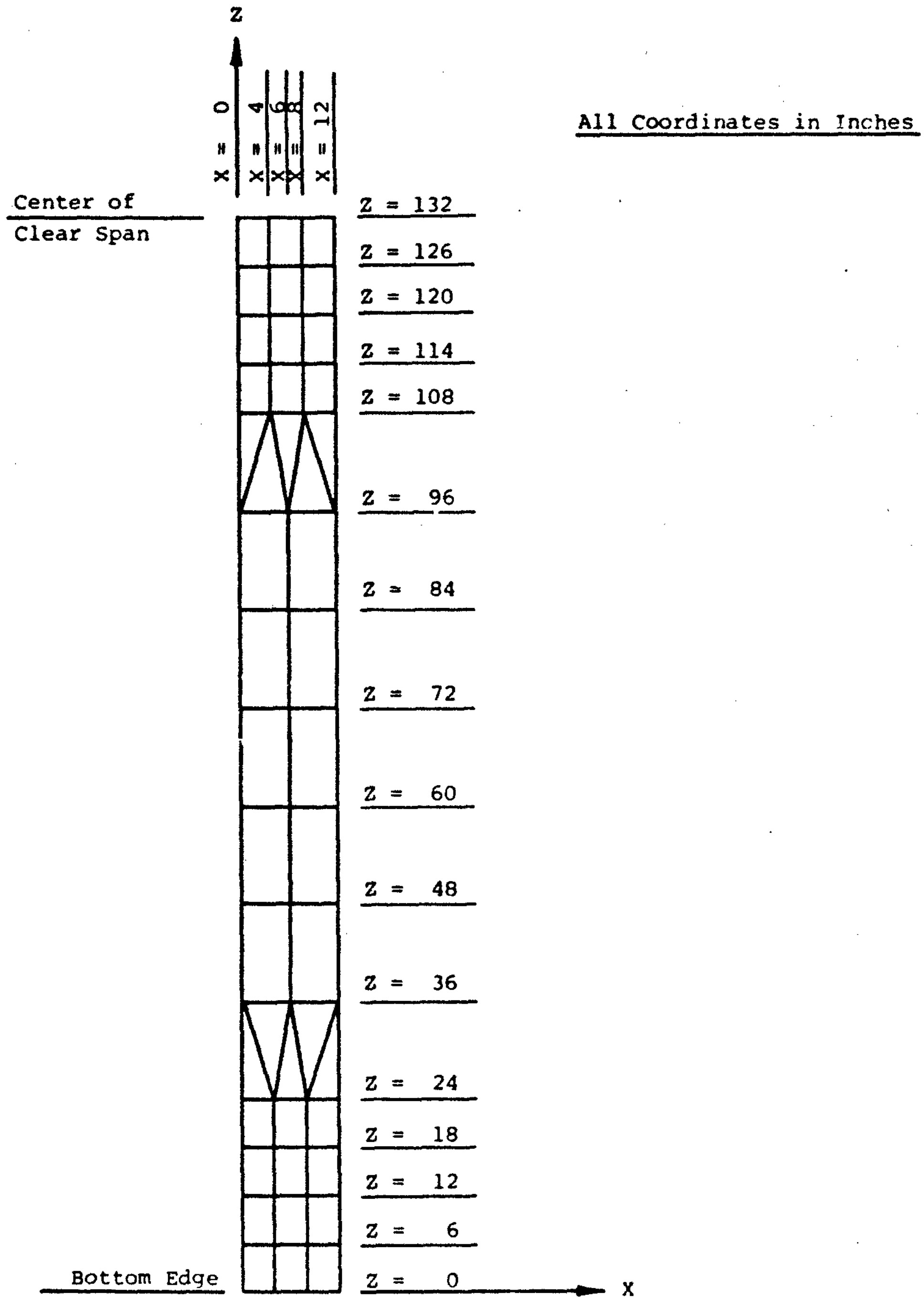
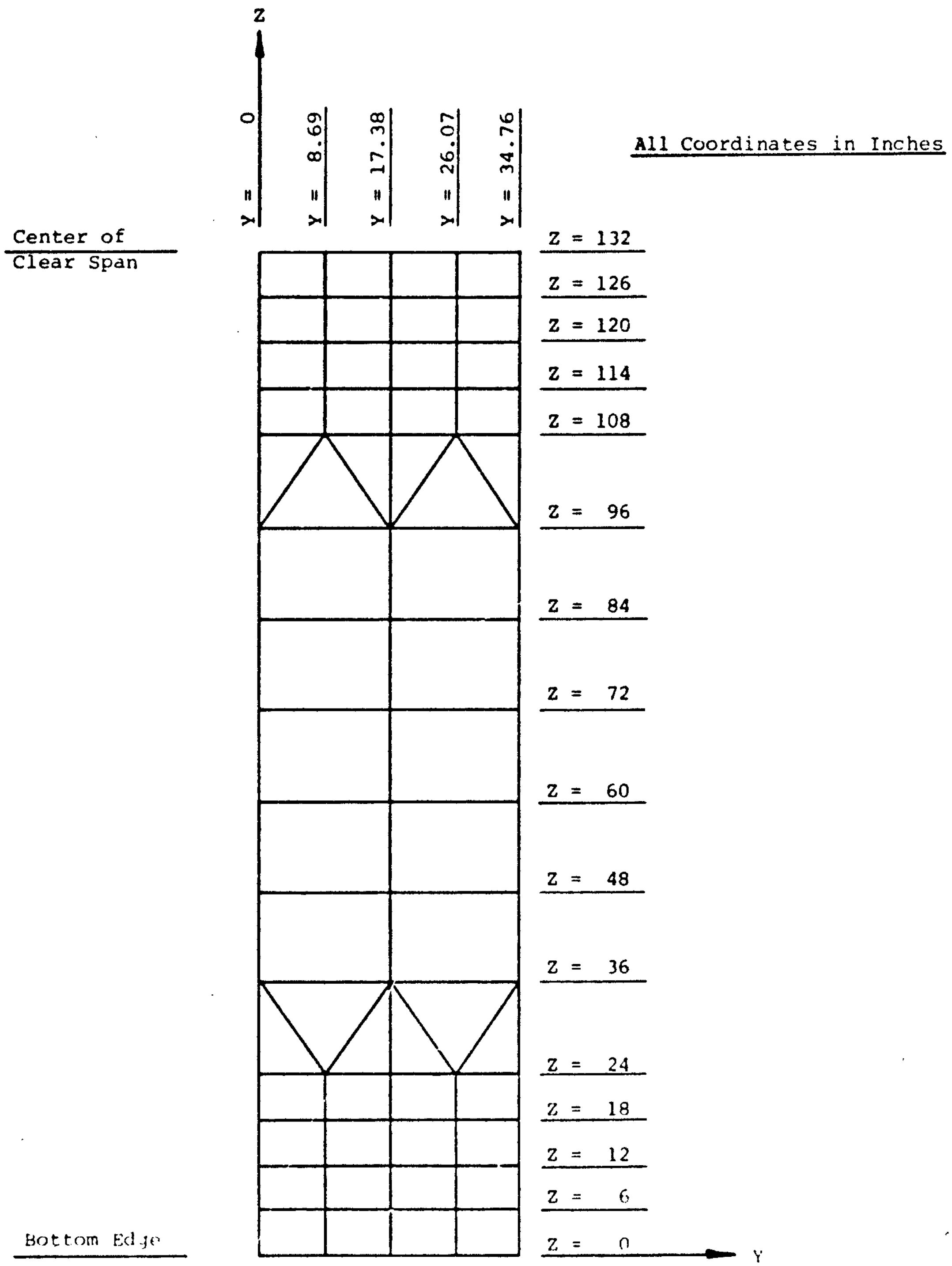


Figure 75. Closure Region Represented by Finite Element Model



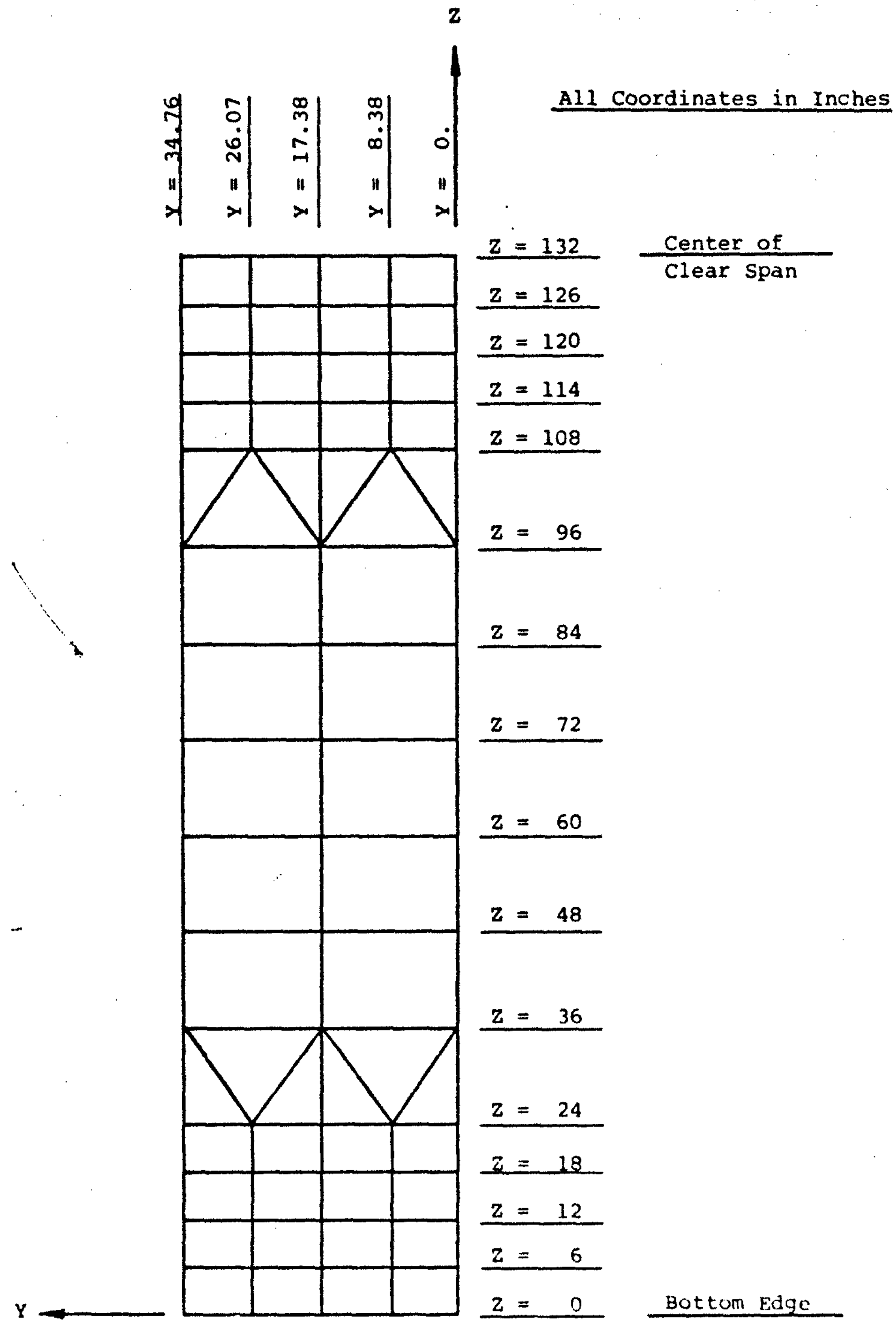
a. Front Face (Y = 0.0 plane)

Figure 76. Finite Element Model of Closure



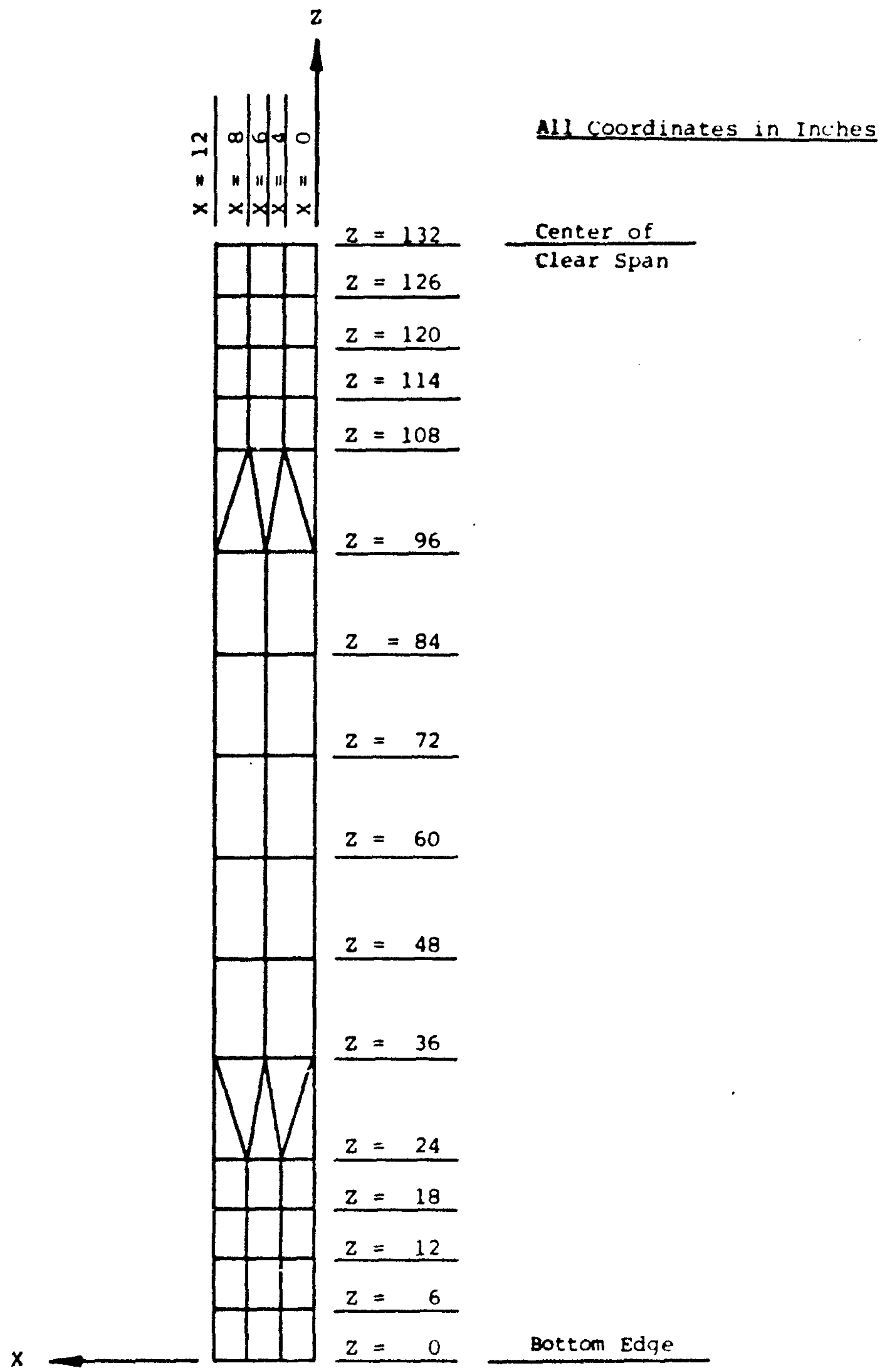
b. Right Face (X = 12. Plane)

Figure 76. Finite Element Model of Closure (continued)



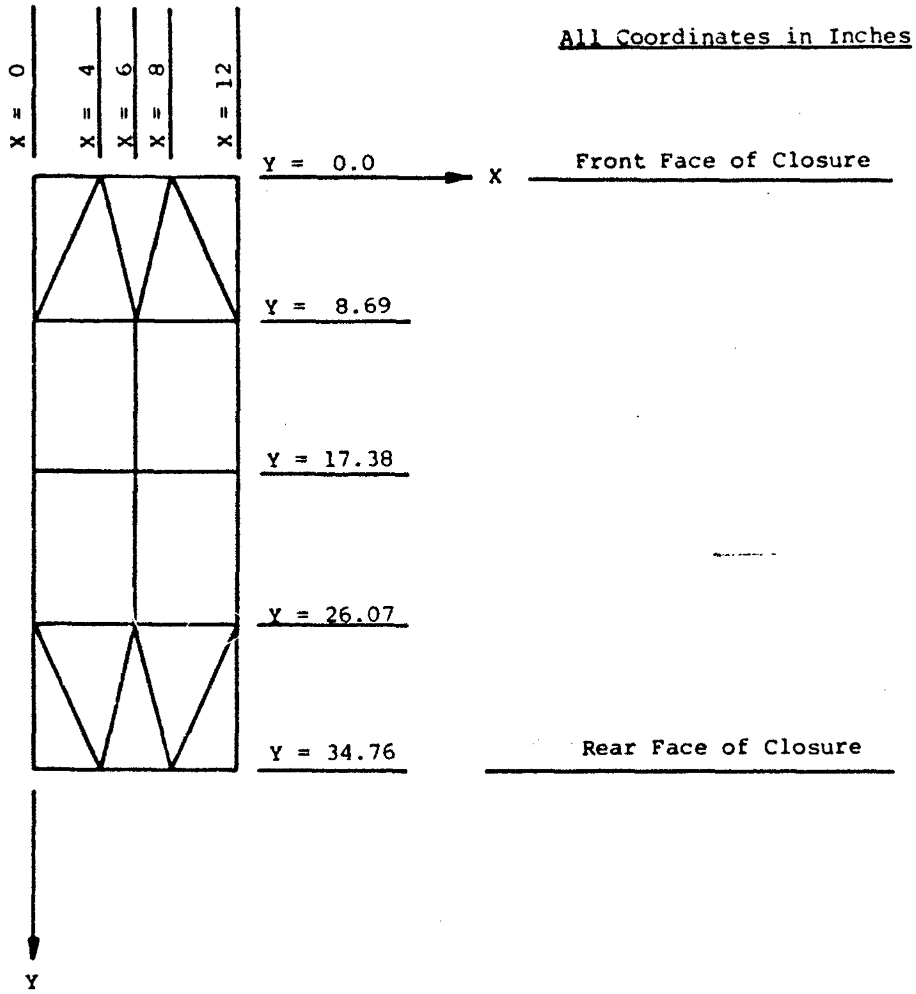
c. Left Face (X = 0.0 Plane)

Figure 76. Finite Element Model of Closure (continued)



d. Rear Face (Y = 34.76 Plane)

Figure 76. Finite Element Model of Closure (continued)



e. Lower End Cover Plate (Z = 0.0 Plane)

Figure 76. Finite Element Model of Closure (concluded)

lower edge boundary conditions represented a simple support with the lowest nodes on the rear face restrained against movement in the direction of the applied load.

The pressure loading function used for the dynamic analysis was derived from earlier dynamic analyses of closures with a grid system of internal stiffeners. These analyses compared the dynamic response of a strip model of the closure to that of a model of the complete upper one-half of the closure. The loading used in the dynamic analysis of the one-half closure model was a blast wave traversing the width of the closure at a velocity corresponding to that of the shock front for 250 psi overpressure. The blast wave had a zero rise time to its peak of 250 psi and an exponential decay to zero pressure. Its duration was that predicted for the weapon yields considered. The half-closure model had a relatively coarse mesh but did provide deflections and stresses in elements for comparison with results from the strip model. The strip model of the closure was constructed with a mesh size approximately equal to the mesh of the one-half closure model. Three different loading functions were applied to the front face of the strip model. Stresses and displacements were obtained for each loading for comparison with results of the one-half closure analysis. The pressure-time histories of the loading functions used, in order of decreasing severity, were

- Zero rise time to 250 psi with an exponential decay.

- Linear rise to 250 psi at 18 msec with an exponential decay.
- Linear rise to 170 psi at 12 msec with an exponential decay.

The first loading function represents the actual free field overpressure-time history. The second adds a rise time equal to the time required for the shock front to travel the length of the closure but retains the 250 psi peak value. The third function was obtained by plotting a linear rise to 250 psi at 18 msec onto a plot of the first function and solving for the point of intersection. The two curves intersected at 170 psi and 12 msec.

All three of these loading functions yielded displacements and stress levels for the strip model which were greater than those from the one-half closure model analysis. The results from the first loading function were much larger than those from the one-half closure model. Results from the last two loading functions showed closer agreement with the one-half closure model. The second loading function yielded only slightly more conservative results than the third. From these comparisons, it was concluded that the second loading function provided the most realistic results in terms of predicting the response of the closure from a simple strip model. The results are not felt to be so overly conservative that the closure is grossly overdesigned.

The second blast loading function was then applied to the mathematical model of the closure constructed from W36 x 150 structural shapes and the dynamic response obtained. Results of that analysis were

- Peak Displacement = 0.921 inch
- Peak Horizontal Shear = 24.1 ksi (web)
- Max Principal Shear = 26.0 ksi (web)
- Peak Vertical Tension = 64.7 ksi (flange)
- Max Principal Tension = 64.7 ksi (flange)
- Peak Average Support Force = 35.5 kip/in

All of these results are within the allowable values and the closure should remain elastic. Buckling of members cannot be checked with the finite element model. Hand calculations in section VIII showed that buckling should not occur for the stress levels predicted there. Since the peak deflection and stress levels in this section are almost identical to the previous hand computations, buckling should not be a problem.

SECTION X

PREDICTED GROUND SHOCK AND RIGID BODY SHELTER MOTIONS

1. INTRODUCTION

The combined effects of airblast, crater-induced ground motions, and outrunning ground motions, subject an aircraft in a buried shelter to a combination of vertical and horizontal ground motions. The purpose of this investigation was to predict the shock environment observed by the aircraft in the shelter and determine whether such motions were sufficient to cause damage to the aircraft.

Shock tolerances are presented in table 11 for four of the aircraft which might be housed in the aircraft shelter. Corresponding data on the other aircraft being considered in the overall shelter conceptual design were unavailable. In most cases peak allowable accelerations in fore-aft, lateral, and vertical directions were specified. In one case, the acceleration-time history was also supplied. All peak allowable accelerations listed in table 11 apply to aircraft in a parked configuration. As might be expected, aircraft are more vulnerable to motions in the lateral direction. This vulnerability is due to the fact that aircraft landing gear are not designed to withstand high side-loads.

Approximate peak free-field ground motions are known for the given threat. However, it is not exactly known how these ground motions interact with the shelter structure. If rigid-body displacements of 2 feet are applied extremely slowly,

Table 11

AIRCRAFT GROUND SHOCK LIMITS

AIRCRAFT	PEAK ACCELERATION	NOMINAL DURATION
F-15	Fore & Aft : ± 8g Lateral : ± 0.5g Vertical : ± 2.5g	Not Specified
F-16	Fore & Aft : 40g Lateral : 15g Vertical : 20g	11 millisec duration, half-sine pulse
A-7D	Fore & Aft : ± 9g Lateral : ± 3g Vertical : + 10.5g (up) - 4.5g (down)	Not Specified
F-111	Fore & Aft : Not Specified Lateral : 0.5g Vertical : 2.0g	Not Specified

the aircraft could move with the structure without causing excessive bending stresses in the landing gear. However, if the rise time to this peak displacement is short, significant landing gear bending stresses could result.

For soil response due to rapidly applied loads, the shelter structure will not directly follow the free-field soil motions. There will be a complex structure-medium interaction, and it is important to determine motion of the structure itself.

In order to simplify the analysis it was assumed that the structure was a rigid body capable only of rigid body translation. Thus, higher modes of response were neglected. It was also assumed that the ground shock can be partitioned into three different types:

- Airblast ground shock
- Outrunning ground shock
- Crater-induced ground shock

The rise times of both outrunning and crater-induced ground shock are normally sufficiently long so that, for most situations, the structure will follow the free field ground motions. As a result, calculations of horizontal and vertical motions of the structure due to these types of ground shock can be based upon free field values. Such an assumption cannot be made for overhead airblast induced ground shock.

Two ground geologies were considered. Both were representative of a homogeneous wet clay geology with water table depths of 1 meter and 20 meters. For the 1-meter water table, the effective seismic velocity, C, was assumed to be 5500 ft/sec. For the 20-meter water table, the seismic velocity was assumed to be 2500 ft/sec. In both cases, a unit weight of 110 lbs/ft^3 was used.

For the 1-meter water table geology, the geology was assumed to be uniform. The 20-meter water table geology was effectively a two layer geology due to the increase in compression wave velocity below the water table. A study of the time of arrival of the ground shock from various sources indicated that the site is superseismic for both geologies considered. As a result, outrunning ground motions were not considered.

2. CRATER INDUCED GROUND MOTION

Waveforms for crater-induced ground motions were approximated using the trapezoidal horizontal velocity pulse shown in figure 77. All calculations were based on the largest nuclear yield in the specified weapon threat. In order to determine the actual velocity trace it was necessary to obtain the peak horizontal crater-induced velocity, v_h, as well as the time, t_p, to peak displacement. This peak velocity is given by

$$v_h = 0.01C_e\left[\frac{R}{V_a^{1/3}}\right]^{-2}$$

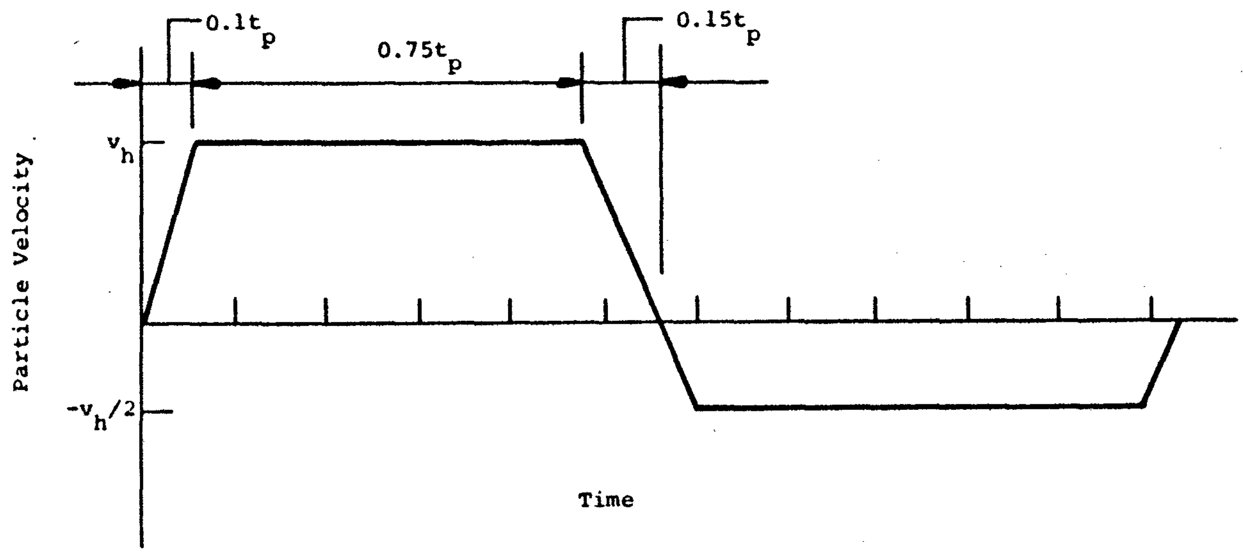


Figure 77. Approximate Crater-Induced Horizontal Particle Velocity Pulse

where

C_e = effective wave velocity

V_a = apparent crater volume

The effective wave velocity for the crater-induced signal was determined to be 5500 fps for the 1-meter and 4700 fps for the 20-meter water table geologies, respectively.

Procedures described in reference 2 were used to calculate apparent crater volumes for the two site geologies. Using these crater volumes and a range corresponding to the 250 psi contour, the following peak crater induced velocities were calculated

v_h = 2.88 ft/sec for 1-meter water table

v_h = 1.88 ft/sec for 20-meter water table

In a similar fashion the time to peak displacement for the crater-induced ground motion was calculated using

$$t_p = \frac{50}{C_e}\left(\frac{V_a^{2/3}}{R}\right)$$

Substituting appropriate values of C_e and V_a, the times were

t_p = 0.706 sec for 1-meter water table

t_p = 0.631 sec for 20-meter water table

The resulting horizontal crater-induced velocity-time traces for the two geologies are presented in figure 78. Corresponding crater-induced displacement time histories for the two geologies were then determined by graphical integration. These displacement curves are presented in figure 79.

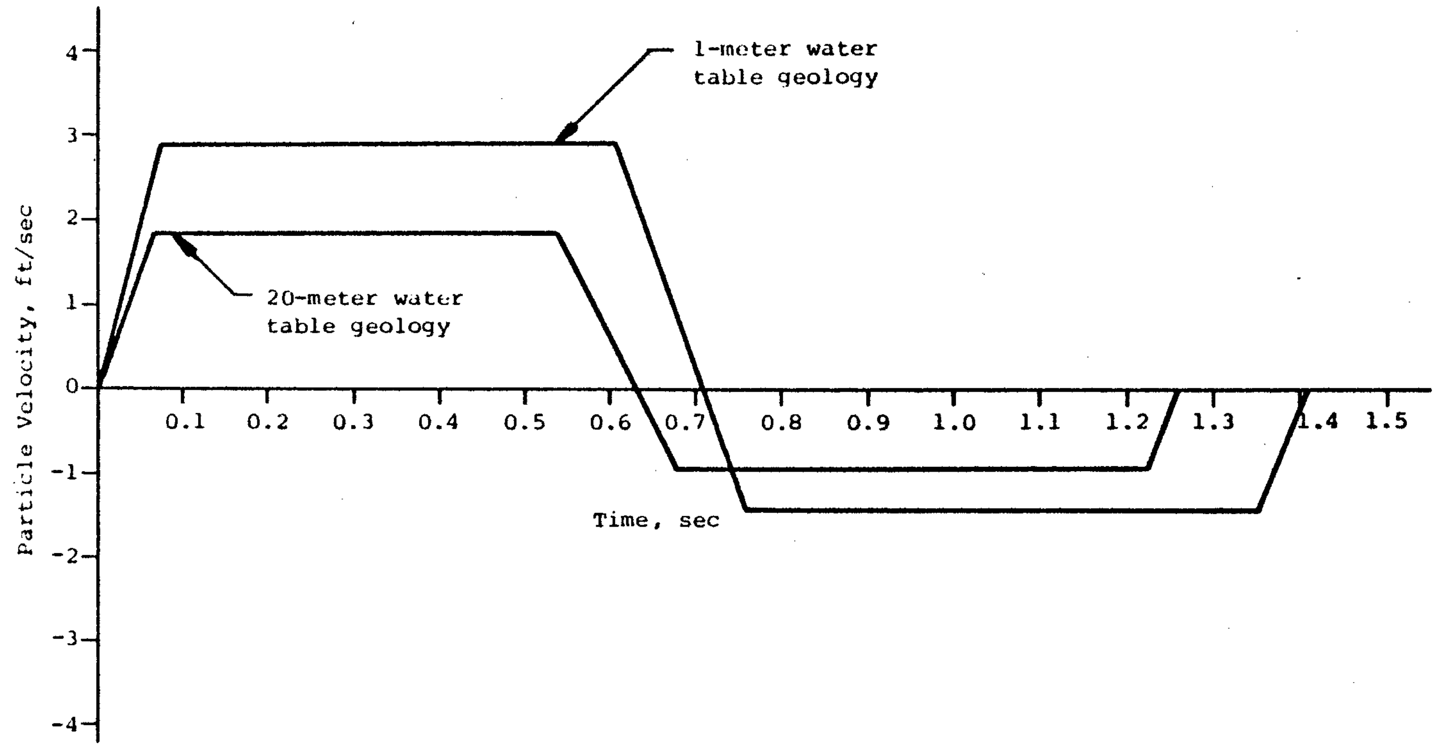


Figure 78. Horizontal and Vertical Crater-Induced Particle Velocity as a Function of Time

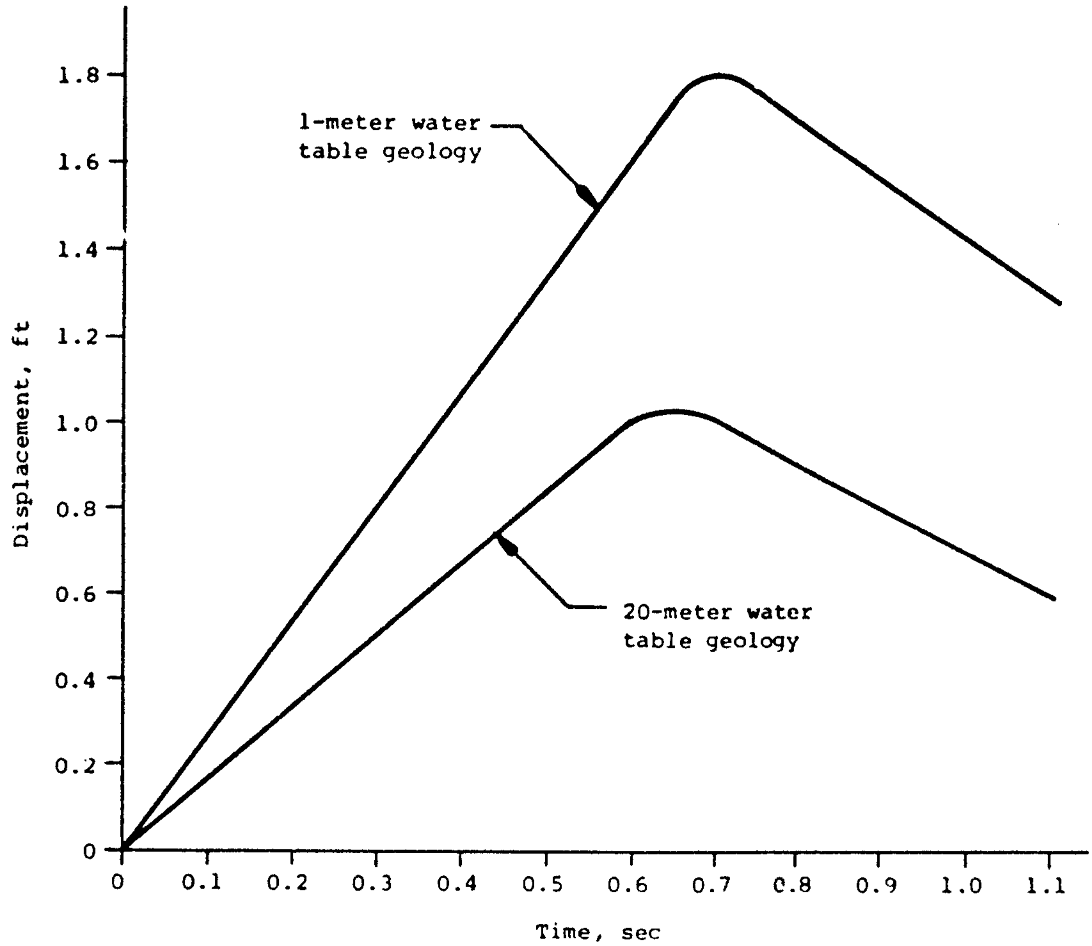


Figure 79. Horizontal and Vertical Crater-Induced Displacement as a Function of Time

3. VERTICAL AIRBLAST MOTIONS

One-dimensional methods presented in reference 2 were utilized to calculate vertical motions induced by overhead airblast.

Figure 80 shows a rectangular structure buried in an elastic soil half-space. It was assumed that the structure is buried at a depth sufficient to ensure that free surface reflections do not influence the response during the time of interest.

The stress on the roof of the structure is given by

$$\sigma_t(t) = 2\sigma_{ff}(t) - \rho C_L v(t)$$

where

$\sigma_t(t)$ = total stress acting on roof
$\sigma_{ff}(t)$ = incident stress wave in the free field
ρ = mass density of the soil
C_L = compressional wave velocity of the soil
$v(t)$ = velocity of the structure

The stress acting on the base of the structure is

$$\sigma_r(t) = \rho C_L v(t)$$

Neglecting shearing stresses on vertical surfaces results in the following equation of motion for vertical motions

$$M\dot{v}(t) + \rho C_L(A_1 + A_2)v(t) = 2A_1\sigma_{ff}(t)$$

where A_1 and A_2 denote, respectively, the projected roof and foundation bearing areas.

The free field stress history, $\sigma_{ff}(t)$, was assumed to

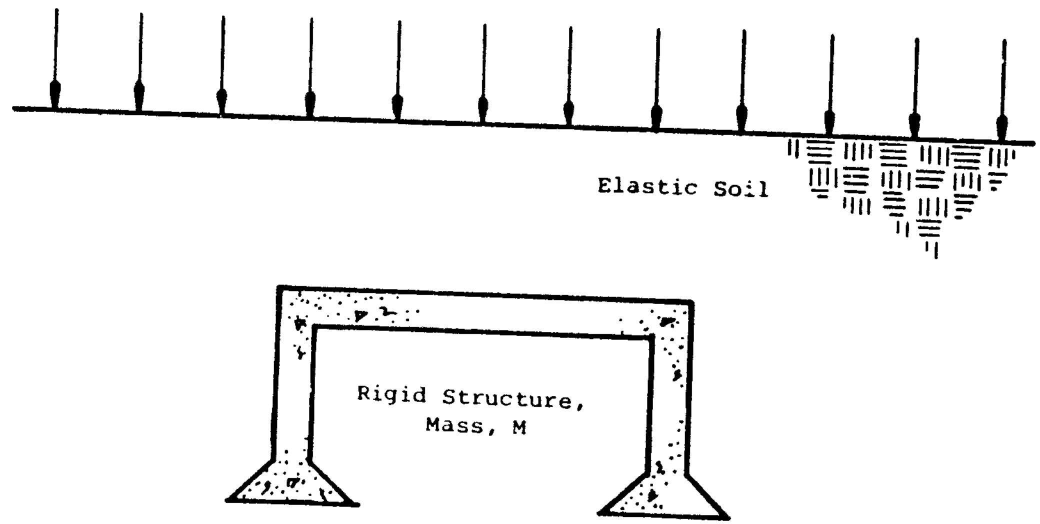


Figure 80. Buried Structure Idealization

be identical to the free field side-on airblast overpressure. This free field airblast overpressure-time behavior was approximated by an exponential pulse of the form

$$p(t) = P_o e^{-\alpha t}$$

fitted to a more exact relationship presented in reference 2. P_o is the peak side-on overpressure.

The equation of motion was then rewritten in terms of displacements and a general free field driving function

$$\sigma_{ff} = \sigma_o e^{-\alpha t}$$

obtaining

$$\ddot{x}(t) + \beta\dot{x}(t) = \gamma e^{-\alpha t}$$

where

$$\beta = \frac{\rho C_L (A_1 + A_2)}{M}$$

$$\gamma = \frac{2A_1\sigma_o}{M}$$

The general solutions of the equation of motion for rigid body shelter displacement, x(t), velocity, v(t), and acceleration, a(t), are

$$x(t) = \gamma\left[\frac{1}{\alpha\beta} + \frac{\beta e^{-\alpha t} - \alpha e^{-\beta t}}{\alpha\beta(\alpha-\beta)}\right]$$

$$v(t) = \frac{\gamma}{(\alpha-\beta)}\left[e^{-\beta t} - e^{-\alpha t}\right]$$

and

$$a(t) = \frac{\gamma}{(\alpha-\beta)}\left[\alpha e^{-\alpha t} - \beta e^{-\beta t}\right]$$

These general solutions were then evaluated for the specific airblast parameters of the problem as well as the two geologies. An approximating exponential function,

$$p(t) = 250e^{-16.09t}$$

was derived from the actual pressure-time history for the maximum specified nuclear weapon yield. Since the free-field soil stress was assumed to vary as the peak overpressure, it was taken as

$$\sigma_{ff}(t) = 250e^{-16.09t}$$

Considering one hangar of the shelter separately, other pertinent input parameters for the two geologies were then as follows.

A_1 = 8500 sq ft

A_2 = 3622 sq ft

M = 3.53×10^5 slugs

ρ = 3.42 slugs/cu ft (both geologies)

C_L = 2750 ft/sec for 1-meter water table (assuming $C_L = 0.5C$)

= 1250 ft/sec for 20-meter water table (assuming $C_L = 0.5C$)

α = 16.09 sec^{-1}

σ_o = 250 psi = 36000 lb/ft^2

From the above parameters, γ = 1733 ft/sec^2 for both geologies and β = 323 sec^{-1} and 146.8 sec^{-1} for the 1-meter and 20-meter water tables, respectively.

Substituting these parameters in the equations of motion resulted in the vertical displacement-time, velocity-time and acceleration-time histories shown in figures 81 through 83.

4. HORIZONTAL AIRBLAST MOTIONS

Horizontal motions of the buried shelter due to the sweeping effect of the airblast wave were next considered. First, the problem was restricted to horizontal motions only. A free body diagram of the structure is shown in figure 84. Note that only horizontal loads are indicated. The loads at early times on the vertical front face toward the burst point are due to the airblast shock wave propagating downward. This wave is propagating nearly vertically downward so that reflections were neglected. It was also assumed that the loading was caused by the free field soil stress. The loading on the vertical (rear) face away from the burst point is initially reactive until the ground shock from the overhead airblast reaches that face. On the bottom and lateral faces of the structure, reactive shear forces act.

The equation of horizontal rigid body motion becomes

$$Mx(t) = F_H^{(1)}(t) - F_H^{(2)}(t) - F_S(t)$$

where

$F_H^{(1)}(t)$ = the total horizontal force due to ground shock acting on the front face.

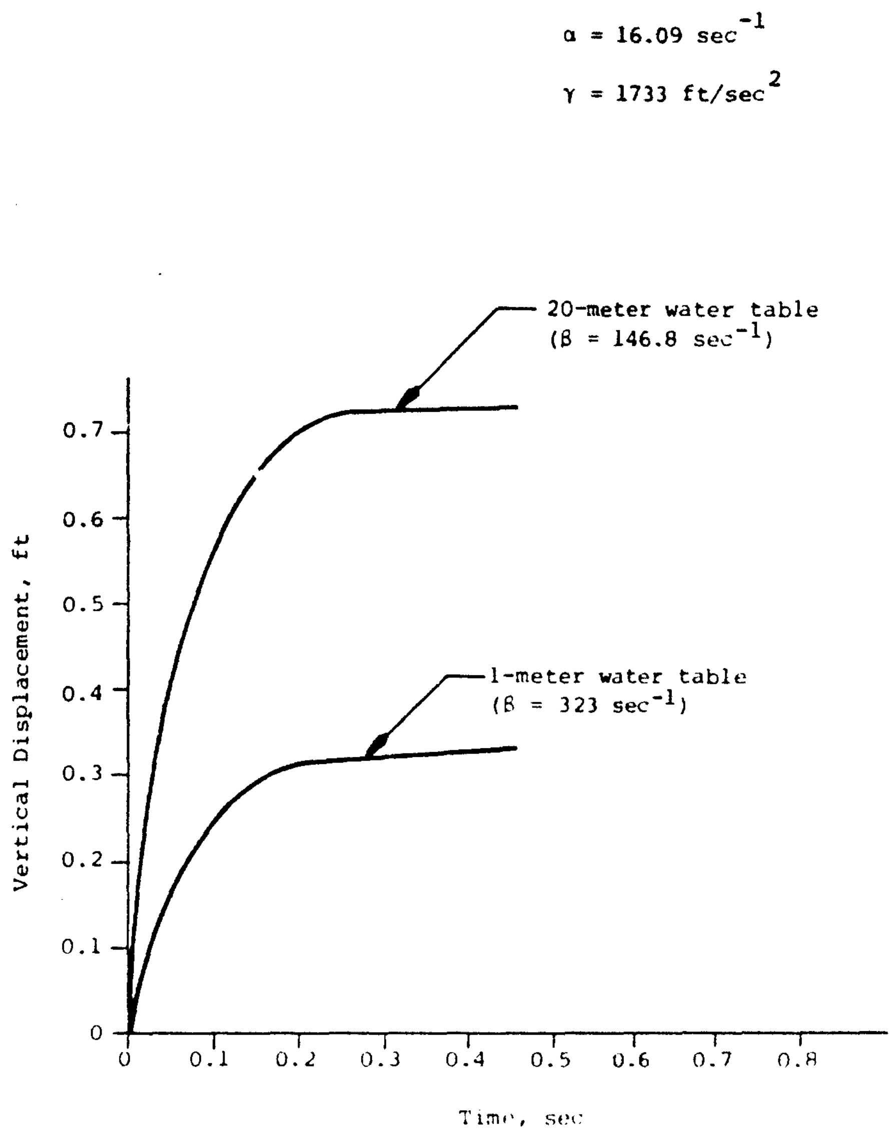


Figure 81. Vertical Displacement as a Function of Time Due to Overhead Airblast Loading

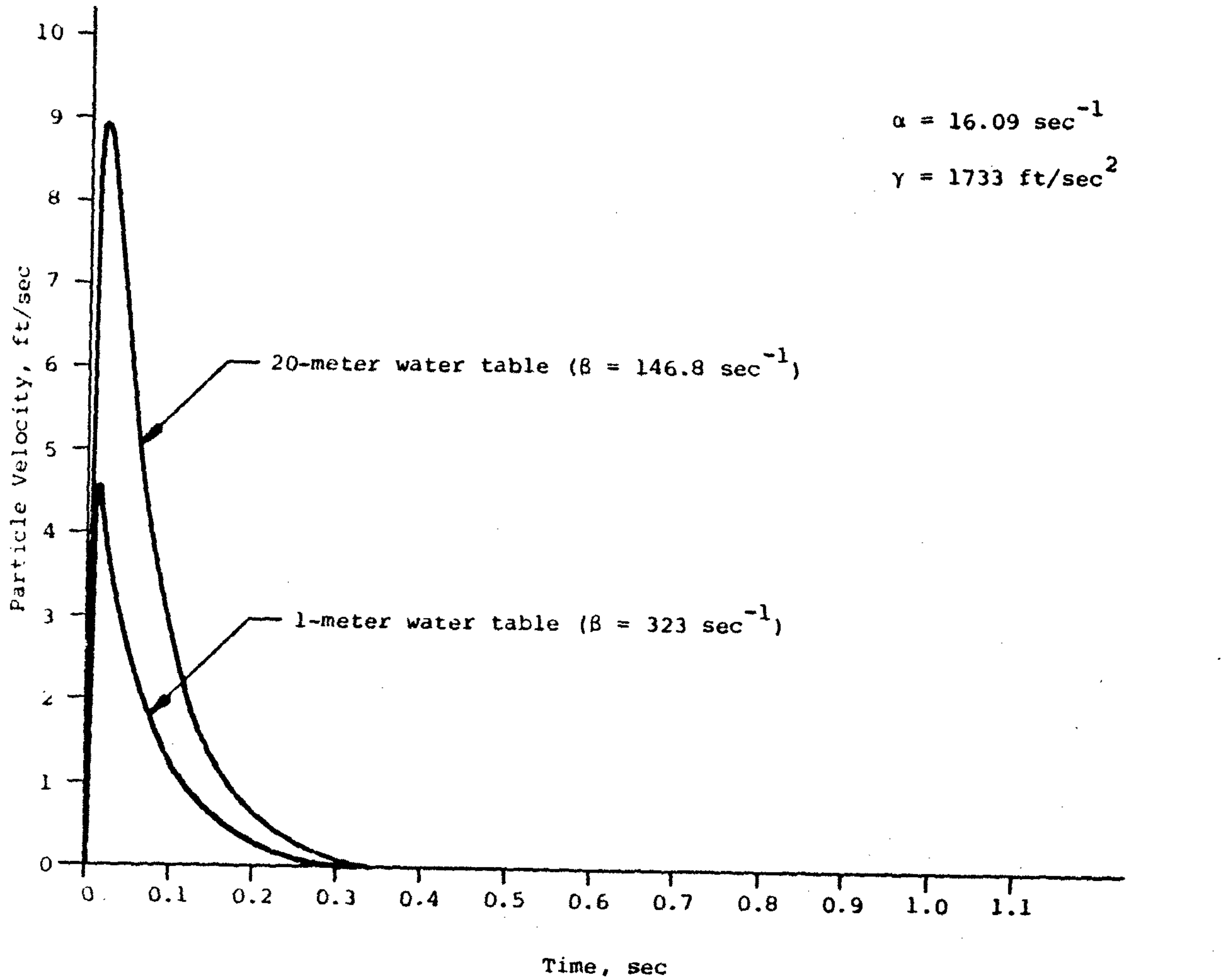


Figure 82. Vertical Shelter Velocity as a Function of Time Due to Overhead Airblast Loading

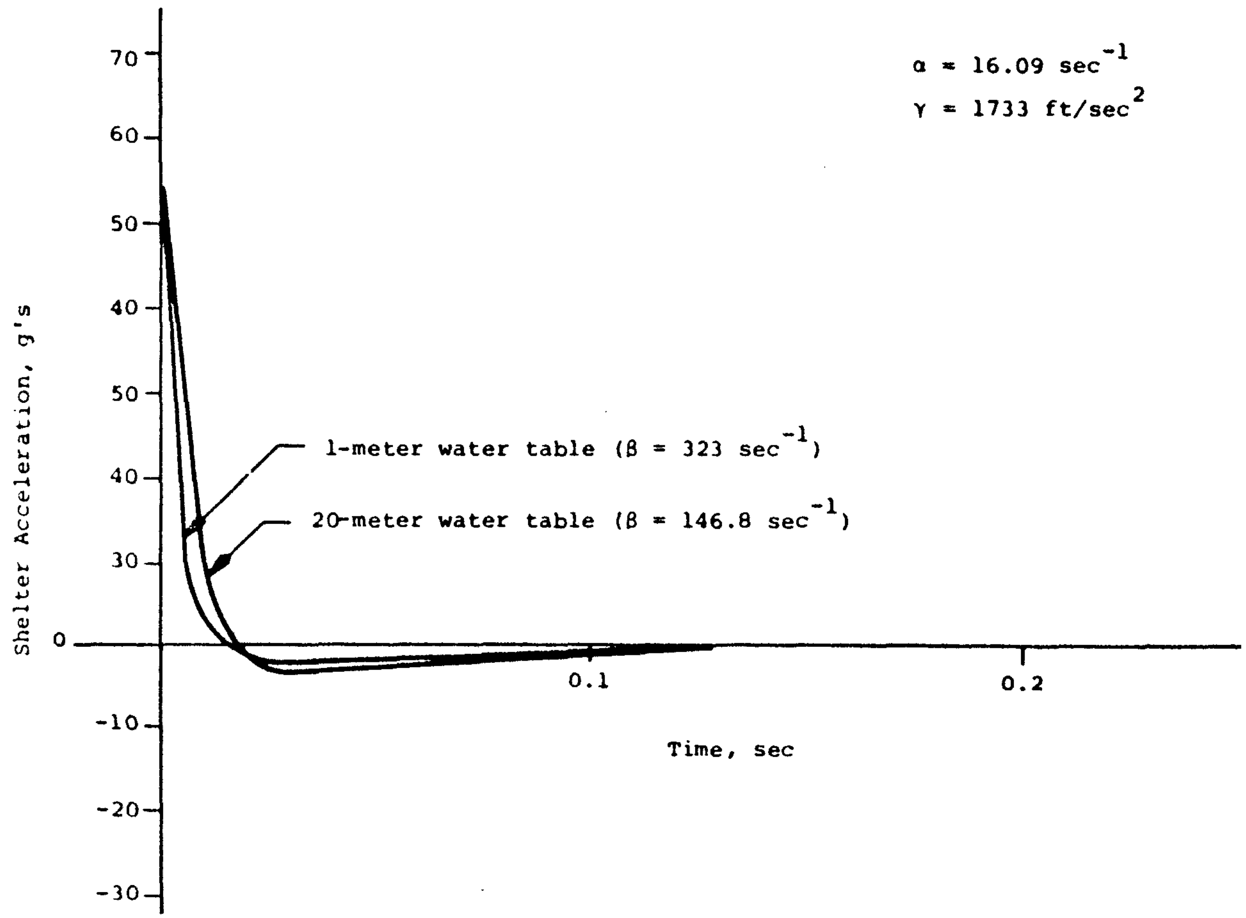


Figure 83. Vertical Shelter Rigid Body Acceleration as a Function of Time Due to Overhead Airblast Loading

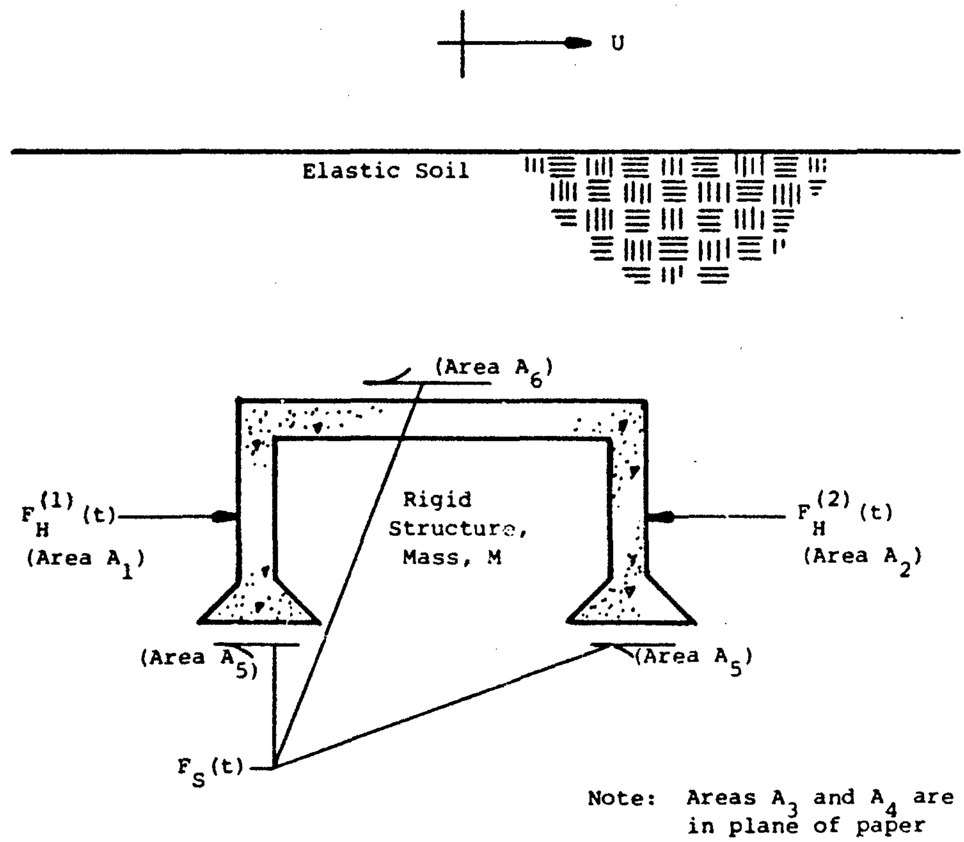


Figure 84. Horizontal Loading of Buried Structure

$F_H^{(2)}(t)$ = the total horizontal force acting on the rear face. This force is a combination of initial reactive force and appropriately delayed ground shock.

M = mass of the rectangular structure.

F_S = the total shear force acting on the sides, projected roof area, and base of the structure.

The total force acting on the front face was taken as

$$F_H^{(1)}(t) = K_o A_1 \sigma_{ff}(t) - \rho C_L v_H(t)$$

where

K_o = coefficient of lateral earth pressure

A_1 = area of the front face

$\sigma_{ff}(t)$ = free field vertical stress history at the mid-depth of the structure

ρ = soil density

C_L = loading wave velocity

$v_H(t)$ = rigid-body horizontal velocity of structure

The free field stress history, $\sigma_{ff}(t)$, was again assumed to be identical to the free field side-on airblast overpressure, and was determined in the previous section to be of the general form

$$\sigma_{ff}(t) = \sigma_o e^{-\alpha t}$$

The horizontal force on the rear face consists of two components.

- A reactive (or radiative) component due to rigid body motion of the structure.

- A component due to airblast. This component is appropriately delayed by the airblast transit time over the structure, L/U, where L is the length of the structure and U is the airblast shock front velocity.

This force then becomes (consistent with the sign convention of figure 69)

$$F_H^{(2)}(t) = C_L v_H(t) A_2 + K_o A_2 \sigma_{ff}\left(t - \frac{L}{U}\right) H\ t - \left(\frac{L}{U}\right)$$

where $H\left(t - \frac{L}{U}\right)$ is the Heaviside step function, and A_2 is the area of the back (reactive) face of the structure. For 250 psi peak overpressure and sea level conditions, the airblast shock front velocity is 4400 fps.

Determination of the total shear, $F_s(t)$, acting on the structure is somewhat subtle. It is approximated here as the total area of the base, roof, and two sides multiplied by a shear stress,

$$\tau_s(t) = \rho C_s v_H(t)$$

where C_s is the shear wave velocity in the soil. The resulting equation of horizontal motion becomes

$$\begin{aligned} M\ddot{x}(t) = {} & \left[K_o \sigma_{ff}(t) - \rho C_L \dot{x}(t)\right] A_1 \\ & - \left[\rho C_L \dot{x}(t) + K_o \sigma_{ff}\left(t - \frac{L}{U}\right) H\left(t - \frac{L}{U}\right)\right] A_2 \\ & - \left[\rho C_s \dot{x}(t)\right]\left[(A_3 + A_4 + A_5 + A_6\right] \end{aligned}$$

where A_3, A_4, A_5, and A_6 denote, respectively, the areas of the base, the two lateral sides, and the roof of the structure.

For a rectangular structure, $A_1 = A_2$ and $A_4 = A_5$, so the equation of motion can be simplified to

$$\ddot{x}(t) + B_1\dot{x}(t) = B_2 e^{-\alpha t} - B_2 e^{-\alpha(t-\beta)} H(t-\beta)$$

where

$$B_1 = \frac{2\rho C_L A_1 + \rho C_s \left(A_3 + 2A_4 \div A_6\right)}{M} , \quad \beta = \frac{L}{U}$$

and

$$B_2 = \frac{\sigma_o K_o A_1}{M}$$

For a structure initially at rest, i.e., $x(0) = \dot{x}(0) \equiv 0$, the solution for rigid body displacement as a function of time is found to be

$$x(t) = B_2 \left\{ \left[\frac{1}{\alpha B_1} + \frac{B_1 e^{-\alpha t} - \alpha e^{-B_1 t}}{\alpha B_1 (\alpha - B_1)} \right] - \left[\frac{1}{\alpha B_1} + \frac{B_1 e^{-\alpha(t-\beta)} - \alpha e^{-B_1(t-\beta)}}{\alpha B_1 (\alpha - B_1)} \right] H(t-\beta) \right\}$$

The horizontal velocity is found

$$v(t) = \frac{B_2}{(\alpha - B_1)} \left\{ \left[e^{-B_1 t} - e^{-\alpha t} \right] - \left[e^{-B_1(t-\beta)} - e^{-\alpha(t-\beta)} \right] H(t-\beta) \right\}$$

and the acceleration is

$$a(t) = \frac{B_2}{(\alpha - B_1)} \left\{ \left[\alpha e^{-\alpha t} - B_1 e^{-B_1 t} \right] - \left[\alpha e^{-\alpha(t-\beta)} - B_1 e^{-B_1(t-\beta)} \right] H(t-\beta) \right\}$$

Horizontal displacement, velocity and acceleration as a function of time are plotted in figures 85 through 87 for both geologies. The following parameter values were used in the motion calculations. Again, values of parameters are based on consideration of one hangar separately.

$M = 3.53 \times 10^5$ slugs

$\rho = 3.42$ slugs/cu ft (both geologies)

$C_L = 2750$ ft/sec for 1-meter water table
1250 ft/sec for 20-meter water table

$C_S = 2062$ ft/sec for 1-meter water table
937 ft/sec for 20-meter water table

$\sigma_o = 36{,}000$ lb/ft^2

$L = 110$ ft

$U = 4400$ ft/sec

$K_o = 1$ for 1-meter water table
1/2 for 20-meter water table

$A_1 = A_2 = 3960$ sq ft

$A_3 = 3622$ sq ft

$A_4 = A_5 = 3096$ sq ft

$A_6 = 8500$ sq ft

$\alpha = 16.09$ sec^{-1}

Values of other pertinent parameters appearing in the equations of motion were

$B_1 = 577.0$ sec^{-1} for 1-meter water table
262.0 sec^{-1} for 20-meter water table

$B_2 = 404.0$ ft/sec^2 for 1-meter water table
202.0 ft/sec^2 for 20-meter water table

$\beta = 0.025$ sec

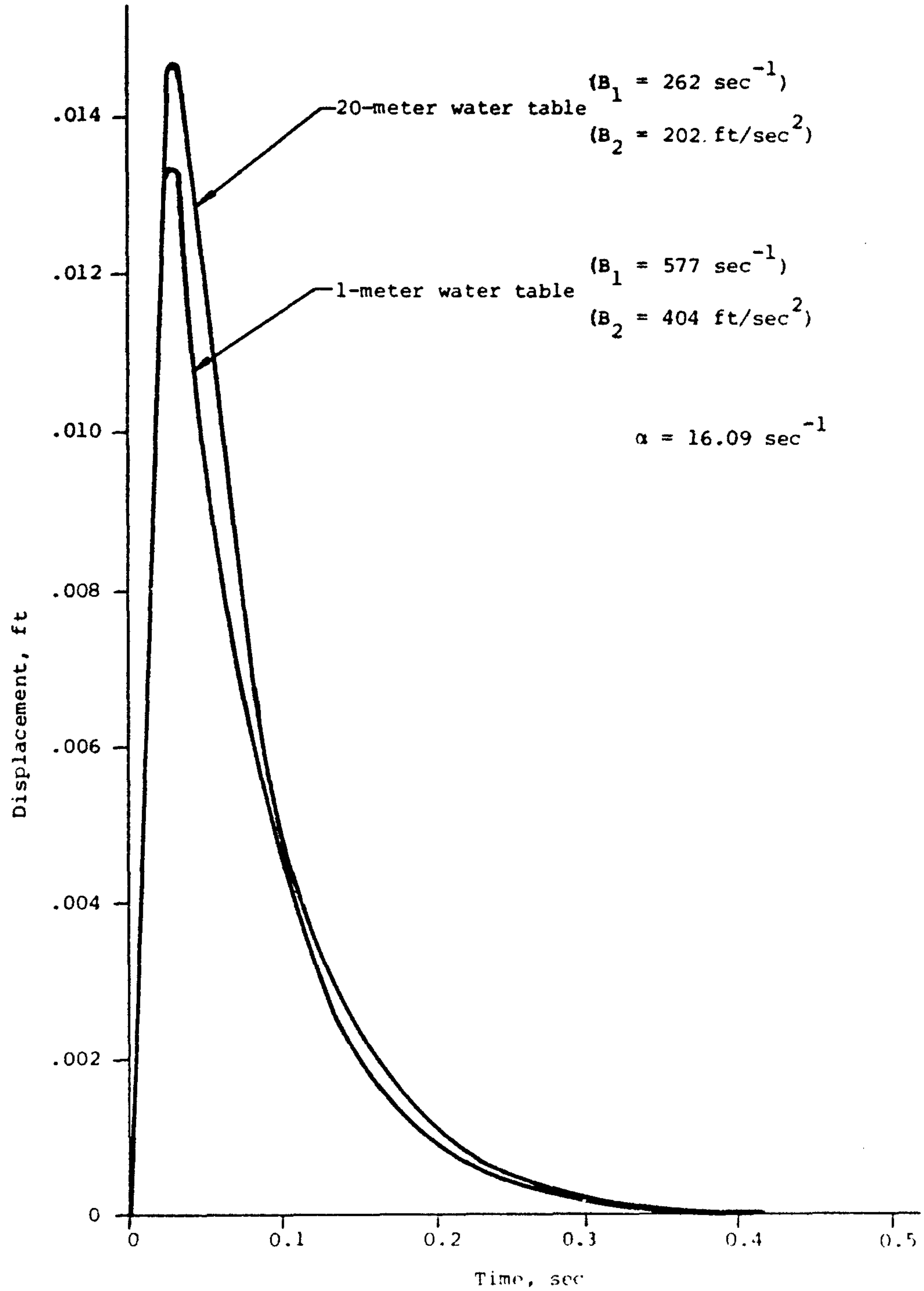


Figure 85. Horizontal Rigid-Body Displacement Due to Sweeping Airblast

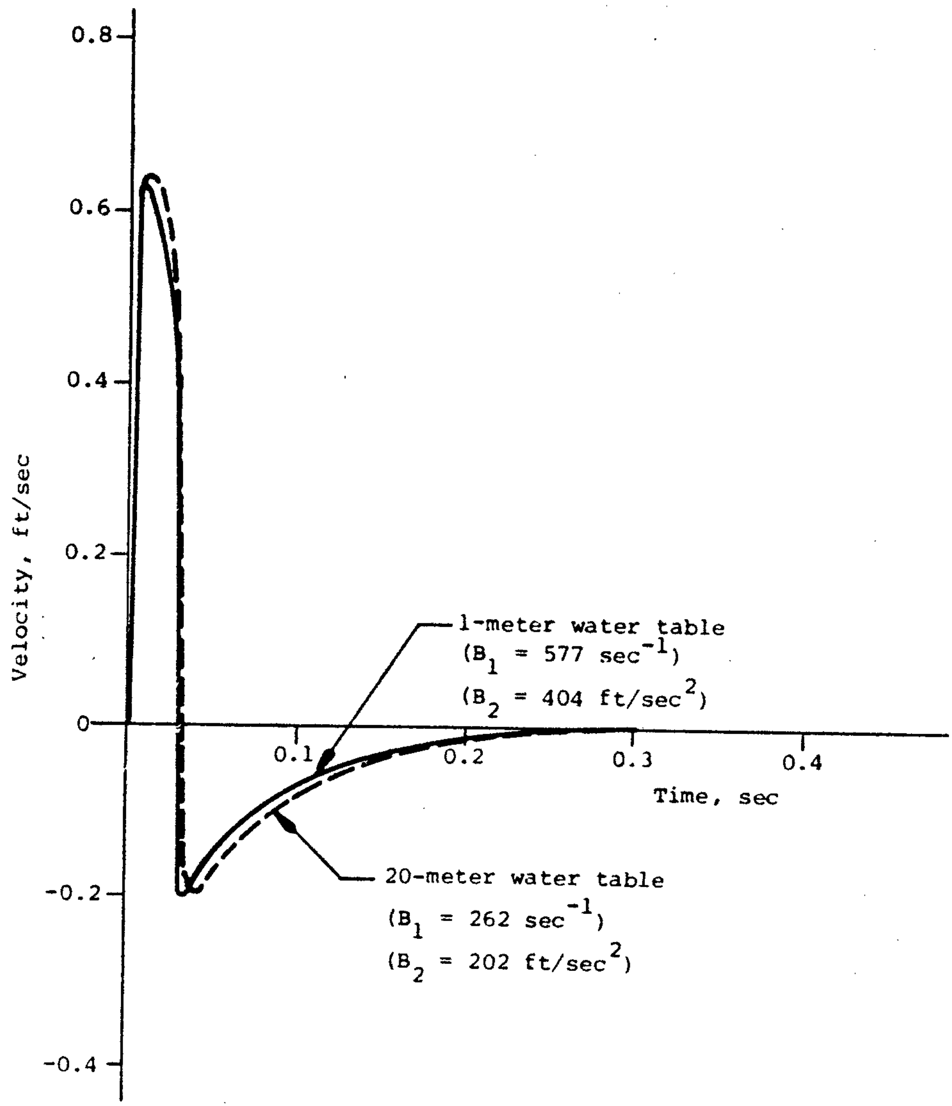


Figure 86. Horizontal Rigid Body Velocity Due to Sweeping Airblast

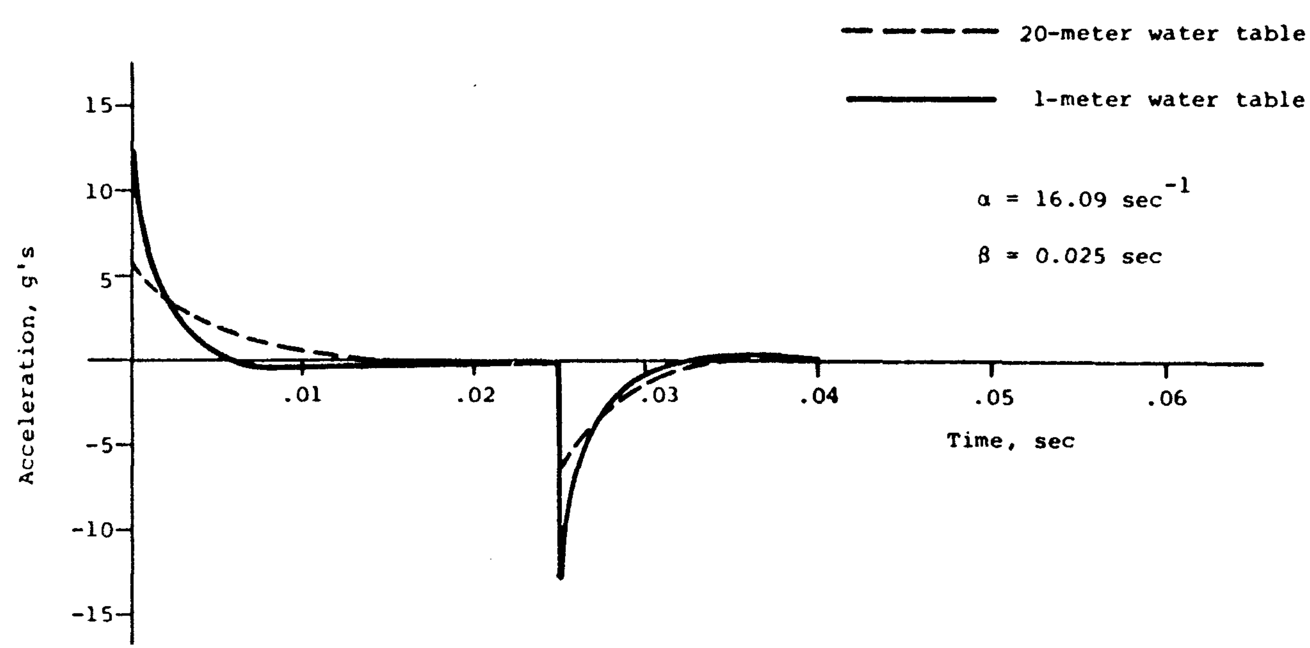


Figure 87. Horizontal Rigid-Body Acceleration Due to Sweeping Airblast

5. SUMMARY

The following observations were based upon the calculated horizontal and vertical shelter motions. Only displacements and accelerations are discussed, as velocities are of lesser importance to this analysis.

a. Horizontal Motions

- For both geologies, the horizontal rigid body displacements induced by the finite duration sweep time of the propagating airblast wave are negligible in comparison to crater-induced horizontal displacements. Thus, horizontal displacements may be based entirely on figure 79, where the peak displacements are 1.8 feet for the 1-meter water table and 1.0 feet for the 20-meter water table.
- Peak horizontal accelerations for crater induced motion are small (maximum of 1.86 g for 1-meter geology) in comparison to the peak values indicated in the sweeping airblast. Thus, horizontal accelerations may be based entirely on figure 87. The peak horizontal acceleration (1-meter water table) is seen to be 12.5 g.

b. Vertical Motions

- Vertical displacements due to crater-induced and airblast-induced sources are of approximately the same magnitude and, thus, must be superimposed (with appropriate phasing) for each geology. The superimposed displacement time histories are shown in figures 88 and 89, respectively, for the 1-meter and 20-meter geologies. It is seen that the total peak vertical displacements are 2.1 ft and 1.75 ft for the 1- and 20-meter geologies, respectively.

- It can be readily shown that crater-induced vertical accelerations are negligible in comparison to vertical airblast-induced rigid body acceleration. For both geologies, peak vertical acceleration is 53.8 g.

Comparisons of the above predicted shelter horizontal and vertical acceleration values with aircraft shock tolerances indicated in table 11 suggest the need for shock isolation of the aircraft in the shelter. One possible approach is the use of shock isolation platforms. However, the shelter floor is not rigidly connected to the shelter at present, so that there will be some degree of isolation of the aircraft from shelter motions with the present shelter design. The maximum displacements indicated by figures 88 and 89 would probably cause severe distress in portions of the shelter complex and could impair post-attack operations. A critical factor would be the degree of simultaneity of the motion.

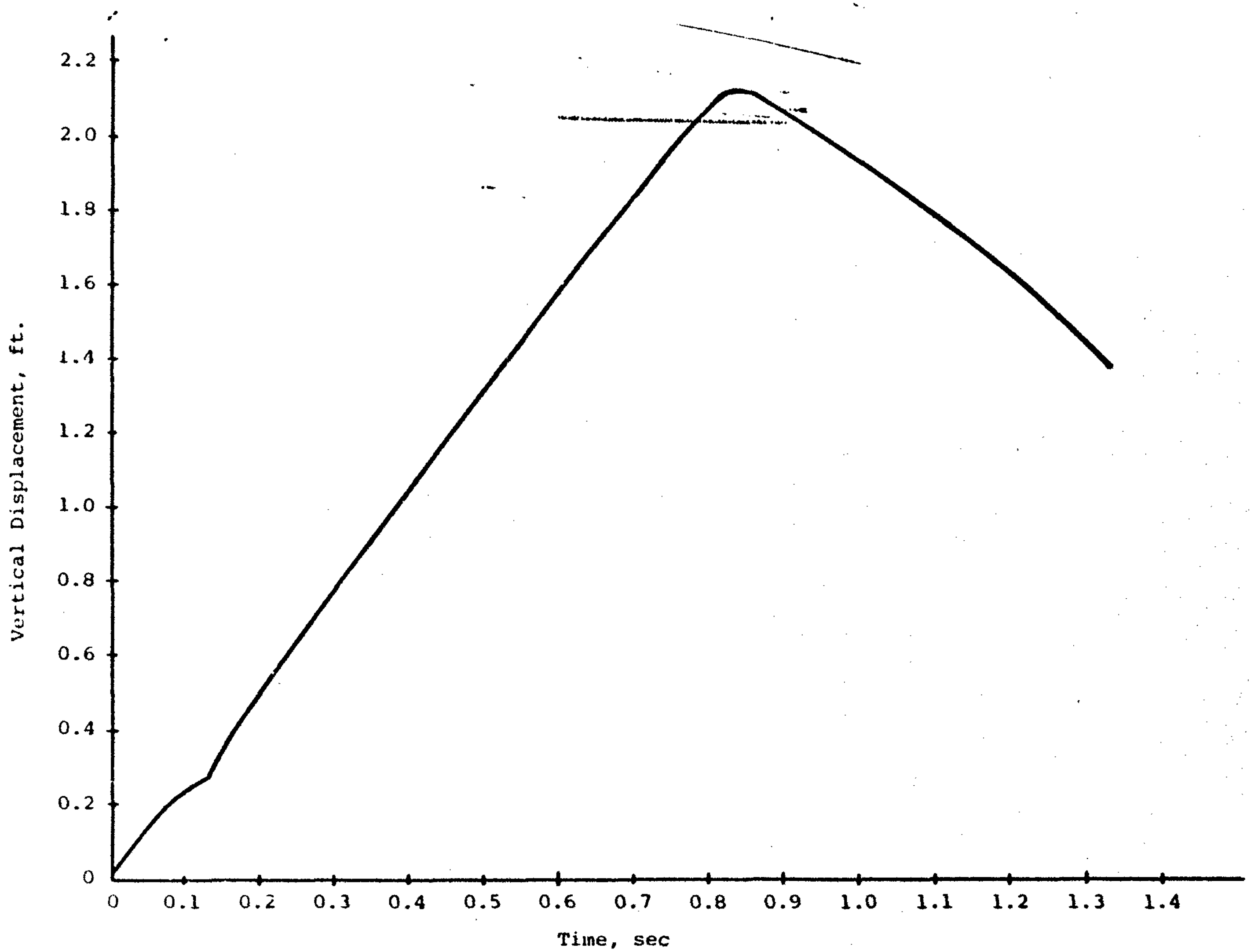


Figure 88. Superposed Vertical Displacement-Time History of Shelter for 1-meter Geology

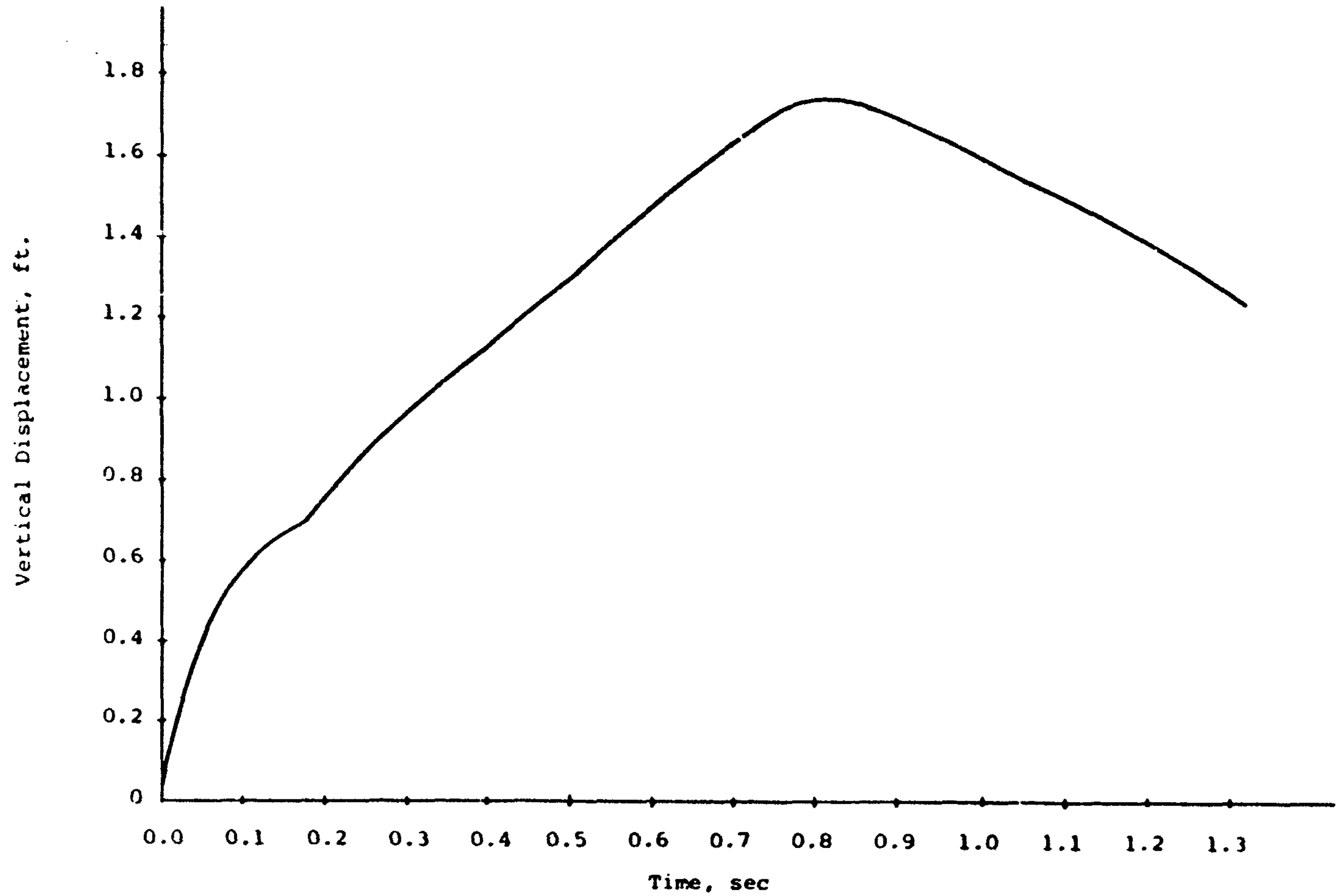


Figure 89. Superposed Vertical Displacement-Time History of Shelter for 20-meter Geology

SECTION XI

COST ESTIMATE

The cost estimate for the star-shaped covered corridor is based on January 1, 1976, Albuquerque, New Mexico prices for material and labor. The items used for the quantity estimate could not be established to the same level of detail as is normally done for construction drawings, and the costs reflect only the major items. The excavation and backfill cost was taken from section VI since this item reflected only a small portion of the total cost. The cost of the shelter complex, the closure and closure pit were derived from quantity estimates for concrete, reinforcing steel, structural steel, forming areas and other large items. Although large quantities of material are included in a single shelter complex, the cost estimate does not include reductions in material or labor costs for large quantity purchases. When more detailed cost estimates are made, a reduction in unit prices for large quantities can be included. The net effect should be a rather small change in the cost estimate given here. Overhead, profit and contingencies were taken as 30 percent of the in-place material costs. Thus, the given cost estimate should be considered a reasonable indication of the in-place cost of a single complex with an allowance for unforeseen costs.

The cost breakdown for the single unit (prototype) is as follows

Shelter Complex

Concrete

Main Corridor	6,400 cy			
Entry Corridors	4,250 cy			
Shelter	13,500 cy			
Endwalls	3,250 cy			
Ribs	800 cy			
	28,200 cy	@ 60	= $1,692,000	

Reinforcing Steel

Main Corridor	380 T			
Entry Corridors	250 T			
Shelters	800 T			
Endwalls	300 T			
Ribs	80 T			
Foundations	830 T			
	2,640 T	@ 700	= $1,848,000	

Steel Plate

1/4 inch thick	280 T			
11 gage	75 T			
	355 T	@ 1,500	= $ 532,500	

Steel Shapes

Tie Beams	245 T	@ 1,700	= $ 416,500	
Formwork	120,000 SF	@ 4	= $ 480,000	
Total Shelter Complex				$4,969,000

Closure, Pits and Bulkheads

Concrete

Pits	2,300 cy		
Bulkheads	200 cy		
	2,500 cy	@ 60	= $ 150,000

Reinforcing Steel

Pits	270 T		
Bulkheads	20 T		
	290 T	@ 700	= $ 203,000

Steel Shapes			
Closures	270 T		
Struts	21 T		
	291 T @ 1,700 =	$ 494,700	
Formwork	20,000 SF @ 4 =	$ 80,000	
Hydraulics	LS	$ 130,000	
Total Closures, Pits and Bulkheads			$ 105,770
Excavation, Compaction and Backfill			$ 27,400
Sub-Total			$6,300,700
30% Overhead, Profit, Contingency			$1,890,300
Total Single Unit Cost			$8,191,000

For the 100-unit construction the cost estimate is changed by using mill prices for carlot and truckload quantities with a 15 percent markup. The unit labor costs were reduced appropriately for concrete and structural steel but the unit labor costs were unchanged for reinforcing steel placement and formwork.

The costs breakdown for a single unit of a 100-unit construction is as follows

Shelter Complex

Concrete		
Main Corridor	6,400 cy	
Entry Corridors	4,250 cy	
Shelter	13,500 cy	
Endwalls	3,250 cy	
Ribs	800 cy	
	28,200 cy @ 50 =	$1,410,000

Reinforcing Steel				
Main Corridor	380 T			
Entry Corridors	250 T			
Shelters	800 T			
Endwalls	300 T			
Ribs	80 T			
Foundation	830 T			
	2,640 T	@ 545 =	$1,438,800	
Steel Plate				
1/4 inch thick	280 T			
11 gage	75 T			
	355 T	@ 1,000 =	$ 355,000	
Steel Shapes				
Tie Beams	245 T	@ 1,200 =	$ 294,000	
Formwork	120,000 SF	@ 3 =	$ 360,000	
Total 100-Unit Construction Shelter Complex				$3,857,800
Closure, Pits and Bulkheads				
Concrete				
Pits	2,300 cy			
Bulkheads	200 cy			
	2,500 cy	@ 50 =	$ 125,000	
Reinforcing Steel				
Pits	270 T			
Bulkheads	20 T			
	290 T	@ 545 =	$ 158,050	
Steel Shapes				
Closures	270 T			
Struts	21 T			
	291 T	@ 1,200 =	$ 349,200	
Formwork	20,000 SF	@ 3 =	$ 60,000	
Hydraulics	LS		$ 130,000	
Total Closures, Pits and Bulkheads				$ 822,250

Excavation, Compaction and Backfill	$ 274,000
Sub-Total	$4,954,050
30% Overhead, Profit, Contingency	$1,486,250
Total Single Unit Cost for 100-Unit Construction	$6,440,300

SECTION XII

OBSERVATIONS

Although this study has provided a detailed preliminary structural design of major elements of the aircraft shelter complex, there are still areas of uncertainty. Some of these areas are complex in nature and deserving of much more detailed analysis than was possible in this conceptual study. The following paragraphs review areas which should be subjected to further analysis or study before structural concepts and designs are finalized.

- A uniform vertical load was chosen for the NASTRAN analysis of the quarter section model because it allowed simplification of the model and analyses. Additional analyses should be made using a blast wave traversing the complex from several directions. This more realistic loading may expose weaknesses or critical areas not occurring under the uniform loading.

- The shelter complex presents a wide range of resistance to deformation under airblast and ground shock loading. This characteristic will probably result in significant relative motions between major elements of the shelter complex. An investigation of these deformations is necessary to assess their impact on shelter performance.

Large relative displacements could cause distress or failure in structural elements or interfere with post attack operation of critical elements. Concept or structural changes may be necessary to either minimize relative deformations or accommodate them at interfaces between shelter elements.

- The critical intersections between the main corridor, entry corridors and shelter arches require further study. The initial structural analyses completed in this conceptual study indicated yielding at some sections of the arch shells and strengthening ribs. The extent of yielding and its effect on shelter performance should be investigated to determine the need for changes in section properties.

- Although the shelter foundation designs proposed by this study are considered adequate, no attempt was made to optimize these designs or fully evaluate the effects of foundation characteristics on shelter performance. There are some uncertainties regarding the effectiveness of inclined foundations and the techniques used to model their response to dynamic loading. The complex interaction between arch, arch foundations, and tie beams should be investigated.

- The personnel access door between the main corridor and the shelter was not subjected to analysis or design under this study. It must be fitted with a blast door at the main corridor end of the access tunnel. The tunnel itself should be designed to accommodate relative motions between the main corridor and shelter.

- Although the simple rigid body analysis of structural motions presented in Section X indicates a need for aircraft shock isolation, the cost of a shock isolation system justifies a more detailed study of the problem.

- Previous studies have shown the desirability of using spall plates for tensile reinforcement. A steel spall plate properly anchored to a concrete core can simultaneously serve as tension reinforcement and spall liner. For this study, it was questionable whether the 0.25 spall liner could be properly anchored to the six-foot thick section of concrete, and its contribution to section capacity was neglected. In view of the fact that approximately 2000 tons of reinforcing steel are used in reinforced concrete sections backed by spall plates, a small test program to establish the tensile reinforcing capability of spall plates appears justified. The cost-saving could be significant.

- The closure is the weak link insofar as neutron and gamma radiation shielding is concerned. The EMP shielding provided by the shelter floor is questionable. Acceptable radiation levels within the shelter should be established and an analysis made to determine the need for increased levels of protection.

APPENDIX A

REPRESENTATIVE AEROSPACE GROUND EQUIPMENT

This description of representative aerospace ground equipment (AGE) used inside tactical aircraft shelters was obtained from reference 1. Engine specifications obtained from the reference are given below. Sketches are included in Figures A1 through A4 to indicate dimensions of the various units and the direction of exhaust flow.

<u>TTU-228E Hydraulic Test Stand</u>

Engine - Continental Motor Corp., Detroit, Michigan, Model PE-150-7. Six-cylinder, overhead valves, air-cooled, four-stroke, 133 net BPH at 2,400 RPM.

<u>AM32A-60A Gas Turbine Generator</u>

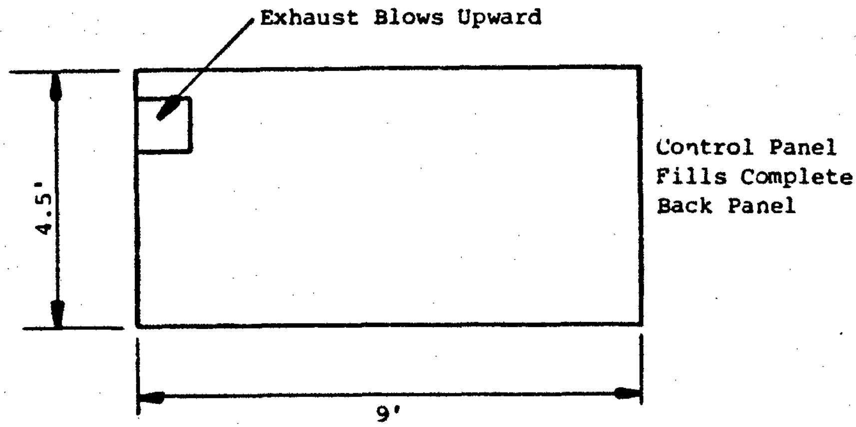
Engine - Air Research Model GTCP85-180, Part No. 380834-1-1. 42,000 ± 100 RPM, 177 hp at rated speed. EGT 649°C (1,200°F) maximum continuous operation.

<u>BT-400 (H-1) Heater</u>

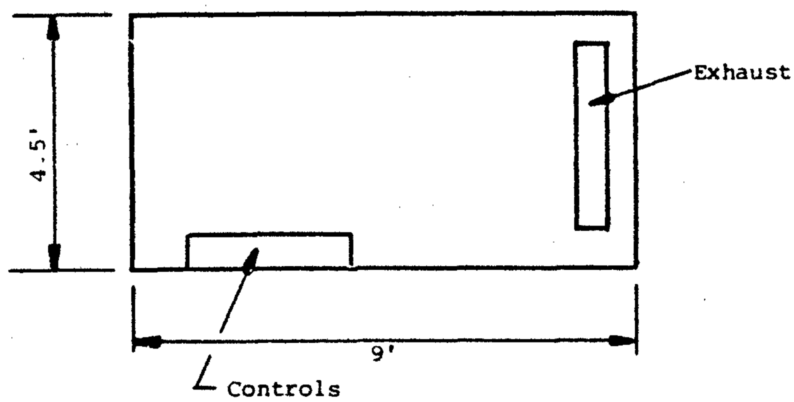
Engine - Continental Motors Corp., Detroit, Michigan, Model AU7B, Spec. No. 339. One-cylinder, four-stroke cycle, air-cooled, 2-1/4 hp at 3,450 ± 50 RPM.

<u>MJ-1 Bomb Lift and MJ-4 Bomb Lift</u>

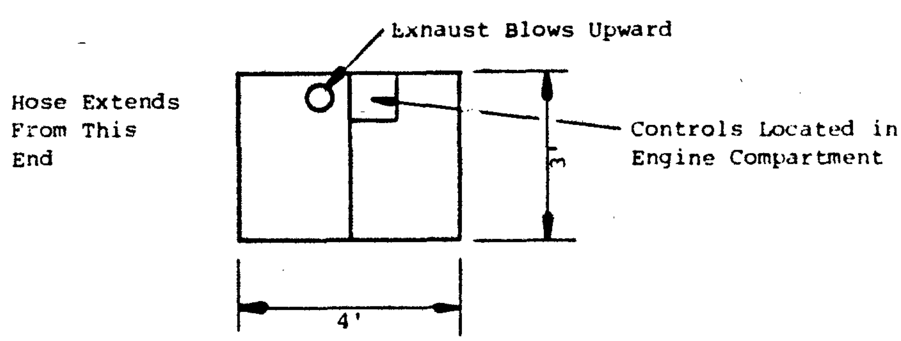
Engine - Wisconsin Model MVF4D, Spec. No. 173764. Four-cylinder, four-cycle, air-cooled, 3.25-inch bore, 3.25-inch stroke, 107.7-cubic inch displacement, 25 hp at 2,400 RPM.



TTU-228E Hydraulic Test Stand

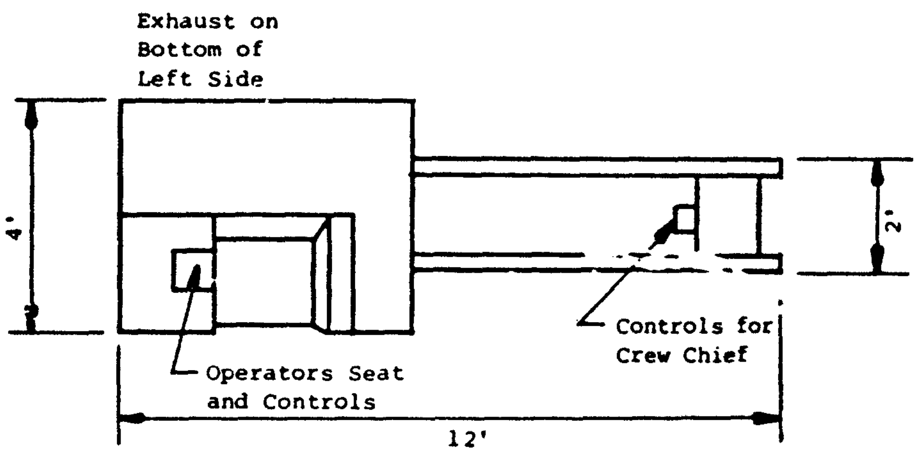


M32A-60 Generator

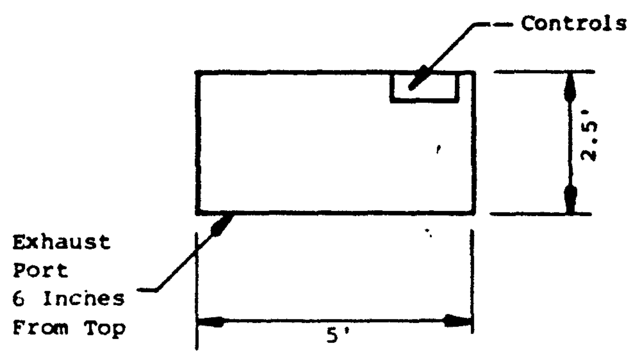


BT-400 Heater

Figure A1. Hydraulic Test Stand, Generator, and Heater Sketches

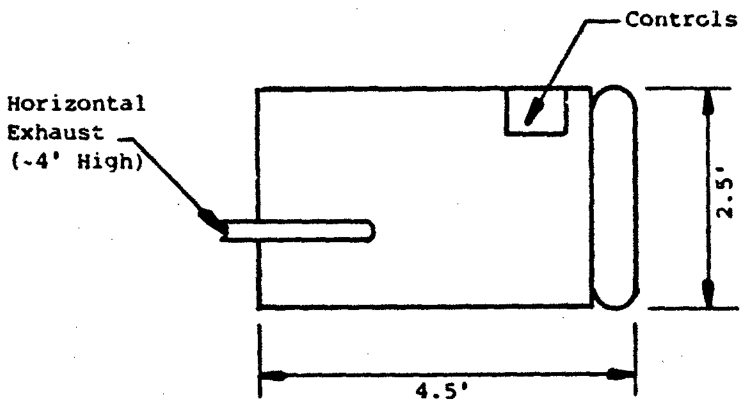


MJ-1 Weapons Hoist

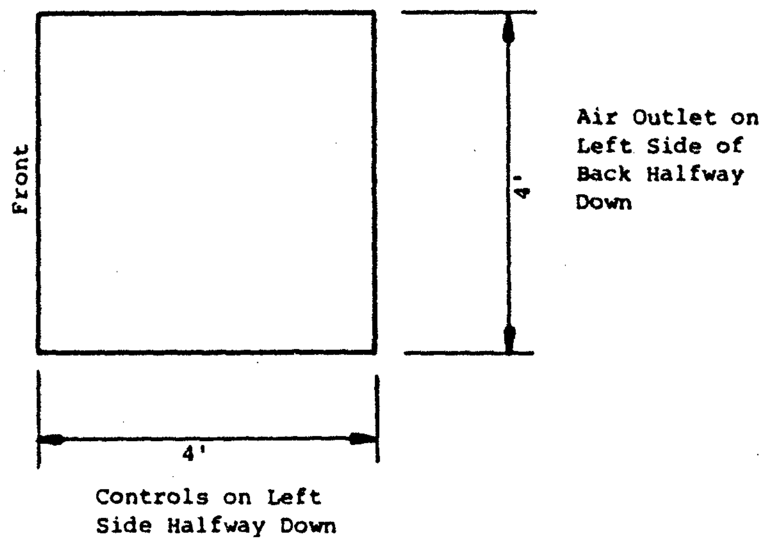


MC-1A Air Compressor

Figure A2. Weapons Hoist and Air Compressor Sketches

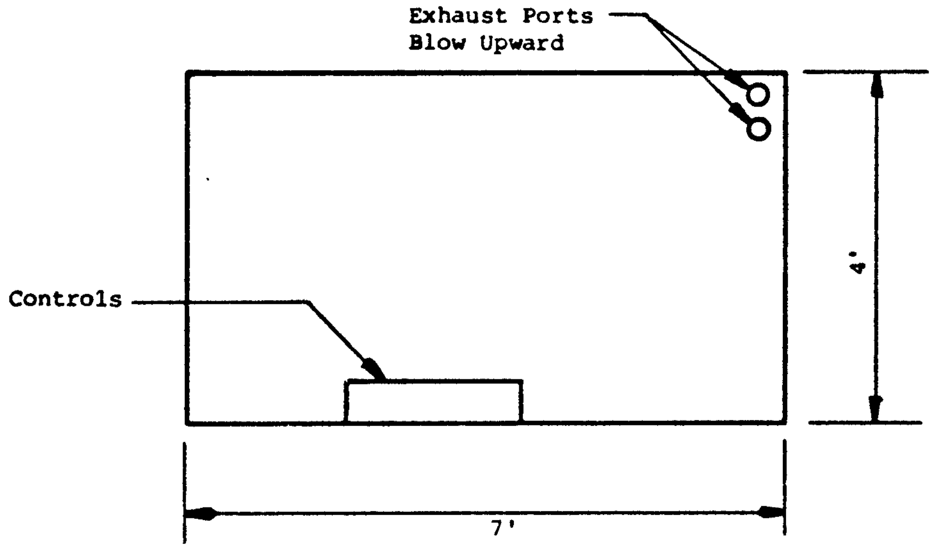


MC-2A Air Compressor

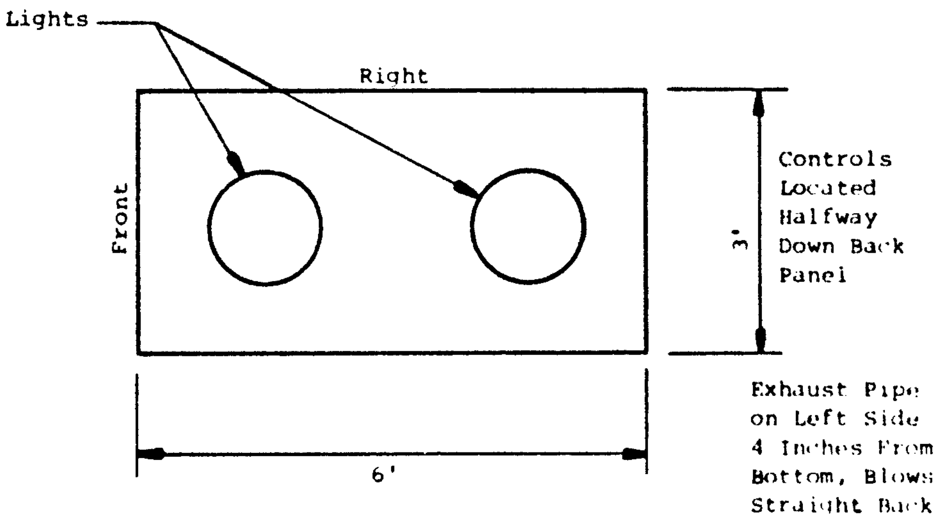


M-32 Cooler (No Engine No Exhaust)

Figure A3. Air Compressor and Cooler Sketches



MD-3 Generator



NF-2 Lighting Stand

Figure A4. Generator and Lighting Stand Sketches

MC-1A Air Compressor (High and Low Pack)

Engine - Air Research Model GTCE85-15, Part No. 376490. Single-stage radial inward-flow power turbine. 42,300 maximum RPM, EGT 1,120°F maximum continuous operation.

MC-2A Air Compressor (Low Pack)

Engine - Onan Division of Studebaker Corp., Minneapolis, Minnesota, Type CCKM-MSV 427E. Two-cylinder, four-cycle, 5.5 to 1 compression ratio, 10.3 hp at 1,800 RPM.

M32 Air-Cooler

Run by turbine exhaust; no combustion engine.

MD-3 Generator Set

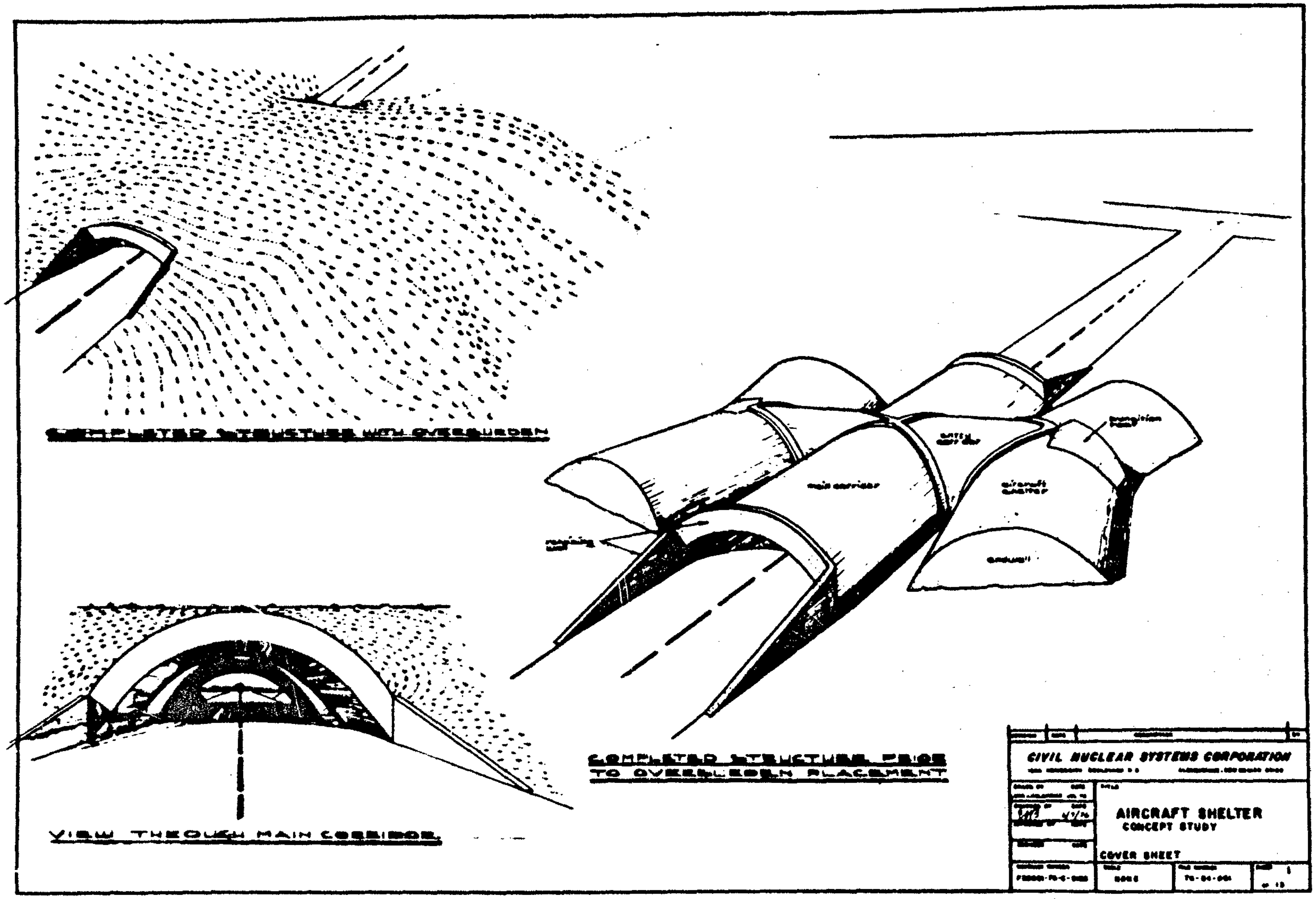
Engine - Continental Motors Corp., Detroit, Michigan, Model PE-150-6. Six-cylinder, horizontally opposed, four-stroke (OTTO) cycle principle, 5-inch bore, 5-inch stroke, 471-cubic inch displacement, 180 hp at 2,400 RPM.

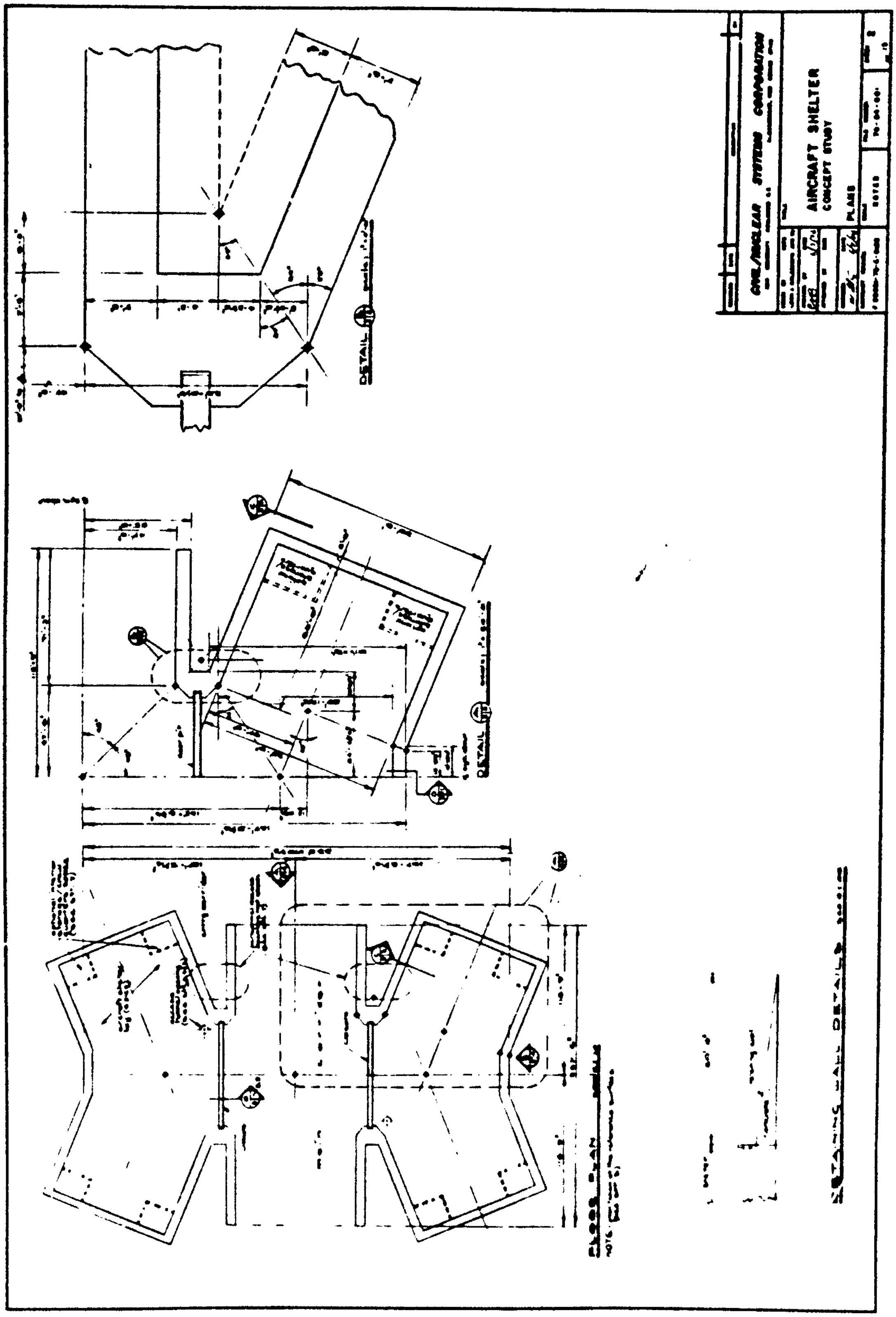
NF-2 Light Cart

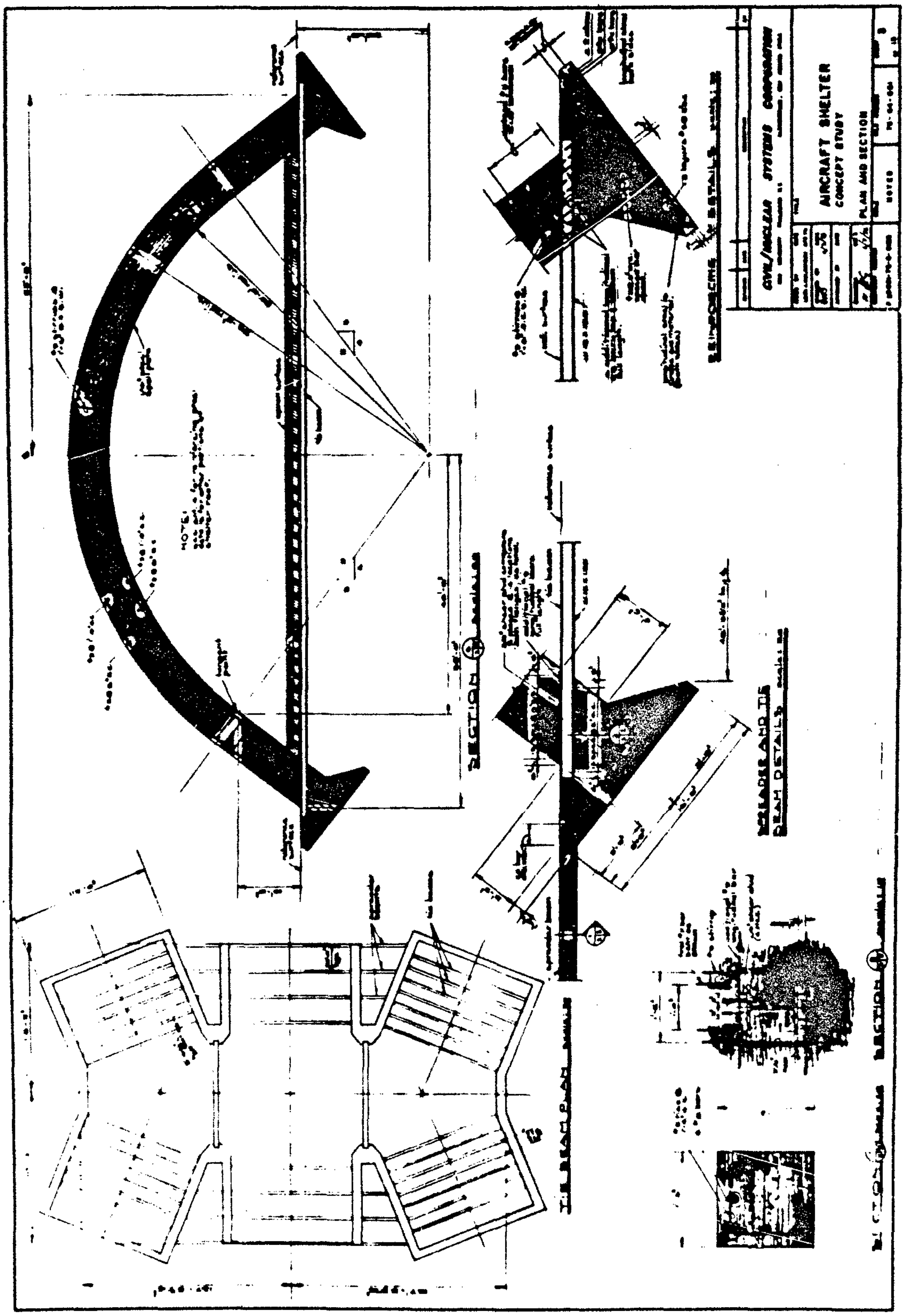
Engine - Onan Division of Studebaker Corp., Minneapolis, Minnesota, Part No. 100A371, Type 2CCK - IRV3/17550. Two-cylinder, four-stroke, air-cooled, 3.25-inch bore, 3-inch stroke, 50-cubic inch displacement, 10.3 hp at 1,800 RPM.

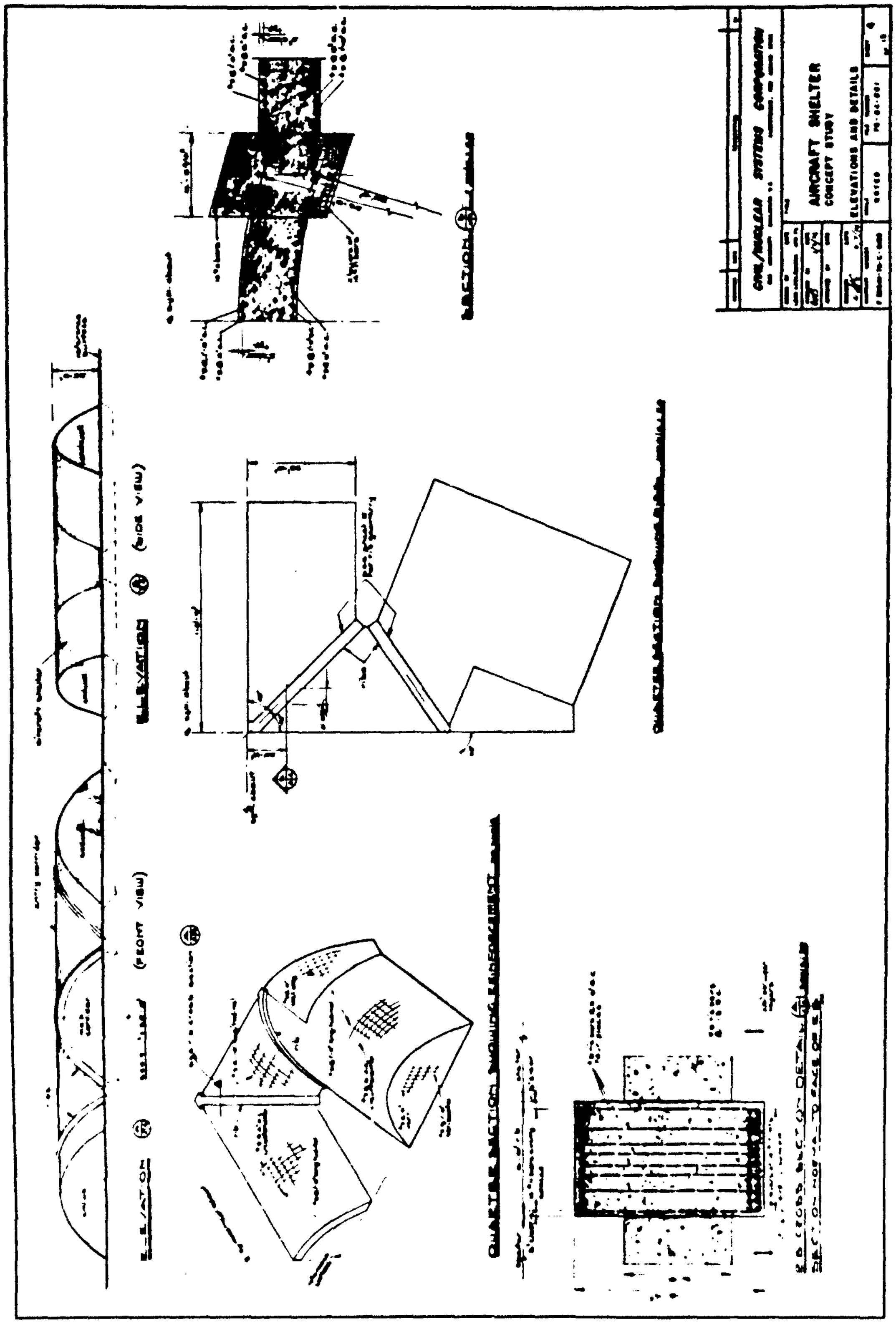
APPENDIX B

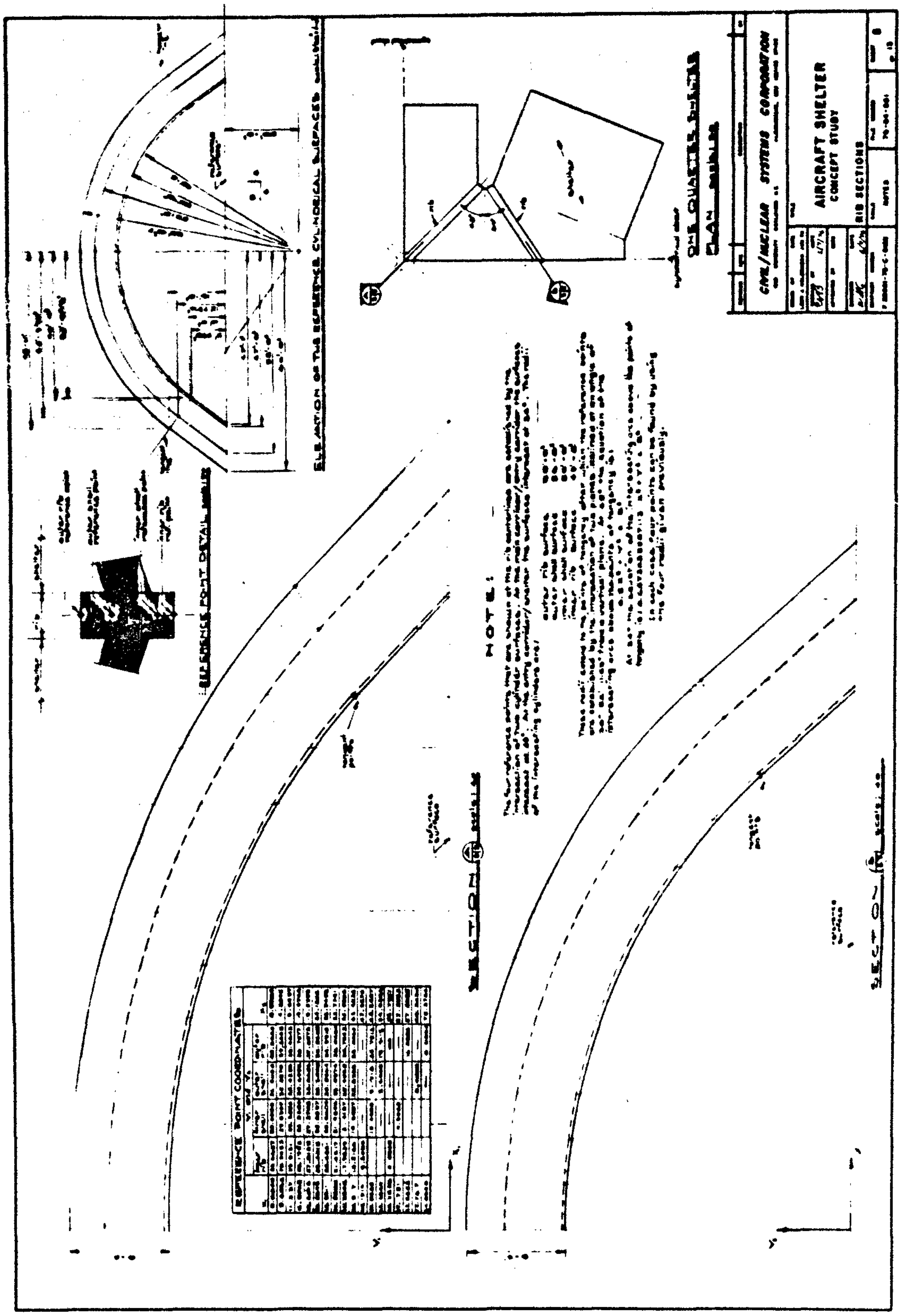
CONCEPTUAL DESIGN DRAWINGS


COMPLETED STRUCTURE WITH OVERBURDEN
COMPLETED STRUCTURE PRIOR
TO OVERBURDEN PLACEMENT
VIEW THROUGH MAIN CORRIDOR
transition
main corridor
aircraft shelter
CIVIL NUCLEAR SYSTEMS CORPORATION
AIRCRAFT SHELTER
CONCEPT STUDY
COVER SHEET


CIVIL/NUCLEAR SYSTEMS CORPORATION
AIRCRAFT SHELTER
CONCEPT STUDY
PLANS
DETAIL
DETAIL
FLOOR PLAN
RETAINING WALL DETAILS


CIVIL/NUCLEAR SYSTEMS CORPORATION
AIRCRAFT SHELTER
CONCEPT STUDY
PLAN AND SECTION
SECTION
TIE BEAM PLAN
HEADER AND TIE BEAM DETAILS
REINFORCING DETAILS
NOTE:


ELEVATION (FRONT VIEW)
ELEVATION (SIDE VIEW)
QUARTER SECTION SHOWING REINFORCEMENT
SECTION
CIVIL/NUCLEAR SYSTEMS CORPORATION
AIRCRAFT SHELTER
CONCEPT STUDY
ELEVATIONS AND DETAILS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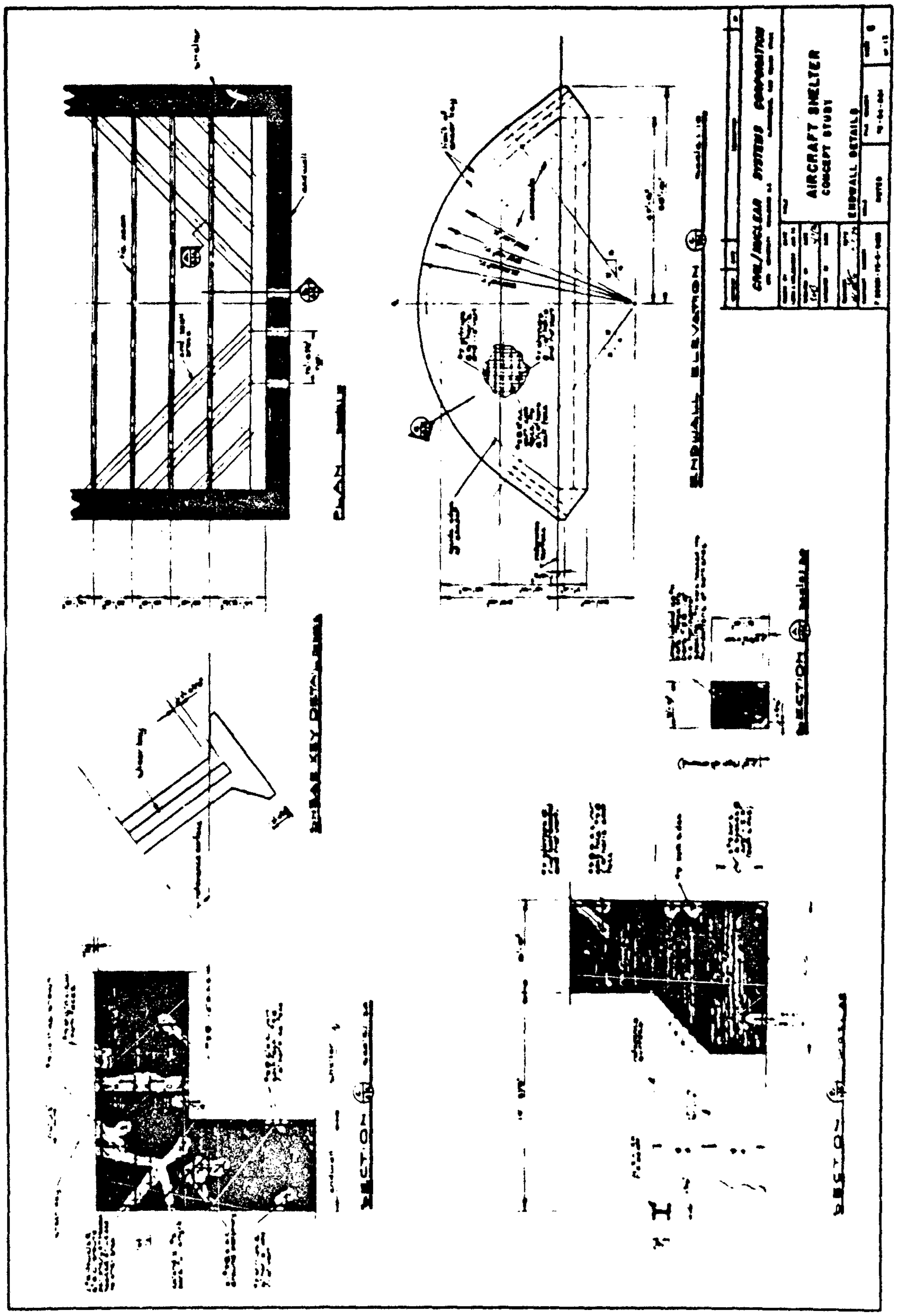
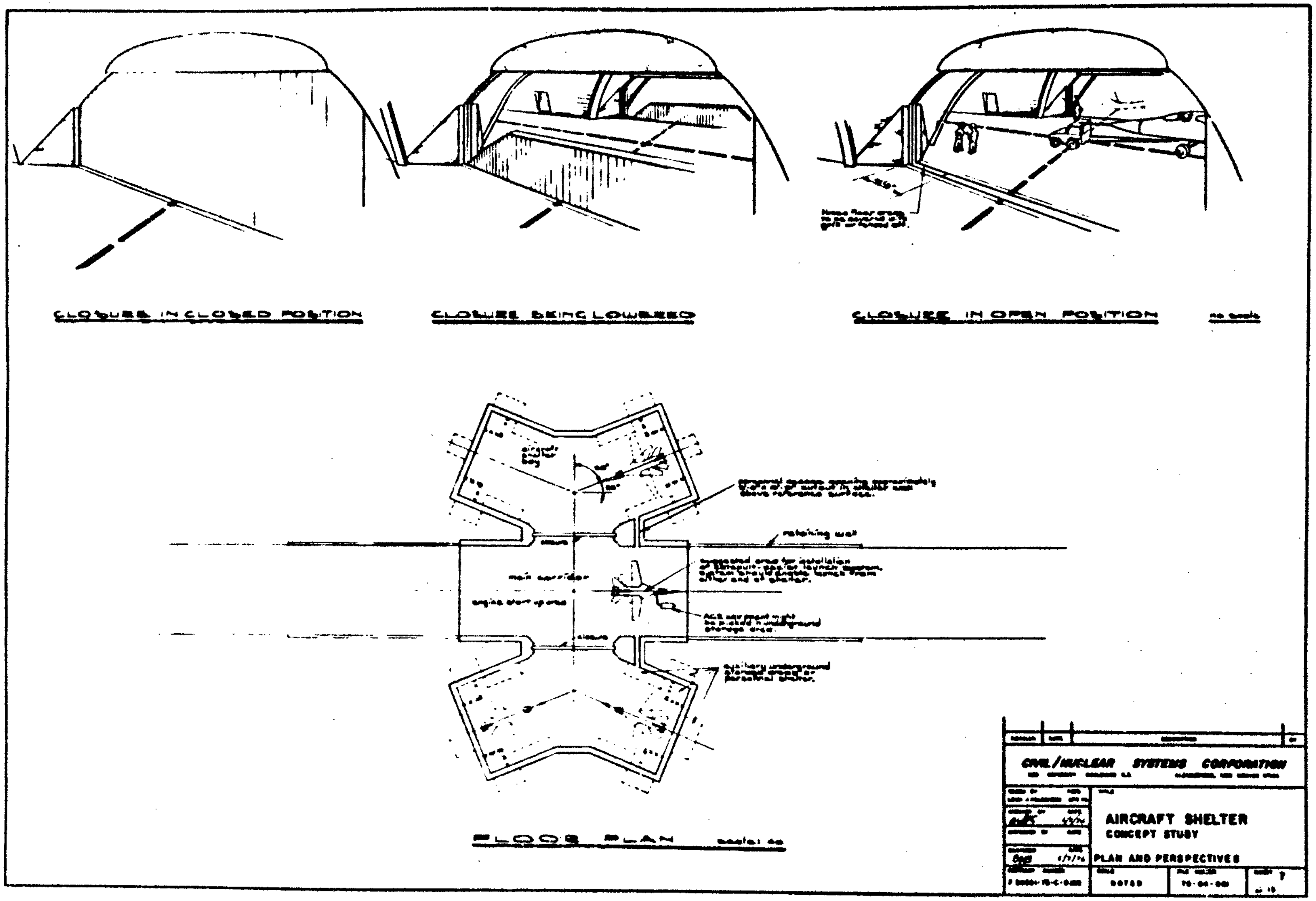
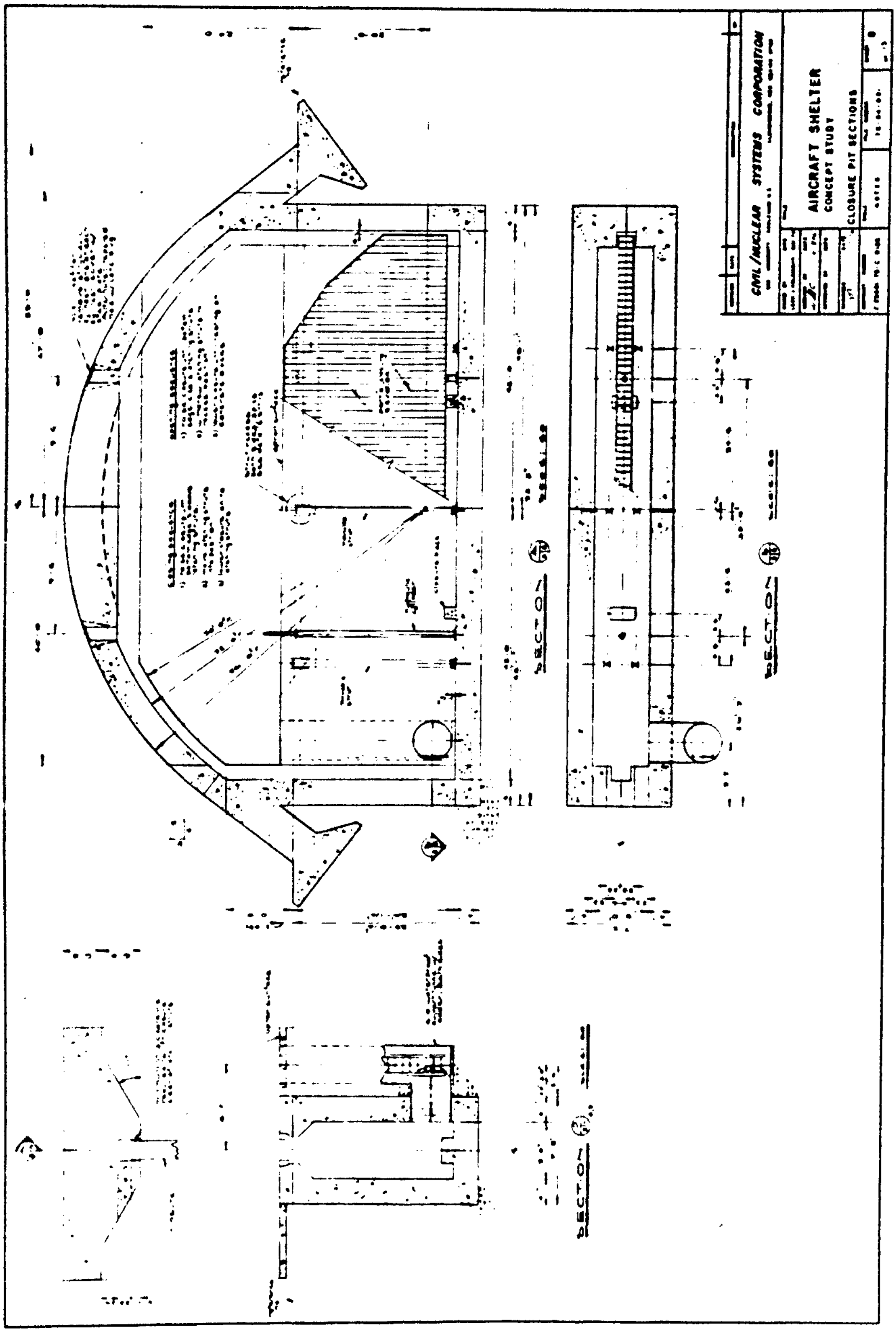
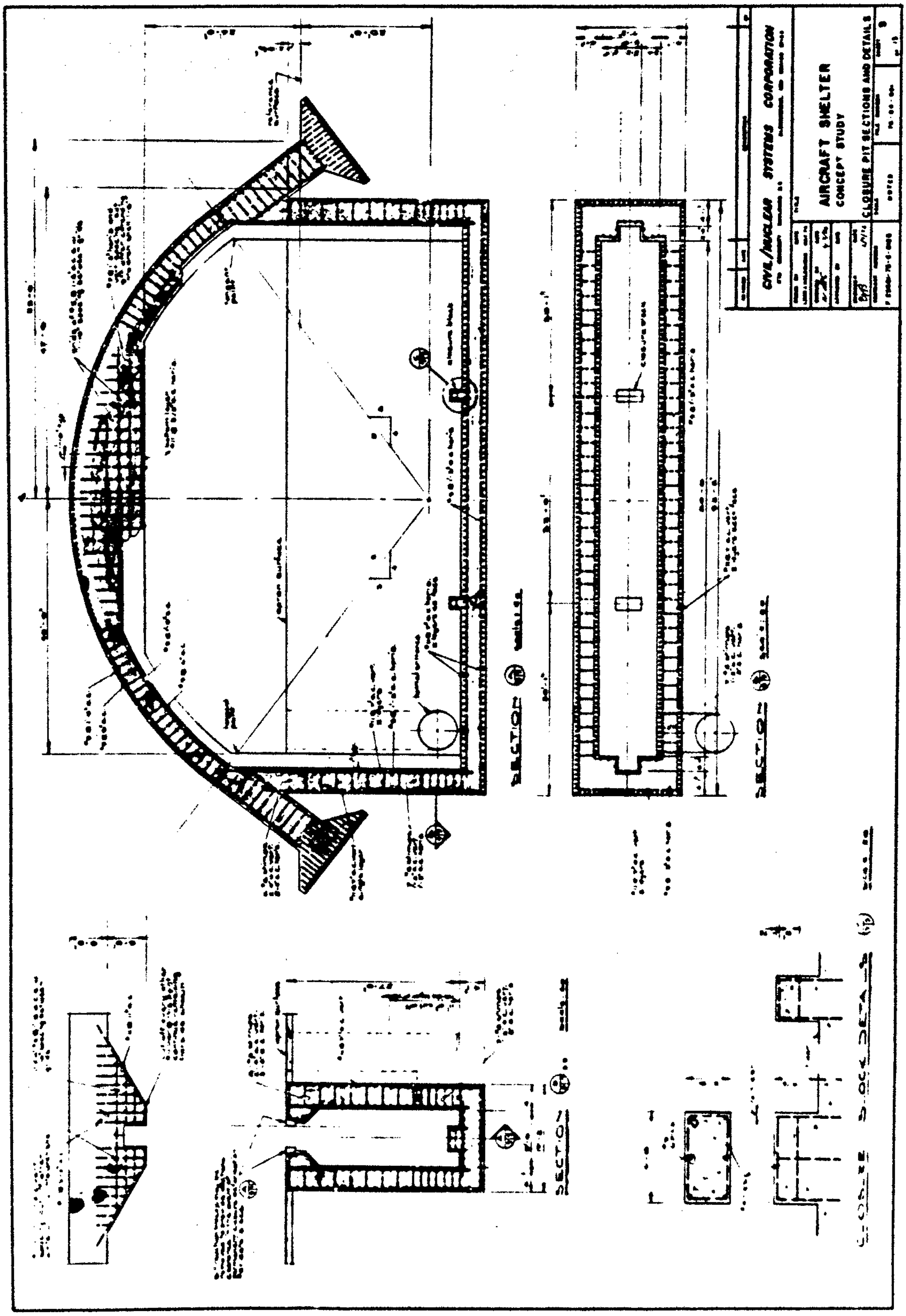
AIRCRAFT SHELTER
CONCEPT STUDY
RIB SECTIONS


CIVIL/NUCLEAR SYSTEMS CORPORATION
AIRCRAFT SHELTER
CONCEPT STUDY
ENDWALL DETAILS
PLAN
ENDWALL ELEVATION
SHEAR KEY DETAIL
SECTION
SECTION
SECTION


CLOSURE IN CLOSED POSITION
CLOSURE BEING LOWERED
CLOSURE IN OPEN POSITION
no scale
retaining wall
main corridor
closure
FLOOR PLAN
CIVIL/NUCLEAR SYSTEMS CORPORATION
AIRCRAFT SHELTER
CONCEPT STUDY
PLAN AND PERSPECTIVES


CIVIL/NUCLEAR SYSTEMS CORPORATION
AIRCRAFT SHELTER
CONCEPT STUDY
CLOSURE PIT SECTIONS
SECTION
SECTION
SECTION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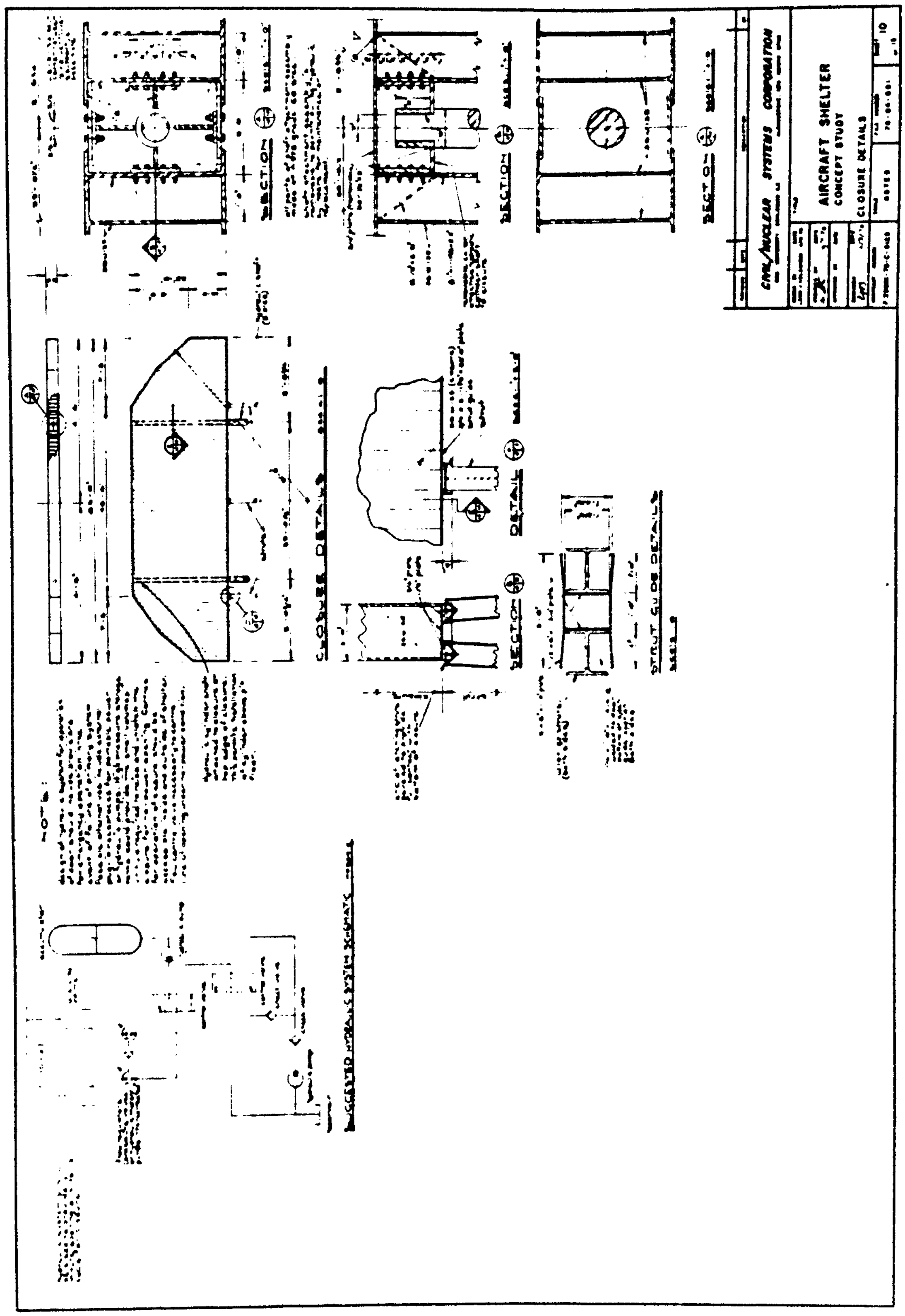
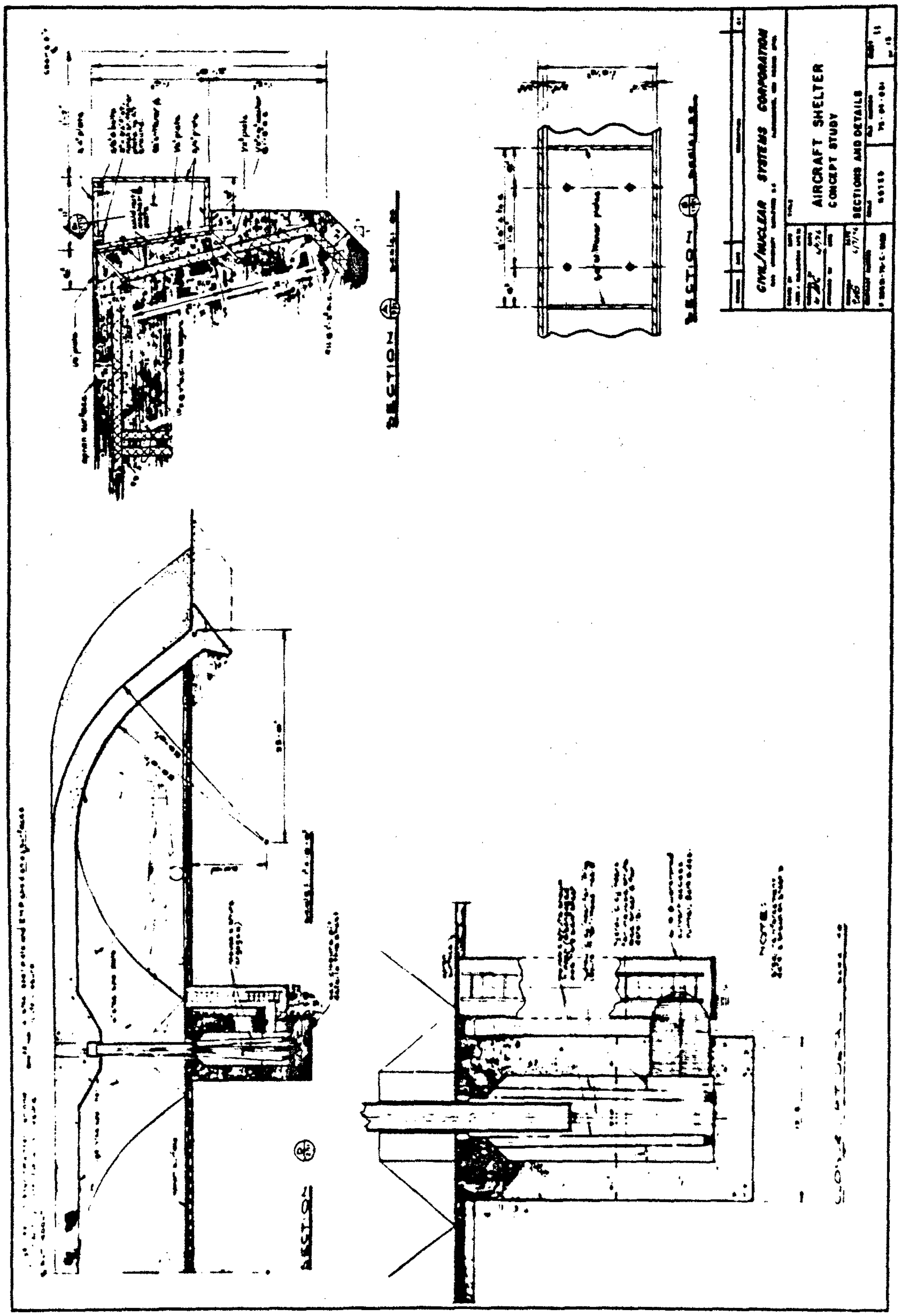
AIRCRAFT SHELTER
CONCEPT STUDY
CLOSURE PIT SECTIONS AND DETAILS


CIVIL/NUCLEAR SYSTEMS CORPORATION
AIRCRAFT SHELTER
CONCEPT STUDY
CLOSURE DETAILS
STRUT GUIDE DETAILS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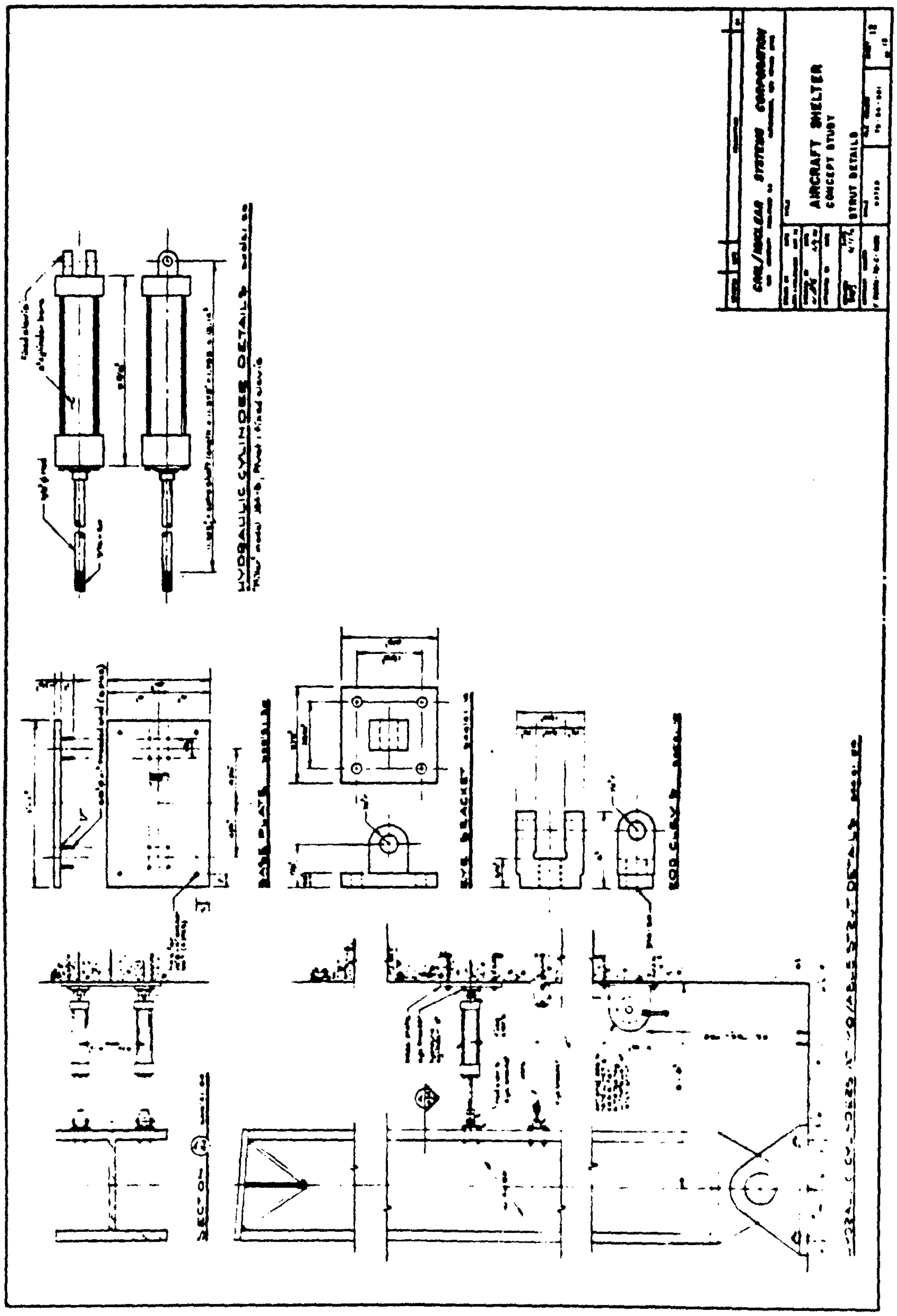
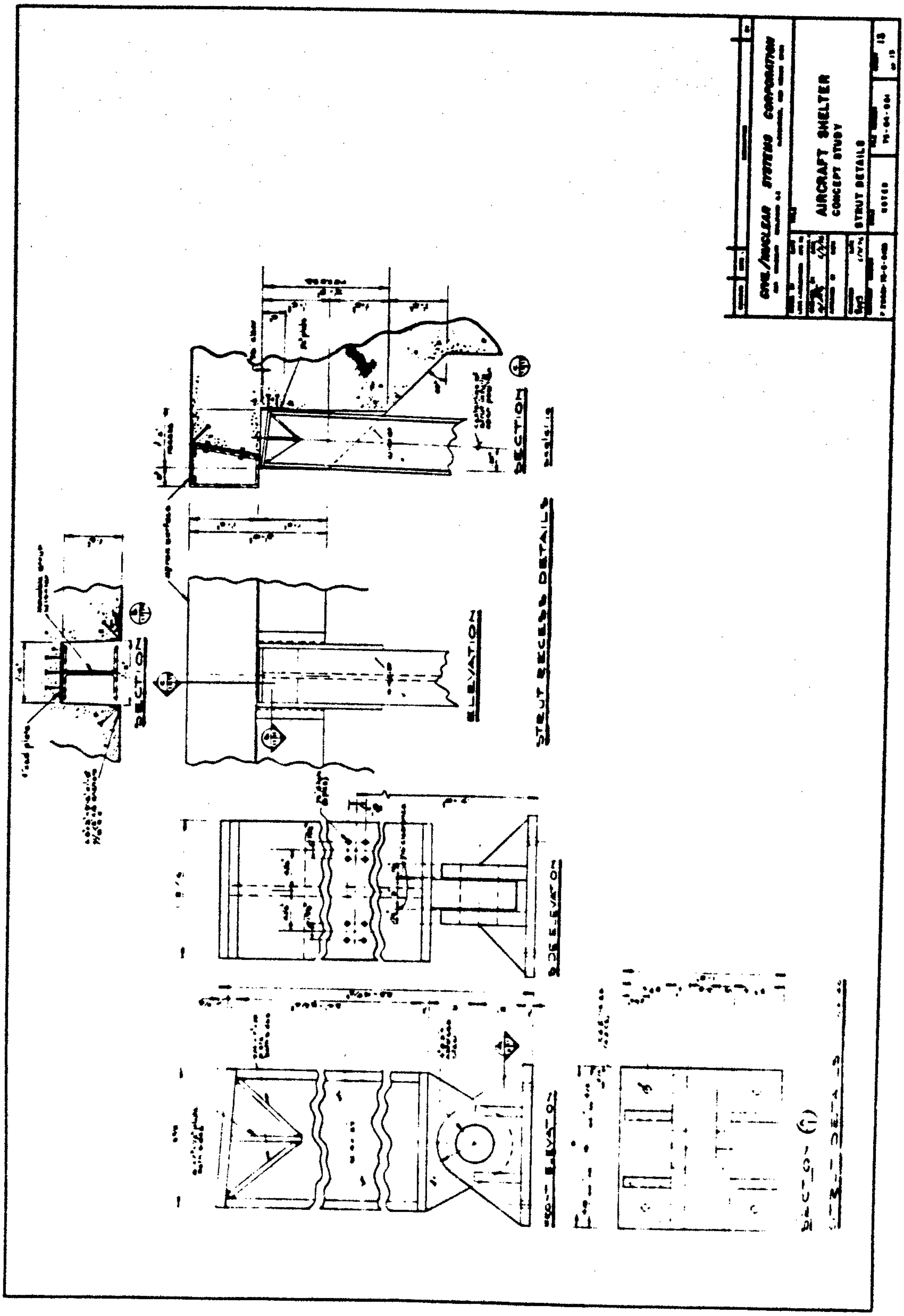
AIRCRAFT SHELTER
CONCEPT STUDY
SECTIONS AND DETAILS
SECTION
SECTION
SECTION



CIVIL/NUCLEAR SYSTEMS CORPORATION
AIRCRAFT SHELTER
CONCEPT STUDY
STRUT DETAILS
HYDRAULIC CYLINDERS DETAILS
BASE PLATE DETAILS
EYE BRACKET
ROD CLEVIS DETAILS
SECTION


CIVIL/NUCLEAR SYSTEMS CORPORATION
AIRCRAFT SHELTER
CONCEPT STUDY
STRUT DETAILS
STRUT RECESS DETAILS
ELEVATION
SECTION
SIDE ELEVATION

DISTRIBUTION

Hq USAF, Wash, DC 20330	
(PREE)	5
(PRE)	2
(RDP)	2
(RDQ)	2
AFCEC (CC), Tydall AFB, FL 32401	5
AFSC, Andrews AFB, Wash, DC 20334	
(DO)	2
(DE)	2
(DLCAM)	1
TAC, Langley AFB, VA 23365	
(DEE)	2
(LGMD)	2
CINCSAC, Offutt AFB, NE 68113	
(DEE)	2
(XPFC)	2
ADC, Ent AFB, CO 80912	
(DEE)	1
(XPQDQ)	2
AU, Maxwell AFB, AL 36112	
(LDE)	1
(ED, Dir Civ Eng)	2
CINCUSAFE, APO NY 09012	
(DEE)	2
(DOA)	2
AFIT, WPAFB, OH 45433	
(Tech Lib, Bldg 640)	2
(CES)	1
CINCPACAF, APO San Francisco 96553	
(DEE)	1
(DOA)	1
USAFA, CO 80840	
(DFSLB)	2
(DFCE)	1
ASD, WPAFB, OH 45433	
(ENYEHM)	1
(Tech Lib)	3
(DEE)	1
AFATL (DLOSL), Eglin AFB, FL	2
RADC (Doc Lib), Griffiss AFB, NY 13441	1
TAC (DEE), Eglin AFB, FL 32554	1
Civ/Nuc Sys Corp, 1200 University Blvd NE, Albuquerque, NM 87102	1
DDC (TCA), Alexandria, VA 22314	2

AFWL, Kirtland AFB, NM 87117	
(HO, Dr. Minge)	1
(SUL)	2
(DEV)	5
(SAB)	2
(SAS)	2
USAMC, RSIC (Chief, Doc Sec), Redstone Arsenal, AL 35809	1
BRL, Aberdeen Pvg Gnd, MD 21005	
(AMXBR-TB)	1
(AMXBR-BL)	1
Dept Army (DAEN-RDM), Rm 5G044, Forrestal Bldg, Wash, DC 20315	2
Dir, USA Eng WWES (Tech Lib), POB 631, Vicksburg, MS 39181	2
ONR (418), Wash, DC 20360	1
Comd, NASC (AIR-52023), Wash, DC 20360	1
Dir, BRL (2027), Wash, DC 20390	1
Cmdr, NWC (753), China Lake, CA 93555	3
Cdr, NSWC (730), White Oak, Silver Spring, MD 20910	3
NCEL, Port Hueneme, CA 93041	3
Dir, DNA, Wash, DC 20305	
(SPSS)	1
(STTL)	2
(STSI)	1
DDR&E (Asst Dir, Strat Wpns), Wash, DC 20301	5
Dir, DIA, Wash, DC 20305	2
OSD, ARPA (NMR), 1400 Wilson Blvd, Arlington, VA 22209	1
FC DNA (FCPR), Stop 44	1
LVLO (FCTCL), POB 2702, Las Vegas, NV 89104	1
WSEGp (Doc Cont), Wash, DC 20305	1
JSTPS (JLTW), Offutt AFB, NE 68113	1
Sandia Lab, Albuquerque, NM 87115	
(Div 1314)	1
(Tech Lib)	1
Off Rcd Cy (Capt Choate, AFWL/DEV)	1