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FOREWORD

This report was prepared by Foster-Miller Associates, Inc., Waltham, Massachusetts under USBM Contract No. H0133050. The contract was initiated under the Coal Mine Health and Safety Act of 1969. It was administered under the technical direction of the Pittsburgh Mine Safety and Research Center with Dr. Richard Stein and Dr. Lew Wade acting as Technical project officers. Mr. Joseph Herickes was the contract administrator for the Bureau of Mines. The program was carried out at Foster-Miller under the general supervision of Mr. Adi R. Guzdar, with Dr. Kenneth R. Maser acting as program manager.

We would like to particularly acknowledge the contribution of the late Mr. Donald S. Kingery, whose insights into the potential application of emergency shelters determined the direction of this program.

COLOR ILLUSTRATIONS REPRODUCED
IN BLACK AND WHITE

SUMMARY

This report describes a research and development program sponsored by the United States Bureau of Mines. The purpose of the program was to develop reusable explosion-proof bulkheads for a cross-cut emergency shelter, and to study the implementation of such bulkheads, and the resulting shelters, as part of an emergency escape plan in a normal coal mining environment.

The first part of this report reveals a number of significant conclusions regarding the application of emergency shelters. A study of the causes of, and the conditions prevailing during a mine emergency indicates that ideal placement of the shelters would be midway along developing entries, and at the intersection of butt and main entries. The proper spacing of shelters is based on the travel capability of trapped miners, which is in turn dependent on the height of coal. The life of a given emergency shelter, based on an analysis of the above information, will be anywhere from 6 months to 10 years. Consequently, some shelters are regarded as "temporary" while others "permanent".

The configuration of bulkheads for an emergency shelter will be based primarily on cost and logistics. The initial notion of having an emergency shelter which is frequently moved to stay close to the working face has been disregarded based on the above analysis, and with it has gone the concomitant requirement for portability. Economics and logistics do favor, however, bulkheads which require minimum labor and material handling.

Based on the above ground rules, a number of explosion-proof bulkhead concepts have been developed and evaluated. The reusable concepts, which are totally new to this in-mine application, are generated from a variety of potential structural systems. The principle discipline evoked is the transfer of technology developed in the aerospace and nuclear fields of engineering to the mining environment. The major technical problem which occurs is the anchoring of an explosion proof

structure to the structurally unpredictable rock and coal which comprises the walls of the crosscut. This must be accomplished while, at the same time, maintaining an air-tight seal. A number of unique approaches to the anchorage and sealing problems are developed.

The alternative to the reusable bulkhead is a permanent structure. Though typically cheaper from the point of view of materials cost, these concepts are generally more intensive in labor, materials handling, and equipment requirements. A good deal of research into the construction of permanent, explosion proof structures in coal mines has been carried out in this country and abroad. Documentation of this work has been reviewed to assess labor and material requirements, construction techniques, test techniques, and potential concepts for emergency shelter application. It is found that most of the bulkheads which have been studied are excessively massive and leakage-prone for the application at hand. One concept, a fly ash-cement bulkhead, was considered to merit further consideration.

Since past investigations turned up little in terms of fruitful permanent structural concepts, two additional concepts were generated - one involving reinforced concrete, and the other involving shotcrete.

A cost analysis compares the overall labor and materials costs for the three above mentioned permanent concepts and the three "best" reusable concepts. The analysis indicates that the reusable and permanent concepts are competitive from a cost point of view, while logistically in terms of in-mine application it is much simpler to deal with a reusable type of structure.

It is concluded that due to the variability of support conditions in the crosscut, no one universal anchorage design for a reusable structure can be proposed for all situations. Consequently, the designs of the three selected reusable concepts - the truss, channel-turnbuckle, and liner plate arch, are developed in detail. A full set of detailed fabrication drawings have been supplied to the Bureau of Mines along with this report.

A system for sealing the irregular interface between the periphery of each bulkhead and the irregular passage walls has been developed. It consists of membrane of light gage aluminum flashing supported by sprayed-in-place frothed urethane foam. Due to the novelty of this sealing system, a test program was carried out to verify its pressure and relative displacement capabilities. The tests indicate that the seal can remain intact when subjected to a static pressure of 40 psi and a relative displacement of 3". This is sufficient to meet the bulkhead design requirements.

It is recommended that the anchorage components such as rock bolts, floor trenches, and rib abutments be tested in-situ before full scale explosion trials are attempted. A simple, portable test apparatus is proposed.

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1. Introduction

1.1 Background

The concept of "barricading" has long been recommended for those cases of emergencies where miners are trapped in by a fire. Barricading, which is the construction of a makeshift shelter in cross-cut or deadend, protects the entrapped miners from toxic gases and enables them to conserve a supply of air. In this manner, they can survive two or three days - hopefully enough time for them to be reached by rescue teams. In the past, many lives have been saved with this technique.

A modern outgrowth of the barricading concept is the establishment of pre-existing emergency shelters. These shelters would be situated at critical locations throughout the mine, such that they would provide readily accessible refuge for entrapped miners during an emergency. The concept of an emergency shelter can be implemented by sealing off both ends of a crosscut with two bulkheads, each fitted with a door. The shelter, which is the space between the bulkheads, could be equipped with emergency provisions capable, for example, of sustaining 7 to 13 men (a section crew) for several days.

The bulkheads themselves would serve to prevent the contamination of the supply of fresh air within shelter by toxic gases on the outside. The bulkheads would also have the structural strength to resist the pressures resulting from secondary explosions occurring after the initial fire or ignition.

There are some important questions which must be addressed before the emergency shelter concept can be practically implemented. These can be summarized as follows.

1. Placement - where shall the shelters be located, and how many are required?

2. Equipment - what supplies and equipment shall be provided in an emergency shelter to maximize the chances of survival of entrapped miners.
3. Bulkhead Design - what kind of structure would most efficiently and economically satisfy the sealing and explosion-proof requirements.

The problem of placement must consider the basic emergency escape plan of the mine. Shelters can be placed near the working face, along developing submains, or along main entries. It is important that the shelters be placed not only where they will be accessible during an emergency, but also where miners are likely to be when they realize that they must seek refuge in a shelter.

The equipment associated with a shelter has received considerable attention in a recent U.S. Bureau of Mines program carried out by the Westinghouse Corporation.^{(6)*} This program involved the development of a completely self contained chamber, the subassemblies of which were maneuvered into a cross cut and assembled therein. All life support provisions were supplied with the chamber. This chamber proved to be extremely cumbersome and labor intensive to erect, and its module seals failed to survive explosion trials. For the bulkheaded crosscut which we are considering here, an additional provision of a surface borehole can be considered. This, of course, is an additional expense, but it does provide an immediate lifeline to entrapped miners. In addition to serving as an access for air and provisions, the borehole can serve as a pilot hole for current state-of-the-art boring and hoisting equipment which can bring about the ultimate rescue of the miners.

The issue of bulkhead design combines structural, economic, and logistical problems. The structural aspects of the bulkhead are primarily involved with its ability to withstand explosion pressures while remaining structurally anchored and sealed to the indigenous strata.

*Numbers in parentheses refer to references listed in the bibliography.

The economic and logistic problems are closely related to bulkhead placement. If an emergency shelter is to remain close to the working face, then economics dictates that bulkheads should be of lightweight, modular, portable construction. If, on the other hand, shelters are to be permanent structures on main haulageways, then it may be more economical to use a massive monolithic structure (eg., concrete, fly ash-cement) similar to those which have been used for sealing off gob areas. Other factors, such as material handling costs, reusability of materials, required construction equipment, and required man hours of labor will also enter into the economic and logistic evaluation.

The following report concerns itself primarily with questions 1 and 3 - ie., the placement of shelters in a mine and the design of the bulkheads. As these two problems are mutually related, we find it natural to treat them simultaneously.

1.2 Scope of Report

The following report describes a research and development program sponsored by the United States Bureau of Mines. The concept evolution during its tenure resulted in a redirection of the initial R and D effort. Initially, the program set out to design an emergency shelter bulkhead and to study how the placement of bulkheads could take advantage of existing ventilation stoppings for optimum placement of emergency shelters. The bulkheads were to be "portable", so that the emergency shelter could follow close to the working face. Subsequent investigation indicated that shelters located in such a manner would be of little value to miners following typical escape procedures. Consequently, the issue of placement became a central issue, and this guided the subsequent bulkhead design effort.

The report treats three major areas of investigation;

1. Review of the Emergency Shelter Concept

This review is presented in Section 2. It deals with the application of emergency shelters in an emergency situation,

as well as the implementation of emergency shelters as a part of the overall mine emergency escape plan.

2. Evaluation of Alternative Approaches to Bulkhead Design and Construction

We reviewed background material and generated conceptual approaches for bulkheads which are either permanent installations or which are moveable, reusable installations. These two approaches are described in Sections 4 and 6 respectively. Section 5 presents an analysis of the in-mine cost of implementation of the proposed concepts.

3. Complete Design Details for Selected Reusable Concepts

The three reusable concepts which were selected are developed into a detailed design package, complete with stress analysis and assembly instructions.

2. Emergency Shelter Analysis

2.1 Definition of Terms

The following terminology will be used in this section.

Rescue Chamber - Refuge Chamber - Emergency Shelter - A pre-established shelter for the protection of entrapped miners after an explosion or fire has occurred.

Permanent Shelter - A shelter or chamber which has a life of 3 - 10 years.

Temporary Shelter - A shelter or chamber which has a functional life of 6 months to 3 years.

Permanent Bulkhead - A shelter bulkhead which cannot be moved without destroying it.

Reusable Bulkhead - A shelter bulkhead which can be disassembled and reassembled in an alternate location.

Portable Bulkhead - A reusable bulkhead specifically designed for frequent relocation.

2.2 Emergency Safety Requirements

Emergency disaster protection in coal mines must include escape-ways from operating faces to fresh air, escape facilities in shafts and slopes, individual protective devices, and as a final resort, established and well-equipped rescue chambers. Research to develop special life support systems and mobile escape vehicles has been initiated; however, the mine worker today must be able to follow planned escape procedures and make individual judgements. Numerous lives have been sacrificed because designated escapeways were not open, men were not trained, and self-rescue equipment was either not available or faulty.

Practically all state mining laws contain special provisions directed toward emergency escape of workers. Yet nearly every year, some disaster occurs and men become trapped in by a fire. The Federal Coal Mine Health and Safety Act of 1969 in provisions 75-1704 and 75-1704-1 specifically deals with escapeways: the number to be provided, maintenance, regular inspection, posted maps, training, and special facilities. Emergency shelters per se are not included as a mandatory provision of the law.

2.3 Mine Operating Practices

Modern coal mine operating practices can be divided nearly equally between continuous miner production and conventional mining. Concentrations of operating sections is practiced with both systems; consequently, three or more producing units may work in adjacent entries utilizing the same secondary transportation system. This concentration of work endangers escape possibilities in the adjacent entry should a serious disaster occur in one entry. Men probably will not be affected by the explosion, yet, a severe shock wave could destroy stoppings and overcasts on connecting main entries. Such a situation could require the use of self-rescuers to reach fresh air.

Mine fires constitute the greatest hazard to trapped workmen. Such fires commonly occur in face areas on mining equipment, on or along belt conveyors, and on main haulage roads from power wires. Such fires are usually extinguished quickly, yet by occurring on intake air, they endanger all workers in by unless a separate intake escape entry is maintained. Such an entry is required under certain provisions of the Federal Law. Fires resulting from explosions can occur and when this happens a very dangerous situation exists. Ventilation is disrupted, methane continues to be released; consequently, a secondary explosion is possible -- often, more violent than the first. Such events necessitate some refuge protection for remaining employees in adjacent operations. There have been several explosions that were not detected in other parts of the mine; one such occurrence involved a second explosion before the mine workers were evacuated.

The height of the coal bed will have a direct bearing on escape-way effectiveness and the possible need for established refuge chambers. It is not physically possible for middle-aged workers to travel long distances on hands and knees or in a crouched position. An emergency shelter established at a reasonable distance from working areas could continue to add to the potential safety of such workers. Present mine ventilating practices provide separate splits for each working section or unit. Such practice removes the danger of contaminated air from passing over adjacent workmen. It also provides time for workers to leave their working areas and execute a prepared escape plan.

2.4 Emergency Shelter Application

Emergency shelters should be used only as a last resort when no possible way out exists, or for seriously injured men that cannot travel and it is known that the air is becoming contaminated.

2.4.1 Emergency Shelter Design

The concept of temporary shelters and their application for "stand by" emergency shelters has both advantages and disadvantages. The principal disadvantage is space when utilizing a single breakthrough. The number of persons that must be sheltered will range from seven to thirteen depending upon the type of mining and transportation systems. The estimated time of entombment before drillhole contact will be three days, possibly more. Using the common figure of 500 cu. ft. per man per day, it is evident that few men or very few hours can be provided for by a single breakthrough emergency shelter, even in high coal. Consequently, larger areas will be required or else some oxygen source must be provided. This may require a special mine plan to provide the maximum area for the minimum bulkhead installation. The following volumes will result from a single 80 foot breakthrough:

<u>Coal Height</u>	<u>Passage Width</u>	
	<u>15 ft.</u>	<u>20 ft.</u>
30"	3,000 cu. ft.	4,000 cu. ft.
42"	4,200 cu. ft.	5,600 cu. ft.
45"	4,800 cu. ft.	6,400 cu. ft.
60"	6,000 cu. ft.	
72"	7,200 cu. ft.	

Providing additional chambers is not practicable because in the event of actual use each chamber will need a surface bore hole connection. A more logical approach is to split the pillar where the chamber is to be located and bulkhead two breakthroughs, as shown in Figure 1. This will provide three equivalent areas.

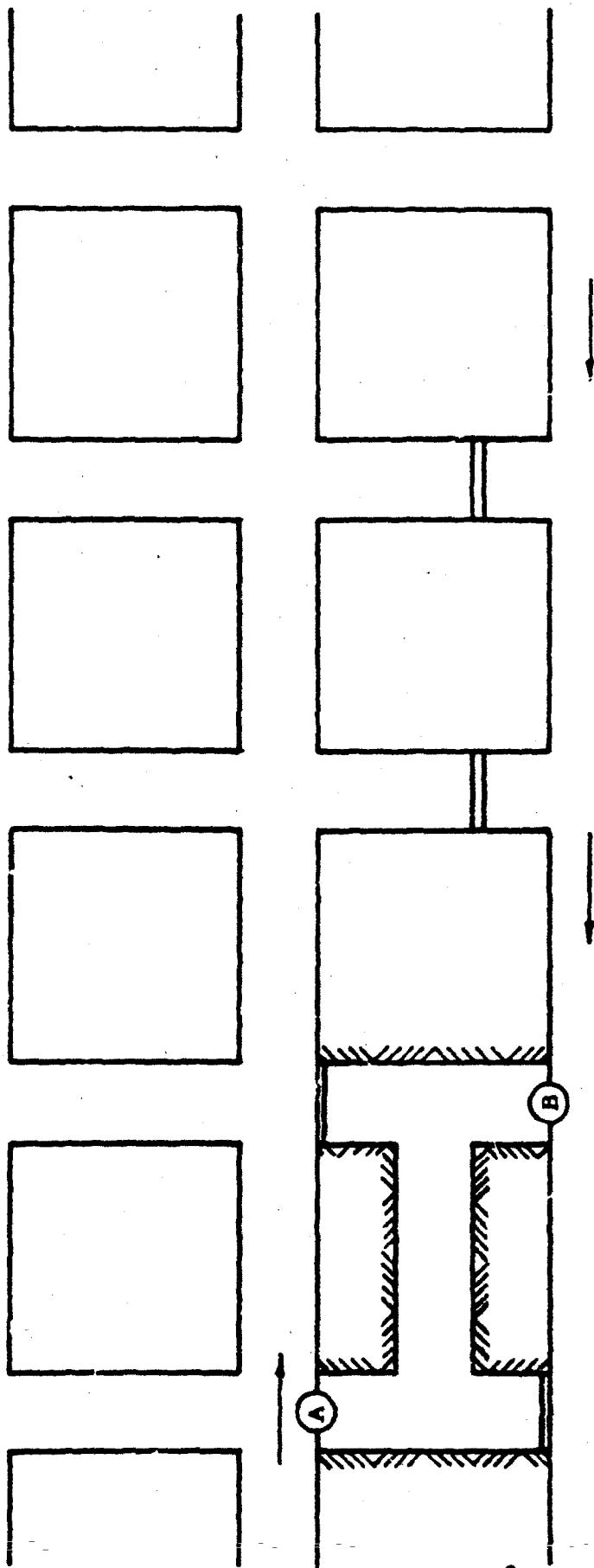
Supplemental oxygen sources such as "O₂" candles could introduce hazards due to methane release within the chamber. Oxygen cylinders could serve as an emergency source of oxygen; however, control and discipline must be used to gain maximum efficiencies. The best arrangement is surface boreholes.

2.4.2 Location and Use

The location and number of emergency shelters needed will depend upon the time available for escape.

The time interval after an explosion-type disaster for escape in most cases will be unlimited, except for those working in the disaster area.

The time interval for escape when a mine fire is involved may be very limited unless some warning system is effective. This is the purpose of law requirements for multiple escapeways. Consequently, the greatest need for emergency shelters are for refuge when trapped inby mine fires. The number of shelters required will depend



Note: A and B to be specially constructed bulkheads with doors. "A" on intake and "B" on return side of the ventilation currents - others to be ventilation stoppings reinforced to meet explosion pressure requirements.

Additional chambers to be installed contingent to height of coal, as entries advance.

Split Block Shelter

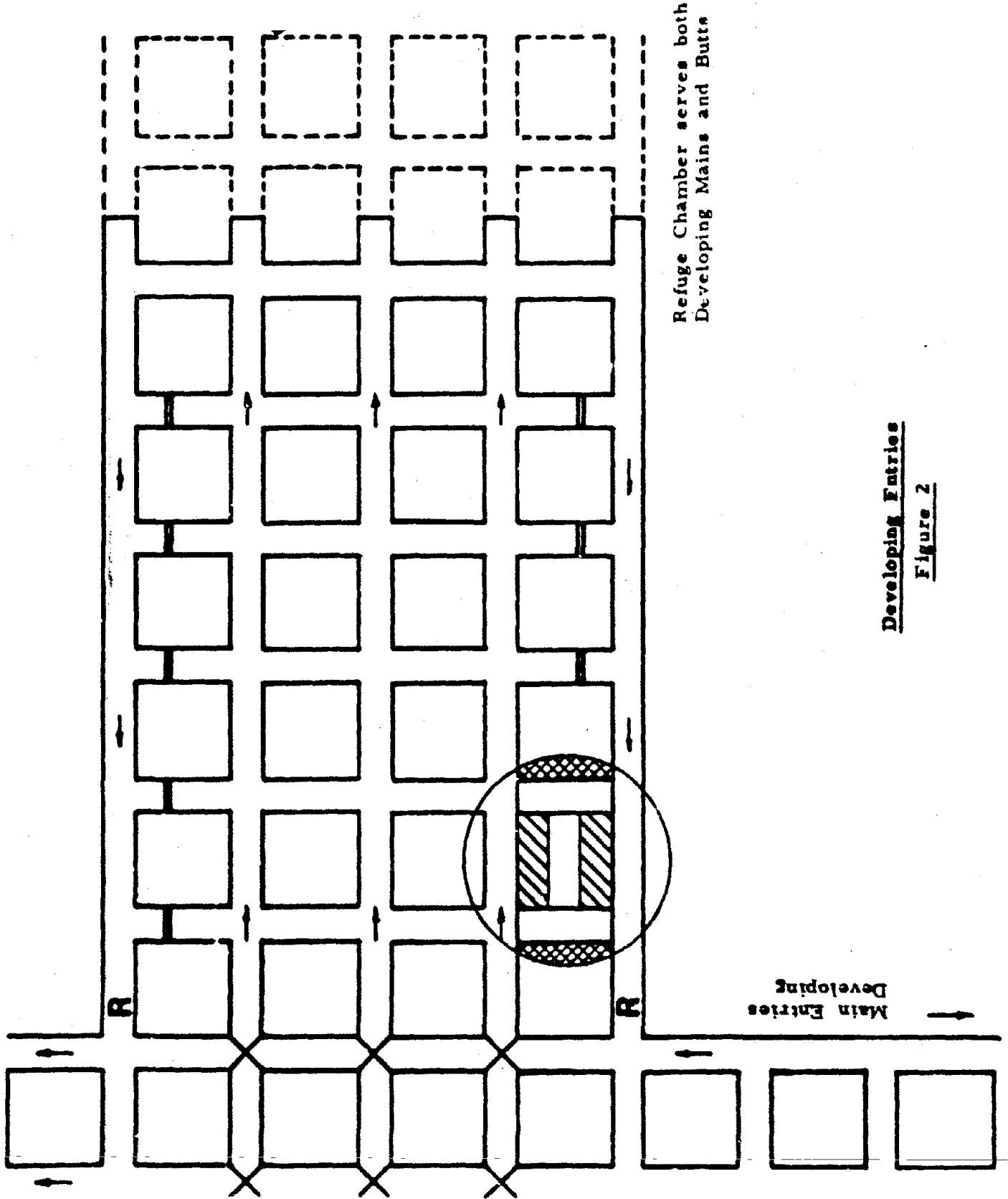
Figure 1

upon the mine development, concentration of operation, the number of operating sections, coal height and the mobility of the individual miners. The ease of travel will be the limiting factor to escape possibilities and refuge chamber proximity. U. S. B. M. Information Circular 8127, Escapeways and Other Emergency Measures in Coal Mines by R. W. Stahl, discusses travel speeds in escapeways and the need to outdistance air velocities when fires occur on intake air courses.

The following emergency shelter intervals are based upon the assumption that those workers not directly involved in a disaster will have a minimum of 60 minutes to make an escape effort. Based upon the travel speeds listed in U. S. B. M. I. C. 8127, the following distances apply:

<u>Height of Coal</u>	<u>Shelter Interval</u>
30 inches below	1,500 feet
42 inches below	2,500 feet
60 inches below	3,000 feet
60 inches above	4,000 feet

It is projected that a shelter can be installed at the start of each development panel or series of butt entries (see Figure 2). This shelter will serve the development work along the butt and also provide standby refuge for further inby operations that find escapeways blocked out by this shelter. Such a shelter may be considered permanent. Its usable life could economically support borehole connections to the surface.



Refuge Chamber serves both
Developing Mains and Butts

Developing Entries

Figure 2

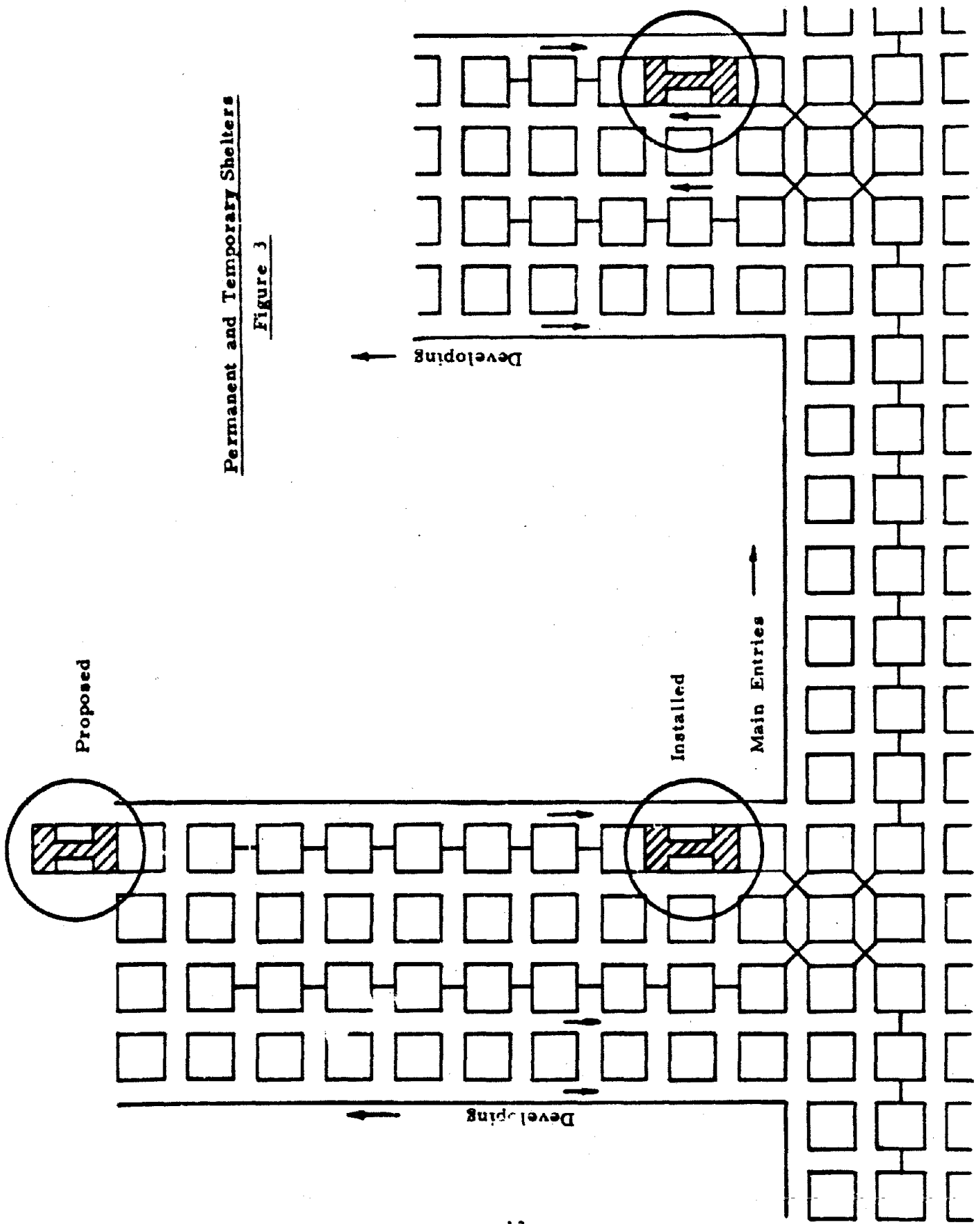
Main Entries
Developing

The temporary shelter concept is best applied to intermediate installations within the panel entries as they advance to prescribed distances (Figure 3). Such shelters will serve further development and retreat operations and can be expected to have a usable life of possibly one year or more. Such shelters probably will not support borehole communications to the surface unless required by law. The number and spacing of such shelters will depend on the coal height, as outlined above.

The type of mining system, whether conventional or continuous miner operations, will not affect shelter locations or design criteria except for space requirements; longwall operations possibly will require shelters in both the head and tail access entries to assure protection for all face workers. Hand loading operations with room development easily can prepare a worked out room for emergency use. Figures 4, 5, and 6 show schematically the layout of emergency shelters for coal seams of three different thicknesses.

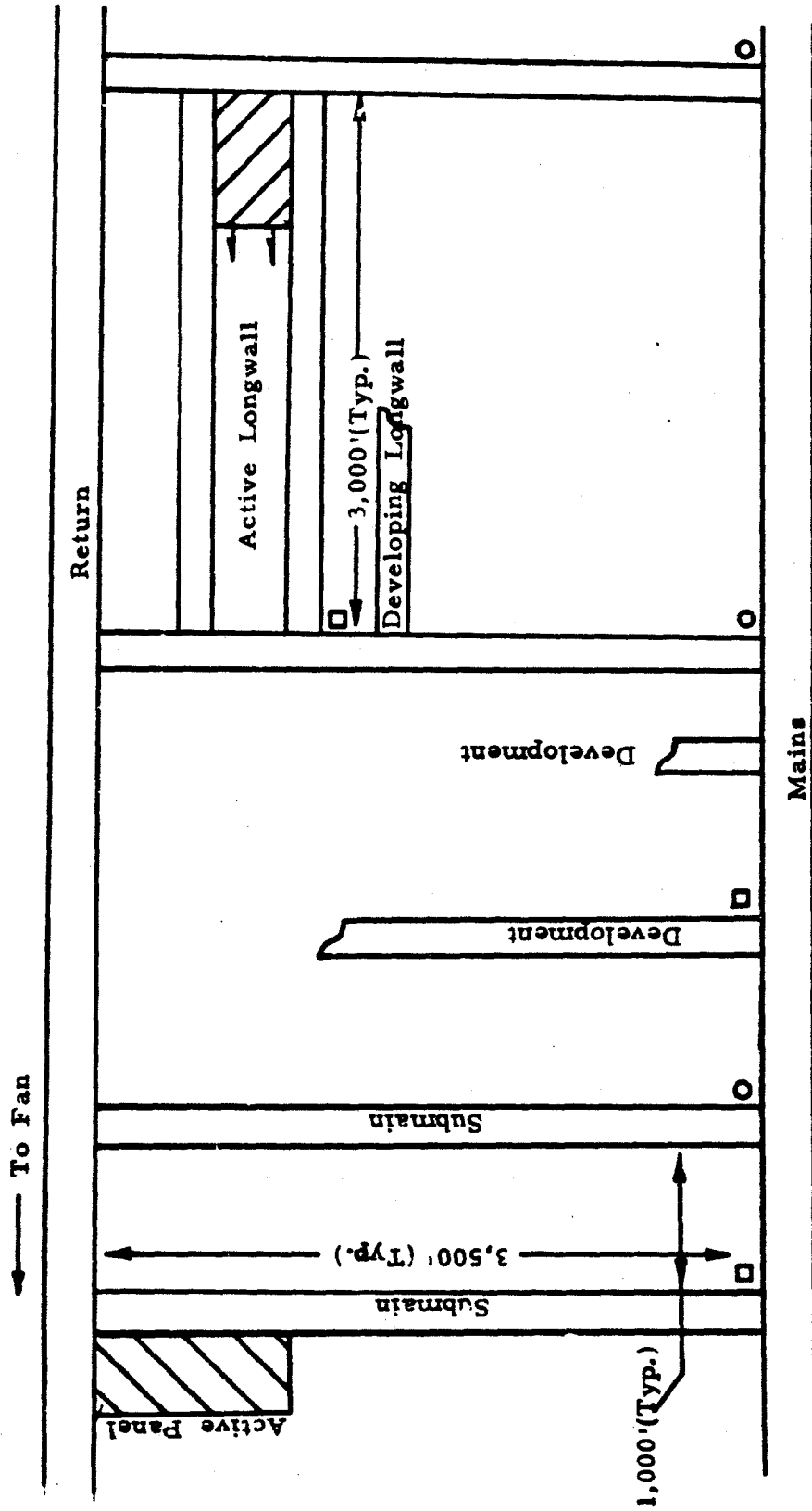
The installation and maintenance of bulkheads and emergency shelters is most important and should be an assigned responsibility to a top mine official. Special construction crews with transportation and special tools and equipment may be beneficial to assure proper erection. Supplies of material in the shelter must be checked regularly and the integrity of the chamber checked in the event of mine roof stress changes.

Each shelter should have a telephone connection to a central point. The telephone lines should be buried or otherwise protected. Another advantage to prepared boreholes is communication as well as oxygen supply.



Permanent and Temporary Shelters

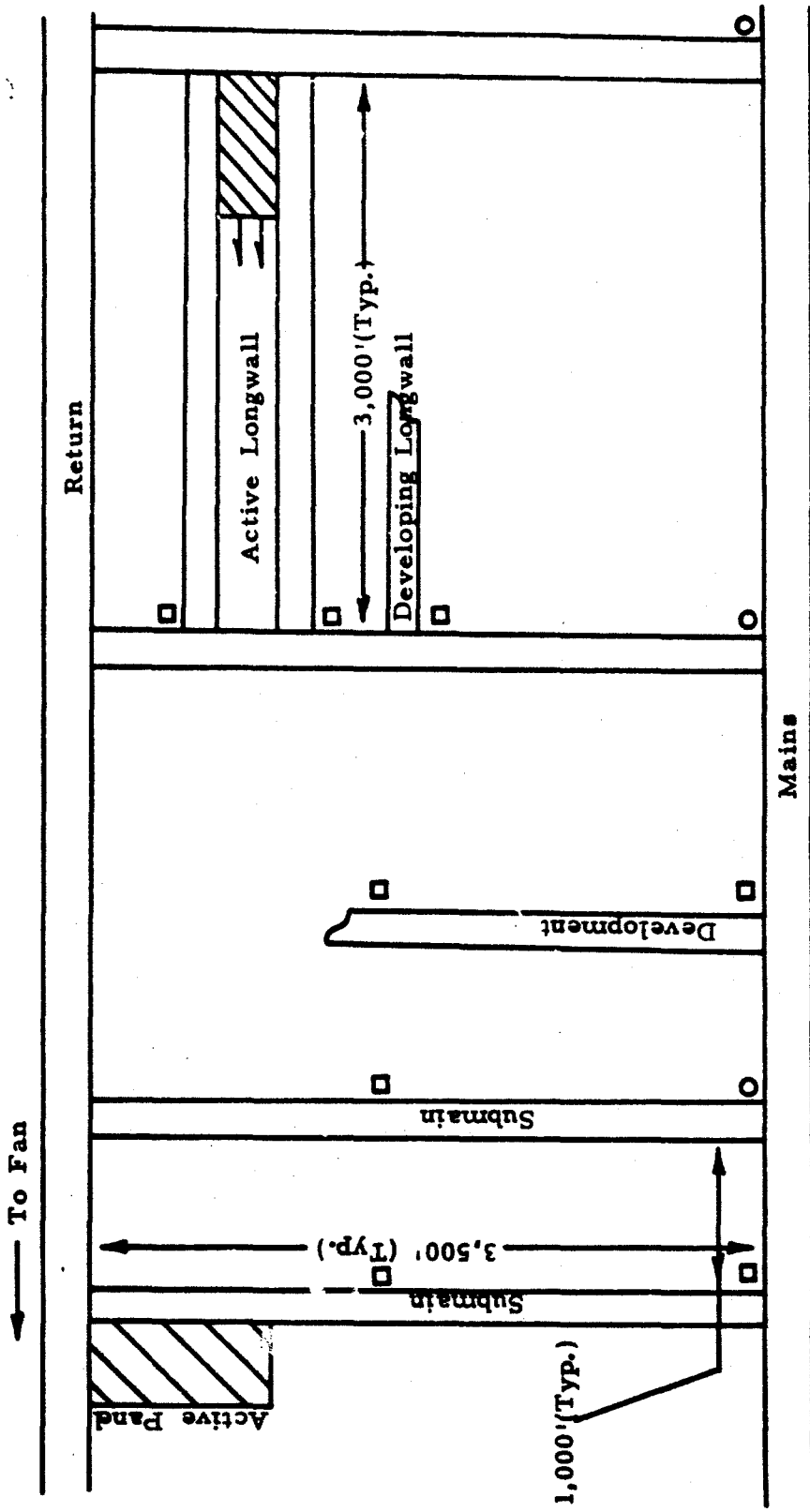
Figure 3



- 3 □ Permanent Shelters
- 3 ○ Reusable Shelters

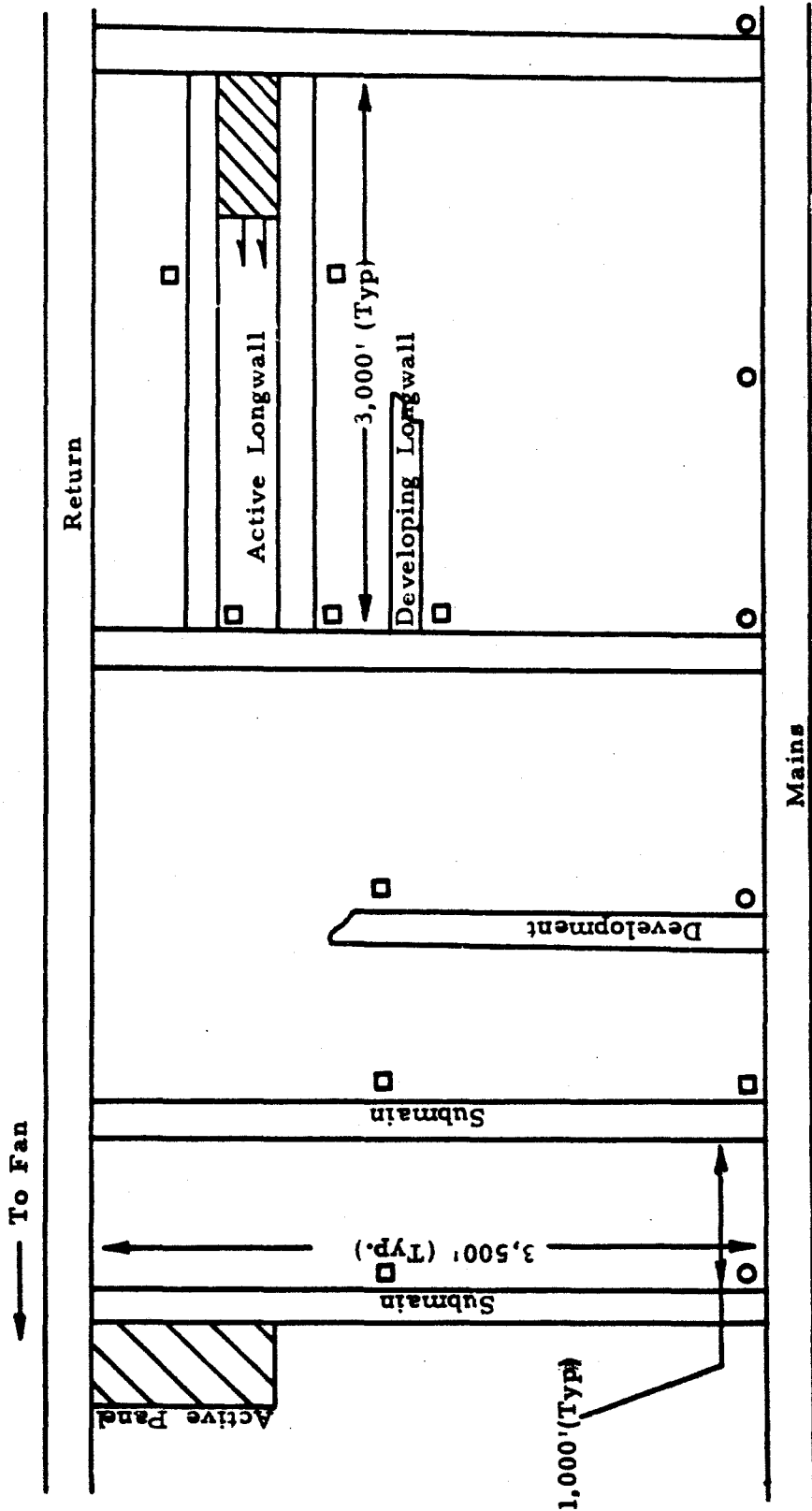
Sample Emergency Shelter Layout - 60+ inch Coal Seam
 3,000' to 4,000' Shelter Spacing

Figure 4



3 ○ Permanent Shelters
 8 □ Reusable Shelters
 Sample Emergency Shelter Layout 48" Coal Seam
 2,500' to 3,000' Shelter Spacing

Figure 5



5 ○ Permanent Shelters
 9 □ Reusable Shelters
Sample Emergency Shelter Layout 36" Coal Seam
2,000' to 2,500' Shelter Spacing

Figure 6

The cost for providing a single emergency shelter is estimated to be within the range of \$12,000 to \$17,000, provided a borehole to the surface is included. Reusable material will reduce the cost of intermediate or temporary bulkheads.

Assuming that such shelters will be required by Federal Law, the number of units based upon present coal mine operations will be tremendous. There are approximately 6,000 operating sections that will require protection. Based on an average of 1.5 shelters per operating unit the total number of shelter installations will approximate 9,000. Annual replacement is estimated at 4,000 per year.

3. Development of Reusable Bulkhead Concepts

The principal thrust of the R and D program on which this report is based was the development of reusable bulkhead concepts. These concepts were primarily oriented toward the temporary shelter concept, so that the economic advantages of material reusability, portability, and ease of construction could be most favorably realized. It is also conceivable that these economic advantages could be sufficient to make a reusable design practical for a permanent shelter.

As will be described in greater detail in this section, we have found that there are numerous design approaches which can be followed in the development of a reusable bulkhead. All, however, have one design problem in common; that of anchoring any explosion-proof structure to the rib, roof, and floor of a mine passage.

We have broken down the design of a reusable bulkhead into three principal design areas:

Main Structure
Anchorage
Sealing.

In this section we shall first review some general design requirements which will be followed in developing a reusable bulkhead design. We shall then review conceptual approaches to the three principal design areas. These conceptual approaches will lead to the development of some specific component designs and these components will be synthesized to form a number of complete bulkhead designs.

3.1 Design Requirements

The criteria and constraints which must be met by a given reusable bulkhead design fall into three categories: those associated with its function as part of an emergency shelter; those associated with the requirements of a rapidly erectable, reusable bulkhead; and those

associated with the survival of such a bulkhead in the normal mine environment once it has been erected. The specific requirements are outlined below.

3.1.1 Functional Requirements

To provide refuge for entrapped miners, an emergency shelter bulkhead must provide a means for rapid entry, and must provide protection from explosions and toxic gases which are generated during a fire. The bulkhead is not intended to provide thermal insulation for entrapped miners. It is assumed that a fire very close to the emergency shelter will transmit heat through, and cause deterioration in the adjacent coal, which would render a heat-shield type bulkhead useless.

3.1.1.1 Loadings Due to Mine Explosions

Peak Pressure Intensity

Considerable experimental research has been carried out on mine explosions at the USBM Experimental mine in Bruceton, Pa. As a general rule from numerous simulated mine explosions, the intensity of an explosive pressure wave 200 feet or more from the origin of an explosion does not exceed 20 psig unless coal dust accumulations are abnormal and the incombustible content of the dust is far less than that allowed by law. On this basis, a 20 psig pressure wave has been used as the design condition (see Appendix A for a more detailed discussion).

The emergency shelter bulkhead will be located in a cross-cut about 5' from the intersection of the cross-cut and the entry. In this configuration, the bulkhead experiences side-on, rather than reflected pressure. This implies that the bulkhead will experience the 20 psig pressure loading, and that this loading will propagate down along the width of the bulkhead at the speed of sound.

Pressure Pulse Rise Time and Duration

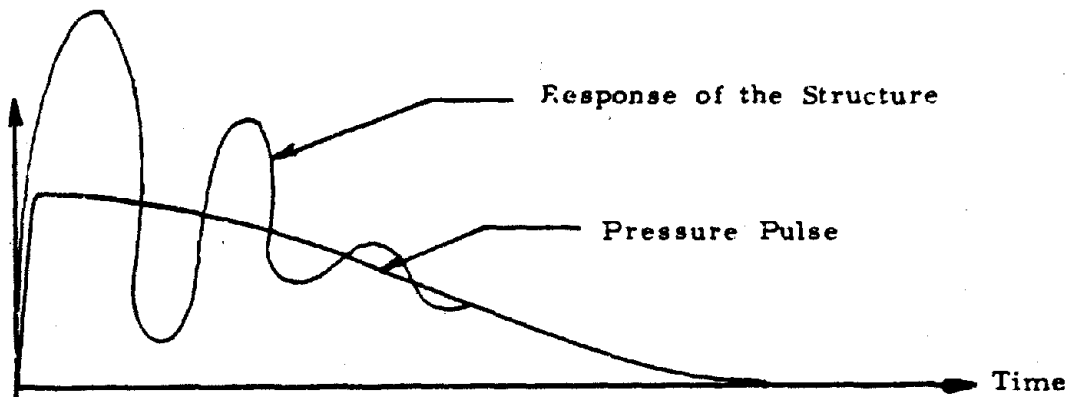
In simulated mine explosions, slow rising as well as shock front type pressure pulses have been produced. Since a shock front, i.e., an instantaneous pressure rise, is the more severe loading, it will be assumed in the design of the bulkheads and their components.

The duration of explosive pressure pulses have been shown typically to range from 0.1 to 1.0 seconds. The typical response time of bulkhead components is on the order of a millisecond. Consequently, the bulkhead sees the applied loading approximately as a step of infinite duration. This characterization is also conservative, and allows for rapid sizing and evaluation of bulkhead components.

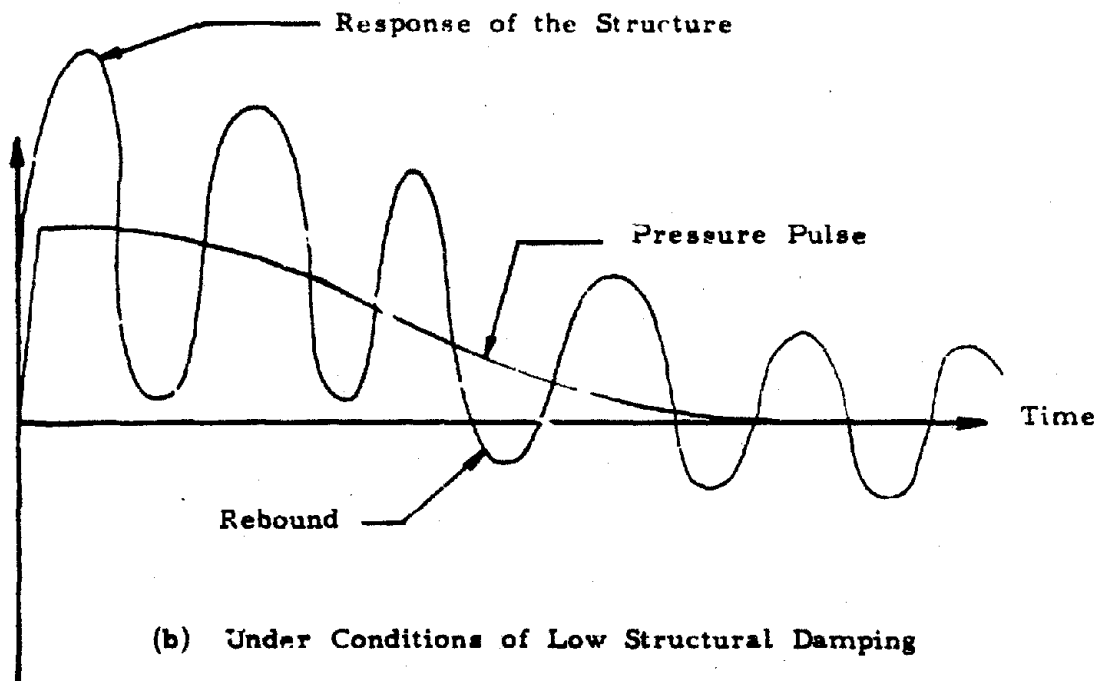
Negative Pressure and Rebound

Following the positive pressure pulse is a negative pressure phase with a magnitude which is generally found to be less than 1 psig. Structural oscillations also cause negative loadings. The degree of this loading depends on the amount of damping in the structure and the duration of the pressure pulse. Since the load duration is very long, there will be perhaps 10 to 100 structural oscillations before the load begins to decay. During this time, damping will decrease the magnitude of these oscillations. If the damping is high, as shown in Figure 7(a), there will be no negative deflections and hence no rebound loading. If the damping is low, as shown in Figure 7(b), the structure will rebound and deflect in the opposite direction. The response of Figure 7(a) is more probable due to the energy dissipation which occurs by inelastic deformation of the rock and coal, and by friction at joints. However, since this can not be quantified, some account must be made for rebound.

Previous structural design criteria^(6,8) have specified a static pressure of 5 psig for the negative pressure load.



(a) Under Conditions of High Structural Damping



(b) Under Conditions of Low Structural Damping

Figure 7 - Influence of Structural Damping on Rebound

This combines both negative pressure phase and rebound. In the absence of any other quantitative information, this criterion has been used for this analysis.

3.1.1.2 Leakage

Our specifications require that men inside the shelter be protected from the encroachment of external toxic gases on their available air supply. This implies that the joints between structural components, as well as the gaps between the structure and the roof, floor, and ribs be tightly sealed. We have not specified an allowable leakage rate.

The requirements for sealants, as interpreted from the above, are summarized as follows.

a. The seal should be capable of withstanding the applied pressure without popping out, cracking, or failing in any other mode.

b. The seal should yield to the relative displacements of the surfaces being sealed without allowing leakage.

c. The seal should survive the passage of an explosive flame front without the destruction of its integrity.

3.1.1.3 Access

The bulkhead is required to be fitted with an access door which has a quick-release latch. The door must survive the over-pressure environment, and must also provide a tight seal.

3.1.1.4 Thermal and Flammability Requirements

As mentioned earlier, sustained high temperatures outside of the chamber would likely be fatal to the miners inside regardless of how well insulated or heat proof the bulkhead may be. Consequently, 200°F has been conservatively chosen as the design temperature for all bulkhead materials.

All exposed bulkhead materials are required to be non-flammable. More specifically, the bulkhead may be exposed to the fireball associated with an explosion. This is essentially a flame front which would pass by the bulkhead in a fraction of a second. This transient flame front is taken as the flame source which must be resisted by all bulkhead components.

3.1.2 Assembly Requirements

The following considerations must be applied in order to develop economical, practically implementable reusable bulkhead design.

(1) Ease of Assembly - The erection procedure must be straightforward, and require a minimum of special skills. On-site cutting and drilling should be minimized. The alignment of various components should either be simple, or should allow for large tolerances. The number of connections should be minimized.

(2) Ruggedness of Components - Components are assumed to be handled roughly in a mine, and hence they should be as rugged as possible.

(3) Height Flexibility - Within a given coal mine, passage height variations of 6 inches to 12 inches are common. It would therefore be desirable to design a bulkhead in such a way that it can adjust to these local height variations. This is not

to say that the same set of bulkhead components designed to fit a 3 foot passage will also fit a 6 foot seam. In this case the same basic design would apply, but different components would be fabricated for different seam heights. Using this design philosophy then, bulkheads will be designed such that a given coal mine should be able to use the same basic units for all of its bulkheads.

(4) Equipment - Designs should be oriented towards equipment and tools that are generally found in a mine, and whose use will be familiar to mining personnel. Such equipment includes continuous miners, cutting machines, hand- and post-mounted electric augers, jackhammers, stopers, small electric drills, roof bolting machines, compressors, and small hand tools. Drilling and shooting is considered to be an acceptable method for trenching the floor and squaring the ribs.

(5) Sensitivity to Workmanship - A structure in which a high degree of accuracy or precision is not required during construction is to be preferred. Structures requiring accurately positioned roof bolts, for example, do not respond well to this criterion. Ditching, if carried out improperly, can destroy the competence of the floor and result in a poorly supported structure. Such operations, although perhaps not completely avoidable, should either be minimized or should be associated with other redundancies to minimize the reliance on the quality of workmanship.

(6) Weight of Components - Rapid assembly by two men suggests that no individual component of the bulkhead assembly weigh more than 150 lbs.

(7) Size of Components - Size of bulkhead components must be such that they can be easily transported on mine vehicles throughout the mine.

(8) Sealing - The structure should be designed to minimize the number and size of the joints which have to be sealed.

(9) Material Availability - Materials and hardware for bulkheads should preferably be drawn from standard, off-the-shelf components where possible. This should minimize problems associated with replacement of damaged or lost components.

(10) Cost of Materials and Labor - Cost of materials and required manhours of erection labor should be minimized. Total cost will eventually suggest the most acceptable bulkhead concepts.

3.1.3 Requirements for Survival in the Normal Mine Environment

Aside from its specialized functions, the reusable bulkhead must endure the normal mine environment without deterioration of its functional characteristics. These environmental requirements are listed below.

(1) Corrosion - The normal damp, acidic atmosphere in the mine tends to accelerate corrosion. The materials of construction of bulkhead components must have adequate corrosion resistance.

(2) Convergence (Squeezing) of Roof and Floor - Normal ground motions which occur after coal is mined include sagging of the roof and heaving of the floor. The total convergence during the life of a reusable bulkhead may be several inches. Bulkheads must be designed such that this convergence does not destroy the bulkhead or its seals.

(3) Rib Sloughing - A seal to an initially solid rib will be destroyed if sloughing occurs. Measures must be taken to prevent sloughing at seal locations.

3.2 Description of Principal Design Problem Areas and Conceptual Solutions

The major design problem areas have been subdivided into main structure, anchorage, and sealing. The specific problems and

conceptual solutions associated with each category are described below.

3.2.1 Main Structure

There are a number of basic structural types which can span the opening of a cross-cut and resist the pressure load. The design of such structures has become standard practice during the past 30 years, primarily due to advances in the fields of aerospace and nuclear blast protection. Our principal challenge is to apply some of this technology to the unique emergency shelter application. Some of the basic considerations are outlined below.

3.2.1.1 Direction of Span

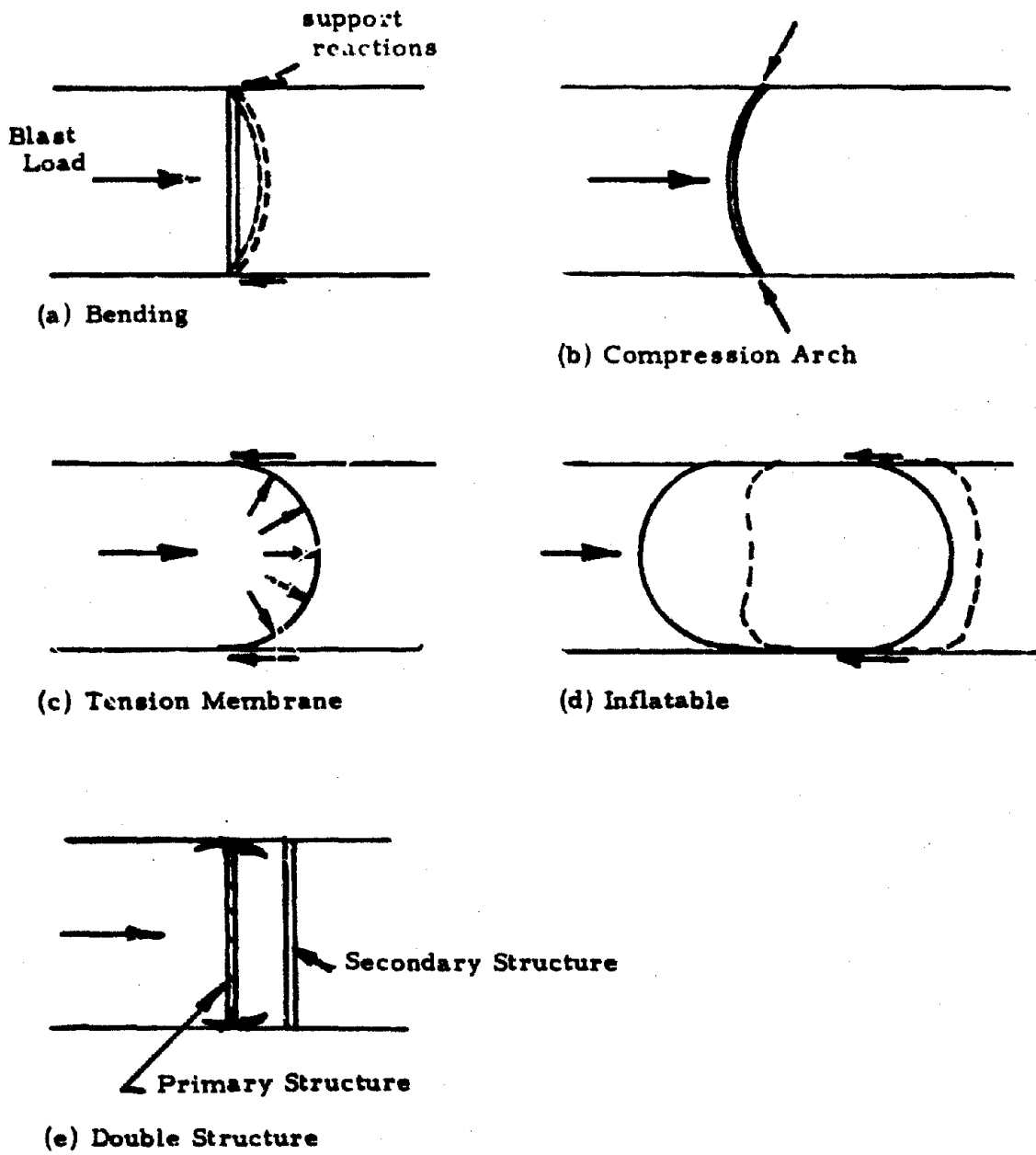
Initial concepts sought to span the mouth of the cross-cut horizontally with beams that overlapped the ribs. However, a coal mine passage is usually much wider than it is high. Consequently, it is more structurally efficient to have the structure span from roof to floor. This will be especially significant in low coal, where the horizontal span can be as much as eight times the vertical span. On the other hand, as will be described later, the anchorage problems associated with vertical spans can be considerably greater, and hence there may be a trade-off between these two approaches.

3.2.1.2 Type of Structure

There are a number of ways in which a bulkhead structure can carry the load. These are depicted in Figure 8, and described below.

(1) Bending

The structure consists of a plane spanning between two surfaces (e.g., roof to floor). It is the most straightforward way of carrying the load. However, it requires support



Alternative Structural Concepts

Figure 8

through high shear loads applied to the rock strata and is thus dependent on the strength characteristics of the strata.

(2) Arching

A circular arch subject to pressure loading transmits loads tangent to its surface to the supporting material. This places the support material in compression, which is a desirable condition for materials such as rock and coal. The arch itself is in compression, and its design is controlled by stability considerations. If the arch spans from rib to rib, then it will be subjected to flexure due to the passing pressure wave, and this may be a more severe design criterion.

(3) Tension Structure

The most efficient way of carrying the load is in tension. This will result in the lightest type of structure. The structure may be either flexible, and shaped into a semicircle on site, or it may be assembled from thin but rigid pre-formed semicircular sections. In either case, shear loads would be applied to supporting rock and coal strata with the inherent advantage that the pressure loading tends to press the edges against the support.

The principle disadvantage of such structures is their lack of inherent resistance to rebound or negative pressure. Also, their fragility, or flexibility, may make them more difficult to handle.

(4) Inflatable Structures

These would be inflated at the site to a nominal pressure required for positioning and stability. Some allowance would have to be made for access into the shelter.

The applied loads on an inflatable structure are resisted by friction at the contact surfaces with

the bordering rock. Excessive movements of the structure may occur before this friction is developed. In addition, local peaks in friction force due to irregularities in the rock surface could easily cause tearing in the inflatable skin. These possibilities are difficult to evaluate without some extensive testing. We believe that the inherent problems associated with inflatables preclude any further consideration within the scope of this study.

(5) Double Structures

One potential concept is to build two structures - a primary structure to resist the blast load, and a secondary structure to provide a gas-tight seal. The purpose is to separate the structural function from the sealing function, and perhaps, in this way, develop a concept which is simpler and cheaper than the single bulkhead approach.

Unfortunately, this concept is only feasible for short pressure pulses. For longer pulses, similar to those which we expect, the leakage of the primary structure can be sufficient to allow pressurization of the intervening space, and thus apply a pressure loading on the secondary structure.

3.2.1.3 Mode of Dynamic Response

The dynamic response refers to the way in which the bulkhead absorbs the energy imparted by the blast. The structure, or various portions of the structure, can respond in an elastic, plastic, viscous, or in some combined mode; each type of response represents a different form of energy absorption.

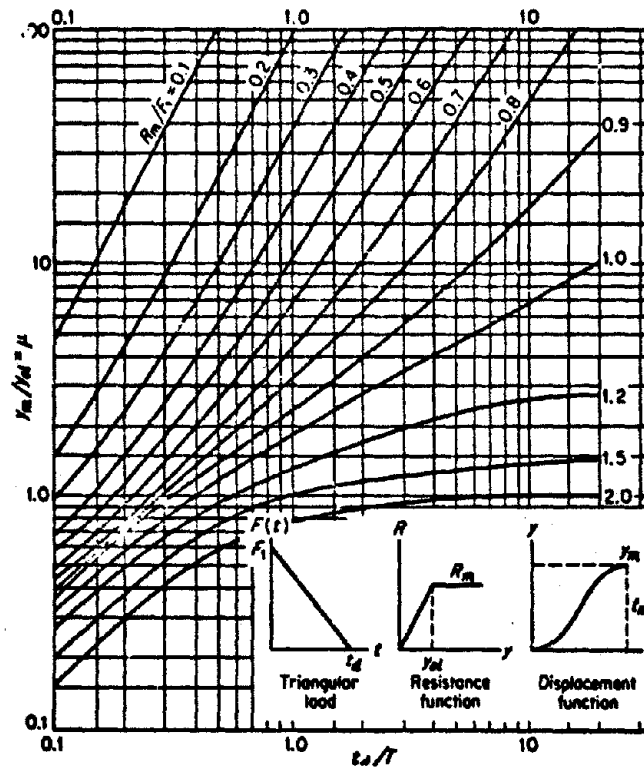
In order to assess the merits of the different forms of energy absorption, one must first develop a dynamic model of the system. For the design of a blast resistant structure, it is sufficient to model the structure as an equivalent one degree-of-freedom system.⁽⁹⁾ Using this type of model, some basic conclusions concerning structural response can be reached.

Figures 9 and 10 illustrate the elastic-plastic response of a one degree-of-freedom system, each with a different model for the load-time function. As mentioned earlier, the duration of the blast loading, t_d , is on the order of one second. The resistance function represents the static load capacity of the structure. The structure responds elastically until it reaches a deflection y_{el} ; its plastic resistance is then idealized by a constant resistance R_m . The maximum deflection of the structure is designated as y_m , and the natural period of the structure is designated as T . Different types of response, as shown in Figure 11, are described below.

(1) Elastic Structure with Rigid Supports

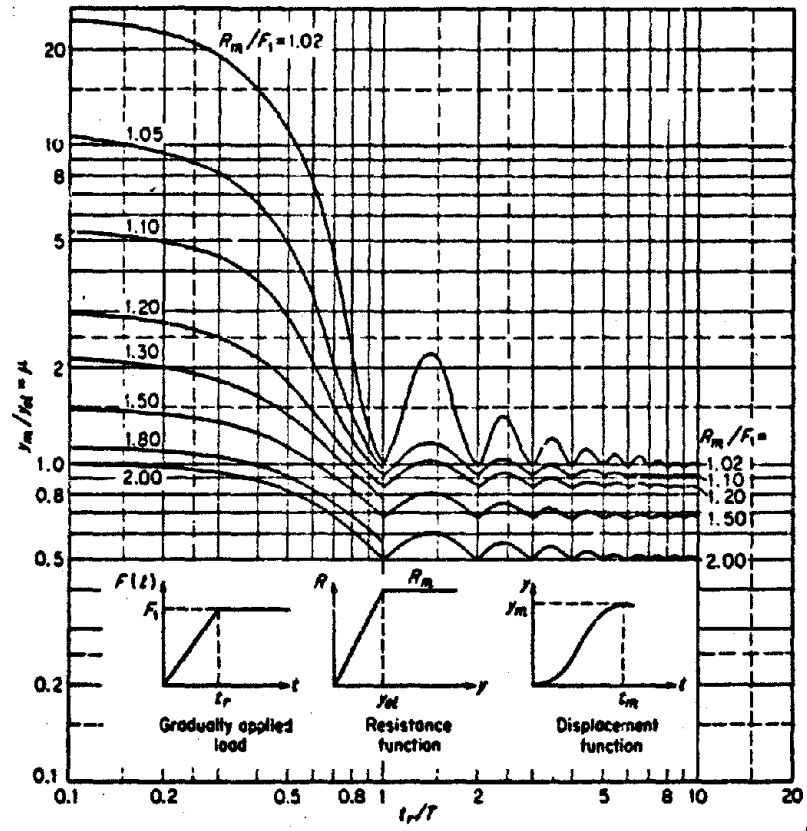
Referring to Figure 11(a), the basic parameter which defines the system response is the ratio of the load duration to the natural period of the structure, t_d/T . A typical steel structure spanning the mine opening will have a period on the order of 1 millisecond (msec), and a concrete structure may have a period on the order of 10 msec. In either case, the ratio t_d/T is large, greater than 100. The requirement for the structure to remain elastic implies that $y_m/y_{el} \leq 1$, and from the design curves, $R_m/F_1 = 2$. This means that the maximum resistance of the structure must be twice the maximum pressure loading; in other words, the structure must be designed for 40 psi static loading.

In the above discussion we have idealized the blast loading as instantaneously applied. In many cases we will have a finite rise time rather than a true shock front. Suppose, for instance, that the pressure wave has a rise time of 1.5 msec. Then referring to the design curves of Figure 10 for an elastically responding structure with $y_m/y_{el} = 1$, and a natural period of 1 msec, $R_m/F_1 = 1.3$. In other words, the structure need only be designed for a static load of $1.3 \times 20 = 26$ psi.



Response Curves for a One Degree-of-Freedom Elasto-Plastic System - Triangular Load with Instantaneous Rise Time (2)

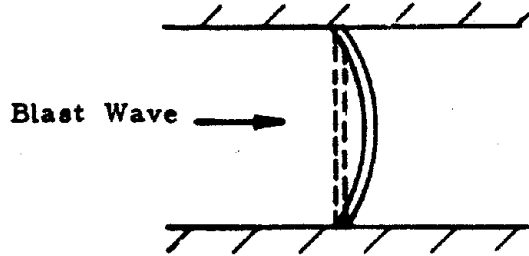
Figure 9



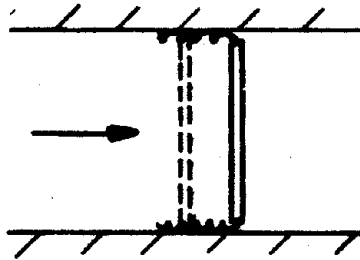
Response Curves for a One Degree-of-Freedom Elasto-Plastic System - Constant Load with Finite Rise Time(2)

Figure 10

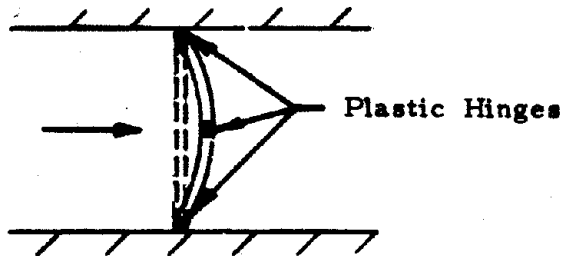
(a) Rigidly Supported Elastic Structure



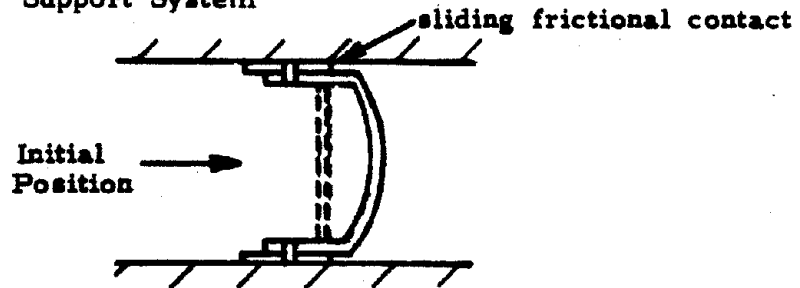
(b) Flexibly Supported Elastic Structure



(c) Elastic-Plastic Structure



(d) "Plastic" Support System



Types of Structural Response

Figure 11

Unfortunately we have little information on the rise times of the pressure waves due to mine explosions. Consequently, we can only take advantage of the fact that the existence of a finite rise time gives us a built-in factor of safety.

(2) Flexibility Supported Elastic Structure

One possibility of reducing design loads is to increase the natural period of the structure by providing flexible supports. This situation is depicted in Figure 11(b). The design curve of Figure 9 indicates that the design load can be reduced to 30 psi ($R_m/F_1 = 1.5$) if the natural period of the structure equals the load duration. Suppose we have a structure with mass M , supported by a flexible support system of stiffness k , and a load duration of 1 sec. Then this condition is expressed as

$$1 = 2\pi \sqrt{\frac{M}{k}}$$

or

$$k = 4\pi^2 M$$

The maximum deflection of the structure, y_{el} , is equal to R_m/k , or

$$y_{el} = \frac{R_m}{4\pi^2 M}$$

A typical reusable bulkhead weighs on the order of 2,000 lbs total. R_m is equal to 30 psi times the total area (say 6' x 15') which is 390,000 lbs. Hence

$$y_{el} = \frac{390,000}{4 \times \pi^2 \times \frac{2,000}{32.2}} = 320 \text{ feet}$$

In other words, in order to reduce the design load by 25 percent, we must supply an elastic support system which allows the structure to move about 320 feet. This is obviously out of the question.

(3) Elastic-Plastic Structure

A technique frequently used in the design of blast shelters is to design the structure to deform into the plastic range (depicted in Figure 11(c)). With this technique, the required resistance of the structure can be reduced. This reduction will depend on the duration of the pressure loading, and the degree of plastic deformation permitted.

Excessive plastic deformations can destroy intra-structural seals, and may also cause jamming of the door. In addition, such deformation will be cumulative for multiple explosions. On the other hand, allowance for small plastic deformations may still reduce the required structural strength considerably, as illustrated in the following example.

Suppose we allow a total deformation equal to three times the deformation which would occur if the structure remained elastic; i.e., $y_m/y_{el} = 3$. (e.g., if $y_{el} = 0.01''$ we allow $y_m = 0.03''$ - still very small). Referring to the design curve of Figure 9, for our typical structure where $t_d/T \approx 100$, we find $R_m/F_1 = 1.2$; i.e., the structure can be designed to resist a static load of $1.2 \times 20 = 24$ psi.

(4) Elastic-Plastic Support System

An alternative to designing a structure which deforms plastically is to design a support system which deforms plastically. Here, the plastic deformation is localized at the outer boundaries, and flexible seals can be used to allow such deformation to take place while retaining a seal. Such a support system

can employ frictional elements, for example, which slide over one another at a constant force level in a manner which is equivalent to plastic deformation. Reference to Figure 9 shows that if we allow such a system to slip a distance equal to 10 times its elastic deformation ($y_m/y_{el} = 10$), we can bring R_m/F_1 down close to 1; i.e., we can approach a 20 psi design load for the supporting elements. The figure also shows that to get down below this value, very large increases in the allowable plastic deformation would be required.

The principal drawback to such a support system is that the slippage in each element would not be uniform, and the structure may either deform in an undesirable way, or become unstable. Allowing the structure to go plastic (option 3), rather than the supports, provides much greater control over the type of deformation experienced by the structure.

It should be noted that "plastic" deformation of the supports includes deformation which occurs in the surrounding material, i.e., in the rock and coal that support the structure. Although these deformations are difficult to predict and control, it is clear that some amount of "give" in the support material will substantially reduce the total load experienced by the supports.

The above discussion has indicated that the design load can vary from 40 psi down to near 20 psi, depending on a number of assumptions, and the type of structural responses anticipated. With the exception of the 40 psi design criterion, all design criteria require specific knowledge of the structural response. For the purpose of rapid selection and evaluation of concepts, we will stick with the conservative 40 psi criterion. Once certain best candidates are identified, these will be reevaluated for possible increases in design efficiency.

3.2.2 Anchorage Systems

The discussion of primary structures in the previous section shows that there are many options available; but

regardless of the option selected, the requirements for the support system which anchors the bulkhead to the passage are still severe. It also will become clear that the more efficient structural concepts require more complex anchorage systems.

3.2.2.1 Loadings

The bulkhead support mechanism must be viewed in light of the range of passage sizes to be considered. For example, in a 6' x 15' passage with a 40 psi design loading, a bulkhead supported at the top and bottom of the passage requires a horizontal reaction of 1,440 lbs/inch of passage width. In contrast, a bulkhead supported at the ribs of the above passage requires a rib reaction of 3,600 lbs/inch of passage height. In low coal passages the above disparity between loads carried through the top and bottom supports versus those carried through rib supports becomes much greater.

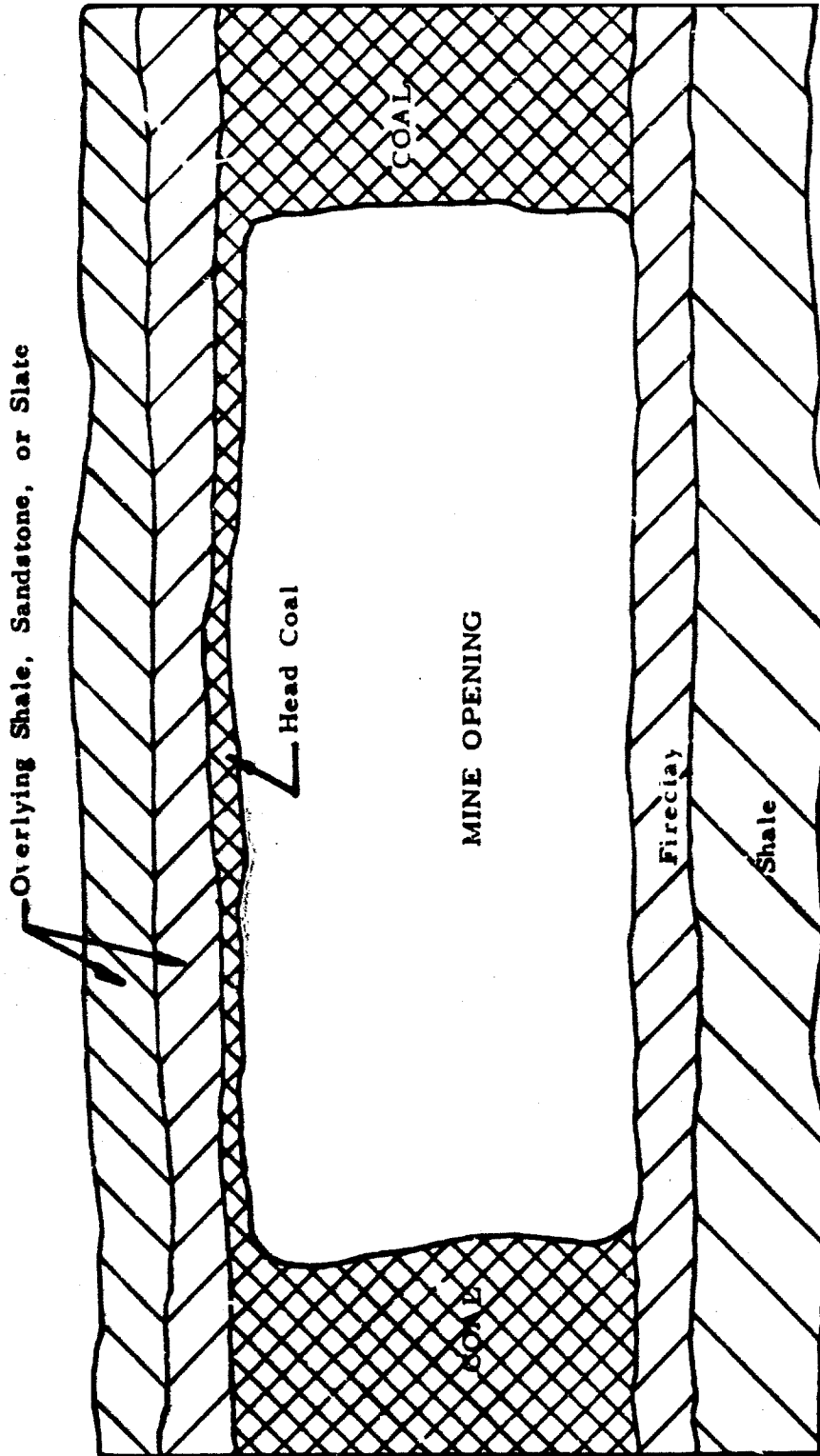
The loadings described above are substantial and require careful consideration of both the anchorage and the support material. These areas are elaborated upon in the following sections.

3.2.2.2 Indigenous Support Material

Figure 12 represents a generalized view of the type of material surrounding the mine opening. Variations on this picture include the following:

- Head coal of up to 6" is usually left in the Pittsburgh seam, and other areas where the roof material is sensitive to weathering (sandy shale, sandstone, and slate). In other areas, where the roof is more durable, either no head coal is left or draw slate is pulled down.

- In some cases, the immediate floor will be shale.



Indigenous Strata Material for Bulkhead Support - Typical Condition

Figure 12

The structural properties as well as the local condition of these materials vary considerably from mine to mine. Therefore it is difficult to predict the competence of a given design. What we have done is examined what is known about these materials, and then prepared designs based on reasonably assumed minimum values for strength properties.

In general, the order of preference for support location is floor, rib, and roof respectively. Floor is preferred, since its condition is the most consistent and predictable. The principal problem at the floor is the existence of wet bottom. It is our opinion that in most cases, suitable locations can be found where this condition will not exist. The rib is second choice to the floor because of the cleated, friable nature of the coal. The roof is the last choice because of the unpredictability of roof conditions.

Below we review the anchorage characteristics of these materials in greater detail, and in the following sections we discuss the principles of some basic anchorage schemes.

1. Fireclay - Fireclay is the most common immediate floor material. When it is in good condition, it is extremely hard, and likely to be a reliable support material. Tests conducted by the USBM⁽³⁾ showed that fireclay has a bearing capacity ranging from 500 to 6,000 psi. However, these tests avoided those mines where there was wet or fractured bottom. Wet bottom causes fireclay to become soft, with virtually no strength. Fractured conditions would cause similar deterioration of strength. It is important, in constructing a bulkhead, to avoid support material which is in poor condition. Either a more suitable location must be found, or an alternate support system must be used.

2. Shale, Sandstone, and Slate - These materials are commonly found in the immediate roof, and occasionally in the immediate floor. The USBM bearing capacity tests mentioned above determined strengths ranging from 2,800 to 6,000 psi.

3. Coal - USBM tests on unconfined coal pillars⁽⁴⁾ indicated a compressive strength on the order of 500 psi, while tests of bearing strength perpendicular to ribs in-situ have yielded strength values on the order of 4,000 psi.⁽¹²⁾ Coal has cleavage planes, and hence strength values can vary considerably. Rib coal is under stress due to the arching action around the mine opening. Consequently, sloughing is always a possibility, and must be provided for.

3.2.2.3 Basic Anchorage Schemes

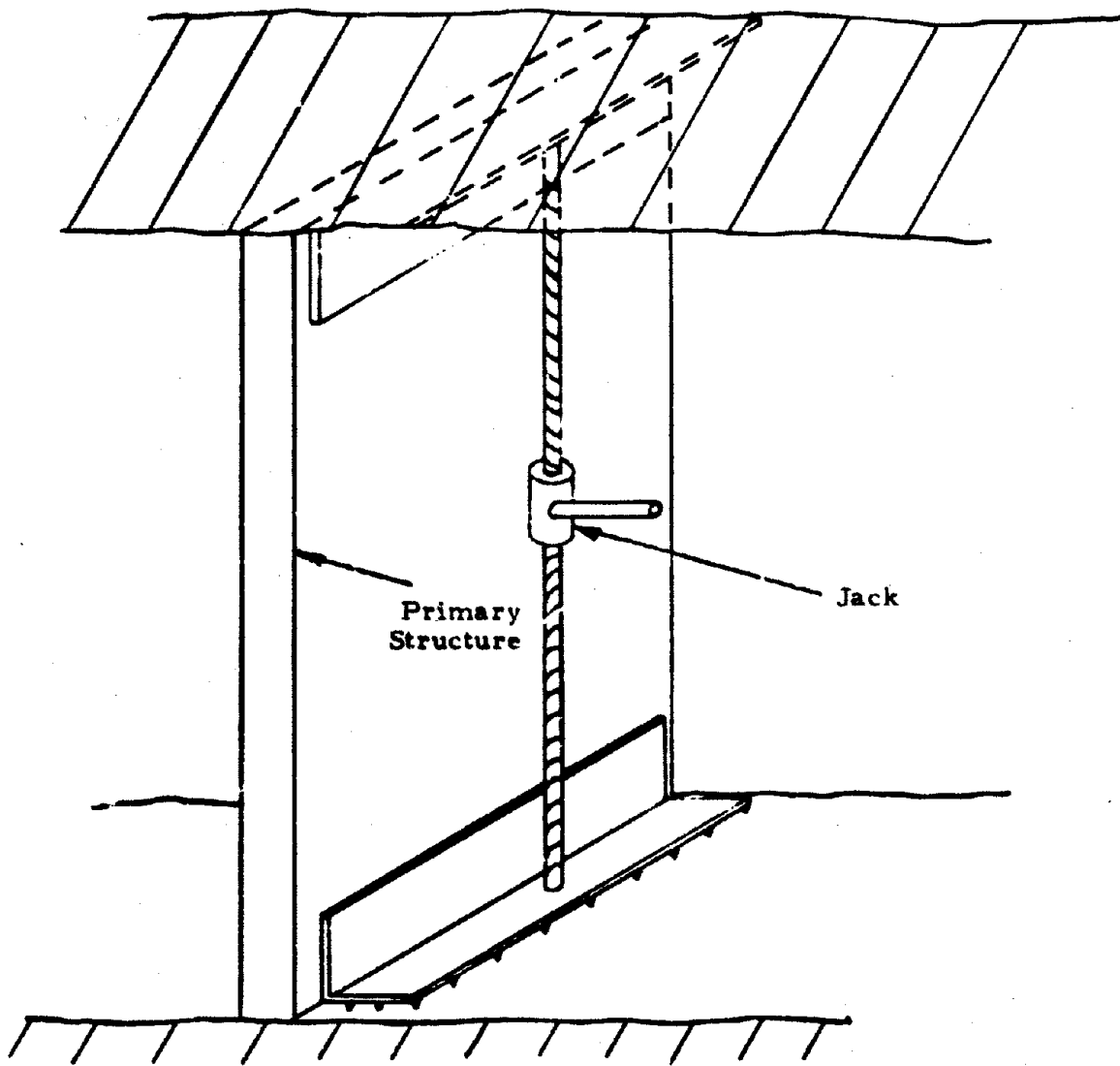
A number of basic schemes can be utilized to support the bulkhead structure. These are described below.

1. Jacks - Because of their simplicity of installation, jacks were considered early in the program as a means of bulkhead support. Figure 13 shows a basic jacking configuration. The principal mode of support is through friction at the roof and floor. Suppose, for example, that we are jacking against a 6" wide plate, and that our jacks are spaced every 4'. Assuming a friction coefficient of 0.5 and our assumed requirement of 1,440 lb/in, the jacking load would be $1,440/0.5 \times 48/2,000 = 70$ tons/jack. This is an extremely high jacking load, and it suggests that jacks would have to be spaced every 6" for more realistic loadings. Such jacks can be either mechanical or hydraulic.

Further consideration indicates two basic problems associated with jacks. These include:

Creep of Surrounding Material

All of the materials surrounding the mine opening tend to creep under constant, long-term loading. Hence, mechanical jacks would lose their load and become ineffective.



Jack Support System

Figure 13

Loss of Hydraulic Pressure

Typically, hydraulic systems suffer leakage in sustaining long duration pressures, and hence constant maintenance would be required.

2. Trenches - Figure 14 shows a typical trench support configuration. The cement filler is added to provide smooth bearing contact between the structure and the rock. Trenching is considered to be a viable means of support at the floor. It is excluded at the roof due to the fact that it would induce a roof fall.

The adequacy of a trench support depends on the shear strength of the floor material. In particular, the shear strength along the failure plane shown in the figure will determine the allowable bearing pressure throughout the depth, d .

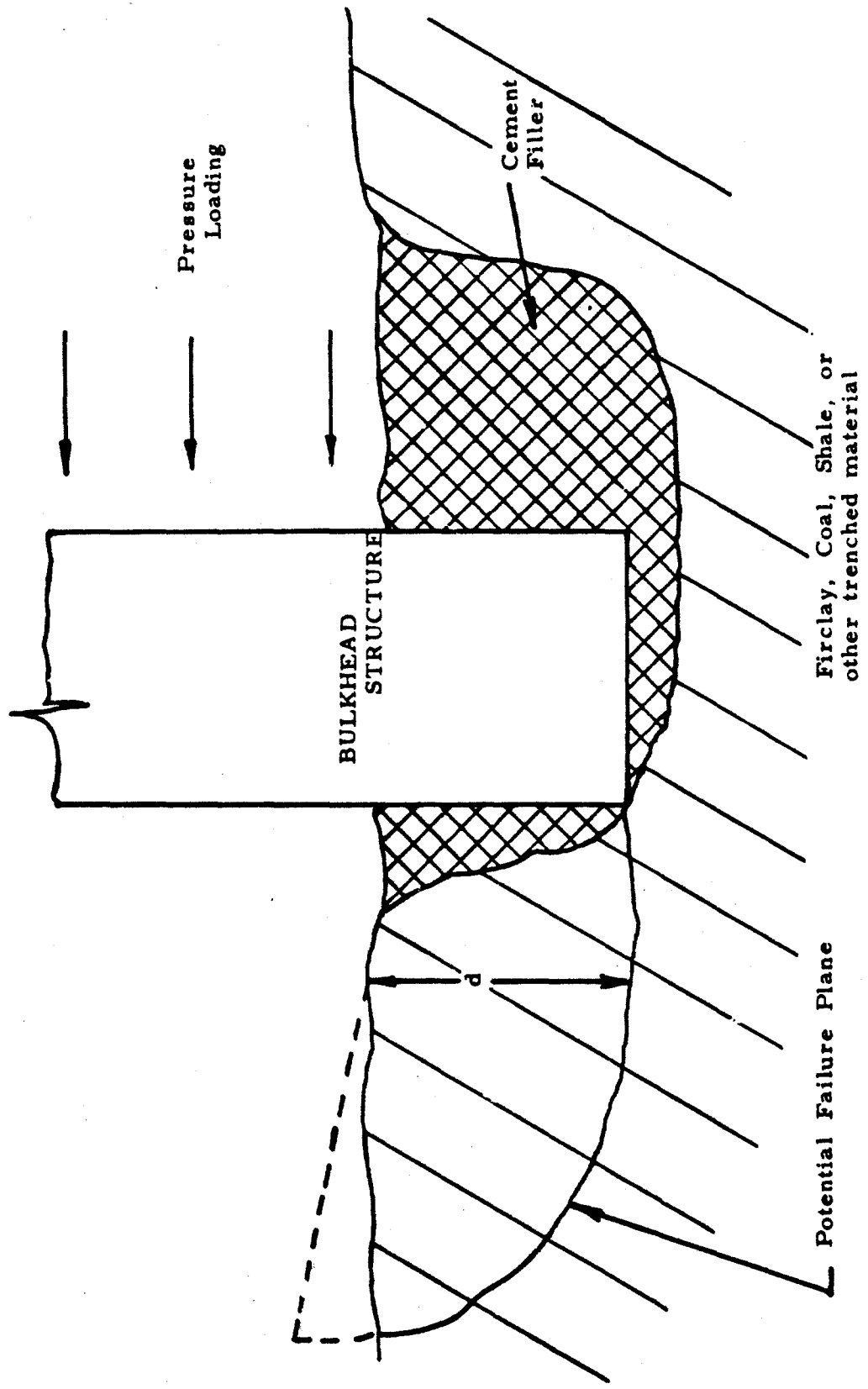
Results from the theory of soil mechanics⁽⁵⁾ show that the shear strength of the floor material can be determined from the USBM bearing strength results as

$$\tau = p/7.4$$

where p is the maximum normal bearing pressure, and τ is the shear strength. The length of a circular failure plane is equal to $\frac{\pi d}{2}$. For the maximum load of 1,440 lbs/in, the relationship between the required trench depth d , and the bearing strength p , is

$$d = \frac{3340}{p}$$

where p is in psi, and d is in inches. For the minimum bearing strength value of 500 psi, the required trench depth is therefore 7".



Trench Support

Figure 14

One principal problem with trenching is the manner in which it is carried out. Available equipment to do the job includes continuous miners, universal cutting machines, jackhammers, and hand picks. For hard bottom, trenching may very likely be carried out by drilling and shooting.

The large pieces of equipment will make an extremely wide trench, and hence will not provide a vertical bearing surface. Drilling and shooting, if carried out haphazardly, will cause failure planes in the surrounding rock and reduce its strength. The basic problem with trenching is that the better floor material is the more difficult to remove, and careful excavation procedures must be used.

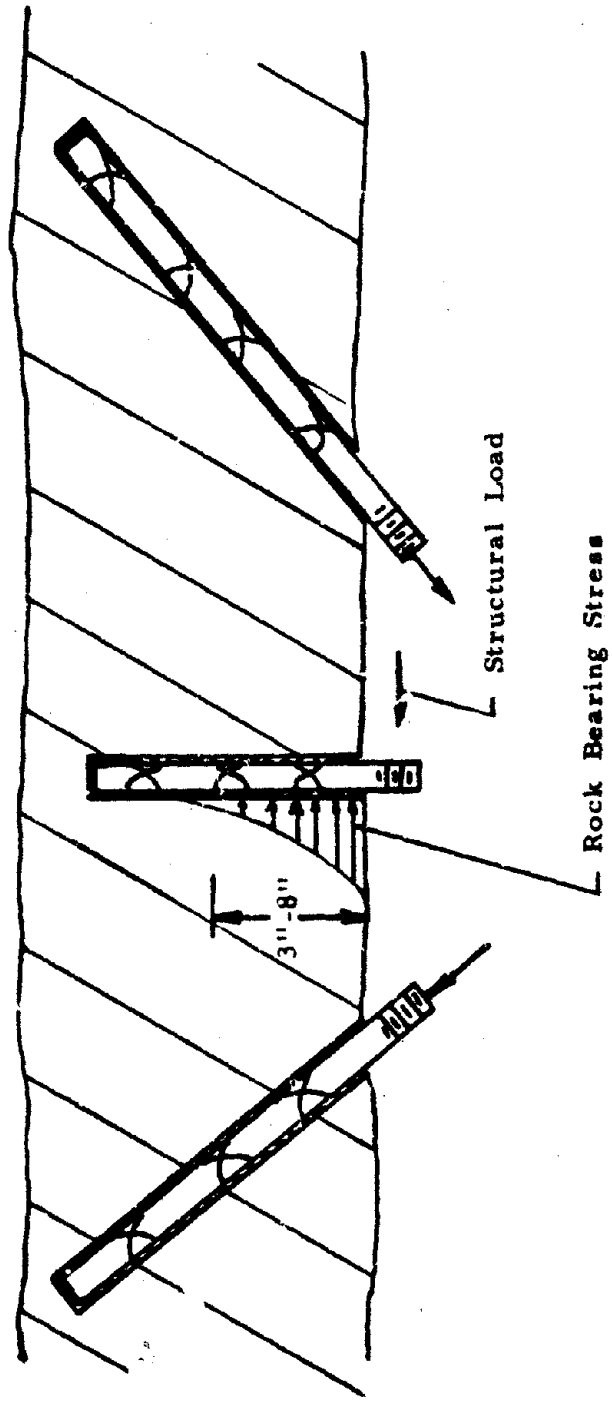
3. Rock Bolts - Bolting into the roof and floor is considered to be a viable approach to the anchorage problem. Figure 15 shows a number of different bolting configurations. The unique problems associated with rock bolting are:

(a) Can a bolt take the required loading in shear?

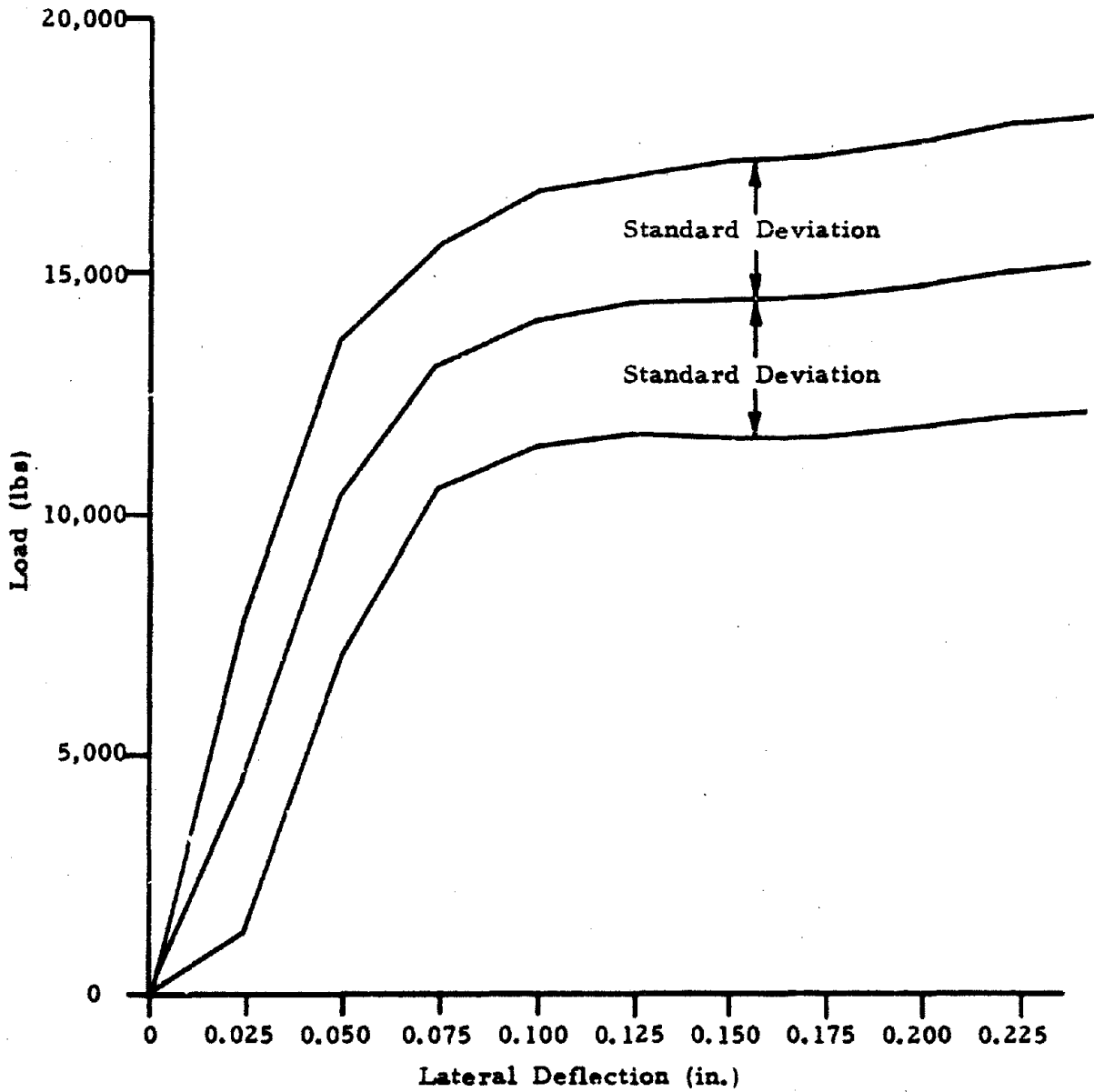
(b) Is there equipment available to place bolt holes in the locations and orientations required?

In response to question (a), rock bolts are typically designed to take load in tension; i.e., as they behave in their roof control function. There is little data on roof bolts in shear, and very little such data on rock bolts in general.

Figure 16 shows some preliminary test results developed at the Bureau of Mines. This figure summarizes the force-deformation behavior for a full column 5/8" diameter resin bolt embedded into, and sheared between two blocks of Indiana Limestone. The data indicates an apparent yield strength which is greater than that due to pure shear of the bolt. This is



Bolting Schemes
Figure 15



Force-Deformation for a 5/8" Full Column Resin Ec2 in Pure Shear

Figure 16

apparently due to the combined action of tension and shear. This non-linear effect also tends to result in a "strain hardening" behavior. Table 1 shows manufacturers data for various rock bolts anchored in concrete. This data primarily reflects the shear strength of the bolt, with some variations depending on the strength of the concrete. We should emphasize that the above data was taken using carefully controlled homogeneous rock and concrete specimens. In actual practice we must deal with laminated, unpredictable conditions, and in many cases this uncertainty will preclude the bolting approach.

An alternative to the shear loading is an anchorage system which loads the rock bolt in either tension or compression. In the case of tension, greater bolt lengths are required to provide the necessary anchorage. In addition, if a large number of roof bolts in a row are loaded in tension, there is the possibility of pulling down the roof.

In any event, the primary resistance is carried close to the loaded end of the bolt, and hence, for bolts in shear, we would anticipate using a bolt length of 1 to 1-1/2 feet. As far as the type of bolt to use, a fully grouted bolt is recommended. This is because there would be a smoother stress transfer between the bolt and the rock, and because the grout may fill some of the faults in the bolted strata.

In response to question (b), we believe that bolt installation equipment will not limit us in selecting an anchorage scheme. We do, however, have doubts that unusual bolting configurations will be practical from a labor standpoint. For example, a previous study⁽⁶⁾ indicated that the drilling of angled bolt-holes in the floor was prohibitively time consuming.

Table 1

Design Data on Concrete Bolts in Shear

<u>Manufacturer</u>	<u>Concrete Compressive Strength (psi)</u>	<u>Bolt Diameter</u>	<u>Ultimate Shear Loading (lbs)</u>
Parabolt	2,000	3/4"	16,400
	3,000	3/4"	19,200
	4,000	3/4"	21,750
Phillips	4,000	3/4"	18,000
Wedjet	3,750	3/4"	17,000
		7/8"	21,500
		1"	26,000
		1-1/8"	34,000
		1-1/4"	40,000
Hilti	3,500	3/4"	19,900
		1"	39,200

4. Bearing Supports

Arch-type structures will transmit bearing loads into the supporting strata. Abutments must be designed such that the resulting bearing pressure is within the allowable bearing limits for the strata in question. Figure 17 illustrates this situation.

For abutments in the rib, a particular complication arises due to the angle of the loading and the fractured nature of the coal. The rib has a natural orthogonal array of fracture planes, called cleats. When a load is applied normal to the rib, the stress state is entirely compressive and the load capacity is not affected by the presence of cleats. Previous data has indicated that under these conditions, an in-situ bearing strength of 4,000 psi was achieved.⁽¹²⁾ With the load applied at an angle, however, the stress state will change and some of the coal will be in tension. This situation is illustrated in Figure 18.

The figure shows that the coal behind the line perpendicular to the edge of the abutting structure will be in tension. An estimate of the coal support capacity can be obtained by assuming that coal has no tensile strength, and that the arch is supported entirely by the compression wedge shown in the figure. The maximum stress in the wedge can be computed from^(42, p. 198)

$$\sigma = \frac{P}{\alpha + 1/2 \sin 2\alpha} \times \frac{\cos \theta}{r}$$

where

α = $\pi/2 - \phi$

ϕ = abutment angle

r, θ = polar coordinates of point whose stress is being computed

P = thrust in lbs/in

σ = compressive stress

Note that this solution is for a point load, and hence $\sigma = \infty$ when $R = 0$. Consequently, we assume that a fictitious concentrated load is buried inside of the structure, by a distance d , which is computed by equating the contact pressure to the above solution.

$$\sigma = \frac{P}{W} = \frac{P}{d} \times \frac{1}{\alpha + 1/2 \sin 2\alpha}$$

where W = bearing width of the structure.

or,

$$d = \frac{W}{\alpha + 1/2 \sin 2\alpha}$$

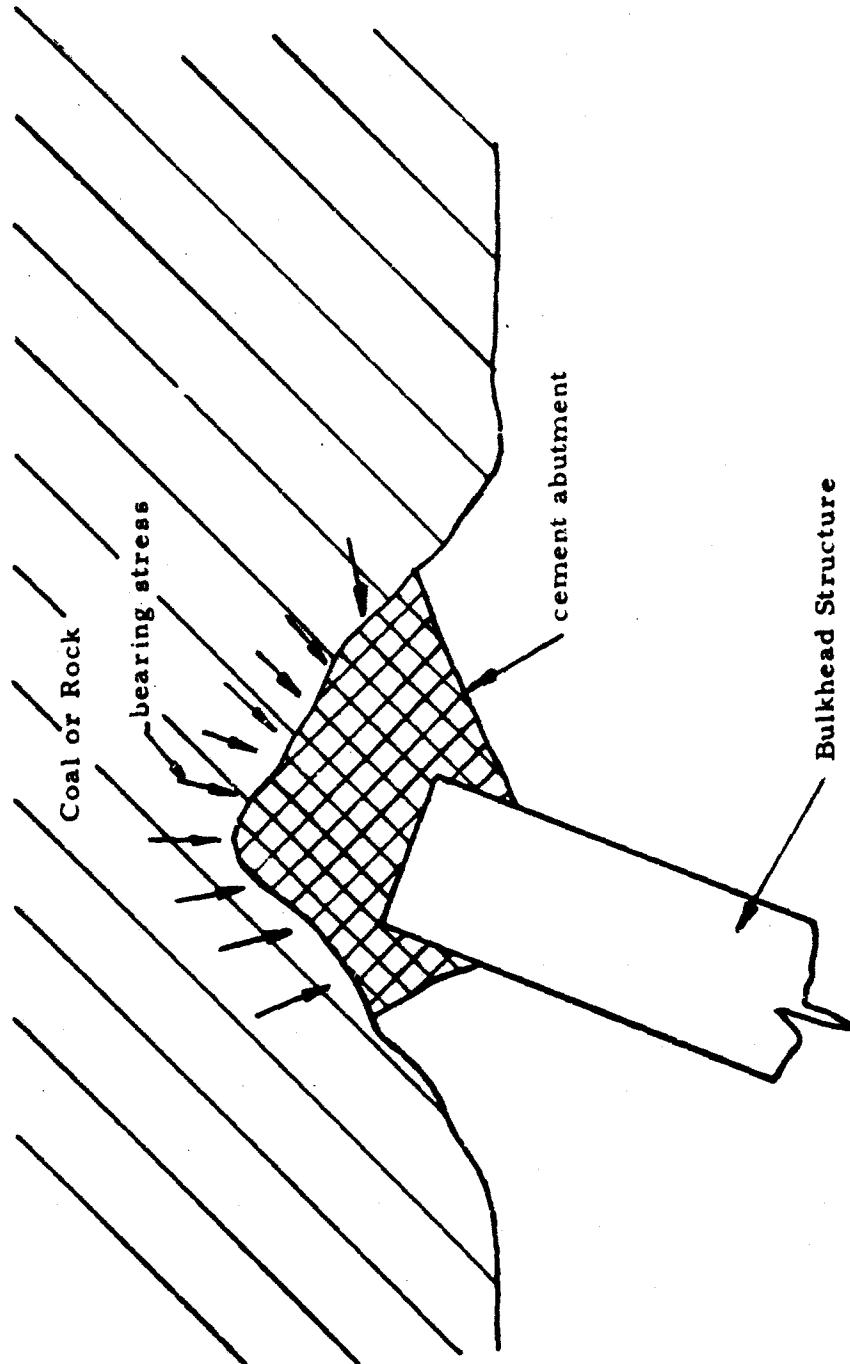
For example, assume a 6" wide structure with a 30° abutment angle, and a 5" thick concrete abutment. Then

$$d = \frac{6}{\pi/3 + 1/2 \sin 2\pi/3} = \frac{6}{.61} = 9.77"$$

$$r = 9.77" + 5" = 14.77"$$

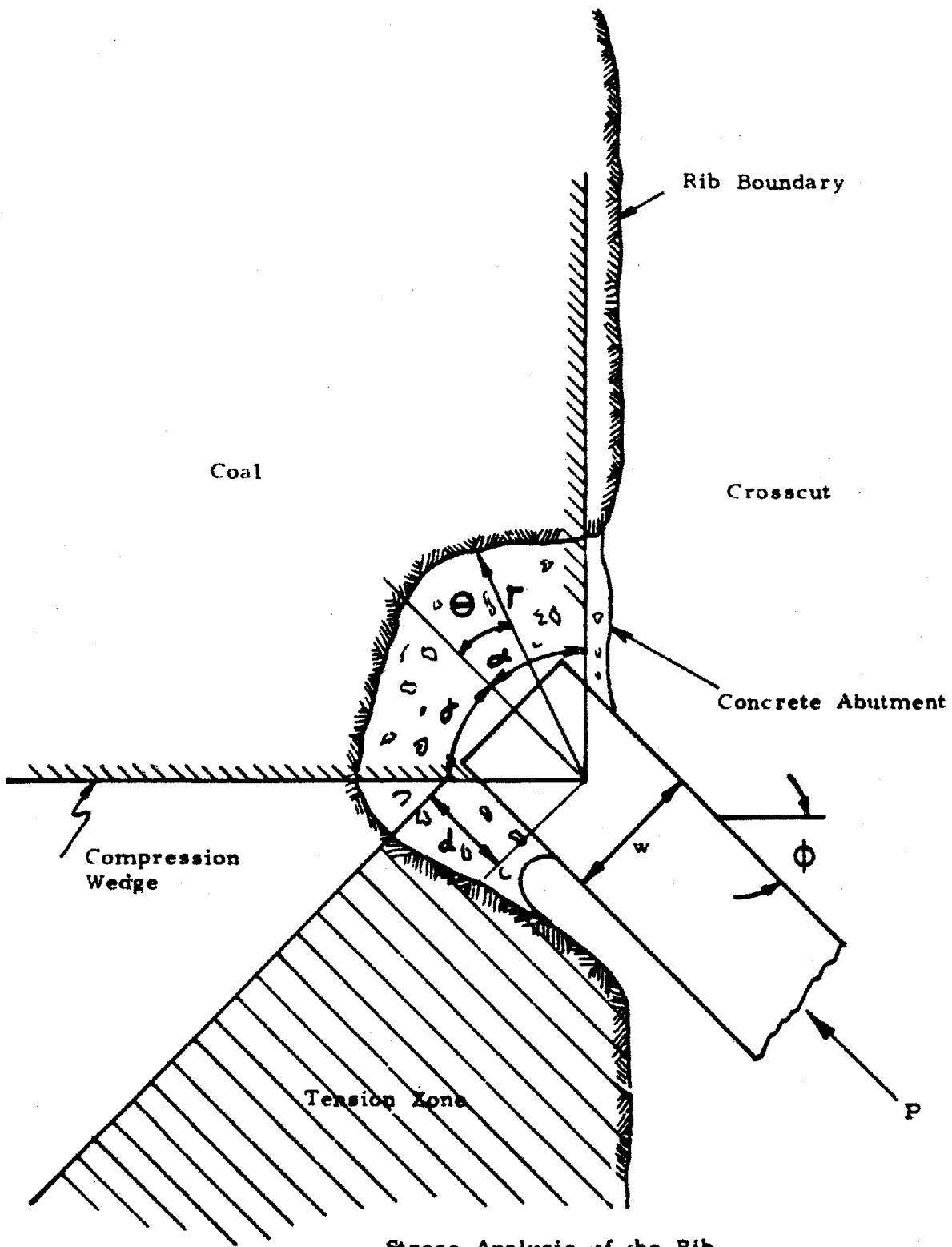
and the maximum stress in the coal is ($\theta = 0$)

$$\sigma = \frac{P}{\pi/3 + 1/2 \sin 2\pi/3} \times \frac{1}{r} = \frac{P}{.61 \times 14.77} = .111 P$$



Bearing Support

Figure 17



Stress Analysis of the Rib

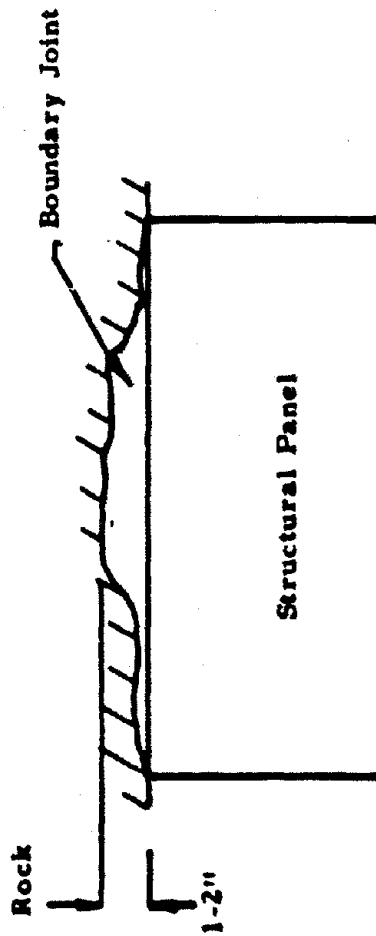
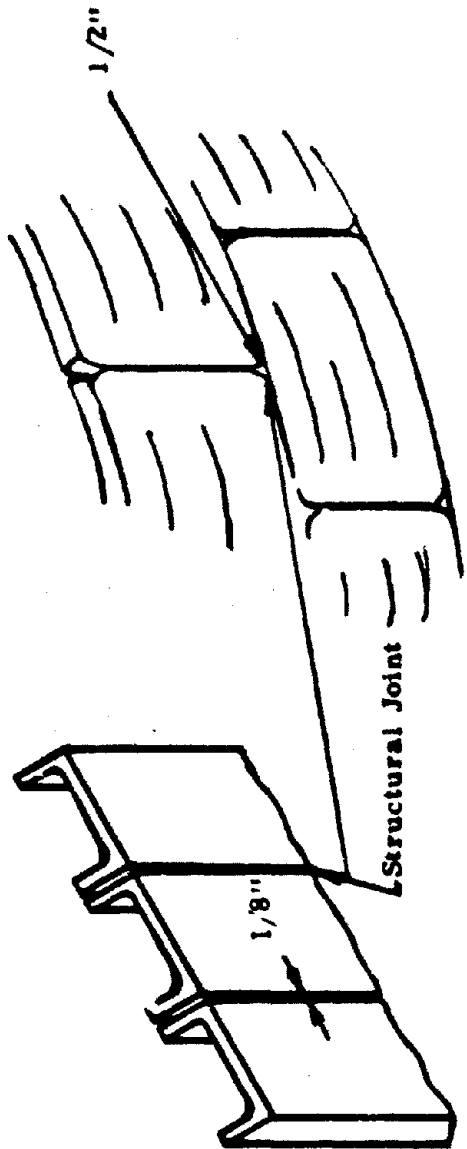
Figure 13

3.2.3 Sealing Systems

The leak-proof specification of the bulkhead requires that all air gaps in the structure be appropriately sealed. Gaps generally occur at joints, which either occur within the structure between structural elements, or at the boundary of the structure between the structure and the surrounding rock and coal. Figure 19 depicts some typical sealing conditions. Typically, structural elements which are not machined will have some surface irregularity and warp, and gaps between such elements will be from 1/8 - 1/4". Structural shapes usually have rounded corners and edges, and when these intersect, gaps on the order of 1/2" may occur. The irregularity at the interface between the structure and the rock and coal is on the order of inches. Locally, this may be one to two inches. If the structure can not adjust to height variations throughout the width, this gap may be as large as 6".

Since sealants are frequently elastomeric, care must be taken in choosing one which satisfies the flame and temperature requirements. Beyond this, the sealant must have sufficient strength so that it will not be blown out by the blast, yet it must have resilience so that it will deform without permitting leakage. The strength of the seal will depend a great deal on the width of the gap to be sealed. A number of basic sealing techniques are listed below.

1. Bead-Putty Sealants - There are a number of bead sealants on the market which remain resilient. They can be applied by a tube or caulking gun. Their principal utility is for gaps 1/8 - 1/4" wide.
2. Gaskets - These can be used in place of bead sealants. They are more subject to abusive handling and improper placement. They do, however, give more depth to the seal.



Locations for Sealing

Figure 19

3. Sprays and Paints - Perhaps the quickest technique is to spray coat the entire structure. This may, however, make disassembly and reuse somewhat more difficult.
4. Foams - Foams are more suited to the 1" - 3" gaps. They are frequently flammable, and hence must either be coated for protection, or must be chosen to be sufficiently flame retardant.
5. Membrane - A technique for sealing the larger gaps is to drape a thin membrane (fabric, sheet metal) between the structure and the surrounding material. This will result in smaller joints which will have to be treated with a final sealant.

Table 2 summarizes the products and materials which were reviewed. Products are identified by trade names where applicable. The full range of sealant functions are covered by the Table. Each particular type of structure will require different sealant functions depending on the type of gaps between the structural elements and between the structure and the passage; therefore, each final concept will have its own set of sealants.

Gasket type sealants have been eliminated from consideration due to difficulties in handling and susceptibility to damage.

TABLE 2
Sealant Materials

Sealant	Composition	Method of Application	Size of Gap	Temperature Specifications	Physical Properties
Ocean 69	cementitious	Spray or trowel (spray requires two men)	1/4"-1/2"	Used as fire-cladding	Hardens like a soft plaster
Superbond	cementitious	Spray or trowel	1/4"-1/2"	-	Excellent adhesion 5000 psi composition strength
Zonolite MSX 100	cementitious	Spray or trowel	1/4"-1/2"	-	-
Mandoseal	cementitious	large spray unit can be trowelled	1/2"-3"	Used as fire-protection	hardens
Surewall	cementitious glass fiber reinforced	Spray or trowel	1/8"-1/2"	Can withstand 250° - 300° F	Excellent adhesion Extremely rigid
Rigiseal	2 component urethane foam	Spray (2 man)	1" - 6"	flammable	Flexible, hardens to about 30 psi strength
Stoppit	Vermiculite, Sodium silicate	Spray (2 man) or trowel	1/2"-3"	non-flammable	Remains flexible
Fireguard 3M	cementitious	trowel or spray (2 man)	1" - 5"	non-flammable	More flexible than cement
Silica Foam	Sodium Silicate glass fibers	trowel or spray (2 man)	3" - 6"	non-flammable	Low strength Inflexible
Bloc-Bond	cementitious, w. glass fibers	trowel	1/8"	non-flammable	Hardens
Scotch-Seal	Elastomer	Putty knife, or pressure extruder	1/8"-1/4"	non-flammable, withstands 200° F	Permanently plastic
Dow Corning Sealant	Silicone Rubber	Caulking gun	1/16"-1"	withstands > 200° F	Permanently resilient
Hornflex	Thiokol	gun or knife	up to 1"	withstands > 200° F	Permanently resilient
CHR Coated Fabrics	Silicon and Viton Rubber Coated Glass	Clamp	6 to 8"	up to 600° F	Pliable - up to 600 lbs/in. strength
Sheet Metal Flashing	Sheet Metal	Clamp or screw attach	6 to 8"	no temperature problem	Pliable - up to 600 lbs/in. strength

3.3 Specific Approaches

3.3.1 Main Structure Concepts

The following section outlines the structural concepts which were considered during the design program, as well as the design principles which were applied.

3.3.1.1 Basic Considerations

One primary rule of thumb is to design structures which use standard sections and shapes. These include commercial, rolled sections, and other special shapes which are mass produced for special structural purposes. Their principal advantage is in cost and availability. Standard structural elements are generally stock items, and replacement can be readily obtained.

Materials for standard elements are usually structural steel (A36 or A242) or structural aluminum (Al 6061-T6). Some elements, such as bolts and corrugated sheeting, are made of higher strength steels.

The use of aluminum, although unacceptable for moving parts in certain mining applications due to sparking, is considered acceptable here, especially where it could enhance the portability of the structure. Ferritic reactions between aluminum and steel components can be avoided by using gaskets, sacrificial plates, or other types of protective coverings.

3.3.1.2 Vertical Span Bending Structures

As discussed in the previous section, the most efficient way to carry the load in bending is to span the shortest available dimension. In the case of a mine opening, this span is from roof to floor. The requirement for an elastic structure is to provide a

sufficient section modulus so that the maximum stress due to the 40 psi loading is below the yield stress of the material. This requirement can be expressed in terms of elastic beam theory, which states that for a simply supported beam, the maximum bending moment occurs at midspan. For the stress at midspan to remain elastic, the required section modulus per unit width of bulkhead is

$$S \geq 5l^2/\sigma_y \quad (1)$$

where S is the section modulus per unit width, l is the height of the passage, and σ_y is the yield stress of the material. This criterion is used in the selection of the following structures.

1. Old Dominion Design

Previous work carried out at Old Dominion University⁽⁷⁾ designed a structure which was a composite of beams and plates. Figure 20 shows a typical steel panel, and Figure 21 shows a typical aluminum panel. Typical panels weigh 200 lbs in both steel and aluminum. The aluminum panel can be divided into two, 100 lb panels. The panels are fixed in length, and are supported by jacks to the rear of the structure.

2. Vertical Beams with Lightweight Facing

An alternative to the modular panel approach is to use a discrete set of substantial vertical beams, and a strong, lightweight facing to span between the beams.

3. Designs from Standard Rolled Sections

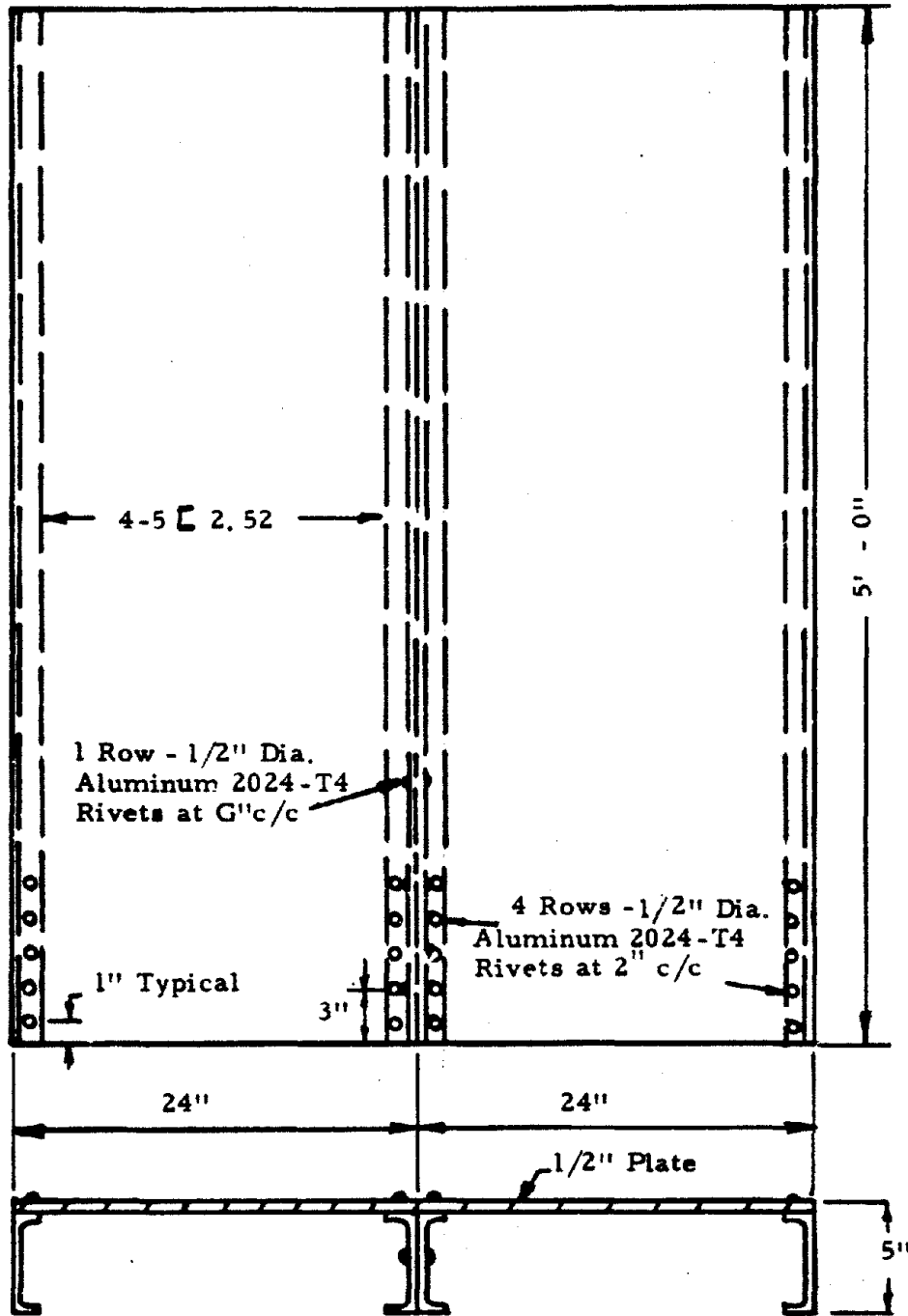
An alternative to large, heavy structural modules is to have a large quantity of small but simply connected structural elements. The use of standard rolled shapes in aluminum and steel is one such approach. Figures 22 and 23 illustrate the types and arrangements of such structural elements.

The various types of structural elements include angles, I beams, Z sections, channels, and box beams. These are stock items at a steel or aluminum supplier. They can be stacked along side of each other and rapidly connected using a number of possible clamping devices. Clamping devices are desirable because they avoid the problem of alignment of bolt holes. The structure, however, loses all strength in the horizontal span. This is undesirable from a stability point of view.

One principal drawback of the standard section approach is that special provisions must be made for a door frame. This will interrupt the span, and will be at least twice as wide as a typical panel. The degree of fabrication involved will be costly, and will detract from the other economic features of this approach.

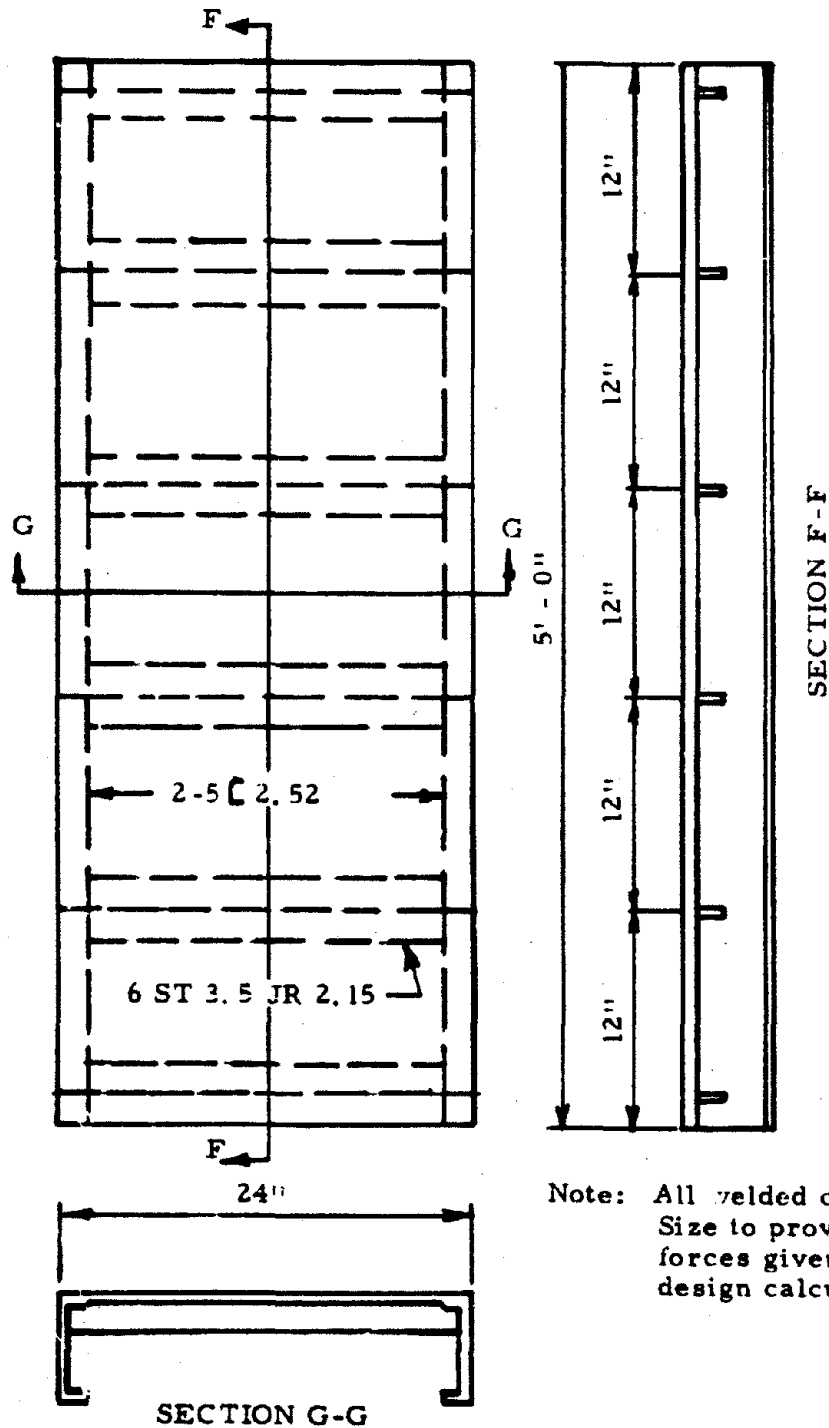
4. Prefabricated Panel Designs

The two approaches described above employ off-the-shelf structural components. For optimal efficiency, structural panels can be completely prefabricated. The expense of fabrication is traded off with lightness of weight.



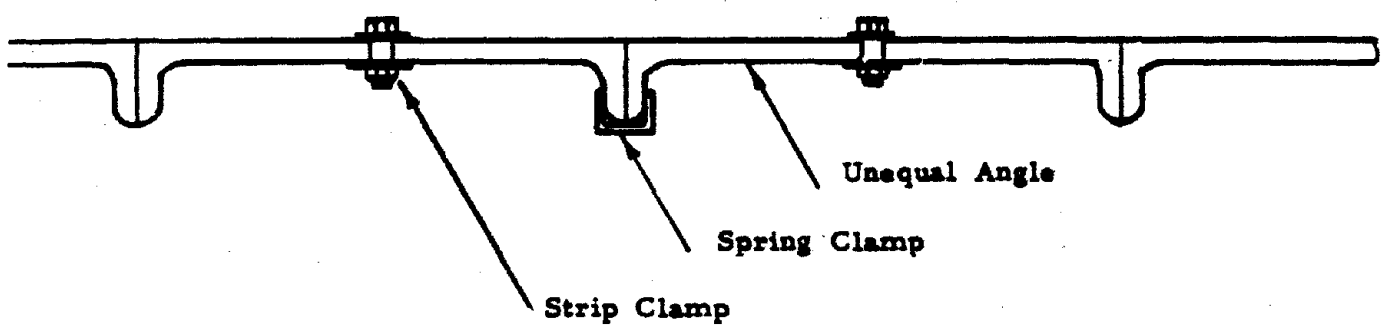
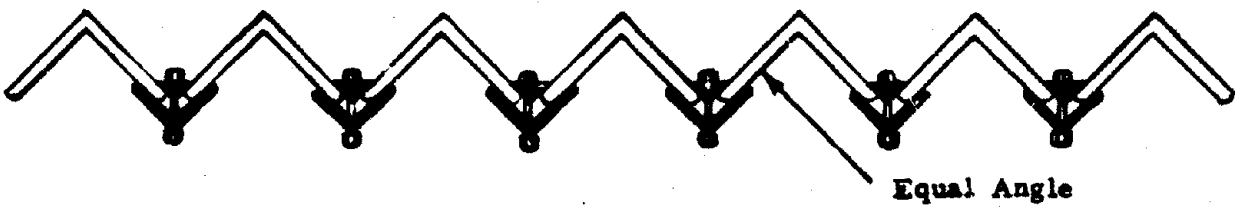
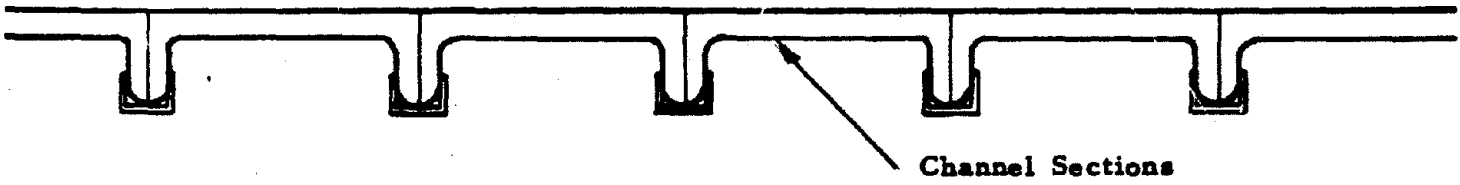
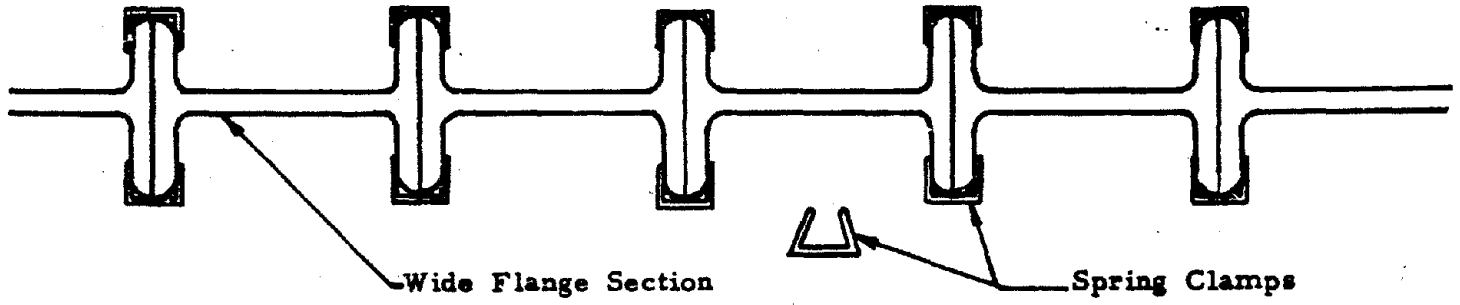
Old Dominion Design
Typical Aluminum Panel

Figure 20



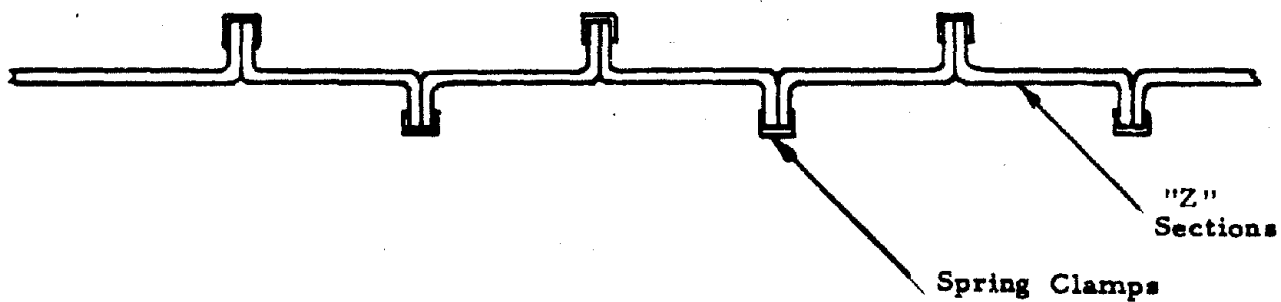
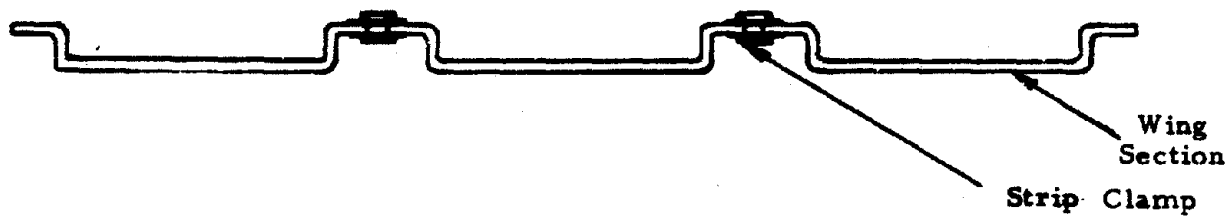
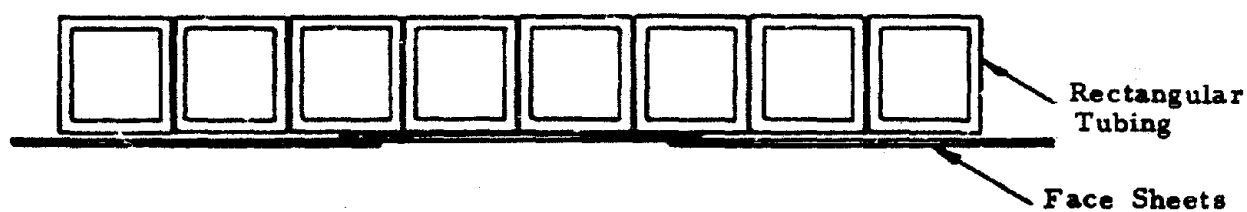
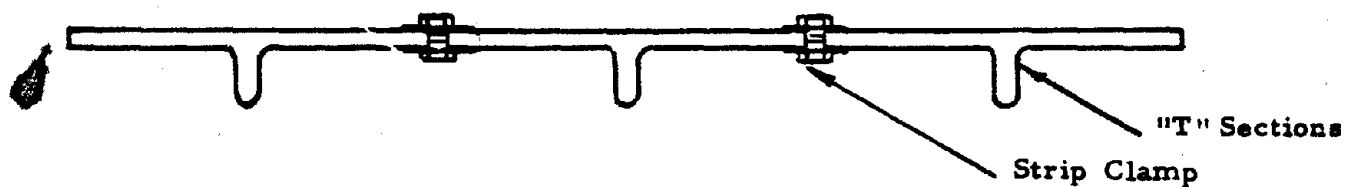
Old Dominion Design
Typical Panel

Figure Z1



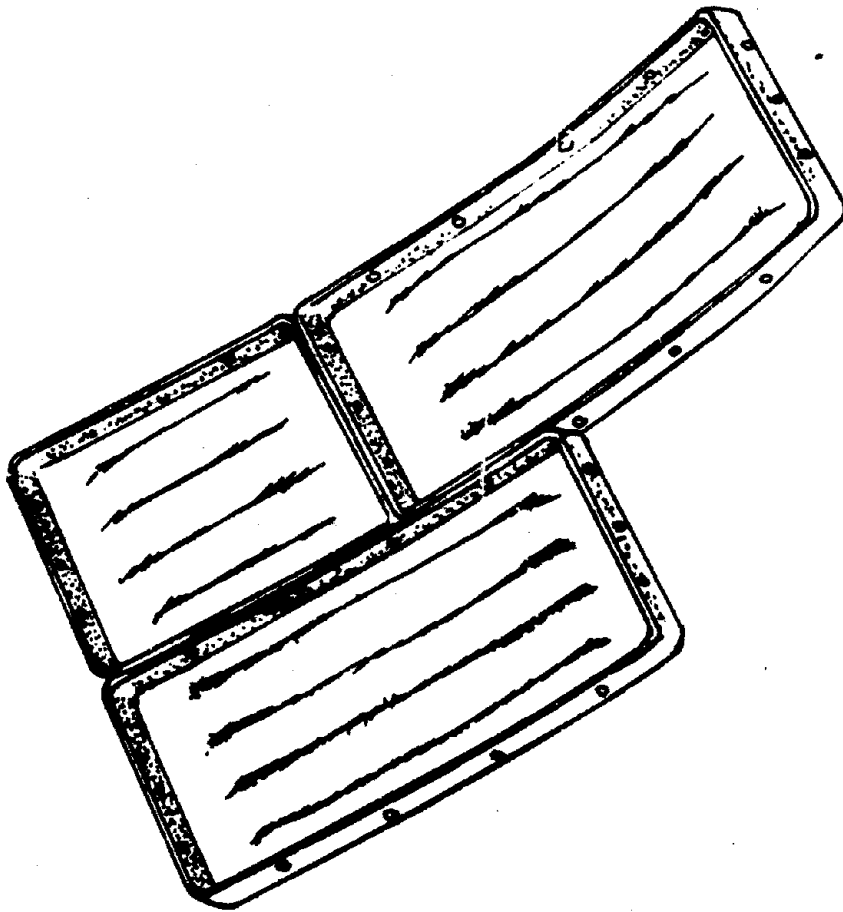
Bulkheads from Rolled Sections

Figure 22



Bulkheads from Rolled Sections

Figure 23



Liner Plates
Figure 24

3.3.1.3 Arch Structures

The common characteristic of all of the bending structures just described is that high shear loads are transmitted to the rock strata in the roof and floor. As discussed in Section 3.2.2.3, the transference of this load is often difficult to achieve reliably. Consequently, arch structures were investigated as an alternative to bending structures.

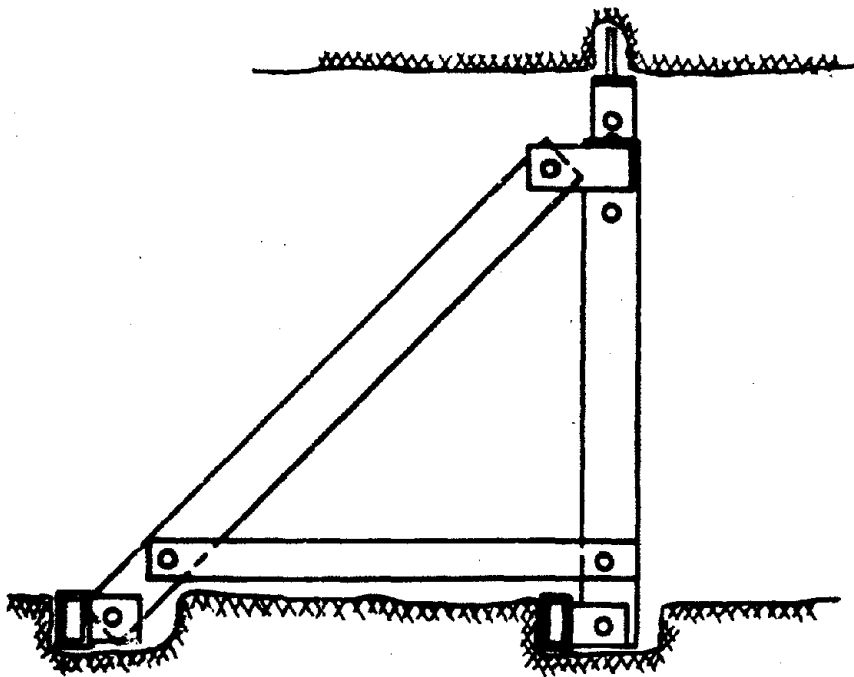
As mentioned earlier, the selection of bulkhead components from standard stock items will help to reduce material costs. For arches, standard tunnel liner plates offer a ready made, off-the-shelf solution. Figure 24 shows typical liner plates. Liner plates typically serve to support the pressure surrounding excavated material during tunnel and deep hole construction.

Curved I beams to provide reinforcing ribs are also standard items in the liner plate business. Liner plates are cast with flanges and holes, and come in widths from 12" to 24", and lengths of either 18.5" or 37". They are rugged, yet light enough to be handled by one or two men.

The required gage of liner plate and size of reinforcing rib will depend on the span and the radius of curvature. These will be calculated for the specific designs.

3.3.1.4 Truss Structures

In order to completely avoid problems of anchorage at the roof, it is possible to design a structure in which all of the loads are carried into the floor. Such a structure looks like a series of simple trusses. A typical truss is shown in Figure 25.



Truss Design
Support using Trenches

Figure 25

The reaction at the top of the front post, which would normally be taken at the roof, is now transmitted to the floor through compression of the diagonal bracing member. The front posts are designed for the usual bending loads, and the spans between the posts can be taken up by corrugated sheeting, hexcel, or other facing panels. The design of the diagonal, for the 6' high bulkhead, is governed by buckling. The base of the triangle is closed to make it an integral structural unit. This prevents the whole unit from failing due to the failure of one support point.

The fact that the structure is no longer supported at the roof makes adjustment in that direction a simple matter. Front posts can be made extensible, to carry the load delivered by additional sections of facing material.

3.3.1.5 Tension Structures

Tension structures are the most efficient in terms of load carrying capacity per unit weight. By being lightweight and thin, they can be flexible enough to conform to the local variations in passage geometry. The usual problems associated with tension structures are finding ways of attaching the structure to the surrounding material, and providing the structure to resist rebound and negative pressures.

One design concept that was considered involved the use of vertically hung "blast mats" to attenuate the shock wave. These could either be made leak proof, as one composite unit, or else be backed at a suitable separation distance by a secondary, lightweight, leakproof structure.

Providing access through the mat, as well as anchorage to the walls, presents the major problems. Most likely a substantial frame will be required, which will then connect to the passage. This will detract from some of the potential simplicity of the concept.

3.3.2 Anchorage Systems

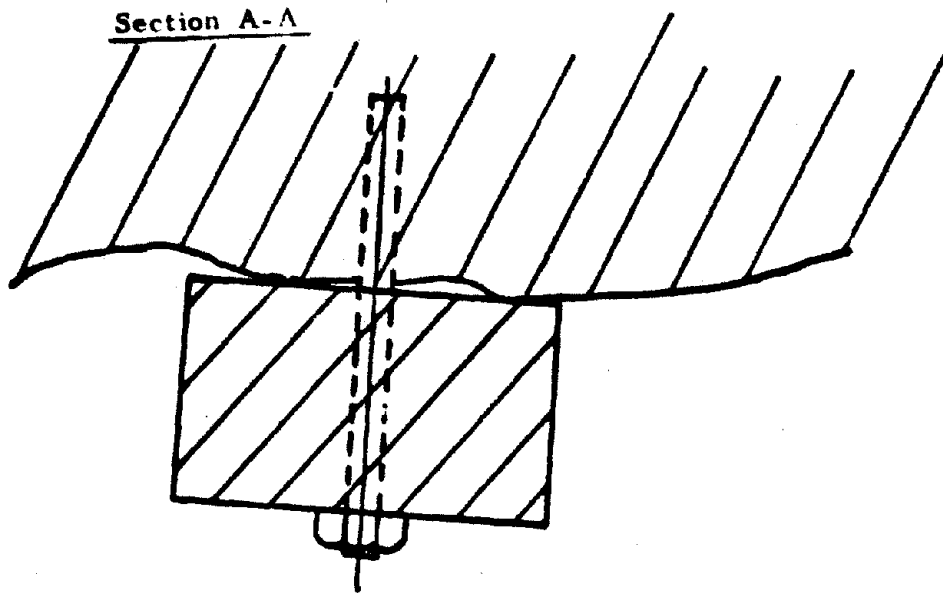
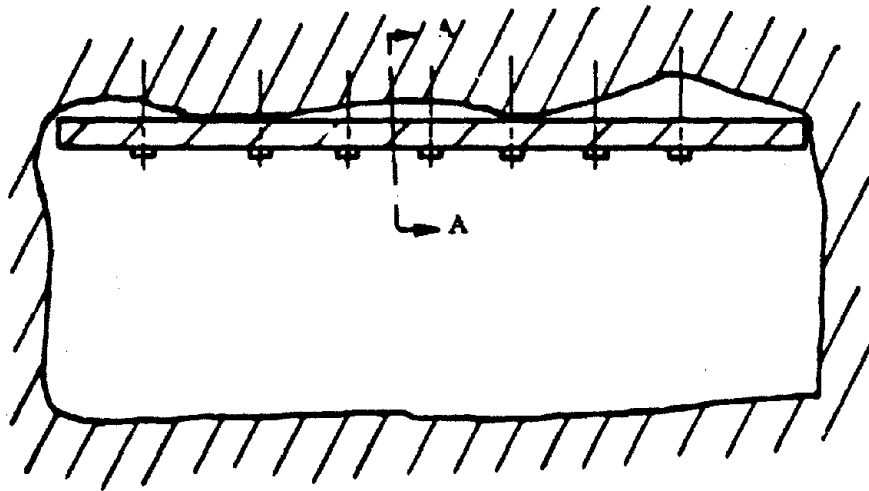
As has been mentioned previously in this report, the anchorage system presents the major technical problem in the bulkhead design. This is primarily due to the geometric irregularities of the passage, and the unpredictability and frequent poor quality of the type of materials available for support.

In addition to the technical problems, anchorage construction also constitutes a major portion of the labor required to construct the bulkhead. The basic goal of anchorage design is to come up with a reliable system which requires minimum labor. A number of potential anchorage schemes have been considered in this design effort, and these are described below.

3.3.2.1 Headers

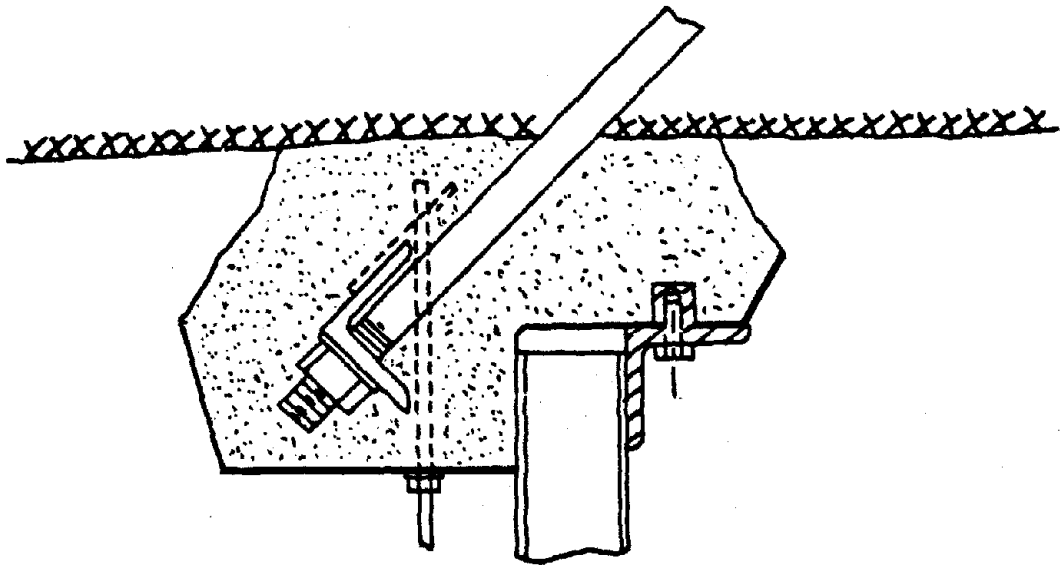
One approach to support at the roof is to provide a beam spanning the width of the passage. Due to the length of the span, the header will be supported along its length using roof bolts, and these carry the load applied by the structure.

One basic problem with a header is that a beam bolted up to the roof will assume a position which is not necessarily alignable with the main bulkhead structure. Figure 26 illustrates this point. Additionally, heavy members will be required to take the local loadings (1,440 lbs/in). The alternate which was investigated here is the use of headers which consist of sheet metal forms filled with mortar or fine-aggregate concrete. The concrete can either be hand troweled, or pumped in as a slurry. In either event, it can be made to conform with the irregularities of the roof. Figure 27 illustrates the concept.



Possible Orientations for Bolted Headers

Figure 26



Example of a Formed Concrete Header

Figure 27

Load transfer from the bulkhead to the roof through a concrete header is complex. The concrete is subject to combined shear, compression, and tension. Reinforcing in the concrete is required to guarantee an adequate support. Ultimately, sound roof material is still required. We investigated potential designs, and concluded that the approach did not merit further consideration.

3.3.2.2 Turnbuckles

This approach involves the direct connection of a bulkhead component to a roof bolt using a turnbuckle. This avoids some of the load transfer complications which were associated with headers. The load transferred to the roof (or floor) bolt is well defined, and then a bolt can be designed having the required capacity. The turnbuckle mechanism allows adjustment so that the roof bolts do not require perfect placement. The turnbuckle ends can be either ball joints, or slotted arrangements, to provide additional freedom of alignment.

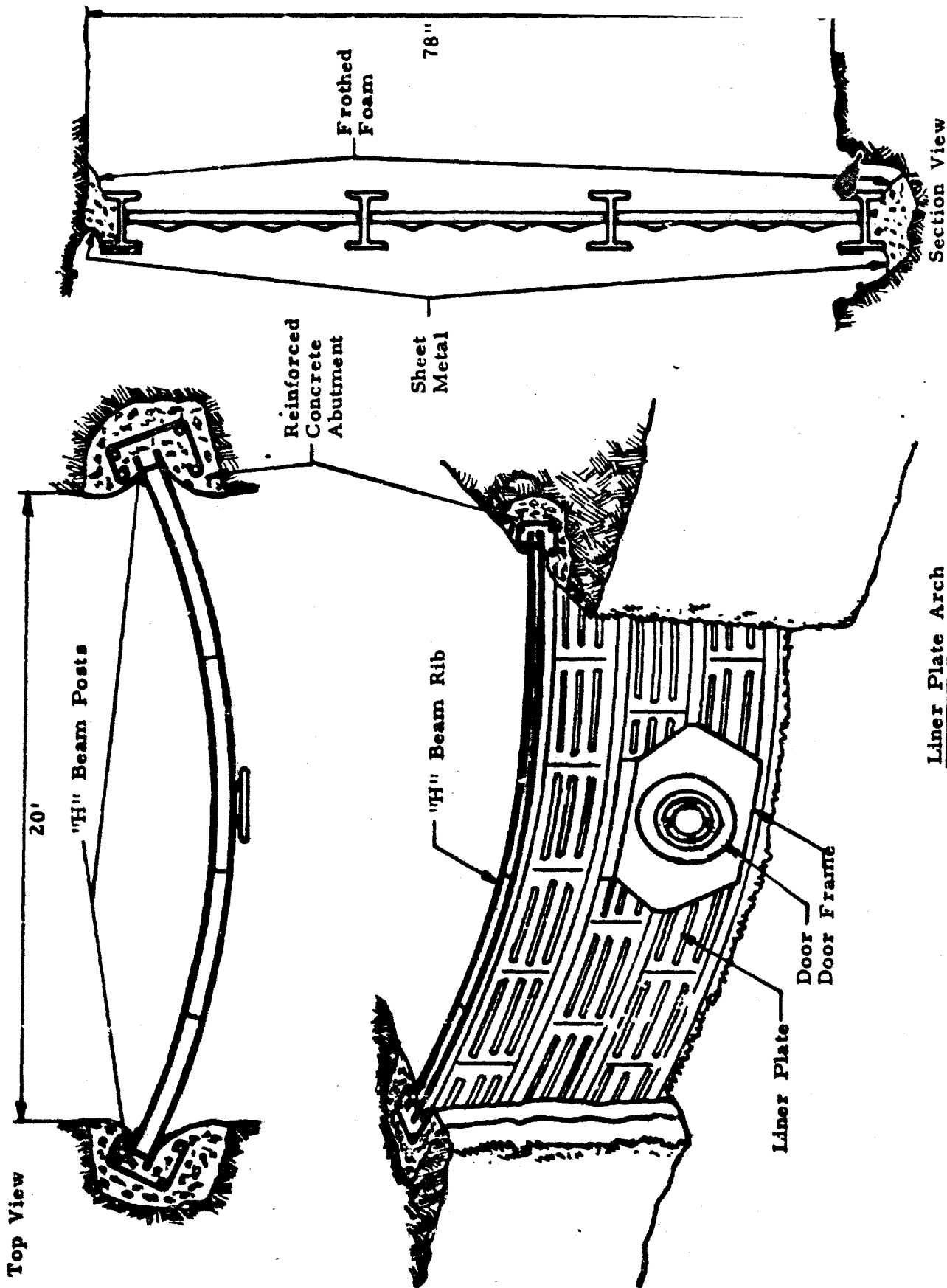
3.4 Synthesis of Complete System

The background information presented in this section has laid the groundwork for the generation of complete bulkhead concepts. Based on the number of alternative component systems which have been present, there are many possibilities for a final concept.

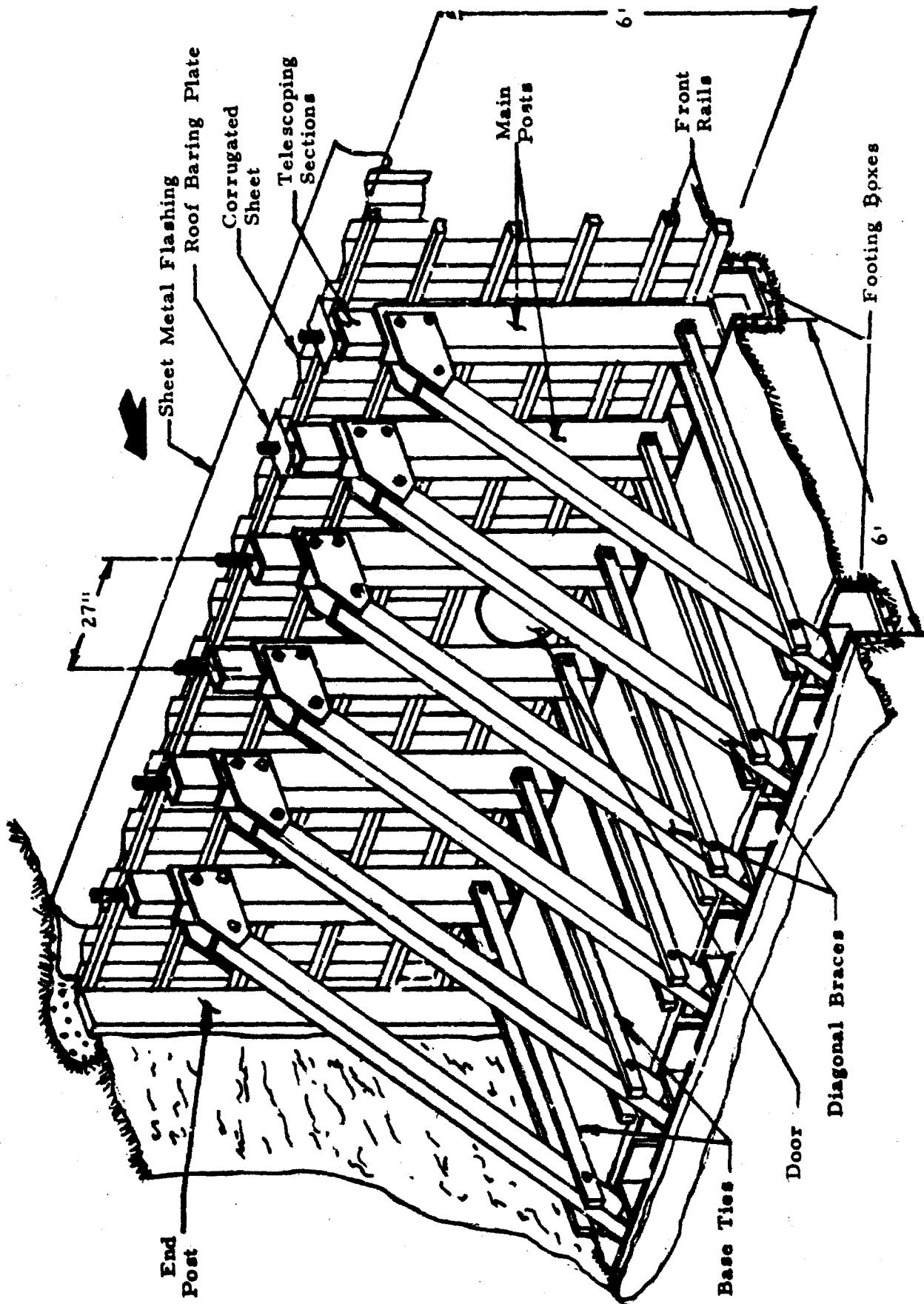
We have selected three concepts, and these are described in detail in Section 6. Three concepts have been chosen because each represents a basically different anchorage scheme, and because it was felt that no one anchorage scheme could suffice for all applications.

The three designs are shown in Figures 28, 29, and 30, and are referred to as:

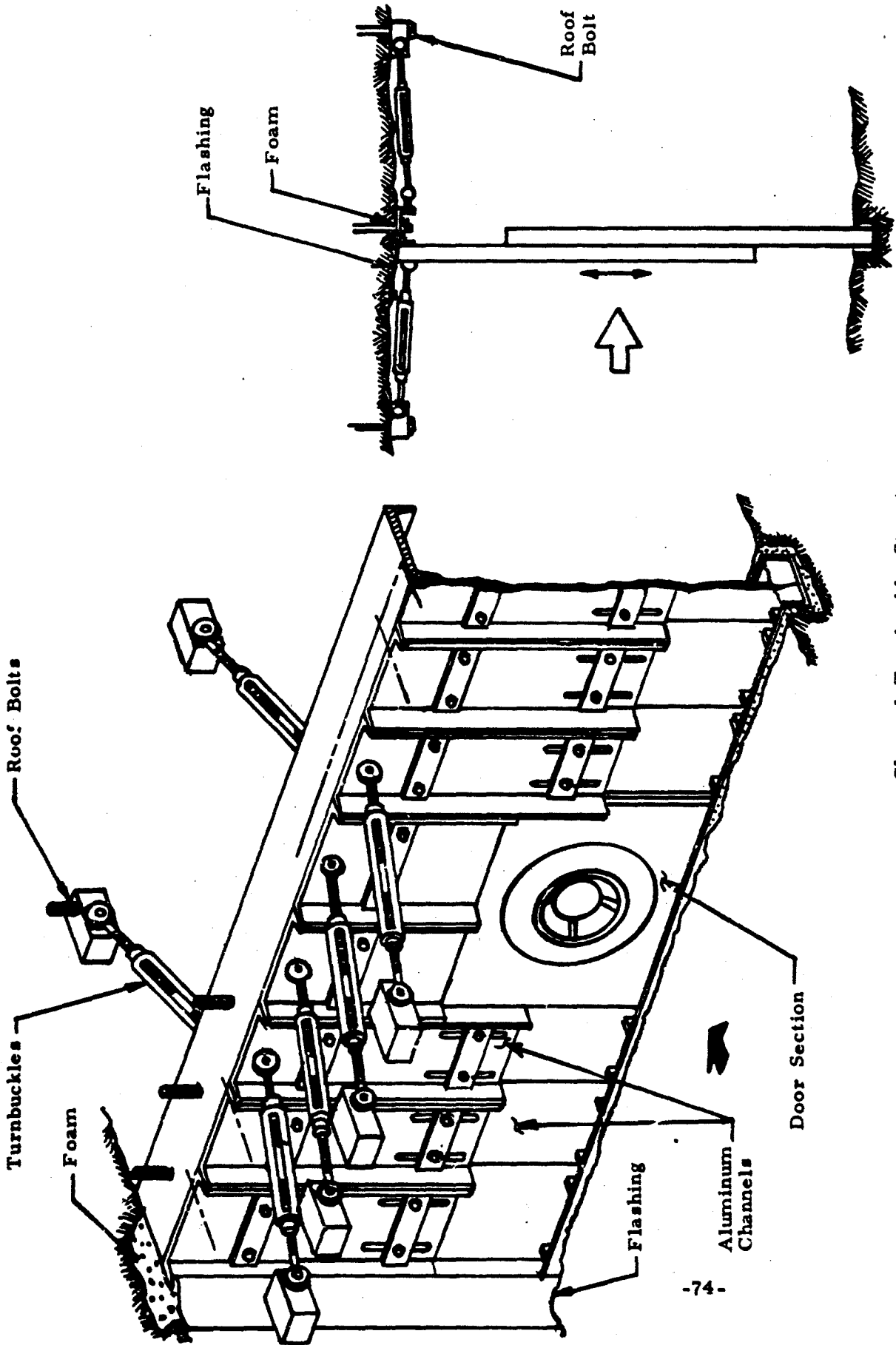
Liner Plate Arch
Truss
Channel Turnbuckle.



Liner Plate Arch
Figure 28

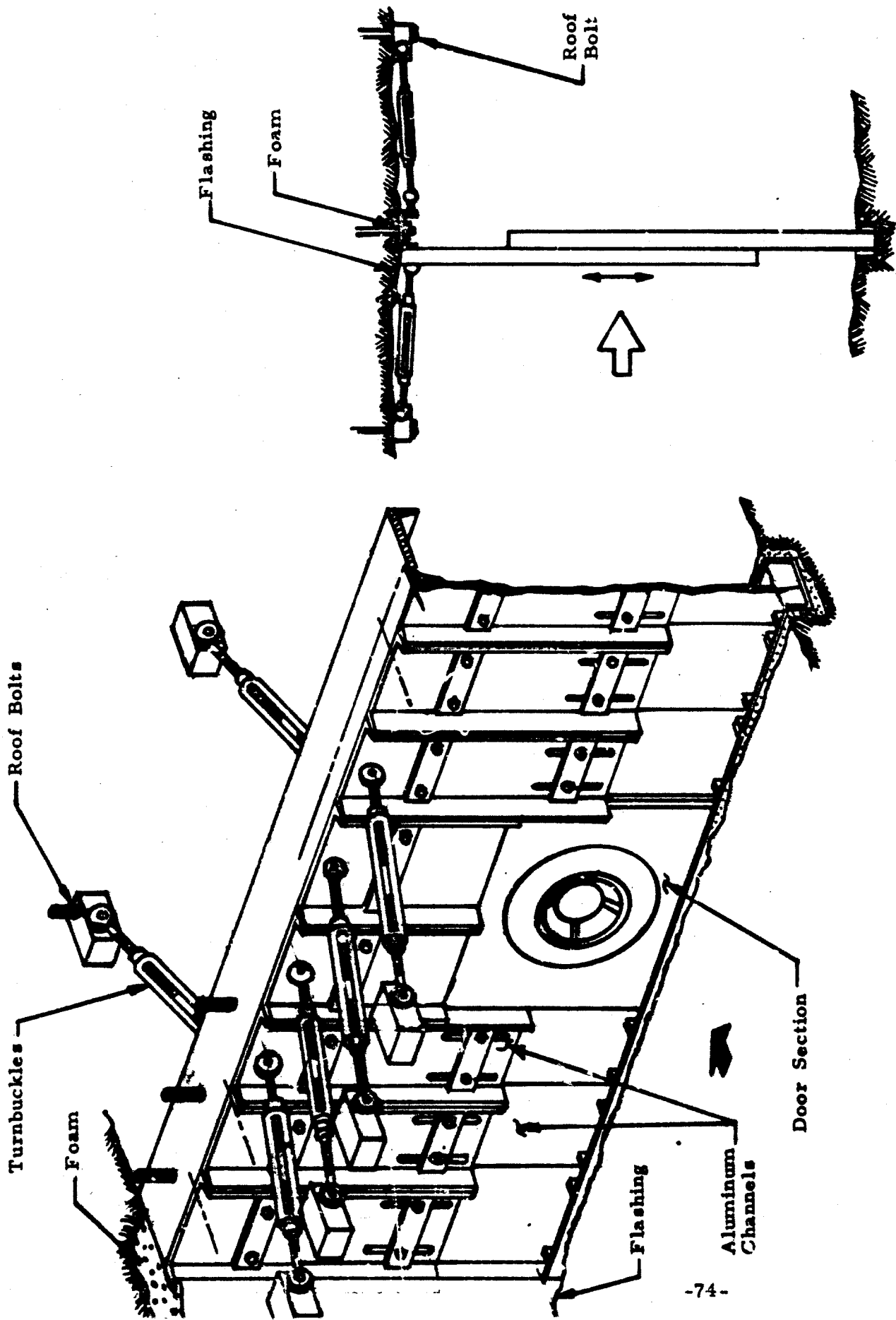


Truss Structure
Figure 29



Channel-Turnbuckle Structure

Figure 30



Channel-Turnbuckle Structure

Figure 30

The detailed description of these designs has been reserved for Section 6, where design specifications, calculations, and assembly procedures are presented.

Some of the major design decisions which dictated the final selections, and which evolved from the considerations presented in this section, can be summarized as follows:

Floor Anchorage - The use of a trench has been preferred to anchor bolts. In the past bolt installation in the floor has been time consuming, and special drills are required for handling the cuttings. For the extra effort involved, the use of a trench provides continuous support and a far greater margin of safety.

Roof Anchorage - The header concept has been found to be impractical and direct connection to roof bolts via turnbuckles has been favored. In this manner, the load on the roof bolt is well defined. The use of the roof for structural support is still viewed with caution, but conceptually there are many competent roof situations which should provide sufficient support. Some in-situ load testing for roof anchor bolts would shed much light on this area.

Sealing - A sheet metal flashing strip backed up by frothed urethane foam has been proposed for sealing the irregular space between the bulkhead and the passage walls. This can be applied simply with minimum skill, and can seal gaps from 1" to 6" in span. The development of this seal technique is described in greater detail in Section 7.

There are a number of adequate sealants for intrastructural connections. Because of its range of applicability, thiokol has been recommended for the three proposed designs.

In the next section we will focus our attention on "permanent" bulkhead concepts.

4. Review of Permanent Bulkhead Concepts

4.1 Information

4.1.1 Purpose

The review of the application of emergency shelters has indicated that the life of a shelter may vary from six months to several years. For a shelter with a long life, the use of a permanent bulkhead seems to be a logical approach.

The purpose of this section is to provide the background information necessary to develop permanent bulkhead concepts which will be competitive with the reusable concepts currently being considered. This background information includes some extensive research and development efforts carried out by various mining organizations in the United States and abroad. This review seeks to identify previously developed concepts which would be applicable to an emergency shelter. It also seeks to review past experiences in choice of materials, materials handling techniques, construction techniques, and overall costs, to serve as guidelines for emergency shelter bulkhead development. Ultimately this information will be used to evaluate the relative merits of a number of proposed bulkhead concepts, both permanent and reusable.

4.1.2 Background

The principal applications of permanent bulkheads in the mining industry have been for ventilation control, for sealing gob areas, and for sealing mine fires. Ventilation stoppings are principally air seals, and are designed to withstand only small (1 psi or less) pressure differentials. Fire seals and gob area seals are designed to withstand an explosive pressure. Design pressures have varied from 20 to 70 psi, depending on the agency and the year in which the specifications were promulgated. Recently, these figures have been based on measurement of pressures produced by simulated coal dust and methane explosions (see Appendix A).

Fire seals generally require a rapid setting material, due to the likelihood of an explosion occurring shortly after the seal is constructed. Gob area seals are designed to withstand explosive pressure from either side. Neither of these requirements apply to emergency shelter bulkheads. On the other hand, since the sealed volume of an emergency shelter is so much smaller than that for the two above mentioned applications, the leakage requirements for an emergency shelter bulkhead are much stricter. Emergency shelter bulkheads will also require a personnel access. Although the requirements for an emergency shelter bulkhead are somewhat different from those for the previous applications, we will look for those concepts which can be appropriately modified (if necessary) to serve the purpose at hand.

The agencies involved in explosion-proof bulkhead evaluation and testing have been the U. S. Bureau of Mines (USBM), the European Community for Coal and Steel (ECCS, Luxemburg), the British National Coal Board (NCB), and the Experimental Mine Company (EMC, Germany). Bulkheads have been constructed of a variety of materials, including concrete gypsum fly ash cement, cement block, sand, rock dust, and others. These bulkheads will be described in greater detail in the next sections.

4.1.3 Scope

This review will cover the development of permanent explosion-proof bulkheads. The review will include a summary of the various materials which have been used (Section 4.2), and a description of the associated construction techniques, including materials handling and erection procedures (Section 4.3). A review of bulkhead design procedures is presented in Section 4.4, and explosion testing techniques are described in Section 4.5. Section 4.6 presents a summary of the results of these test programs, and Section 4.7 presents conclusions and recommendations for our current emergency shelter application.

4.2 Permanent Bulkhead Materials

4.2.1 Requirements

Bulkhead materials have been traditionally selected on the basis of cost, availability, ease of handling, and strength, in that order. Frequently, lack of strength has been compensated for by an increase in bulkhead mass, when the other factors are favorable. Other physical properties which influence material performance include shrinkage, dry compaction, deformability, and in the case of fire seals, setting time.

4.2.2 Summary of Materials

Sandbags - Sandbags have been a traditional mine sealing method. For an explosion proof design, loose sand is usually packed in 65 lb. jute sacks and stacked in the mine passage. Typical seal thicknesses are between 12 and 18 feet, and the adopted standard is: $T = 1/3 W \cdot H$ (where W and H are the passage width and height, respectively).⁽¹⁶⁾ Girders must be used at the top of the seal to prevent shifting of the top bags, and air leakage may often be excessive as the sand compacts.

Masonry - Masonry refers to limestone (10" x 20" x 10") or concrete (6" x 6" x 8") blocks bonded with cement mortar. For adequate explosion strength, the masonry walls are constructed in two or three layers (courses) and results in a wall thickness of 1.5 to 3.0 ft. Recessing in the ribs and floor has improved strength considerably.

Concrete - Concrete has been used both plain and reinforced. Typical mixtures have contained cement, sand, and gravel in proportions of about 1:2:4. These proportions have yielded compressive and tensile strengths of the order of 3,000 psi and 350 psi respectively. For bulkhead construction, the water content is kept

low to limit shrinkage, hasten setting time, and improve strength.

Early USBM investigations indicated that a plain concrete bulkhead recessed into the ribs could withstand the required explosion pressures, and that reinforcement was unnecessary.⁽³⁰⁾ This fact has much to do with the degree of support provided by the rib coal, as will be discussed in Section 4. Nevertheless, there have been no reports of subsequent investigations into the use of reinforced concrete.

U. S. regulations specify⁽²⁸⁾ a bulkhead thickness of $(W \text{ or } H)/4$ (whichever is larger) for plain concrete, and $(W \text{ or } H)/10$ for reinforced designs. Shrinkage, and slumping away from the roof remain as major drawbacks, however, and for these reasons the use of concrete has been discouraged in Europe.

Fly Ash-Cement - Mixtures of fly ash, cement, and water (62 percent fly ash by weight, 7 percent cement, and 31 percent H_2O) have been investigated in the U. S. The availability of fly ash makes it an attractive bulk material and the mixture provides strengths comparable to gypsum products (flexural strengths approximately 100 psi).⁽¹⁷⁾ Slight shrinkage occurs on setting, however, and a minimum setting time of three days makes the fly ash-cement mixture unsatisfactory for emergency constructions. Standards have established bulkhead thicknesses of $(W \text{ or } H)/4$ for explosion proof design.⁽¹⁸⁾

Gypsum Products - Various forms of gypsum ($CaSO_4 \cdot 2H_2O$) have been used as bulkhead structural materials. During manufacture, raw gypsum is heated to eliminate the water of hydration. The resulting plaster of paris ($CaSO_4 \cdot 1/2 H_2O$) is mixed with water to reform the gypsum crystal structure. Since the recrystallization occurs rapidly (1 to 15 minutes), retarding agents have been added to slow the setting time and enable the mixture to be used in bulkhead construction. The proprietary products used in bulkhead construction are Hardstem (15 minutes setting time) and Hardstop (30 minutes or 90 minutes).

Saarilit ($\text{CaSO}_4 \cdot 1/2 \text{H}_2\text{O}$) is a similar product, produced in Germany as a by-product in the preparation of phosphorous acid. Additives again retard the setting time.

Synthetic anhydride (CaSO_4) is a by-product in the preparation of hydrofluoric acid and reacts with water to produce gypsum. The setting times for anhydride are greater than those for plaster of paris and no retarding additives are used.

Practical mixtures of plaster of paris typically require 50 percent water by weight of solids. Flexural strengths vary from 600 psi to 100 psi depending on the mixing method.⁽¹⁷⁾ The latter value is more typical of field conditions. Up to 1 percent of shrinkage may occur.

Synthetic anhydride can produce a higher strength gypsum, but it appears to be more sensitive to the water/solids ratio (0.06 gal/lb. gives a flexural strength of 240 psi, 0.09 gal/lb. gives 100 psi),⁽¹⁶⁾ and does not provide any major advantage over plastic of paris.

Gypsum deforms plastically before failing,⁽¹⁷⁾ and hence the possibility of plastic accommodation will improve the performance of gypsum subject to both repeated explosions and ground motions.

Water Bags - The use of water bags for the construction of mine seals has received some attention in Europe. A single, or multiple bags are inflated, filled with water and then supported by brattices and ropes. The bag strength is the key structural element. These have been made from nylon reinforced neoprene, and polyvinyl chloride coated polyamid. Some fire protection (e.g. asbestos) must be provided.

Composite Bulkheads - Some explosion barriers have been designed in which the load carrying capability is distributed between two or more materials. In a typical composite bulkhead, a large volume of filler material (rock dust, mine debris, etc.) is sandwiched between wooden or masonry retaining walls. In this configuration, the filler provides an inexpensive, deformable support for the retaining walls.

Wooden walls with rock dust fill have been investigated, but do not provide adequate strength to withstand even small explosions. With the use of masonry end walls, composite designs have demonstrated excellent explosion strength and air sealing capability.

4.3 Bulkhead Construction Techniques

Different bulkhead designs may require different material handling and construction techniques. The required bulkhead location, and distribution of these locations throughout the mine, and the local passage geometry will determine which transportation and construction methods are the most economic for a given application. For example, bulkheads located on or near mains may be constructed from materials delivered in bulk, while remote locations off mains may favor hydraulic or pneumatic transport. Bulkheads constructed in low coal may not permit the use of standard mixing and pumping equipment.

For emergency shelter applications, the bulkheads will be placed both near main haulage lines and up to 4,000 ft. down developing entries. Bulkheads will be required near working sections, which may be concentrated, or which may be at opposite ends of the mine. They will be required in both low and high coal. The tradeoffs between the alternate material handling and construction techniques must be evaluated with these factors in mind.

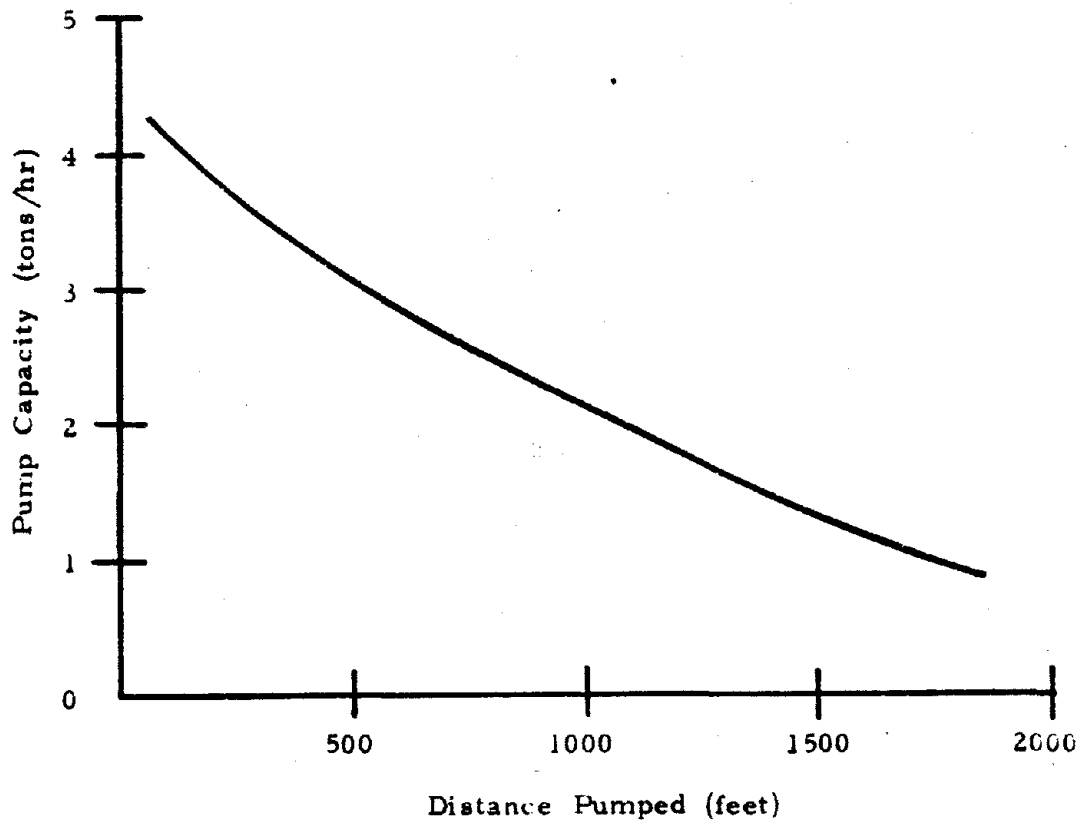
4.3.1 Materials Handling Methods

4.3.1.1 Bulk Delivery of Materials

All of the prospective material handling methods require some bulk delivery. The bulkhead materials are delivered to the mine by rail or truck, transferred to mining cars and transported to the passage level. From this point, the materials may be transferred to the bulkhead site in shuttle cars, and the materials may be mixed directly at the site. For sand bag or masonry bulkheads, transport of the materials directly to the site is the most economic method. For monolithic bulkhead designs which include a dry powder and water, pneumatic and hydraulic transport methods have a definite advantage.

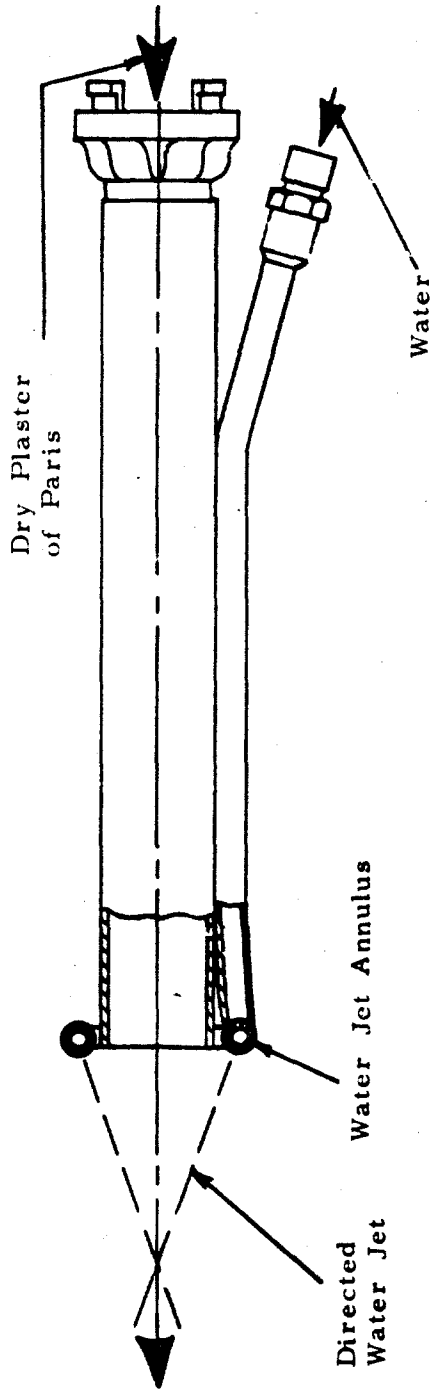
4.3.1.2 Pneumatic Transport

Pneumatic pumps can be used to transport dry powders over long distances. Rock dust and plaster of paris have been pumped pneumatically to bulkhead construction sites as far as 2,000 ft. ⁽²⁰⁾ Typical air requirements for a ton of powder are 400 ft³/ton powder for a 50 foot pumping distance, and an additional 7,000 ft³ for each additional 330 feet of travel. Figure 31 indicates the relation between pumping capacity and distance. Pneumatic transport is essential for plaster of paris since it is too fast setting to permit hydraulic pumping. In the construction of a gypsum bulkhead, the water and plaster of paris are mixed at the site with a specially designed nozzle (Figure 32). The major drawback of this method is that the flow of plaster of paris exiting the nozzle cannot be regulated uniformly. The water/solids ratio in the mixed gypsum varies and the strength of the bulkhead will suffer. Laboratory tests ⁽¹⁷⁾ indicate that pneumatically mixed gypsum has a flexural strength of only 106 psi, while properly mixed gypsum has strengths of 575 to 597 psi.



Pneumatically Pumped Master of Paris: The Effect of Pumping Distance on Pump Capacity

Figure 31



Pneumatically Pumped Plaster of Paris: Water/Plaster Mixing Nozzle

Figure 32

4.3.1.3 Hydraulic Transport

With the development of slow setting plaster (Hardstem, Hardstop, Saarilit) and synthetic anhydride for use in bulkhead construction, hydraulic pumping of a gypsum slurry became feasible. The slurry is produced in a slurry rockduster or any electric or compressed air mixer. The controlled mixing of the powder and the water enables a uniform and high-strength bulkhead to be constructed. Fly ash-cement-water slurries may be mixed in a similar fashion. In European studies,⁽²⁰⁾ slurries have been hydraulically pumped over 650 feet with a pump capacity of 200 to 300 cu. ft. per hour.

4.3.2 Construction Methods

Particular construction methods are strongly dependent on the shape and support available in the mine passage. Wide passages or soft, friable coal will require a greater bulkhead thickness for the same blast resistance. There are some general construction procedures that are followed for each bulkhead type.

Masonry Seals - Materials for masonry seals are typically prepared at the bulkhead site. The wall is constructed without framing. Rib and floor recesses of 16 and 12 in. deep⁽¹⁷⁾ are common, and the floor is usually leveled by laying a cement footing for the brickwork. For a bulkhead width of 16 or more feet, USBM researchers have experimented with the use of a reinforcing "pilaster" at the center of the bulkhead for additional structural support. Should this be required, the construction cost estimates based on similar bulkhead widths would have to be increased.

Formed Bulkhead Construction - All bulkheads that require setting, and those bulkheads made from filler materials like rockdust and mine debris, require a similar construction of brattices and bracing. In addition, recesses are often included to improve bulkhead strength when homogeneous materials are used.

The bulkhead frame is usually constructed of plywood or planks with reinforcing braces every 3 feet. A typical brattice construction design is illustrated in Figure 33.

If access, ventilation or drainage is required through the bulkhead, the necessary tubing must be installed in the brattice before the bulkhead is poured. A ventilation and access tube (usually 2 to 3 feet in diameter), a water drainage pipe and an atmosphere surveillance tube are frequently required.

Before the pumping of the slurry or powder fill, glass fiber or brattice cloth is packed into all areas for potential leakage. For slurry bulkheads, it is important not to have any free water in the bulkhead space which may dilute the pumped mixture.

It is important that the bulkhead be constructed in one continuous pouring. Any prolonged interruption (greater than 10 minutes) may adversely affect the bulkhead strength. Concrete and fly ash-cement bulkheads are less vulnerable to this problem since they require setting times of several days. Figure 34 indicates a typical pumping operation for gypsum bulkheads.

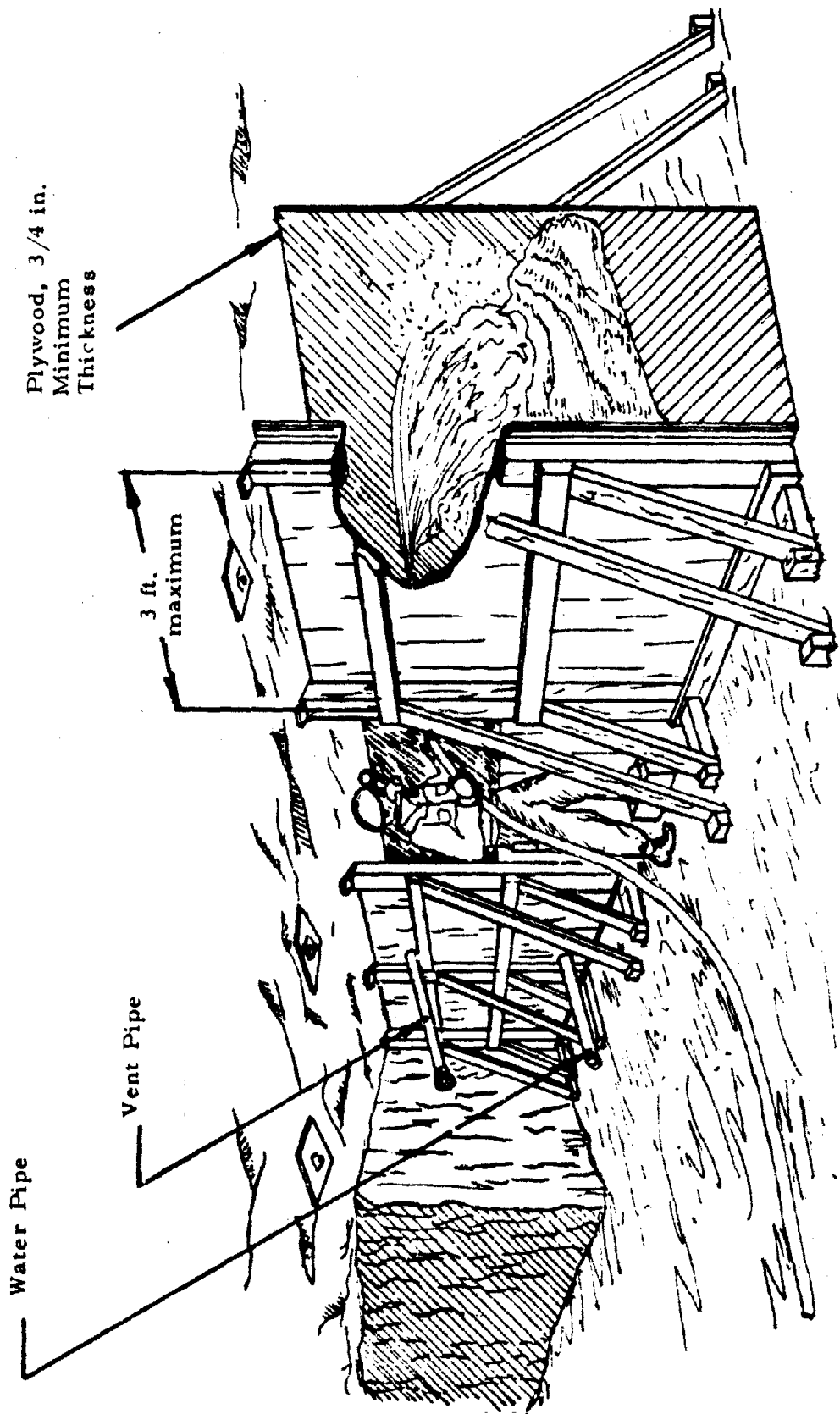
4.4 Bulkhead Design Techniques

Although most bulkhead development work has been through trial-and-error full scale testing, it is important to acknowledge those design techniques which can be used and to appraise their applicability and limitations. Some design techniques have been used in previous studies, and these will be discussed below.

4.4.1 Bulkhead Structural Behavior

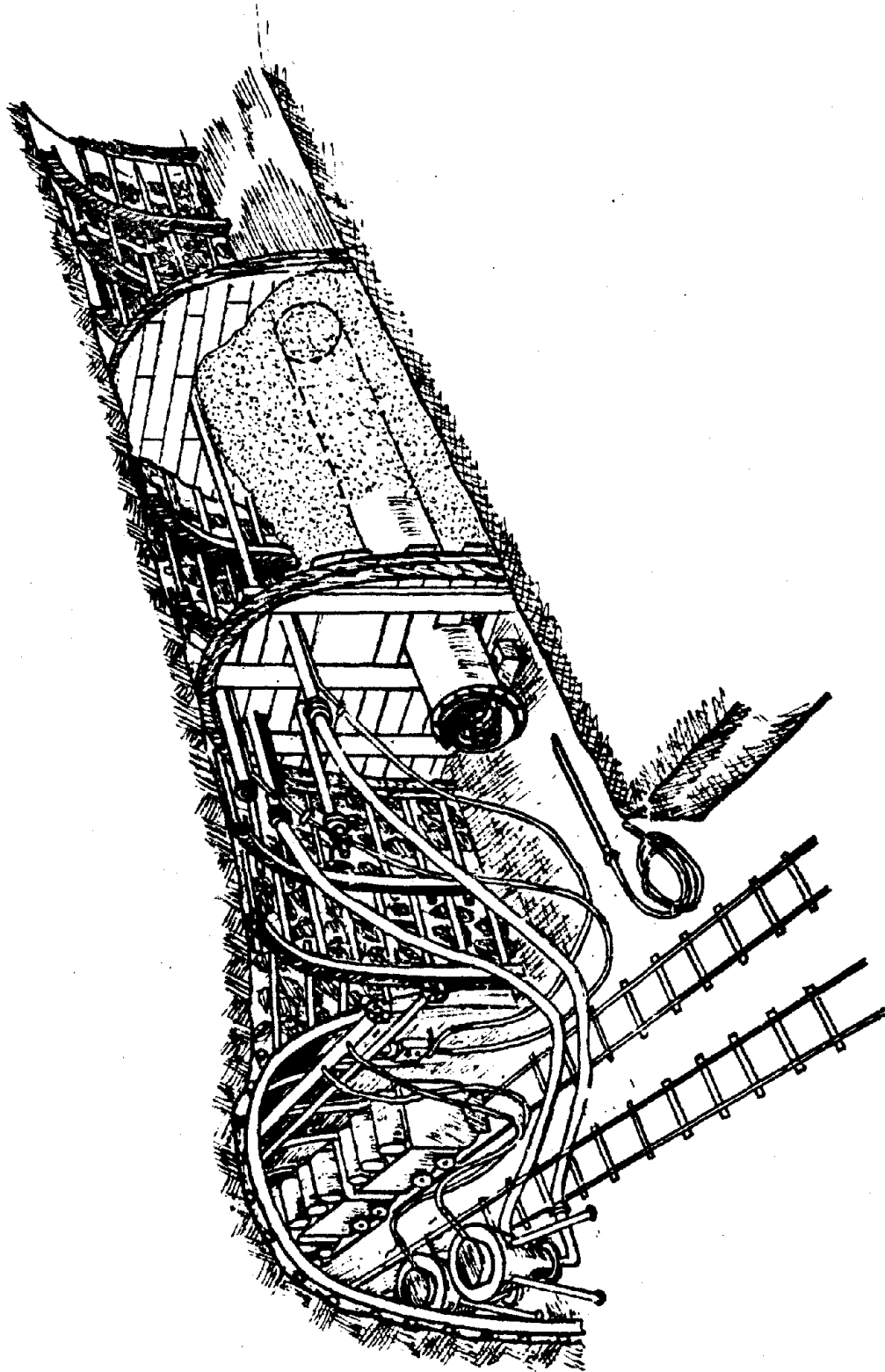
Bulkheads developed and tested to date can be structurally categorized as either a plug, a baffle, a thin plate, or a thick plate, with the following characteristics.

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Formed Construction Method

Figure 33



Typical Pump Installation

Figure 34

Plug - A plug bulkhead has a thickness of the order of its width. It resists explosive forces with its inertia and through friction developed with the passage roof, floor, and ribs.

Baffle - A baffle resists explosive forces by absorbing energy through reflection or through a deformable medium. Composite bulkheads, consisting of two walls separated by a large amount of deformable filler, are an example of this type of bulkhead.

Thin Plate - Here, the bulkhead is a wall whose thickness is less than one tenth of its width. Its structural behavior is characterized by plate bending, with simple or fixed supports.

Thick Plate - A thick plate structure has a thickness which is greater than one tenth of its width. The extra thickness induces an arching action which is superimposed on the bending behavior.

4.4.2 Design Calculations

Evaluation of bulkheads has been primarily through full scale explosion testing. Calculations depend on the load-time characteristics and the material response, both of which are extremely uncertain for a bulkhead in a mine environment. Design calculations do, however, provide a rational context for understanding experimental results and for extrapolating to new situations, and some calculation techniques have been used.

The techniques which have been developed for bulkhead design involve plate-type structures. These techniques are static analysis techniques. This can be modified for dynamic blast loading by recognizing that the maximum structural response of a structure subjected to a stepped loading is twice that which would occur if the load were applied statically.

Thin Plate Analysis - The simplest design calculation assumes the bulkhead to act as a simply supported thin plate spanning the long dimension of the passage. Here, failure is in bending at the center of the span, and this bending failure is governed by the tensile strength of the material. Using this method, the required bulkhead thickness is predicted to be

$$T = 0.865 a \sqrt{p/s_f} \quad (1)$$

where

- T = bulkhead thickness
- a = maximum dimension
- p = design pressure
- s_f = tensile (flexural) strength

In actuality, this technique is extremely conservative, since it ignores the other two wall supports, and it neglects a supporting wall thrust, which will be described in greater detail in the following paragraphs. Mitchell has observed⁽¹⁶⁾ that most bulkheads designed by the above method have withstood much higher (up to a factor of 10 times the design pressure) explosion pressures. This observation, however, does not support the conclusion that bulkheads which are designed for a static load can withstand a much higher dynamic load. Rather, it suggests that the static design technique could be modified to more realistically represent the response of the structure.

Thick Plate - Arch Analysis - USBM investigations^(29,30) found that bulkheads recessed into the floor and ribs attained strengths surprisingly higher than the pressures for which they were designed. They concluded that this was due to an arching action which took place through the thickness of the bulkhead, and which applied a lateral thrust to the coal ribs. Design techniques have been developed to consider this action. Genthe⁽²⁰⁾ used what he referred to as the "support line method." Whitney et. al.⁽³¹⁾ developed an arch model

which treats the bulkhead deforming as two rigid blocks, cracked at two sides and along the midspan. Assuming rigid walls, their conclusions can be represented as:

$$T = \beta \cdot a \sqrt{\frac{p}{S_c}}$$

where

S_c = compressive strength of the bulkhead material

$$\beta = \frac{1}{\sqrt{2}} \left[1 - \frac{S_c}{E} \frac{\sqrt{1 + 4\alpha^2}}{\sqrt{1 + 4\alpha^2} - 1} \right]$$

E = elastic modulus of bulkhead material

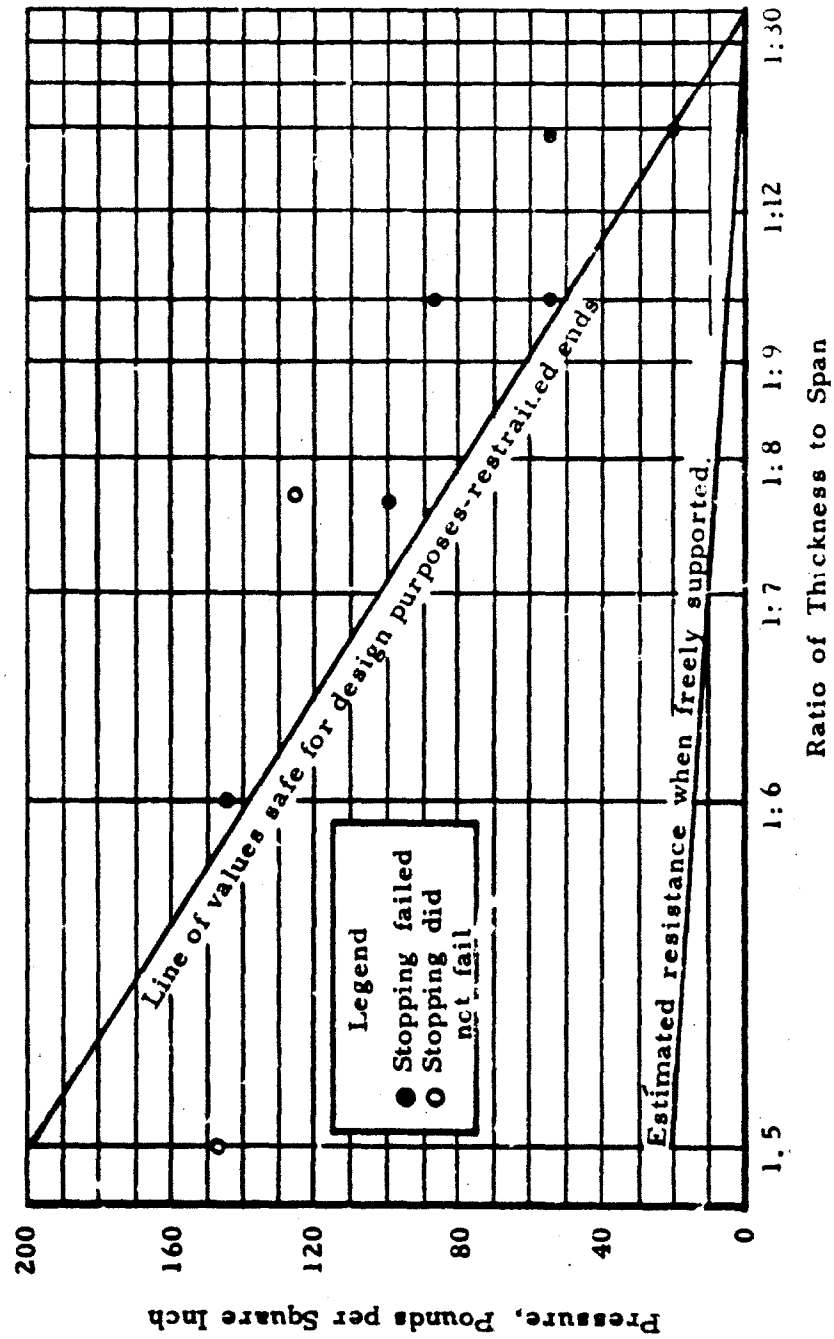
α = T/a

For thickness to width ratios (α) of 1/10 or greater, β is approximately 0.707, and the thickness formula is approximately

$$T = 0.707 \cdot a \sqrt{\frac{p}{S_c}} \quad (2)$$

We see that this is very similar to the bending formula presented earlier, with the exception that S_f , the tensile strength, is replaced by the compressive strength S_c . For most bulkhead materials, the S_c is approximately 5 to 10 times S_f . This implies that (2) permits a reduction of design thickness of 50 percent to 70 percent from those values predicted in (1). In other terms, formula (2) states that a bulkhead can actually withstand 5 to 10 times the design pressure used in (1). Figure 35 shows an experimental curve which supports this conclusion.

The use of recesses has helped to achieve the increase in strength described above. The recesses insure that the



Relation of Strength of Plain Concrete Stoppings Tested to Ratio of Thickness to Span

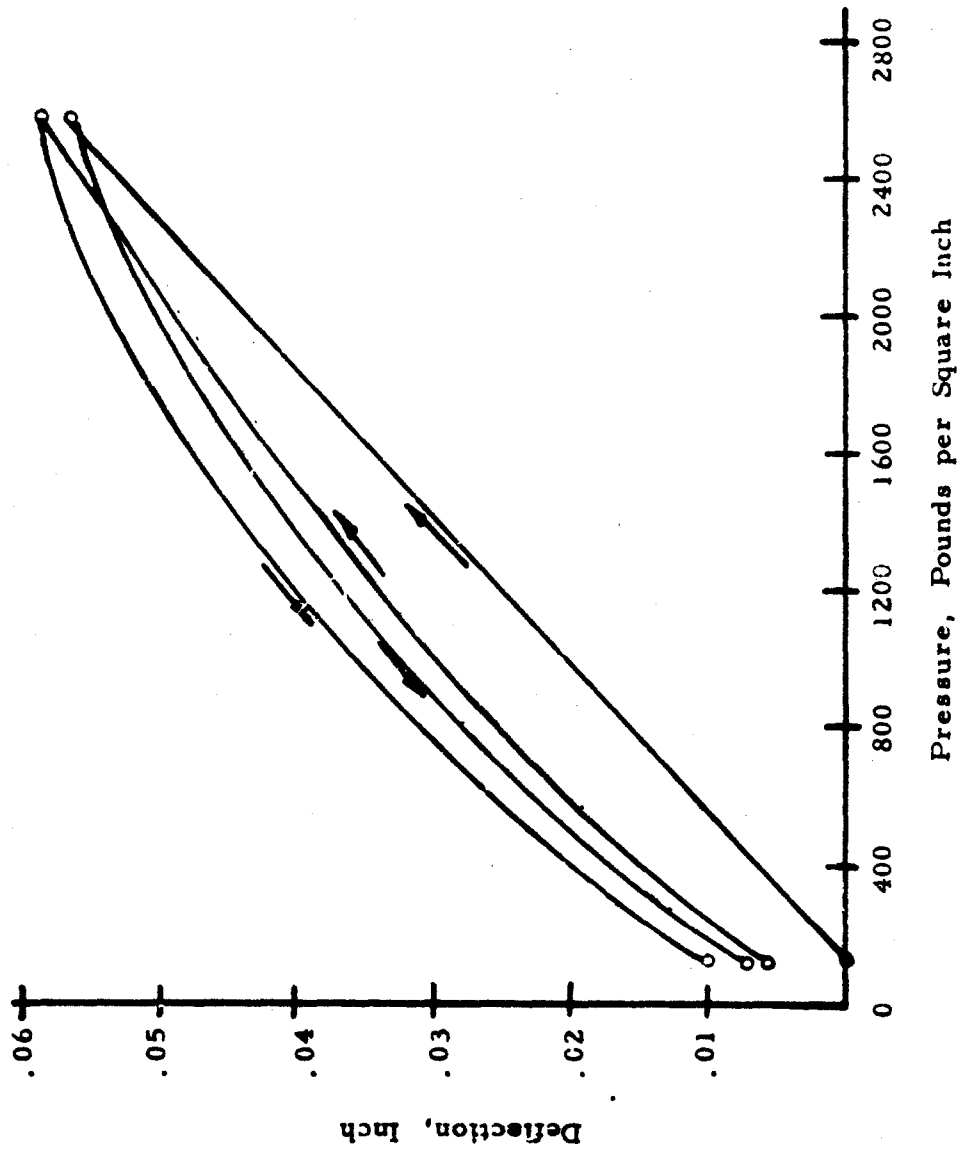
Figure 15

thrust is distributed through a concrete abutment before it is transmitted to the coal, and hence is fully developed. Concern has been given to the available thrust capacity of the coal, and tests have been conducted along these lines.^(29,12) These tests have determined in-situ coal compressive strengths and compressibility of coal. Figure 36 shows a typical load-deformation curve. The recess serves to lower the unit pressure, and hence minimize deformation. Future design efforts can include coal deformation in the design formula, and design the size of the recess to insure adequate bulkhead strength.

The "thick plate" design approach has two principle drawbacks. First, it relies on the strength and stiffness of the coal in place. This is certainly an unreliable source of support, and future design efforts should use lower bound values for these quantities. The second problem, relating specifically to emergency shelters, is that the arching behavior does not occur until significant cracking has occurred. This may result in a post-explosion bulkhead which is still standing but which is leaking excessively. The thicker the bulkhead, the smaller the cracks will be. This, then, is another design factor which must be considered for future application of permanent bulkheads to emergency shelters.

4.4.3 Small Scale Experiments

Small scale testing of bulkhead plates^(20,29) has provided information regarding the response of various types of bulkheads to different loadings, different rates of loading and different methods of restraint. For example, a short series of model tests⁽²⁹⁾ demonstrated that restraining the edges of a bulkhead will cause a dramatic increase in strength, much greater than one would expect from plate theory. Another series of model tests⁽²⁰⁾ showed that some bulkheads were able to support higher dynamic peak pressures than sustained static pressures. In general, small scale experiments have been useful for qualitative observations, but have little value for quantitative measurements. Such measurements require scaling of both the pressure-time curve and the material properties.



Typical Load-Deflection Curve for Rib Coal Loaded In-Situ

Figure 36

The following two sections review past work in explosion testing of bulkheads.

4.5 Full Scale Test Techniques

4.5.1 Philosophy

Full scale testing has been considered in the past to be the only satisfactory method of evaluating alternative bulkhead designs. The basic technique is to install the prototype bulkhead in an actual mine and subject it to the forces of an artificially created explosion. The most important pieces of information obtained are the pressure required to fail the bulkhead, and the maximum pressure withstood by the bulkhead.

Much of the information supplied with previous test research is incomplete. Frequently omitted is a description of where pressure measurements are taken, (e.g., along the passage wall, or on the bulkhead face). It has been assumed here that most measurements are at the bulkhead surface. Also frequently absent is a pressure-time trace, or a measure of total impulse.

The following sections will summarize the various techniques used in full scale experiments.

4.5.2 Test Procedures

4.5.2.1 Types of Explosions

All testing requires an explosion source. Laboratory tests have used compressed air and blasting powder,⁽²⁹⁾ but full scale tests usually employ coal dust and methane/air mixtures to simulate typical mine explosions. In a methane/air simulation,⁽¹⁹⁾ 1,500 to 3,500 ft³ of methane and air (7 to 10 percent methane), are retained between two plastic or paper screens 100 to

250 feet down the passage from the test bulkhead. Ignition is provided by one or more cartridges of black powder and gun cotton.

In coal dust explosion simulations, 1,000 to 2,000 lbs. of dust are usually piled on racks which are distributed along the passage length. A small methane/air or black powder explosion is used to disperse and ignite the coal. Bags of coal dust and black powder igniters have also been used.⁽²⁸⁾

4.5.2.2 Bulkhead Location

Apart from the explosion intensity, the two variables that determine the peak loading on the bulkhead are the passage length between the bulkhead and the explosion source, and the orientation of the bulkhead with respect to the mine passage.

Increasing the distance of the bulkhead from the explosion source can either increase or decrease the peak pressure, depending upon what distance is required for a shock front to coalesce. Bulkheads parallel to the direction of the pressure wave will experience a side-on pressure equal to that of the wave, while bulkheads perpendicular to the pressure wave will experience additional reflected pressures which can more than double the pressure of the wave. With the exception of the recent USBM research⁽¹⁷⁾ and one other location,^(20, Hagenbeck mine) all bulkheads described have been of the latter variety. In the Bureau's work, the bulkhead was recessed five feet into a cross-cut perpendicular to the passage in which the explosion was ignited.

Some test bulkheads have bounded a confined chamber. Since the expanding gases have nowhere to go, the bulkhead experiences a much longer pressure pulse and a much greater impulse than would occur in an open-ended passage.

4.5.3 Test Parameters and Measured Results

Testing programs have taken a given type of bulkhead and subjected it to a number of explosions of different intensity until failure occurred. The same procedure has been carried out varying the bulkhead thickness, material formulation, and degree of restraint.

Associated with each test is a pressure-time record experienced by the bulkhead. As mentioned earlier, usually only peak pressures are recorded. After each test, the bulkhead is examined for deflections, cracks, and any other manifestations of damage.

Air leakage measurements are a relatively new means of evaluating the integrity of a bulkhead. Since leakage of combustible or inert gas into a rescue chamber may endanger the personnel using it, one requirement of structural integrity is that air leakage rates remain satisfactorily low. Methods have recently been developed⁽¹⁷⁾ which can monitor air flows for very low (10^{-3} psi) differential pressures, but there is little experimental data on its use.

4.6 Summary of Previous Bulkhead Testing

The U.S. Bureau of Mines, the British National Coal Board (NCB) and the European Community for Coal and Steel (ECCS) have performed independent evaluations of bulkhead designs during the last fifty years. Since they were done independently, test methods and objectives have been different. A general summary of full scale, explosion testing of bulkheads is compiled in Appendix B. While none of these studies have been oriented towards the design and construction of an emergency shelter, the test configurations and results are directly applicable to these requirements. For example, some of the bulkheads tested have ventilation tubes large enough for men to crawl through, and sampling tubes that permit remote inspection of the atmosphere on the other side of the seal.

4.6.1 U.S. Investigations

4.6.1.1 Initial Research (1920 - 1930)

The U.S. Bureau of Mines became involved in testing explosion-proof bulkheads as early as 1923, as a result of legislation⁽²⁹⁾ regulating coal mining in the public domain. Bulkheads were required to withstand a 50 psi coal dust explosion. Two series of tests were performed on reinforced and plain concrete bulkheads at the Bureau's experimental mine, and a number of "laboratory" tests were performed at the Bureau of Standards in Washington, D. C. As was mentioned earlier, the strengths of the bulkheads tested indicated that under loading they behave as arches in compression or other than as simple beams. Since the strength of an arch is in part determined by the crushing strength of the surrounding material, tests were made on the compressibility and crushing strength of the Pittsburgh Coal Bed.

Blasting powder was used to test the bulkheads instead of coal dust. Its use was justified by a comparison of their pressure/time curves which showed that for the same maximum pressure, the pressure rise rates are similar, and the total impulse is much greater for the blasting powder. The tests indicated that restrained concrete beams are 8 to 10 times stronger than unrestrained ones. The mode of failure was the same for all stoppings, originating in a vertical, full length crack at the center of the outby face with a depth of at least 75 percent of the bulkhead thickness. This investigation demonstrated that plain concrete was adequate to withstand the required loading and the following standards were adopted for plain concrete bulkheads.⁽²⁴⁾

$$\text{Thickness (T)} \geq \frac{\text{width(w)}}{10}$$

$$\text{Rib Recess (R)} \geq \frac{w}{10}$$

For Soft Coal:

$$T \geq \frac{w}{8}$$

$$R \geq \frac{w}{5}$$

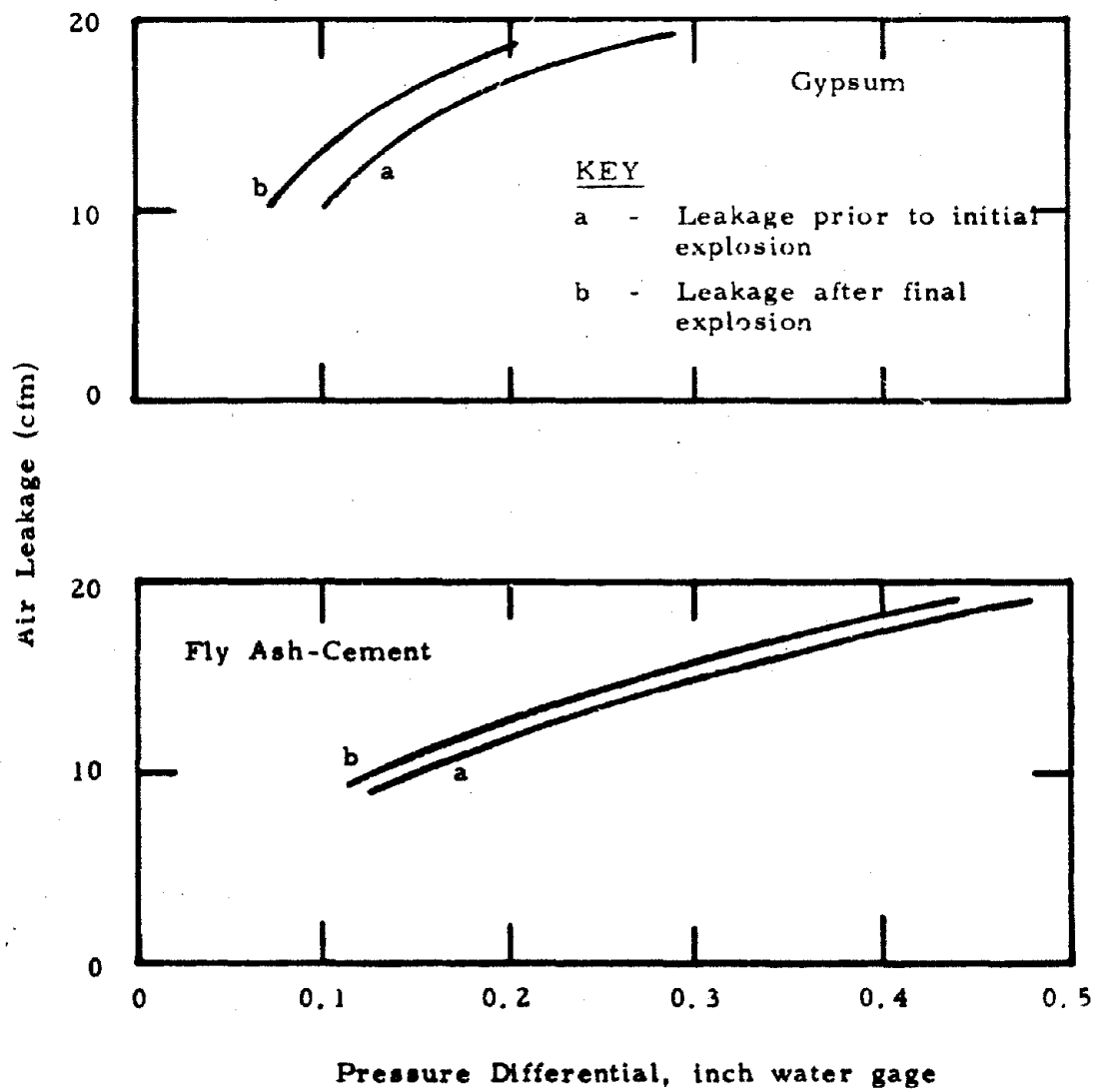
For all Bulkheads:

$$T \geq 12 \text{ in.}$$

4.6.1.2 Current Research (1970 - 1973)

In accordance with the Federal Coal Mine Health and Safety Act of 1969 (P. L. 9H73), further study⁽¹⁷⁾ into bulkhead design and construction has been carried out. Laboratory and mine tests were performed to evaluate new materials and construction techniques. Twenty three slurry materials were evaluated in the laboratory and explosion trials were carried out on six bulkheads, constructed of gypsum, fly ash and cement, and concrete block. Air leakage rates and flexural strength were the major criteria for evaluating the various designs. Flexural strengths were determined by measurements of sound velocities and resonant frequencies of the bulkhead material in accordance with ASTM Method of Test C-215. The data obtained is correlated with data taken from laboratory specimens taken from the same pouring, and the flexural strength is determined by ASTM Method of Test C-78.

The explosion trials demonstrated that all of the proposed designs could withstand a 50 psi (7 psi-sec impulse) pressure pulse. Air leakage rates for the fly ash-cement and gypsum bulkheads were small and not affected by the explosions (Figure 37). As a result of this test program, specifications for bulkhead design and materials preparation have been constructed. For the given materials, the minimum bulkhead thicknesses are given in Table 3.⁽¹⁶⁾



Air Leakage Rates for Two Bulkheads

Figure 37

Table 3

Acceptable Bulkheads for Normal Mining Situations

USBM Recommended Standards

<u>Type</u>	<u>Minimum thickness*</u>
Concrete	$t/4$
Concrete, reinforced	$t/10$
Concrete block	16 inch
Fly ash	$t/4$
Gypsum	$t/4$
Rock, grouted	$W + H/2$
Rock, packed	$2t$
Sand bags	$WH/3$

* t = W or H , whichever is greater, where

W = Average width of passageway and

H = Average height of passageway, plus depth of recess
for concrete block and reinforced-concrete bulkheads.

4.6.2 British Investigations

The Institution of Mining Engineers has investigated^(27,28) the design and construction of explosion proof bulkheads for the containment of underground fires. The stoppings must be explosion-proof since the danger of explosion from accumulated methane increases greatly after the stopping shuts off the mine ventilation.

In their research, Willett, et. al. estimated that the stoppings must withstand pressures between 20 and 50 psi.^(28, p. 16)

Successful stopping designs have been based on sandbags, concrete, and Hardstem. Water dams have also been successful in containing fires, though no information on explosion protection is available. A two-foot diameter ventilation tube is recommended to insure that an explosion will not occur during the construction of the stopping.

Gypsum was first used as a mine sealant in England in 1961. Its suitability from a structural and materials handling viewpoint prompted the I. M. E. to perform some tests to determine its blast resistance. A 10 ft. x 8 ft. x 10 ft. thick bulkhead was constructed and ventilation, sampling, and drainage tubes were installed. The Hardstem was premixed and hydraulically pumped to the bulkhead site. Black powder and coal dust were ignited in a 30 ft. long, closed chamber behind the bulkhead.

Although these tests demonstrated the feasibility of gypsum explosion-proof bulkheads, the excessive thickness of the seal, and the fact that its explosion strength was not exceeded during the tests suggests that they are excessively oversized.

4.6.3 European Community Investigations

A number of agencies within the European Community for Coal and Steel (ECCS) have investigated alternative

explosion-proof fire seal designs and materials. Some independent work has been made,⁽²⁰⁾ but the majority of the research has been sponsored by the ECCS directly or by one of their mine research and safety organizations.^(19, 21-26)

The laboratory testing took place at the Experimental Galleries at Dortmund-Deine. Although a large percentage of the mine tests were performed at the Tremonia Experimental Mine, a large number of mines were involved in more limited testing.

4.6.3.1 Bulkhead Types

Several bulkhead materials were evaluated as alternates to sandbag designs. Single wall masonry bulkheads were constructed of two to three layers (1.7 to 2.5 ft.) of stone. Double wall masonry bulkheads with rockdust fill were also tested for explosion strength. Monolithic bulkheads of cement and rock dust, and gypsum products were investigated most extensively. Gypsum dams were constructed from pneumatically pumped plaster of paris, and hydraulically pumped Hardstem, Saarilit, and synthetic anhydride. The materials handling advantages of homogeneous bulkhead materials was, in part, responsible for the interest shown in them. Fast setting time and good structural strength also made gypsum attractive for emergency fire seals. Water bag seals were discussed but no testing programs had been completed. Concrete, plain or reinforced, was not studied by the European investigators. The setting requirement of several days was considered excessive for any possible use in an emergency application.

4.6.3.2 Major Findings

Several single masonry walls demonstrated satisfactory explosion strength. Depending on the passage geometry and wall thickness, pressures from 20 to 40 psi were withstood by unrecessed stone walls. However, repeated loadings tended to reduce

the strength of masonry walls. In one experiment^(19, Hagenbeck Mine), a 1.7 ft, two layer stone wall (135 ft² passage cross section) was capable of withstanding 15 explosions up to 26 psi, but it later failed at 19 psi.

Double masonry walls with rock dust fill proved to be extremely strong. Two 1.7 ft thick masonry walls, sandwiching 15 ft (86 ft² passage cross section) of rock dust and mine debris was capable of withstanding a pressure of 319 psi. The center of the inby wall was displaced 8 inches, but no structural failure occurred.

A mixture of cement, rock dust and water provided an adequate seal if sufficient setting time was permitted. A 19 ft thick x 86 ft² seal⁽²¹⁾ was able to withstand a 57 psi explosion pressure. However, allowing only 8 hours for setting, a similar seal was unable to withstand 28.5 psi.

Both pneumatically and hydraulically placed gypsum bulkheads demonstrated adequate blast resistance. Pneumatic pumping was found to be less satisfactory^(19,20) since maintaining a uniform plaster of paris flow proved impossible. The variations in the water/solids ratio that inevitably resulted reduced the material strength and required excessive bulkhead thicknesses. In addition, early investigations⁽²¹⁾ believed that thin "membrane" configurations, in which the thickness is much smaller than the width, would produce excessive tensile loads on the wall. Reflecting this, early standards⁽²¹⁾ required the thickness of a gypsum bulkhead be equal to the passage width.

Two developments have contributed to a reduction in this requirement: hydraulic pumping of the slurry, and the use of rib and floor recesses. The use of slower setting Saarilit, Hardstem and synthetic anhydride have enabled accurate metering of the water/solids mixture before it is pumped to the bulkhead site. The effect of recessing on bulkhead strength has already been demonstrated^(24,30), and later European testing^(19,20) incorporated them into

the test designs. The assumption that excessive tensile loads will cause failure in thin bulkheads appears to be ill-founded. The success of thinner designs substantiates the conclusion drawn by the early USBM investigators that a loaded bulkhead behaves as an arch, and fails in compression rather than tension. These modifications in the design helped produce strong, thin seals. A 4.8 ft seal of Saarilit in a 220 ft² passage withstood pressure up to 70 psi,⁽¹⁹⁾ and a 3.25 ft seal in an 86 ft² passage withstood loads up to 276 psi.

4.7 Conclusions and Alternatives

The previous review has indicated that explosion proof bulkheads which have been developed in the past are either excessively massive or highly prone towards leakage after an explosion. Of the bulkhead types discussed, we believe that the fly ash-cement would be the most suitable for an emergency shelter bulkhead. This is primarily due to low cost and simple construction technique. The design would have to be modified to include a door, and a culvert tube for a 25" diameter door (see Section 6.1). The crawl tube would be a 25" diameter pipe with a flange for door mounting. This pipe would also serve as a structural member to make up for the loss of structural continuity.

The primary disadvantages of the fly ash-cement bulkhead are:

Large amount of bulk material delivery.

Requirement for 2 slurry rockdusters (most mines don't even have one).

Requirement for 4 men.

We believe that before evaluation of the use of permanent explosion-proof bulkheads can be made it will be necessary to look at designs other than those which have been developed through

previous mine research. In the following sections we have developed preliminary designs for two alternate permanent concepts - a reinforced concrete wall and a shotcrete arch.

4.7.1 Reinforced Concrete Wall

Figure 38 shows a sketch of a reinforced concrete design. A qualitative description of this design will be presented here. Design calculations can be found in Appendix C.

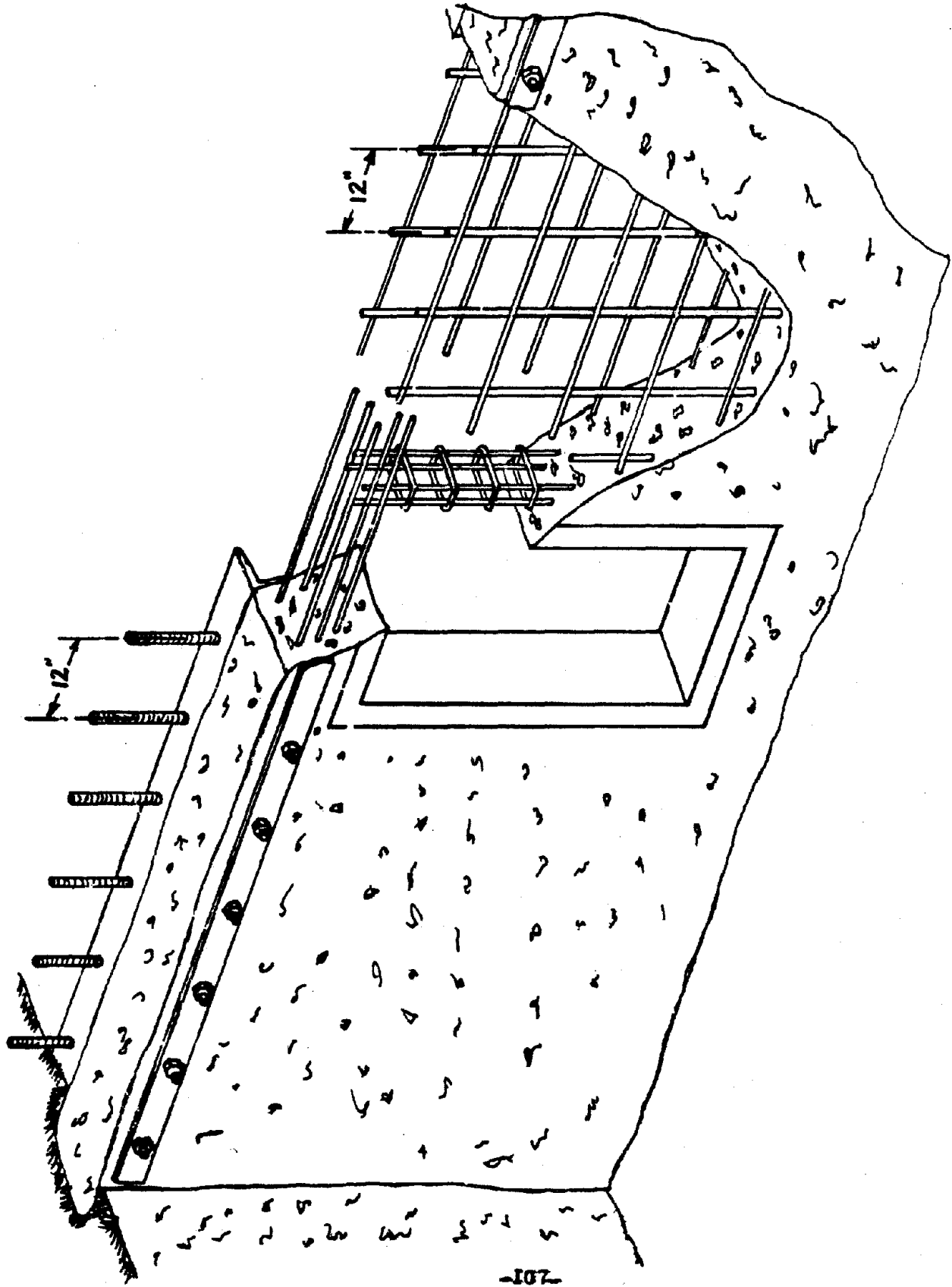
The reinforced concrete bulkhead is designed for a 6' high passage. It has an overall thickness of 16", and uses no. 6 rebars vertically and no. 5 rebars horizontally, on 12" centers. The concrete is recessed into a trench at the floor, and special anchorage provisions have been made at the roof. This has been done so that the center portion of the bulkhead will span the short, vertical dimension, and thus minimize the concrete thickness and the area of steel required. Extra reinforcement has been placed around the door opening to account for the loss of continuity.

The support system at the roof consists of a 6 x 4 x 7/8 backup angle bolted to the roof at one foot intervals. This provides a reaction to the incident pressure. Bolts running through the concrete support a strip on the front face which provides reaction to rebound loads. Additional support at the vertical sides is obtained by recessing the concrete about 6" into the coal.

An angle frame is set into the concrete to act as a mount for the door.

The following assembly procedure is used in the construction of the reinforced concrete bulkhead.

1. Cut recesses into ribs and floor, and scale loose material from roof.



Reinforced Concrete Bulkhead

Figure 38

2. Drill roof bolt holes, and install rear angle brace.
3. Construct complete form work.
4. Install reinforcement, door tube, and top front retainer strip.
5. Mix and pour approximately 6 yards of concrete.
6. Remove forms and grout top.

The principal problem with the reinforced concrete bulkhead is the mixing and placing of concrete. If all materials were delivered to the bulkhead site, using an electric mixer, it would probably take about one man hour for 6 cu. ft, or approximately 4 man shifts to mix and pour the 6 yards of concrete required. This time span would result in partial curing during the pour, and hence result in construction joints not favorable to the overall design.

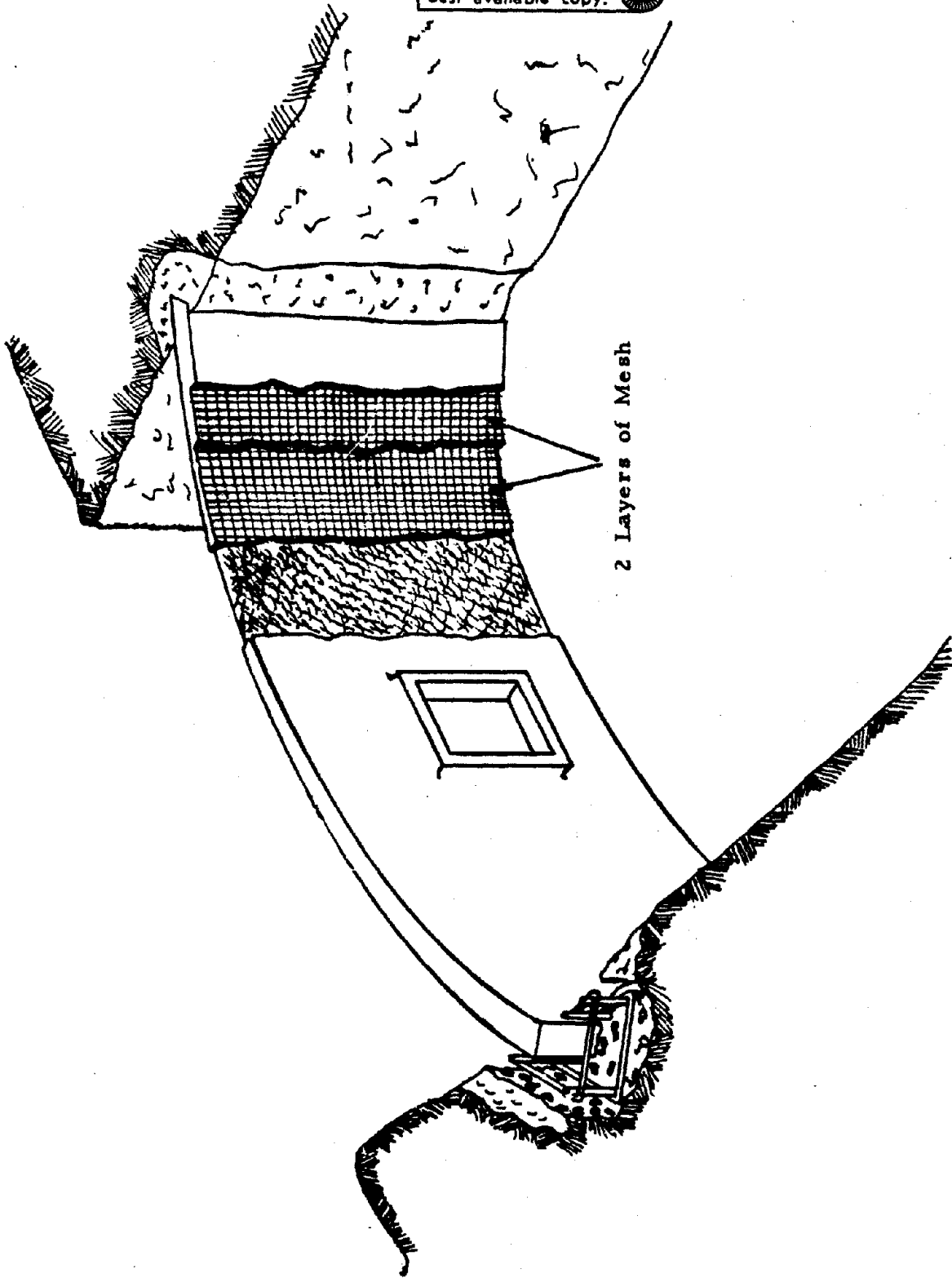
A more efficient approach would be the use of a continuous mixing and pumping system. The dry materials could be dropped off at the nearest convenient point to the bulkhead site. A continuous pug mill would mix and deliver the mixed concrete into a concrete pump, which could pump the mixed concrete several hundred feet to the bulkhead location.

This type of mixing and pumping equipment is not typically found in a mine, and hence would involve a capital outlay by the mine company.

4.7.2 Shotcrete Arch

In an attempt to seek a permanent bulkhead design requiring a minimum amount of material, we have designed the arch structure shown in Figure 39. This arch would be constructed

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2 Layers of Mesh

Shotcrete Arch
Figure 39

using shotcrete. This would minimize the amount of foam work which would have to be constructed, as well as the construction time.

The design calculations are presented in Appendix C. Qualitatively, the arch is similar in design to the liner plate arch described in Section 6 of this report. The radius is 17' and, for a 20' passage width the opening angle is about 70°. The concrete is 9" thick, with two layers of high strength steel wire mesh reinforcing, each having a cross sectional area of 0.5 sq in/ft.

The arch is reacted at the coal ribs by two reinforced distribution beams, which spread the load out through recesses into the coal ribs in such a fashion that it can be supported by the in-situ coal. These beams are poured concrete, rather than shotcrete.

The arch is constructed in the following manner:

1. Clean off loose material on the roof and floor in the neighborhood of the arch location.
2. Cut recesses into the coal ribs.
3. Place reinforcement and pour distribution beam.
4. Scribe outline of arch on floor and roof.
5. Place extensible vertical bars (2 rebars with a clamp) every foot along the length of the arch and clamp each into place.
6. Attach flexible lath form work to outside of vertical bars.

7. Attach one layer of wire mesh to inside of vertical bars, and insert a form for the door opening.
8. Spray on one 3" layer of shotcrete. Allow 3 hours and then apply a second 3" layer.
9. Layout second layer of reinforcing mesh.
10. Apply final 3" layer of shotcrete.

The shotcrete bulkhead will require a shotcrete application unit and a compressor. In addition, a small electric concrete mixer will be required to pour the approximately two yards of concrete required for the distribution beams. The entire operation can be carried out by two men.

The operation could be carried out by an outside contractor or by specially trained mine personnel. It may be desirable from the mine company's point of view to use their own people. If this is the case, the mine would have to purchase the shotcrete unit and train the personnel. These costs must somehow be considered in the total cost picture.

5. Cost Analysis of Alternative Bulkhead Concepts

Earlier it was suggested that the primary trade offs between permanent bulkheads and reusable bulkheads would be economic and logistic. Permanent bulkheads tend to be made from cheaper materials, but they frequently involve larger materials handling costs and a large capital investment in equipment. Reusable bulkheads, although more expensive from the materials point of view, are more conveniently handled and their construction would tend not to disrupt the normal mining cycle. The number of reuses also decreases the material investment.

In this section we shall quantify the economic tradeoffs between alternative bulkhead concepts. We have selected six bulkhead designs - three which are basically monolithic, non-reusable structures ("permanent") and three which are modular, assembleable("reusable") structures. These concepts have been chosen from the reviews presented in Sections 3 and 4. The six concepts are listed below.

Bulkhead Concepts for Cost Analysis

<u>"reusable"</u>	<u>"permanent"</u>
Liner Plate Arch	Fly ash-cement
Truss	Reinforced Concrete
Channel-Turnbuckle	Shotcrete

We would like to point out that many of the figures presented in the section are preliminary estimates based on engineering experience. Much of the labor involved, for example, is not standard procedure in normal mine work. In the final analysis, field trial will be necessary to verify these figures.

5.1 Basic Data

In order to facilitate the cost analysis, we have summarized some of the basic cost variables below.

5.1.1 Labor Cost

Table 4 shows a typical breakdown for mine labor markup. For convenience in our subsequent calculation, we shall use a nominal value of \$100/man shift. We realize that due to contractual changes over the past year, this value may be somewhat higher.

5.1.2 Unit Operations

An examination of the proposed bulkhead concepts indicates that there are certain construction operations which are common to more than one. For uniformity in our cost estimates, we have estimated the labor requirements for these typical construction operations. These estimates, and subsequent labor estimates, are based on the best estimates of several experienced mining engineers. They are listed in Table 5.

In our cost evaluations we shall consider bulkhead installations in both "high" and "low" coal. Although these are general terms, we can arbitrarily set a dividing line at about 48". The labor figures just presented apply to high coal. In general, all labor operations will be slower and more difficult in low coal. For the purpose of cost estimating, we have prepared Table 6 showing approximate labor multiplicative factors for a variety of tasks.

5.1.3 Materials Handling

This includes delivery from the surface to the bulkhead site of structural components and materials in bulk and bag. For either rubber tired, rubber-rail, or rail equipment, we can assume the following for one trip:

Table 4

Estimated Labor Cost

<u>Item</u>	<u>Percentage of Daily Wage</u>
Unemployment	3
Workmans Compensation	4
FICA	6
Black Lung	5
Overtime	5
Portal and Lunch	20
Miscellaneous (Vacation, Holidays, etc.)	<u>7</u>
Subtotal No. 1 (Fringes and Payroll Taxes)	50
Base Rate	<u>100</u>
Subtotal No. 2	150
Supervision 20% of Subtotal No. 2	30
Supplies and Equipment Maintenance at 50% of Base Rate	<u>50</u>
Total	230

or 2.3 x Base Rate

Classification Grade Base Rate as 11-12-73 x 2.3

Bratticeman No. 2	42.75	98.32/shift
Kersey Operator No. 3	43.25	99.48/shift

Table 5

Unit Operations

	<u>Man Shifts</u>
1. Bring basic equipment to the site. Jackhammer Compressor Hoses Small Cement Mixer	0.5
2. Smooth and Scale Passage Walls	0.2
3. Cut recess in rib with jackhammer 1' deep by 1-1/2' wide x 6-1/2' high	0.75
4. Cut trench in floor with jackhammer, pick, and shovel 8" deep by 1-1/2' wide by 18' long 12" deep by 1-1/2' wide by 18' long	2.0 4.0
5. Install 20 1-1/4" of resin grouted roof bolts	0.5
6. Mix and pour one cubic yard of concrete with small (2 cu. ft.) mixer	0.75
7. Construct a Double Walled Form with Door Portal	4.0

Table 6

Low Coal Factors

<u>Item</u>	<u>Time Factor</u>
Equipment Setup	1.5
*Smooth and Scale Top (25' instead of 18')	2.0
Smooth and Scale 4 Sides	1.5
*Trench Ribs (4' instead of 6')	1.0
*Trench Bottom (25' instead of 18')	2.0
Build Forms	1.5
Mix and Pour Cement	1.5
Install Bulkheads	1.5
Remove Forms and Equipment	1.5
Drill and Install Roof and Floor Bolts	2.0
Material Handling	2.0

*We have assumed typical "high" coal cross sectional dimensions of 6' x 18', and typical low coal dimensions of 4' x 25'.

Load	0.75 Man Hour
Haul In	0.80 Man Hour
Unload	0.75 Man Hour
Return Outside	<u>0.80</u> Man Hour
	3.10 Man Hour/Round Trip

A typical rubber-rail trailer 10' long by 5' wide with 10" side boards carries a recommended load of 3 tons. Considering an 8 hour shift and a cost of \$100/shift, the material handling cost is computed as

$$\frac{\text{cost}}{\text{ton}} = \frac{3.10}{8} \times 100 \times \frac{1}{3} = \$13$$

5.1.4 Materials Cost

Estimated materials costs delivered to the mine site are summarized in Table 7. Certain items require further clarification.

Concrete - Typical concrete applications in a mine use bagged premixed material (sometimes referred to as "Sakrete"). This tends to be more expensive, but is much more manageable than the delivery of bulk sand, cement, and gravel underground.

Urethane Foam - The most simple and convenient vehicle for the application of urethane foam in a mine is the disposable foam pack ("Froth Pack", or "Rigi-Pak" are among the trade names). These come equipped with throw away plastic nozzles, and require little operator skill.

Rockbolts - These costs are estimated from current prices for Dupont "Fastloc" full column resin grouted bolts.

Table 7

Materials Costs

Concrete (premix bags) 1 bag = 1 cu. ft.	\$2.00/cu. ft.
Fly ash (delivered in bulk)	\$7.00/ton
Urethane Foam Back (1 pack = 26 lbs)	\$138/pack
Rockbolts and Resin 1-1/2' long, 1-1/8"	\$4/bolt
Aluminum Flashing (16 ga.)	\$0.30/ft
Straight Rebar	\$0.25/lb
Bent Rebar	\$0.35/lb
Forms	
Door Frame	\$250
Door	\$300
Fabricated Steel	\$1.50/lb
Reusable Bulkhead Components	
Channel-Turnbuckle	\$5,200
Truss	\$4,500
Liner Plate Arch	\$3,000
Shotcrete	\$0.70/board ft. applied

Door - We have considered an off-the-shelf commercially available pressure-resistant door. This particular one is a 4-dog scuttle produced by Railways Specialties Corp.

Reusable Bulkhead Materials - Cost of components for reusable bulkheads have been obtained from quotations from potential fabricators. These quotations are based on the final design package that resulted from this program, and are based on manufacturing in lots of 100.

5.1.5 Equipment Costs

The bulkhead designs have attempted to minimize the use of special equipment wherever possible. We must recognize, however, that equipment which is standard for one mine may be special for another. In evaluating the cost impact of installing emergency shelter bulkheads, the capital outlay, rental costs, or logistic problems involved in using special equipment must be considered.

In Table 8 we have summarized the cost of "special" equipment which has been referred to in the construction procedures for the proposed bulkheads. The cost of repair and maintenance of this equipment should also be considered.

5.2 Bulkhead Cost Summaries

In this section we present cost summaries for each of the six proposed bulkhead concepts. The data used in these summaries is drawn on the information presented in the previous section. The following information will help to clarify the logic behind the cost summaries.

5.2.1 Materials

Materials have been identified as either reusable or expendable .

Table 8

Special Equipment Costs

Slurry Rockduster	\$8,000
Pneumatic Rock Dust Distributor	10,000
Concrete Pump and Hoses	\$9,000
Continuous Concrete Mixer	6,000
Compressor (200 cfm, 100 psi)	\$6,000
Jackhammer	\$1,000
Jackdrill and Feed Leg	\$2,500
Shotcrete Spray Unit	\$7,000
Small Electric Concrete Mixer	\$ 300

Reusable Materials - These include all components which can be removed with minimal effort and carried to another location. They include doors, and structural components of reusable bulkheads. They do not include any structural elements which are cast into concrete. We have assumed that reusable components can be amortized over four uses. We feel that this is conservative, because even in the rugged mine environment, it would be difficult to damage the structural elements which comprise these designs.

Expendable Materials - These include grouted rock bolts, sealants, grout, grouted structural components, and cast in place structural materials.

We have, for the purpose of this cost analysis, assumed that material costs for a low coal and high coal bulkhead will be the same. This can be superficially assumed from the similarity in overall cross sectional area. In fact, this is not the case, since the designs will change with the dimension of span. Thus, vertically spanning structures are more economical weight-wise in low coal, while the opposite is true for horizontally spanning structures. We have chosen to use the high-coal material figures for the low coal designs, bearing in mind the differences which may be associated with this assumption.

5.2.2 Labor

Labor includes material handling and assembly labor. Assembly labor has been estimated for high coal, and low coal figures have been generated using the low coal labor conversion factors of Table 6.

5.2.3 Crew Size and Required Equipment

These do not factor directly into the bulkhead cost, but they do reflect the logistic and equipment requirements for the

implimentation of a given concept. Logistically, a small crew (e.g. 2 persons) and simple equipment will minimize the disruption to normal operations. In addition, as mentioned before, some of the equipment mentioned will not be found in every mine - this either means that the equipment must be purchased (an implied cost) or the concept cannot be implemented.

Tables 9 to 14 summarize the cost for each of the six bulkheads, and Table 15 summarizes all of the resulting costs.

5.3 Conclusions

The following conclusions can be drawn from these cost summaries.

1. The reusable and permanent concepts are competitive from a total cost point of view.
2. There are equipment cost and/or availability considerations which favor the reusable concepts.
3. Logistic considerations favor the reusable concepts. Materials can be dropped off one day and assembled when convenient. Continuity of assembly labor is not important.

The remainder of this report is devoted to the development of detailed designs for each of the three reusable concepts.

Table 9

Cost Summary - Fly Ash Cement Bulkhead

		<u>Materials</u>	
Reusable			
Forms	\$70	(2 uses)	
Door	75		
Expendible			
Fly Ash - 17 tons	119		
Cement - 41 bags	82		
Culvert and Door Frame Assembly	<u>250</u>		
Subtotal	\$596		
		<u>Labor</u>	
<u>Material Handling</u>		<u>High Coal</u>	<u>Low Coal</u>
Bulk Material - 20 tons at \$13		\$260	\$520
<u>Other Labor</u>		<u>Man Shifts</u>	
		<u>High Coal</u>	<u>Low Coal</u>
Bring in 2 slurry rock dusters, Lay and connect transport hoses, waterlines.		2.0	4.0
Smooth and scale walls		0.2	0.3
Build forms, insert door frame, culvert, and reinforcement		4.0	6.0
Mix and Pump Slurry		4.0	6.0
Remove forms and equipment		<u>2.0</u>	<u>4.0</u>
Total Man Shifts		12.2	20.3
Subtotal Dollars		\$1,220	\$2,030
Total Bulkhead Cost		\$2,076	\$3,146
Crew Size Required	4 men		
Special Equipment Required			
2 MSA Slurry Rockdusters, modified for 2 component mixing			
1 6 ton pneumatic rock dust distributor			

Table 10

Cost Summary - Reinforced Concrete Bulkhead

Materials

Reusable

Forms	\$70	(2 uses)
Door	75	
Roof Angle	114	

Expendable

Rebar	160
Door Frame	250
Concrete - 6 yds	324
Roof through Bolts	20
Roof Bolts	72
Subtotal	\$1,085

Labor

Material Handling

Approximately 13 tons

High Coal

\$169

Low Coal

\$338

Other Labor

Man Shifts

Bring in mixer and water line	0.5	0.75
Cut Recesses, ribs	1.5	1.5
Floor	2.0	4.0
Drill and Install 18 roof bolts and angle brace	1.0	2.0
Build Forms	4.0	6.0
Install Reinforcement, door frame, etc.	1.0	1.5
Pour Concrete	4.5	6.75
Remove forms, grout top, install door	<u>1.0</u>	<u>1.5</u>
Total Man Shifts	15.5	24
Subtotal Dollars	\$1,550	\$2,400
Total Cost	\$2,813	\$3,823

Crew Size - 2 men

Special Equipment - Cement Mixer
Roof Bolter or Jack Drill

Table 11

Cost Summary - Shotcrete Bulkhead*

Materials

Reusable

Door \$ 75

Expendable

Shotcrete (in place) 1,200 board ft. 915

Abutment Beams (2 yds concrete) 108

Wire Mesh at \$.50/lb 124

Rebar 116

Metal Lath 36

Door Frame 250

Materials Subtotal \$1,624

Labor

Materials Handling

8 tons \$105

Assembly Labor - Man Shifts

Bring Equipment to Site 0.50

Cut Recesses in Ribs 1.50

Place Reinforcement in Abutment .25

Form and Pour Abutment 2.00

Layout Shotcrete Form and Reinforcement,
Spray Foam 2.00

Total Labor \$625

Labor Subtotal \$730

Total Cost \$2,354

Crew Requirements - 2 men - experienced in Shotcrete

Equipment - Shotcrete Machine, Compressor, Jackhammer

*We do not believe that shotcrete equipment can be handled in low coal. Consequently we have omitted this application.

Table 12

Cost Summary - Liner Plate Arch

Materials

Reusable

Main Bulkhead Assembly	\$750
Door	75

Expendable

Reinforcing Steel (300 lbs)	93
Concrete (2 yds)	108
End Posts	150
Aluminum Flashing (36')	11
Foam Pack	<u>138</u>
Material Subtotal	\$1,250

Labor

	<u>High Coal</u>	<u>Low Coal</u> *
Assembly Labor (Man-Shifts)		
Deliver and Set Up Equipment	\$ 0.75	\$ 1.13
Cut Ribs	1.00	1.00
Trim Floor and Roof	0.50	0.75
Assemble Bulkhead	4.00	6.00
Pour Concrete	1.50	2.25
Final Sealing	<u>1.00</u>	<u>1.50</u>
Total	8.75	12.63
Assembly Labor Cost Subtotal	875.00	1,263.00
Materials Handling		
Bulkhead (2 tons)		
Concrete (4 tons)	<u>60.00</u>	<u>120.00</u>
Total Cost	\$2,185.00	\$2,633.00

Crew Size Required - 2

Equipment - Jackhammer, Cement Mixer

* These low coal estimates assume the high coal design components; i.e., a passage width of 18'. For a 25' low coal passage, the components would be more massive and unmanageable in low coal.

Table 13

Cost Summary - Channel Turnbuckle Bulkhead

Materials

Reusable		
Main Assembly		\$1,184
Door		75
Expendable		
Footing Boxes		665
30 Roof Bolts		120
50' Flashing		15
Foam Pack		138
8 cu. ft. grout		<u>16</u>
Materials Subtotal		\$2,213

Labor (Man Shifts)

	<u>High Coal</u>	<u>Low Coal</u>
Deliver and Setup Materials and Equipment	0.5	0.75
Layout and Drill Holes	0.5	1.0
Cut Floor Trench, Trim Ribs	2.0	4.0
Install Roof and Floor Support Systems	1.0	2.0
Assemble Bulkhead	2.0	3.0
Final Sealing (Foam, Flashing, Grout, Sealant)	<u>1.0</u>	<u>1.5</u>
Total Manhours	7.0	13.0
Subtotal Labor Cost	\$700	\$1,225
Total Cost	2,913	3,438
Crew Size Required - 2 men		
Equipment - Jackhammer		
Jackleg and Drill		
Compressor		

Table 14

Cost Summary - Truss Bulkhead

Materials

Reusable	
Main Assembly	\$940
Door	75
Expendible	
14 Footing Boxes	720
Foam Pack	138
50' Flashing	15
16 cu. ft. grout	<u>32</u>
Subtotal	\$1,920

Labor (Man Shifts)

	<u>High Coal</u>	<u>Low Coal</u>
Equipment and Material, Delivery and Set Up	0.5	0.75
Dig 2 Trenches, Trim Ribs	4.0	8.0
Install End Posts and Footing Boxes	0.5	0.75
Assemble Bulkhead Wall	4.0	6.0
Final Sealing (Foam, Flashing, Grout Boxes)	<u>1.5</u>	<u>2.25</u>
Total Man Shifts	10.5	17.75
Subtotal Labor Cost	\$1,050	\$1,775
Total Cost	\$2,970	\$3,695

Crew Size - 2 Men

Special Equipment - Jackhammer, Compressor

Table 15

Summary of Bulkhead Costs for a Temporary Shelter

<u>Bulkhead Type</u>	<u>High Coal</u>	<u>Low Coal</u>
Fly Ash Cement	\$2,070	\$3,146
Reinforced Concrete	2,813	3,823
Shotcrete Arch	2,354	-
Liner Plate Arch	2,185	2,633
Channel - Turnbuckle	2,913	3,438
Truss	2,970	3,695

6. Design Description of Three Reusable Bulkhead Concepts

In this section we shall present detailed design descriptions of the three reusable concepts which evolved from conceptual study of Section 3. The reasoning behind the selection of three concepts relates to anchorage. The variability and unpredictability of rock and coal conditions in any given mine precludes the development of one universal design.

The three concepts which are developed here each represent a different anchorage philosophy, and each would be most favorably applied under a different set of strata conditions. Thus, the three designs attempt to cover a broad spectrum of strata conditions.

The three designs are:

a. Liner Plate Arch - An arched shaped structure spanning the passage width, and made up of standard tunnel liner plates and reinforcing ribs.

b. Truss - A structure supported by a series of vertical trusses, made up of corrugated sheet, tubular rails, and tubular posts and diagonals.

c. Channel-Turnbuckle - A series of vertical aluminum channels, bolted side by side and supported at the roof by adjustable turnbuckle rods and roof bolts and at the floor by footing boxes in a trench.

The three designs are described in detail along with the design rationale. Design Calculations and construction procedures are presented in Appendices C and D respectively.

The principle design requirement for each bulkhead is that it withstand an instantaneously applied, long duration, pressure pulse of

20 psig, and a negative static pressure of 5 psig. Design calculations presented in the Appendix show that assumptions of 400 psi coal bearing strength, 160 psi floor shear strength, and 3,750 psi roof compressive strengths are required for the viability of the different anchorage systems. Although these are conservative values based on available test data, the actual range of strengths found in nature is not well established. Thus, to gain confidence that the emergency shelter bulkheads will be supported in a wide range of actual materials, testing must be conducted to document the variation of strengths that can be expected under various coal and rock conditions. For this reason, an in-situ test program is recommended to obtain this information.

6.1 Bulkhead Access

During the design effort, investigation was made into the incorporation of a commercially available blast resistant door. We felt that this would both simplify the design and reduce costs.

A survey of commercially available blast doors indicated that most of them were expensive (\$2,000 - \$6,000) and/or heavy (300 - 1,000 lbs). This survey included a set of existing designs prepared for the army for application to nuclear blast shelters.

One door, produced by Railways Specialties Corporation, turned out to be the ideal candidate. This door is 25" in diameter, and weighs 105 lbs. It costs \$300. The door is described as a 4-dog scuttle. The dogs lock the door in place, and are centrally operated by a handwheel. The door is designed for a nominal pressure of 35 psi, which means that it has a proof load of 70 psi. We consider it to be adequate for our purposes, and we have adapted it to the three bulkhead designs.

A layout drawing of the door design is shown in Appendix D.

6.2 Summary of Design Specifications

The three designs have been sized to meet the following specifications:

Height - 5.5 - 6.5 ft, adjustable

Width - Approximately 18'

Pressure Loading - 20 psi positive, applied instantaneously
5 psi negative, applied statically

Leakage - Nominally leak proof. Seals must survive explosion environment

Convergence - Structures are to withstand 2" of roof sag and 2" of floor heave

Assembly Crew - no more than 2 persons.

On Site Operations - No cutting or welding. Sheet metal drilling acceptable.

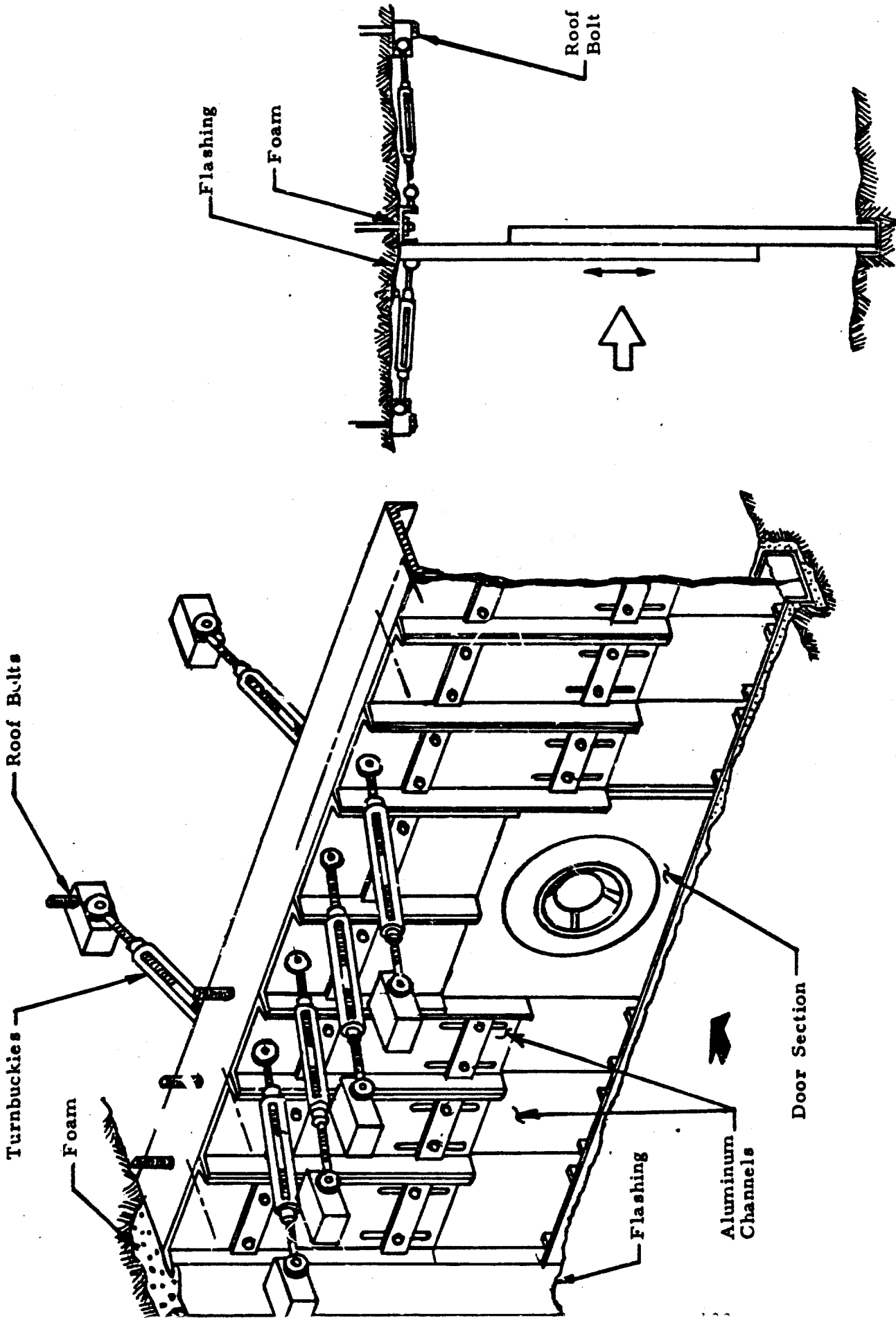
Maximum Component Weight - 150 lbs.

6.3 Design Description and Configuration Rationale

6.3.1 Channel-Turnbuckle Bulkhead

6.3.1.1 Design Description

The channel-turnbuckle bulkhead is designed to be installed in passageways where both the roof and the floor are considered to be structurally sound. The bulkhead is shown isometrically in Figure 40, and a design layout is presented in Appendix D. The bulkhead can be installed in passageways up to 18 ft wide and 5-1/2 - 6-1/2 ft high. The assembly is formed by a series of 1 ft wide facing channel pairs which are located vertically across the full passageway width. The height of each individual facing channel pair is adjustable over a \pm 6 inch range. Each facing section is attached at its base to a footing box which is set in concrete in a floor trench.



Channel-Turnbuckle Structure

Figure 40

At the roof the facing section is supported by a series of butting lateral roof channels which are attached to the roof by roof bolts. Primary structural support for the roof channels, and therefore the top of each facing channel, is provided by roof bolt anchored turnbuckles, located fore and aft of the roof channel. Each facing section is structurally connected to its neighbours by bolts through the butting flanges and also by the interconnecting roof channels and footing boxes.

The door assembly is 3 ft wide and is equivalent to three facing channel pairs. It comprises a 3 ft wide plate structurally stiffened by two edge located vertical channels and topped by three adjustable height, facing channels. The door is bolted to the lower plate and provides a 25 inch diameter entrance. The door latch is operable from both sides of the closed door by a 13 inch diameter handwheel requiring six to eight full turns to operate.

The bulkhead is sealed to the passageway by use of aluminum flashing supported by frothed urethane foam.

6.3.1.2 Configuration Rationale

The intent of this design is to support the bulkhead in those applications where the roof and the floor are regarded as structurally sound elements. Full column resin bolts loaded in shear have been selected for the roof structural attachment. Roof bolting equipment and operators are readily available in most U.S. coal mines. The roof bolts are structurally connected to the bulkhead by adjustable length turnbuckles. These provide good positional tolerance for the lateral alignment of the bulkhead. A similar arrangement of bolts and turnbuckles was initially considered for the floor attachment but this was rejected since drilling and bolting into the floor may cause difficulty and is less reliable. A lateral continuous trench has been selected. The trench runs the full width of the passageway and is approximately 9 inches deep x 1 ft wide. The cutting of such a trench in the passageway floor using a jackhammer is considered a relatively simple task.

To minimize unit part weights the bulkhead has been designed into individual 1 ft wide sections. Each section is made up of two 1 ft wide aluminum channels bolted back to back. The channels, each weighing approximately 60 lbs, are bolted together with a large overlap region. The overlap allows for large slots in each channel which allow vertical height adjustment without compromising the pressure seal. The overlap region also doubles the channel assembly stiffness at its centre where the maximum bending moment occurs under pressure loading.

Each facing channel assembly is bolted at its base to a footing box. Two lengths of footing boxes are provided - six 3 ft long boxes and two 1 ft long. These allow varying width passageway bulkheads. Each steel footing box has a bearing plate to which the corresponding facing channel is bolted. Under positive pressure loading half of the total pressure load is transmitted by the footing boxes to the rear face of the lateral trench where it is reacted by the floor. Under the less severe negative pressure loading the footing box is designed to deform beneath the facing channel until the facing channel bears on the two forward footing box webs. The load is then transferred to, and reacted by, the front face of the floor trench. The footing box which is set in concrete is not reusable.

Upper facing channel support is provided by the turnbuckle supported roof channel. The roof channels are in 3 ft and 1 ft lengths and are each held against the roof by a roof bolt. Large roof bolt clearance holes in the channels allow for alignment of the roof channels, and splicing plates bolted to the rear channel flanges make the roof channels structurally continuous. Each facing channel is attached to the forward flange of the roof channel by an eyebolt. To each eyebolt is attached a turnbuckle with a 12 inch adjustment range and at the forward end of the turnbuckle is a roofbolt which comes with a turnbuckle mount fitting. Attached to the rear flange of each roof channel is a similar eyebolt, turnbuckle and roofbolt arrangement. The forward turnbuckles, spaced 1 ft apart, transfer the positive pressure

loading to the roof while the rear turnbuckles, spaced 3 ft apart, carry the negative pressure loading. The forward flange of each of the 3 ft long roof channels is in contact with four facing channels and is attached by eyebolts to three of them. This structural arrangement provides adequate load paths to ensure that should an individual roof bolt fail, the load is transferred to the two adjacent roof bolts.

To minimize the effects of galvanic corrosion, which could occur should the bulkhead be installed in a damp acidic environment, the following precautions have been taken. Corrosion resistant 6061 aluminum alloy is used throughout. The aluminum plates and channels and the steel footing boxes are coated with zinc chromate primer. All steel hardware that interfaces with the primary aluminum structure is galvanized. Sacrificial aluminum plates are used between the footing boxes and the facing channels and aluminum bearing plates are used at the facing channel pair junctions.

The bulkhead structure is sealed to the passageway by aluminum flashing supported by frothed urethane foam. At the roof interface the flashing is clamped between the facing channel and the roof channel, is then tucked between the roof and the roof channel and then led forward of the bulkhead and attached to the roof by spads. Foam is applied to the back of the roof channel and, where clearance exists between the channel and the roof, the foam travels forward until it contacts the flashing. The foam supports the flashing and fills the void. The tuck in the flashing allows the seal integrity to be maintained for bulkhead deflections of up to 2 in. Under maximum pressure loading a deflection of one inch is anticipated in this region.

A similar foam backed seal arrangement is used at the bulkhead sides. The flashing is clamped between the two facing channels in the channel overlap region and between the channel and bolted clamping plates at the top and the bottom. The flashing is then dog-legged forward and attached to the wall with spads.

The entire area bounded by the facing channels, the flashing and the indented wall is then filled over the full bulkhead height with frothed urethane foam. The deflection capability of this seal is 3 in. and the anticipated maximum deflection is 1-1/2 in.

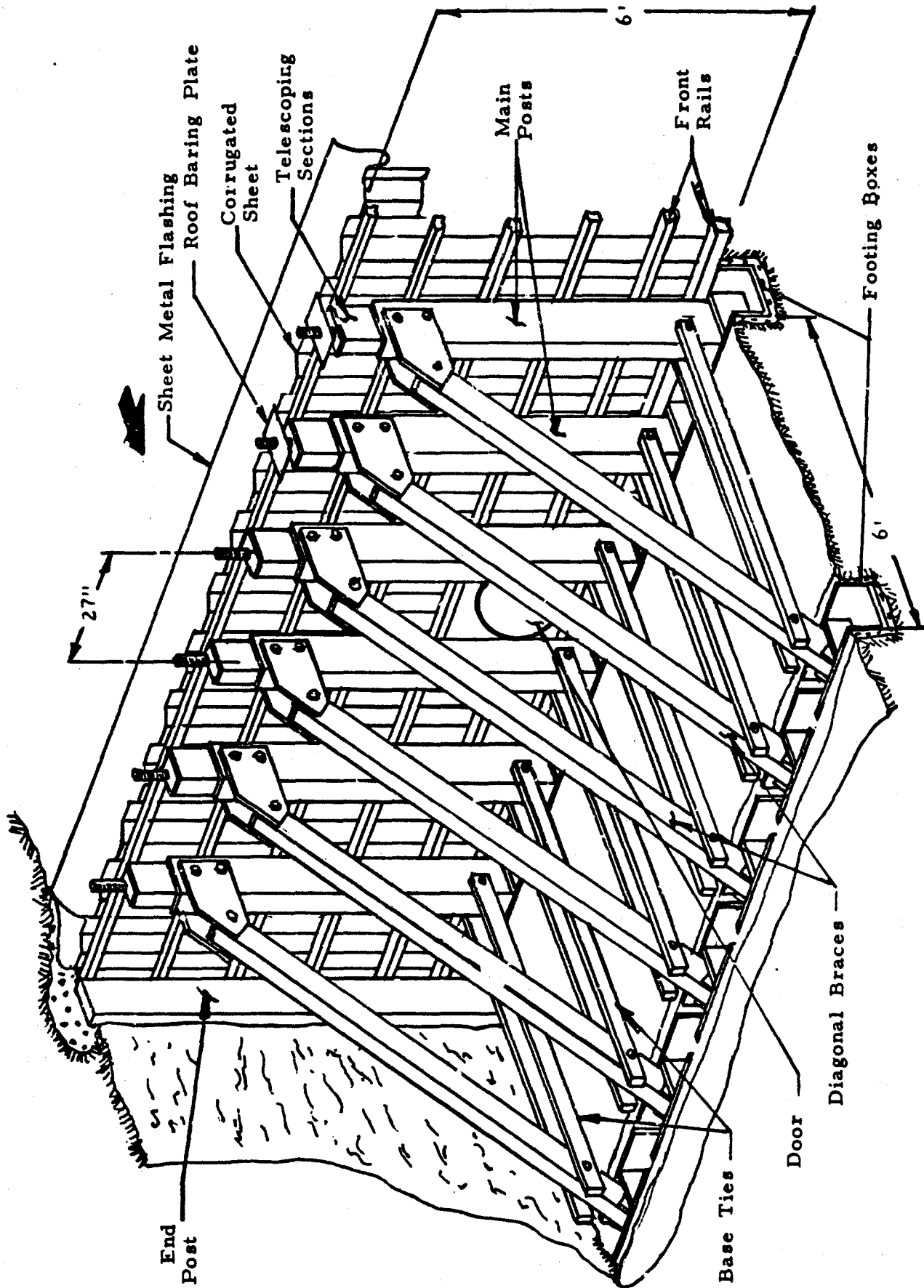
At the bulkhead floor interface the assembly is essentially sealed by the concrete set footing boxes. Minor leakage paths both in this area and for the entire bulkhead are sealed with Thikol Sealant.

6.3.2 Truss Bulkhead

6.3.2.1 Design Description

The truss bulkhead shown isometrically in Figure 41, is designed to be installed in passageways where the floor is considered to be the most structurally sound element of support. The bulkhead can be installed in passageways up to 18 ft. wide and 5-1/2 to 6-1/2 ft. high. The assembly comprises a series of adjustable height posts, equally spaced across the passage width, which support vertical sheets of corrugated steel decking. Each post assembly includes a trussed support and the post and the support base are anchored to the floor in two lateral trenches into which concrete footing boxes are set. Each post assembly includes a pinned slide which allows for a \pm 6 inch height adjustment. At the top of the slide is a jacking stud which is jacked into a locating hole in the passage roof. A corresponding height adjustment is available in the decking and is achieved by adjusting the overlap between two decking sheets. Gross width adjustment is achieved by selecting the appropriate number of trussed post and decking assemblies.

The decking is supported between the posts by square tubing ribs. These ribs are integral with the decking sheets and these decking assemblies are bolted directly to the posts.



Truss Structure

Figure 41

The post and truss strut (diagonal) are each pinned to an individual footing box; the strut is pinned to the upper portion of the post and the two are tied together at the bottom by two horizontal tie bars. Integral truss assemblies are spaced 27 inches apart across the passageway. The decking sheets adjacent to the two walls are supported at the wall by untrussed, adjustable height wall posts.

The door assembly is placed between two of the centre passageway posts and comprises a flat steel plate to which the door is attached. The door plate attaches directly to the posts. The door provides a 25 inch diameter entrance located approximately 2-1/2 ft above the passageway floor.

The bulkhead structure is sealed to the passageway by aluminum flashing supported by frothed urethane foam.

6.3.2.2 Configuration Rationale

The primary load support for the truss bulkhead is the passageway floor. Two lateral trenches 15 in. wide x 9 in. deep spaced 6 ft apart are used to provide this support. Steel footing boxes are placed in each of the trenches and set in place with concrete. The footing boxes and the trenches are sized such that under maximum pressure loading the bearing capability of a single trench is sufficient to withstand the full load. The footing boxes support the main bulkhead structure which comprises a series of trussed posts spaced 27 inches apart across the passage width. The base of each post is pinned to a footing box in the front trench and the base of the truss strut which supports the post is pinned to a second identical footing box in the rear trench. The bases of the post and truss are structurally connected by two tie bars which are sized to transfer the forward footing box load to the rear footing box in the event of a forward trench failure.

Positive pressure loading induces a moment about the rear footing box. This moment is reacted by the passageway roof at the post stud. A predrilled hole in the passage roof is used to locate the post stud. The stud is slid into this hole during erection to a depth where the post bearing plate contacts the roof. The post slide is pinned at this extended portion and the bearing plate is jacked into firm contact with the roof by turning the jacking nut.

The supporting truss assemblies are faced with corrugated high strength structural steel roof decking. The decking, which is stiff in the vertical plane, is supported on 13-1/2 in. centers by horizontally positioned square tubing ribs. These ribs are welded to the decking and bolted to the truss supports. Each bulkhead bay is faced by two overlapping decking assemblies, the degree of overlap being a function of the required height of the bulkhead.

The truss bulkhead structure is sealed to the passageway with aluminum flashing structurally supported by frothed urethane foam. At the passage walls the flashing is attached to the decking edges with self tapping screws and a bearing strip. The flashing is dog-legged forward and is attached to the passage wall with spads. The full height volume bounded by the wall and the flashing is then filled with frothed urethane foam. At both the top and the bottom of the bulkhead the flashing is attached to the decking with self-tapping screws and bearing strips. The corrugations in the decking are filled with a contoured strip of neoprene and the screws pass through the neoprene and into the decking and the ribs. The flashing is routed from the attachment towards the bulkhead centre then away from centre to attach to the roof/floor with spads. The enclosed volume is then filled with the frothed urethane foam. The seal at the bulkhead sides will accommodate a 3 in. deflection which is twice the maximum anticipated deflection of 1-1/2 inch. Similarly the top and bottom seals are designed for 2 in. deflections compared to a maximum load deflection of 1 in.

6.3.3 Liner Plate Arch Bulkhead

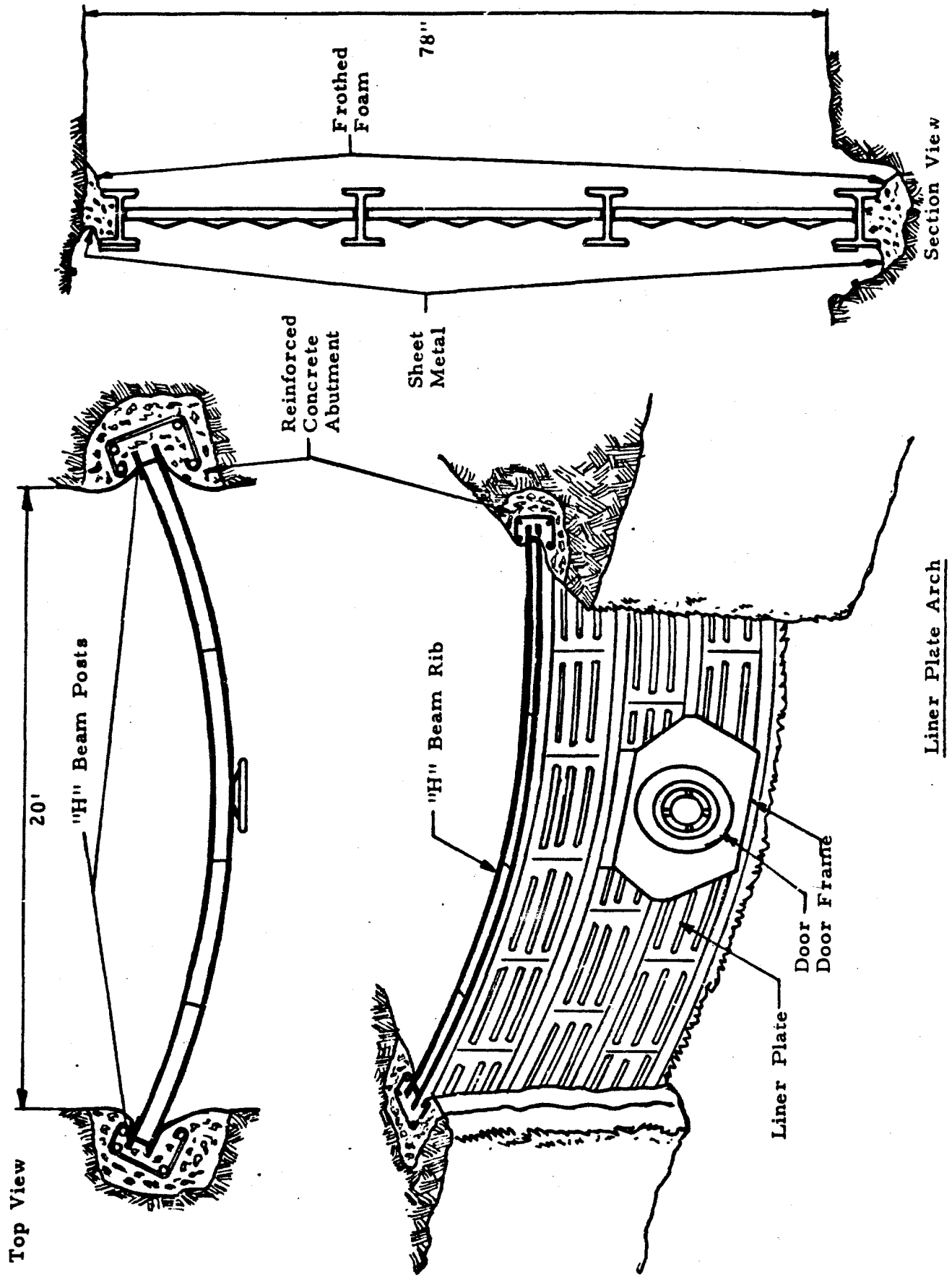
6.3.3.1 Design Description

The arch bulkhead, shown in Figure 42, is designed to be installed in passageways where the ribs are judged to be structurally sound. The bulkhead can be installed in passageways up to 19 ft. wide and 5-1/2 to 6-1/2 ft. high. The assembly comprises four equispaced horizontally radiused I-beam ribs which are anchored at their ends to vertical wall posts which are recessed into the passage walls. Between the four ribs are located three courses of radiused liner plates. Each liner plate, 2 ft. high and approximately 3 ft. long, is curved to a 17 ft. radius and is flanged all around. The plates are structurally linked horizontally by bolts through the butting flanges and vertically by bolts through the flanges and the sandwiched I-beam web.

Each rib comprises five I-beam sections which are butted at their ends and spliced together with splice plates attached to both forward and rear flanges. Holes are punched through the I-beam web centres for attachment of the liner plates.

The 25 in. diameter door is attached to the bulkhead at floor level between the first and third ribs. The second rib is kinked over the door to remain structurally continuous.

The two bulkhead wall posts are recessed into the passage wall and are permanently set into reinforced concrete. The concrete provides the pressure seal at the bulkhead sides. The seal at the top and bottom is formed by aluminum flashing which is clamped between the I-beam and the forward flange splice plates. The flashing is attached to the roof or floor and is backed by frothed urethane foam.



Liner Plate Arch

Figure 42

6.3.3.2 Configuration Rationale

The liner plate bulkhead has been designed to be of fixed height and width. Recesses must be cut into the passageway walls to both ensure adequate wall support and, where necessary, to provide the passage width adjustment. Passage height adjustment is achieved by cutting a trench across the passageway width. The liner plates and ribs used in this design are identical to those which would be used in constructing a 34 ft diameter tunnel except that the ribs have been made shorter to hold their individual part weight to 100 lb. Standard liner plates are used in conjunction with half-width liner plates. This combination allows for 'brickwall' type staggering of adjacent courses. A similar rib junction staggering pattern is achieved with the I-beam ribs by use of two differing lengths of rib.

The door assembly is essentially one liner plate in width, and occupies a significant portion of two liner plates in height. One rib is displaced vertically by approximately 13 in. by the door. Rib structural continuity is maintained however by use of two vertical door frame channels which connect the door top rib to the two ribs beneath it. This channel essentially provides rib web continuity. Flange continuity is provided by a curved door mounting plate on the front face of the bulkhead and by splice plates on the rear face. The displaced rib above the door is connected to the standard rib above it by one standard width, non-standard height liner plate. The door mount plate attaches to the floor rib, the two vertical door channels and the second rib over its region of displacement. The plate has an angle hoop welded into a central cut-out. This hoop provides the door mount face.

Each of the four rib assemblies are attached at their ends by splice plates to the two wall posts. These wall posts are surrounded by a steel rod cage and the whole is set into the passage wall recess with concrete.

The structure assembly is sealed at the sides by the concrete and at the top and bottom by aluminum flashing and foam. The flashing is attached to the top and bottom ribs by clamping it between the I-beam and the six splice plates. The flashing is then routed back behind the rib and is then bent forward to the roof/floor. The flashing is attached to the passageway with spads. During this operation the lower rib is held off the floor with suitable blocks. The region between the flashing, the rib and the roof/floor is filled with frothed urethane foam. This "cushion" of foam prevents structural damage during minor heave. The flashing is configured to maintain a seal under conditions where the bulkhead deflects 3 inches. It is anticipated that under maximum pressure loading the bulkhead will deform 2 inches at its centre.

6.4 Comparison of the Three Designs

6.4.1 Support Requirements

One principle feature which differs among the three designs is the manner of support. This feature becomes extremely important when one considers the character and variability of the rock and coal available for support. The principles of anchorage support have been discussed in Section 3.2.2, and will not be repeated here. Some of the important design consequences are discussed below.

The turnbuckle anchorage design is based on rock bolts supplying high shear loads, up to approximately 20,000 lbs/bolt. This requires that the roof be either massive sandstone or shale.

Should the roof consist of thinly layered shale, or drawslate, or should head-coal be present, the bolts will bend under the shear loading and their load capacity will be reduced to an unacceptable value. Here, the truss design can be applied. The major truss loads are carried into the floor, which is assumed to be hard

fireclay or shale. An additional measure of safety is obtained by recessing the floor anchors into trenches. Here the shear loads are applied to a larger bearing surface, and the required strength of the floor material is significantly less than that which would be required of "competent" roof.

Should the floor be considered inadequate (unavoidable wet soft bottom, for example) as well as the roof, support from the coal ribs can be obtained using the liner plate arch. The arch applies high lateral thrust into the coal abutments. Although coal is not a very sound material (i.e., it has cleavage planes, low crushing modulus, etc.), it exhibits its maximum resistance and minimum deformation when loaded in confined compression as it is here. Consequently, abutments have been designed to provide adequate support.

6.4.2 Influence of Seam Height

The designs presented herein are for a coal seam of height approximately 6'. Since most applications will be at this height or less, we will discuss the influence of reduced seam height.

The designs which span from roof to floor (truss, channel-turnbuckle) become more efficient at lower heights. The loads per unit width are reduced linearly with height, and hence the anchorage loads are reduced accordingly. The bending moments, and hence the maximum stresses in the vertical structural members, will reduce as the square of the height.

For the truss design, this means that the trusses can be spaced at greater distances, and the vertical posts and diagonals can be significantly reduced in cross-section. For the channel turnbuckle, the load on each roof and floor bolt will be reduced, and, hence, the rock strength requirements are relaxed. In addition, the section properties of the channels can be decreased.

The liner plate arch design cannot take advantage of lower seam thicknesses. In fact, since lower seams tend to have wider passages, the arch span will increase and ribs with greater section properties will be required.

6.5 Design Techniques and Allowable Limits

6.5.1 Blast Design

Structures to resist blast loadings have been studied extensively for the past 30 years.⁽⁹⁾ The most typical application has been for protective shelters from nuclear blast or accidental industrial explosions. These structures are typically monolithic reinforced concrete structures.

Blast loadings, in most cases are not continuous service loadings, but rather high intensity, short duration emergency loads. Once a structure had survived one, or multiple blasts, its life may be terminated. Consequently, the allowable stress and deformation limits are considerably relaxed from those for typical engineering structures. One such relaxation which greatly economizes blast resistant structural design is the allowance of plastic deformation. This technique will be discussed in Section 6.5.3.

6.5.2 Computation of Stresses and Deformation

Stresses and deformations in a structure subjected to an explosive loading are determined from a dynamic analysis of the structure. Such an analysis can be extremely complex, when one considers the many degrees of freedom to be considered and the time dependence of the loading. For design purposes, the analysis is simplified considerably by making certain assumptions about the dynamic behavior of the structure and its components, and about the coupling of the various degrees of freedom of the structure. The assumptions and techniques used herein are described below.

6.5.2.1 Idealization of the Structure

A continuous structure, such as a beam, has an infinite number of dynamic modes, and, hence, an infinite number of degrees of freedom. If one assumes that the deflected shape of the structure is the same as that which would occur under static application of the load, then the beam can be treated as a one degree-of-freedom (spring-mass) with an equivalent mass and spring constant (see references 2, 9 for a more complete description). The dynamic behavior, both elastic and plastic, for a one-degree-of-freedom system is well understood, and dynamic responses (both force and deflection) can be obtained from design charts.

6.5.2.2 Response of Idealized System

Figure 43 shows the response curves used for most of the analyses presented herein. The response is due to an instantaneously rising and linearly decaying (triangular) loading. The notation is as follows.

F_1 = peak value of loading (lbs)

R_m = maximum resistance of structural component (e.g., the load which produces the maximum moment, lbs)

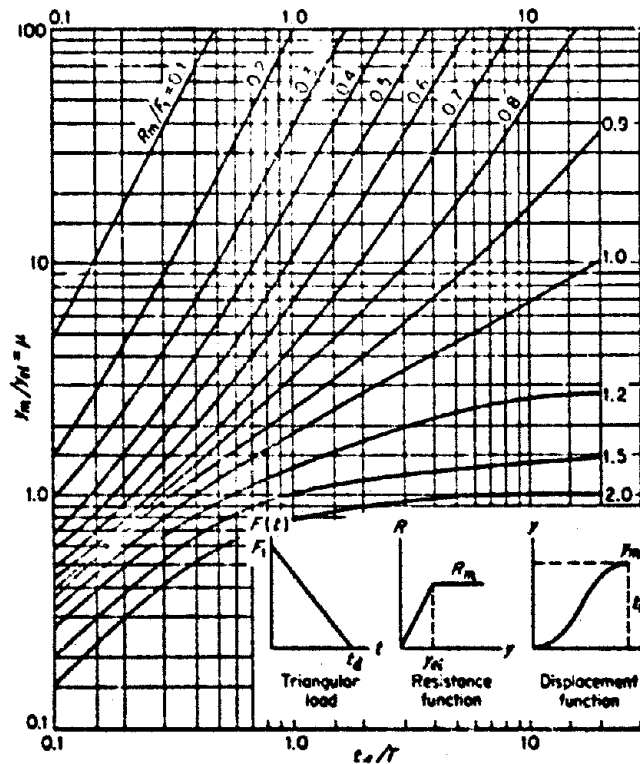
y_{el} = maximum elastic deflection of structural component (in.)

T = natural period (sec.)

y_m = maximum deflection of structural component, (in.)

t_d = duration of pressure loading (sec.)

The natural period of the structural component is obtained from its one-degree of freedom equivalent using



Response Curves for a One Degree-of-Freedom
Elasto-Plastic System - Triangular Load with
Instantaneous Rise Time⁽²⁾

Figure 43

the relationship,

$$T = 2\pi \sqrt{\frac{K_{LM} M}{k}}$$

where

M = total mass of component (lb sec²/in.)

k = stiffness of component (e.g., load required to produce a unit central deflection for a beam (lbs/in.))

K_{LM} = factor which converts the real system to the equivalent system

For the most of the bulkhead components to be considered, the duration of the load t_d is much greater than the natural period, so that t_d/T is always greater than 10. For system with independent dynamic components (Truss design), it is important to determine the period of each component, to see whether or not their dynamic responses can be uncoupled.

The dynamic reactions, or support forces, of a beam must consider the distributed mass of the beam. For a uniformly loaded beam, this is⁽²⁾

$$V = 0.39 R_m + 0.11 F \text{ (lbs)}$$

6.5.3 Allowable Limits for Stress and Deflection

Allowable limits for Stress and Deflection are based on reasonable assumptions regarding the function of a blast resistant structure. Structures for protection of equipment can tolerate large plastic deformations and still serve their function. The bulkheads under consideration are personnel shelters, and hence they are subject to more stringent requirements. Deflection limitation

for emergency shelter bulkheads arise from the following considerations.

1. Large deflections will cause destruction of seals.
2. Permanent deformations are cumulative for multiple explosions.
3. Permanent deformations may cause jamming of the door and prevent exit.

In spite of these limitations, small plastic deformations can be tolerated. Two possible design procedures are described below.

6.5.3.1 Elastic Design

The simplest design procedure is to assume that the maximum stress in the structure is less than or equal to the yield stress of the material. This procedure is simplest and most conservative, since the ability to deform plastically provides considerable reserve strength. In Figure 43, this condition is equivalent to requiring $y_m/y_{el} = 1$. For the blast loading that we are considering, $R_M/F = 2$, and hence we should design for a static load of 40 psi. This procedure has been employed in the preliminary sizing all of the bulkhead structural components.

6.5.3.2 Plastic Design

This procedure permits a fixed amount of plastic deformation. As can be seen in Figure 43, if we allow $y_m/y_{el} > 1$, we can reduce the required resistance R_m/F , and hence design for a lower static loading. The basic design criterion is the allowable plastic deformation which can be tolerated. For our designs, we have selected $y_m/y_{el} = 3$, i.e., we can tolerate plastic deformations equal to 3 times the maximum elastic deformation.

The plastic design method is commonly used in monolithic reinforced concrete structures which have constant cross sectional properties throughout. For metal structures assembled from different components, the section properties of each component must be more carefully controlled so that an unbalance of resistance will not occur.

6.5.4 Dynamic Material Properties

The yield stress of steel and aluminum increases with increased rate of strain. Based on the typical natural periods of the components being considered, the strain rates are in the range of 0.10 - 1.00 sec.⁻¹ For these strain rates, a conservative estimate for the increase of yield strength is 20 percent.⁽⁹⁾ This increased yield stress has been used in the plastic design calculations. For elastic calculations it represents a factor of safety.

Static and dynamic rock and coal strengths are not well known, and vary considerably with local conditions. We have based our designs on what we believe to be the minimum strength which can be expected in most cases. Further testing will be required to verify that these strength values can consistently and reliably be achieved.

6.6 Summary of Design Calculations

The detailed design calculations are presented in Appendix C. The results of these calculations are summarized in Tables 16, 17, and 18. These tables yield an equivalent static pressure capacity based on the strength of each structural component. The safety factor for each component can then be inferred.

For components which have been designed elastically, 40 psi is the base value for safety factor comparison. For elasto-plastic design, the value is less, depending on the degree of plastic deformation which has been allowed.

Table 16

Truss Pressure Capacities

<u>Component</u>	<u>Loading Type</u>	<u>Pressure Capacity at Yield (psi)</u>	<u>Rebound Capacity at Yield (psi)</u>
1-1/2" bolts	Direct Shear	54.6	54.6
Bearing Areas (general)	Bearing	40 (min)	40 (min)
Horizontal Members	Compression	86	-
Height Adjustment Bolts	Direct Shear	101.6	101.6
Front Rails	Bending	39 (elastic) 48 (plastic)	20
Decking	Bending	60	-
	Weld Tension		12.5
Trench	Shear	67 psi (based on a 2,000 psi floor bearing strength)	
Roof Stud	Bending		12.5
Front Post	Bending	52	52

TABLE 17

Channel - Turnbuckle - Pressure Capacities

<u>Component</u>	<u>Loading Type</u>	<u>Pressure Capacity at Yield (psi)</u>	<u>Rebound Capacity at Yield (psi)</u>
Facing Channel	Bending	40 (77.0 ultimate)	40.0 (77.0)
5/8" \emptyset Bolts	Tension	200	200
Eyebolt	Tension	42	16
Turnbuckle	Tension	42	16
7/8" \emptyset Bolts Upper Channel Splice	Tension	120	--
Door Plate Assembly	Bending	40	40
Roof Bolt	Shear	72	27
Trench	Shear	116	116

Table 18

Arch Design - Stress Summary

<u>Component</u>	<u>Loading Type</u>	<u>Pressure Capacity*(psi)</u>	<u>Rebound Capacity (psi)</u>
Ribs	Bending	16.4	16.4
	Axial	96.0	96.0
Door Section	Axial	40.2	40.2
Abutment	Compression	36.0	-
	Flexure	44.4	44.4
	Tension	44.4	44.4

*Based on modified pressure pulses for arch design.
Maximum flexural pressure - 14.1 psi
Maximum compressive pressure - 22 psi

6.7 Required Verification Testing

The design discussion of the previous section has indicated that certain assumptions must be made about the rock and coal properties in order to determine the adequacy of each anchorage design. These are reasonable assumptions based on available test data. The available data, however, does not cover the broad range of conditions which occur in coal mines. Consequently, well defined limits for strength property values can not be stated. These assumptions and their implications are discussed below.

- a. The rib coal can support at least 400 psi with negligible deformation (Arch design).
- b. The roof and floor rock has compressive strength of 3,750 psi. (Turnbuckle design).
- c. The floor has a minimum shear strength of 160 psi. (trenches).

Although these are reasonable engineering assumptions, there may be situations where these values are not achievable. A sound design requires a knowledge of the likelihood of these situations occurring.

There are a number of ways in which these design assumptions may prove to be inadequate. For example, the available coal support strength could be much less than expected due to excessive cleavage or weathering. U.S. Bureau of Mines tests on coal ribs⁽¹²⁾ found average strengths of 4,000 psi. These tests, however, were conducted in a specially prepared passage in the Bruceton Experimental Mine. This value could vary considerably in different locations.

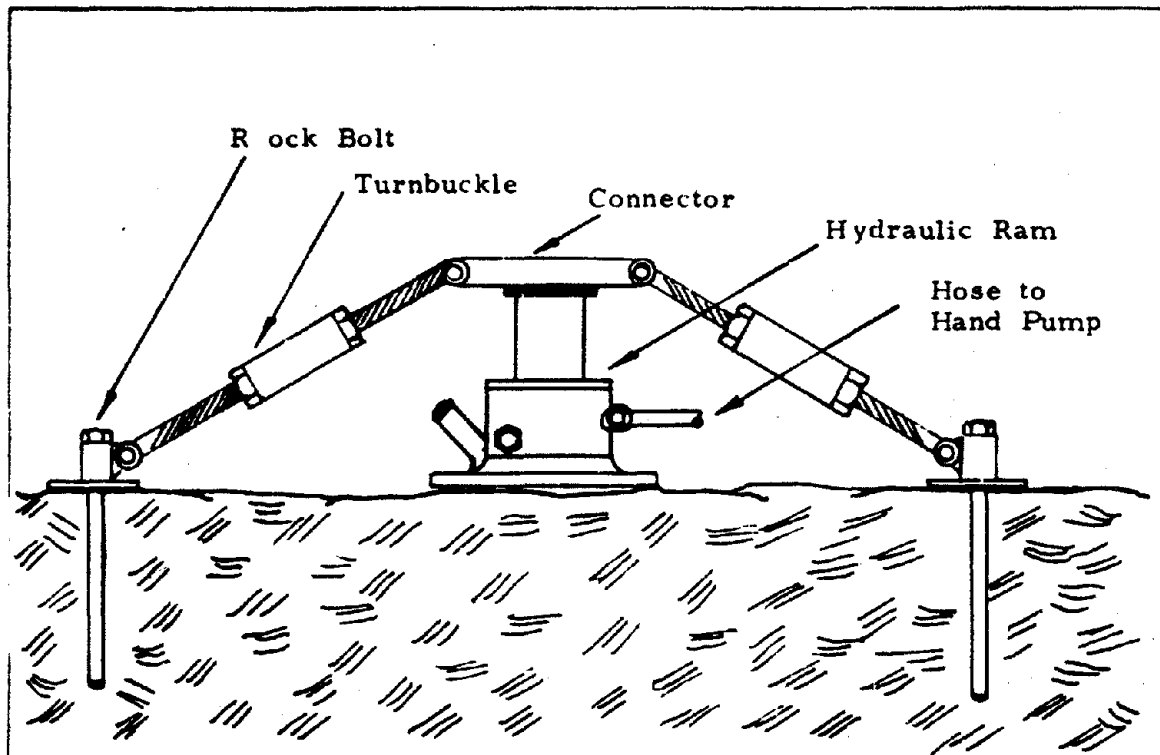
The assumption of 3,750 psi compressive strength in roof and floor strata may not be satisfied. This will cause some bending in

the bolt under shear loading, and will reduce the ultimate capacity of the bolt. Stratification will also cause a reduction in the bolt shear capacity. These influences however, are not well documented, and should be tested for the particular bolt configurations being considered.

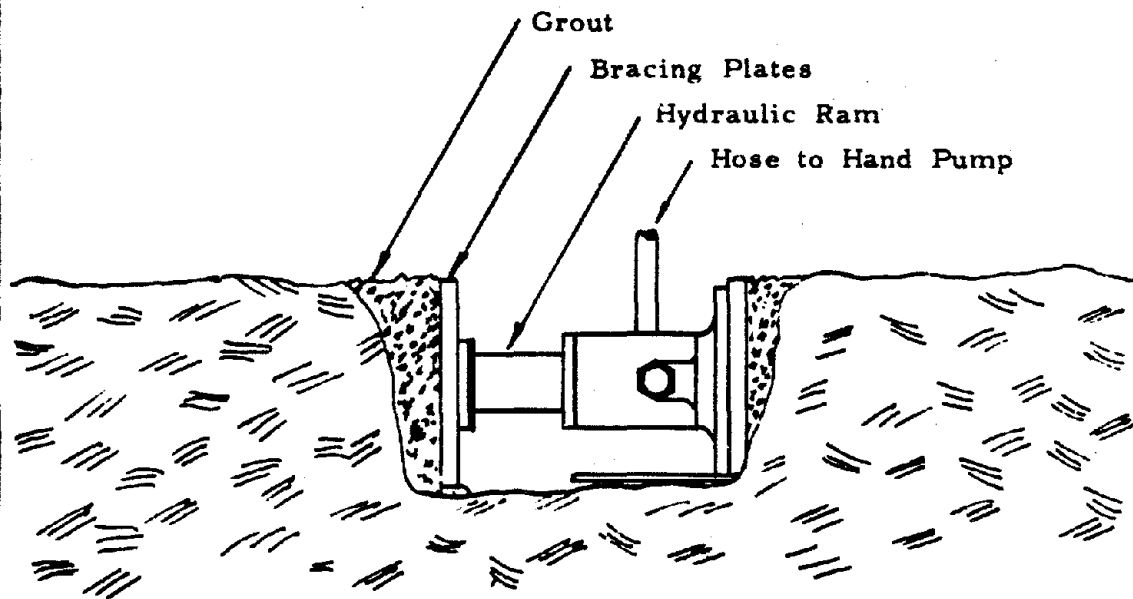
The 160 psi shear capacity requirement for trench supports is easily met, as long as the trenching does not fracture the surrounding rock. This fracturing may be difficult to avoid using impact tools in a brittle rock. Tests should be conducted to determine just what kind of support strength can be anticipated in the trenched configuration.

We believe that a test program of these various components would add a necessary assurance to the design assumptions, while avoiding the expense of failure during full scale explosion tests.

The testing should be conducted in-situ, and can be carried out using a portable testing unit. The basic loading unit would consist of a hand pump powered hydraulic ram. This would be fitted with different adaptors to suit the different types of tests. Figures 44 and 45 show three different test arrangements.



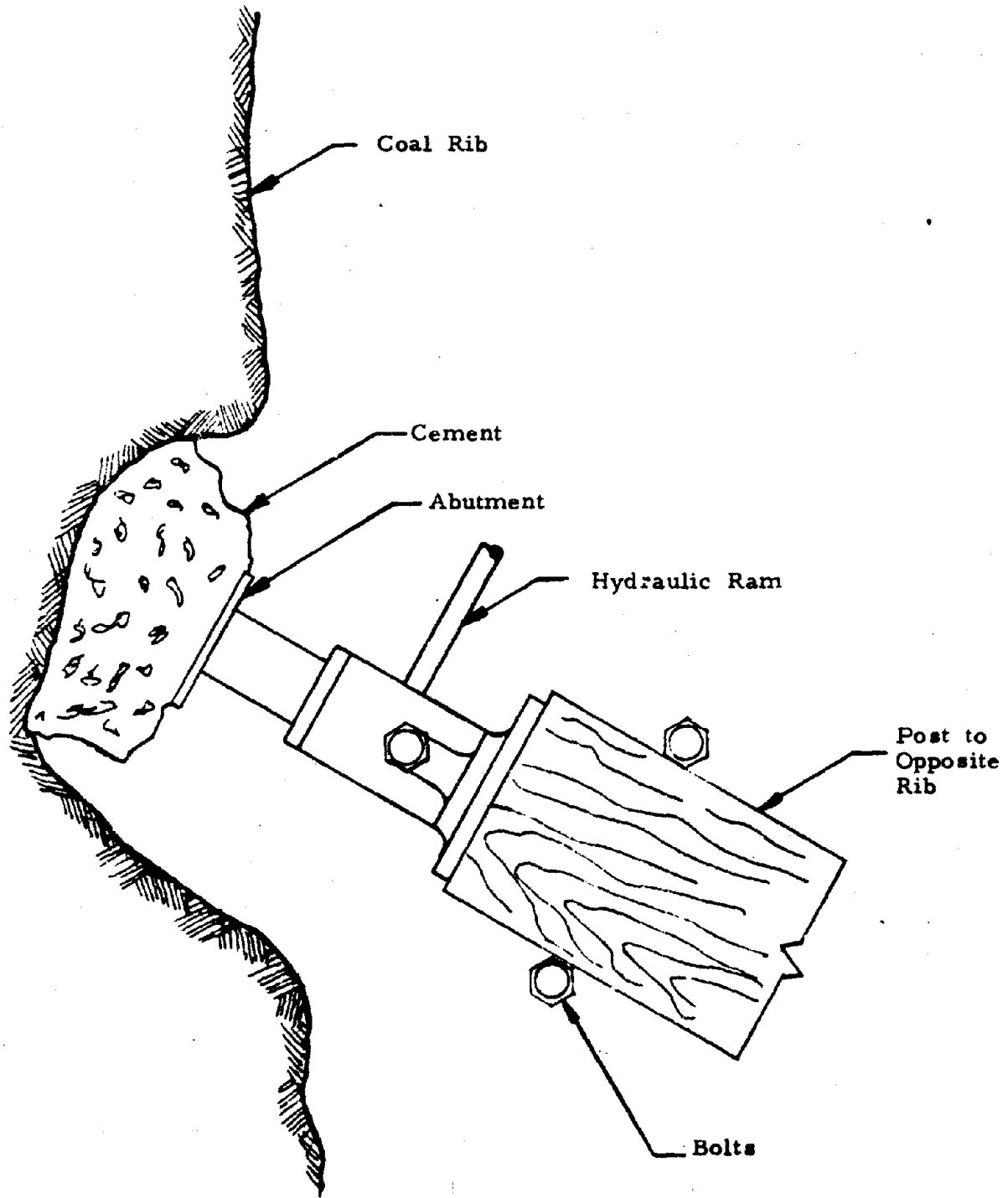
(a) Turnbuckle Anchor Test (Roof or Floor)



(b) Trench Bearing Test

Floor and Roof Anchor Tests

Figure 44



Rib Abutment Test

Figure 45

7. Foam-Flashing Seal System

7.1 General Description

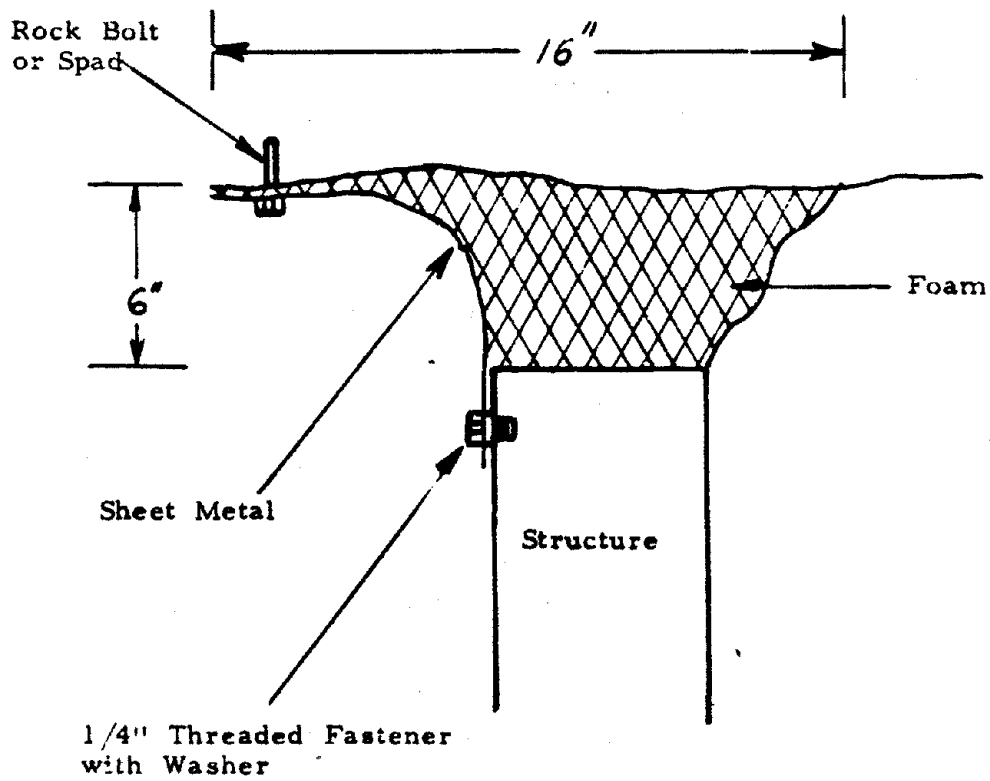
As indicated in the previous section we have proposed a unique foam-flashing seal system to seal the irregular gaps between the bulkhead structure and the entry. The basic sealing mechanism involves a strip of sheet metal (flashing) and frothed urethane foam. The sheet metal is bolted to the structure, and bent around flat against the rib, roof and floor. Froth foam is sprayed into the space behind the sheet metal. Figure 46 shows a cross-sectional view of the basic concept.

The combination of a membrane and a foamed plastic, similar to that which is described above, has seen application as a technique for packaging against impact.⁽³²⁾ In our application, the sheet metal membrane picks up load in tension, and the foam provides both support and a seal. The sheet metal also minimizes the exposure of the foam to a possible flame front. Frothed urethane foam has been selected due to its simple applicability and its good adhesive characteristic. The actual foam can be applied using convenient, portable foam packs currently available on the market. This type of foam is already in use in coal mines for sealing ventilation stoppings.

7.2 Preliminary Design Calculations

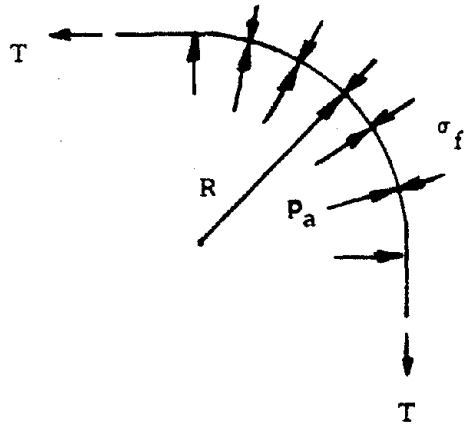
Figure 47 shows the loading diagrams for the sheet metal and the foam. p_a represents the applied pressure due to an explosion, T represents the tension in the sheet metal, σ_f represents the compressive strength of the foam, and τ represents the adhesive strength of the foam.

We can consider p_a as 40 psi, the equivalent static loading for a 20 psi step loading. A typical value for σ_f for rigid

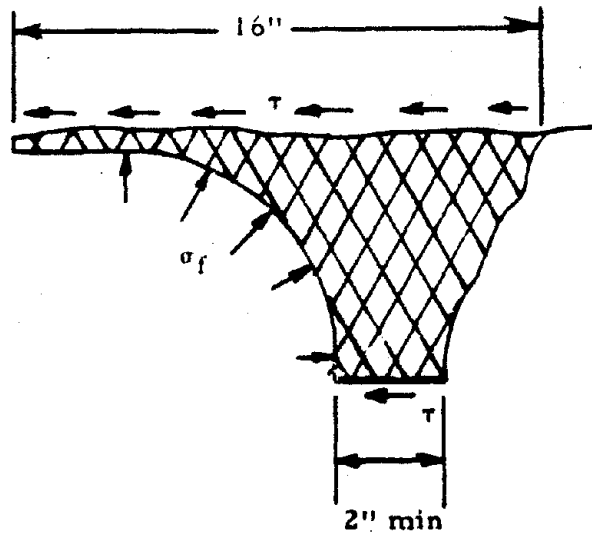


Section View of Foam-Flashing Seal System

Figure 46



(a) Sheet Metal



(b) Foam

Force Diagrams for Foam and Flashing

Figure 47

urethane foam is 30 psi.⁽³⁰⁾ Hence, equilibrium of the sheet metal states

$$T = (p_a - \sigma_f) R = 10 \times 6 = 60 \text{ lbs/in.}$$

For 30 ga. sheet, the tensile stress is

$$\sigma_T = T/0.012" = 5,000 \text{ psi}$$

This is well below the yield stress for typical aluminum and steel flashing.

The foam is supported entirely by its adhesion to the coal or rock, and to the structure. The 2 in. dimension represents the width of the narrowest structural element of the three designs (the top and bottom rails of the truss design). If we assume that the spads carry no load, equilibrium of the foam requires that

$$\sigma_f R = (16" + 2") \tau$$

or

$$\tau = \frac{30 \times 6}{18} = \frac{180}{18} = 10.0 \text{ psi}$$

The adhesion of urethane foam to mine entry surfaces has been found, from tensile pullout tests,⁽³⁴⁾ to be approximately 12.5 psi. This requires that the foamed surfaces be clean and dry. It is reasonable to use this value will be a minimum for the shear adhesion capability due to the favorable contribution of wall roughness. Therefore, the foam seal will have adequate shear strength.

An alternative to rigid urethane foam is "semi-rigid" urethane foam. This type of foam can accommodate larger displacements

while still retaining a seal. The static strength of semi-rigid foam is about 5 psi, but under dynamic loading, rapid compression of the air gaps in the foam enables the development of strengths of up to 40 psi.⁽³⁵⁾ We have investigated this possibility, and we have found that the state-of-the-art in field dispensing equipment for this type of foam is still in the developmental stage.⁽³³⁾ Consequently, we have not pursued it here.

In addition to being preliminary in nature the above analysis has not included the influence of relative displacement between the bulkhead structure or the passage walls. The combined effects of this relative displacement and the applied pressure load will produce a complex stress and deformation state which is beyond analysis. For this reason, we have carried out a test program which verified the competence of this type of seal. This is described in the next section.

7.3 Seal Test Program Description

This section describes the test program which was carried out to both establish the final seal configuration and to verify its performance capabilities. The basic design requirement was that the seal should remain intact and allow little or no leakage when subjected to the following conditions.

1. Ground Motion - The implication is that convergence which may occur at the bulkhead site during its life will not detract from its performance capabilities.
2. Explosion Pressure - Our 20 psi step loading has been applied, and interpreted as a 40 psi static load.

3. Relative Displacements - We consider the effect on the seal of both structural displacements and anchorage displacements.

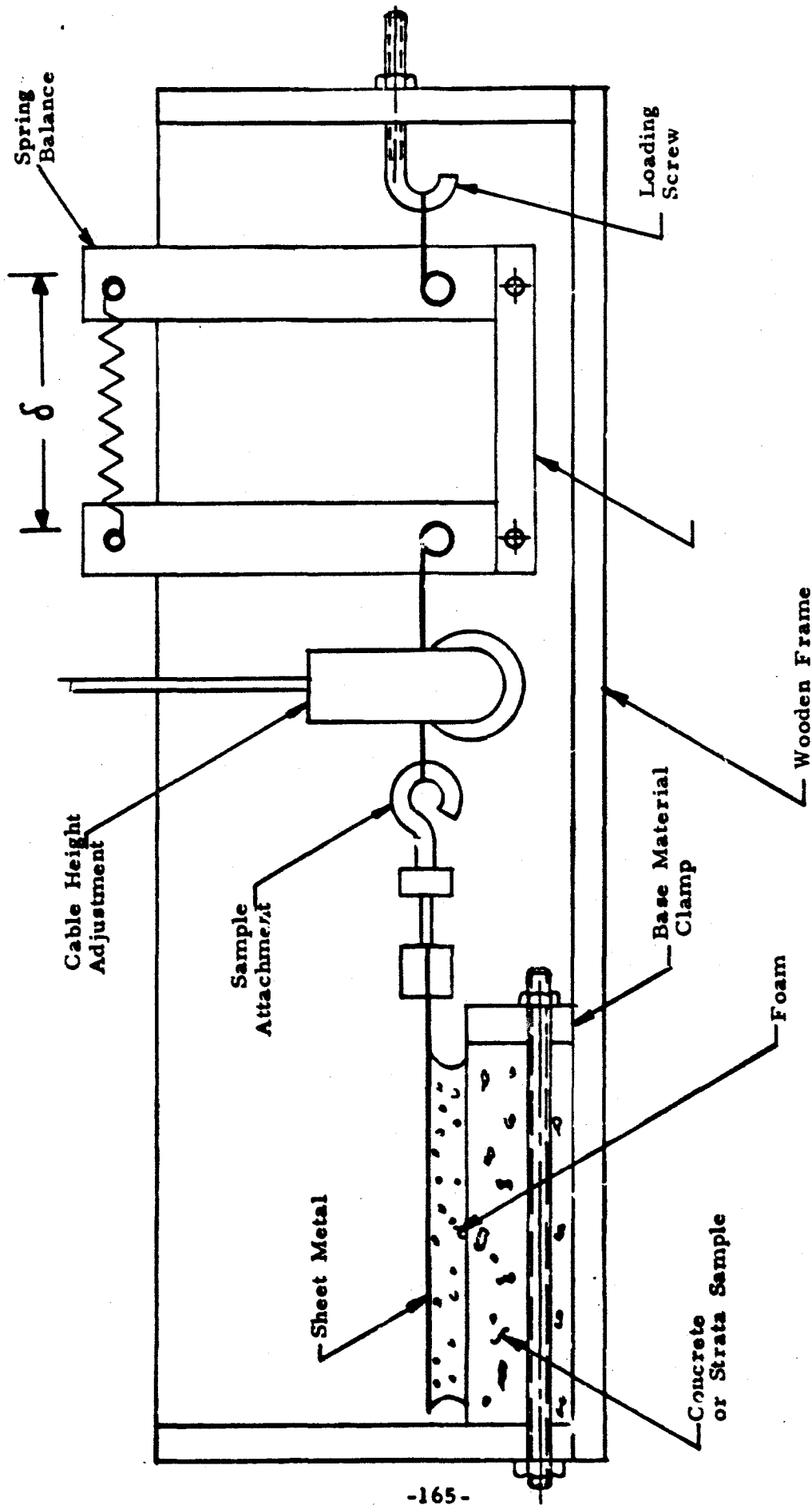
A two part test program, described below, served both the purpose of providing the data required to finalize the design of a seal to meet the above stated requirements and the purpose of subjecting several seals of that design to the closely simulated effects of the postulated explosion overpressure and resulting anchorage displacement, with or without prior convergence.

7.3.1 Preliminary Tests

The first two tests were performed to obtain the data necessary for the selection of a flashing material and a specification of the required geometry in which the foam is to be applied behind the flashing. Our assumption that shear failure is controlled by the foam shear strength was also tested here. The tests included shear bond measurements of foam-flashing and foam-rock interfaces.

A simple test rig (see Figure 48) was constructed for these tests. It included provision for securely clamping mine strata material in place and applying a measured force up to 2,000 lbs to a flashing or a foam sample.

Figure 49 shows the configuration for test 1. In this test shear force-deformation data was collected to compare the adhesive strength between possible flashing materials and the foam. The displacement of the leading and trailing edges of the flashing material were measured as the force was increased in regular increments. When the sample failed, the force at failure was noted and the area of the shear failure surface was measured. In this way, the shear stress at failure was calculated. From the displacement data, the shear strain at failure was estimated. The location of the failure,



Preliminary Test Rig
Figure 48

i.e., in the foam or at the flashing-foam interface, was recorded. The following flashing materials were tested.

1. Aluminum
2. Lead
3. Galvanized Steel
4. Uncoated Fiberglass Cloth
5. Silicone Rubber Treated Fiberglass Cloth.

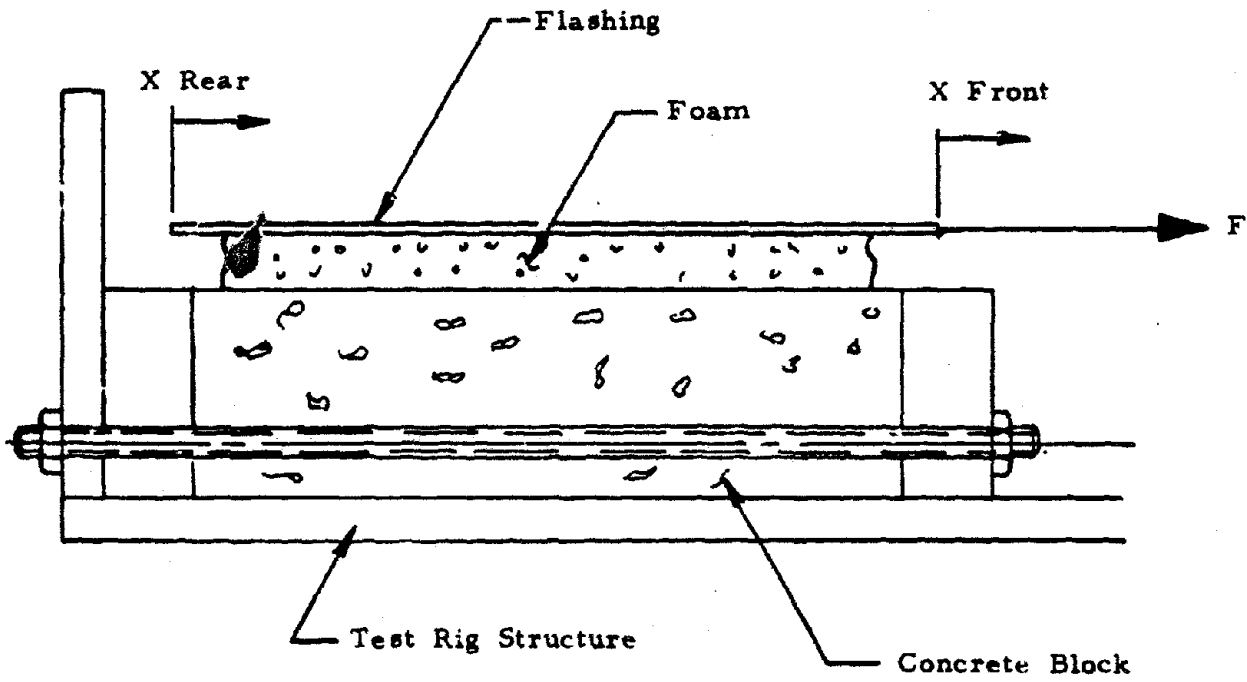
Figure 50 shows the configuration for test 2. In this test, shear force-deformation data was collected to measure the adhesive shear strength between the foam and the various passage strata materials. The shear force was applied to the foam by means of a metal grid embedded in the foam. As the force on the grid was increased in regular increments, the displacement of the grid was measured. When the sample failed, the shear failure area was measured and the failure shear stress was computed. The location of the failure was noted. The strata materials which were tested included the following.

1. Coal
2. Fireclay
3. Slate.

7.3.2 Pressure Chamber Simulations

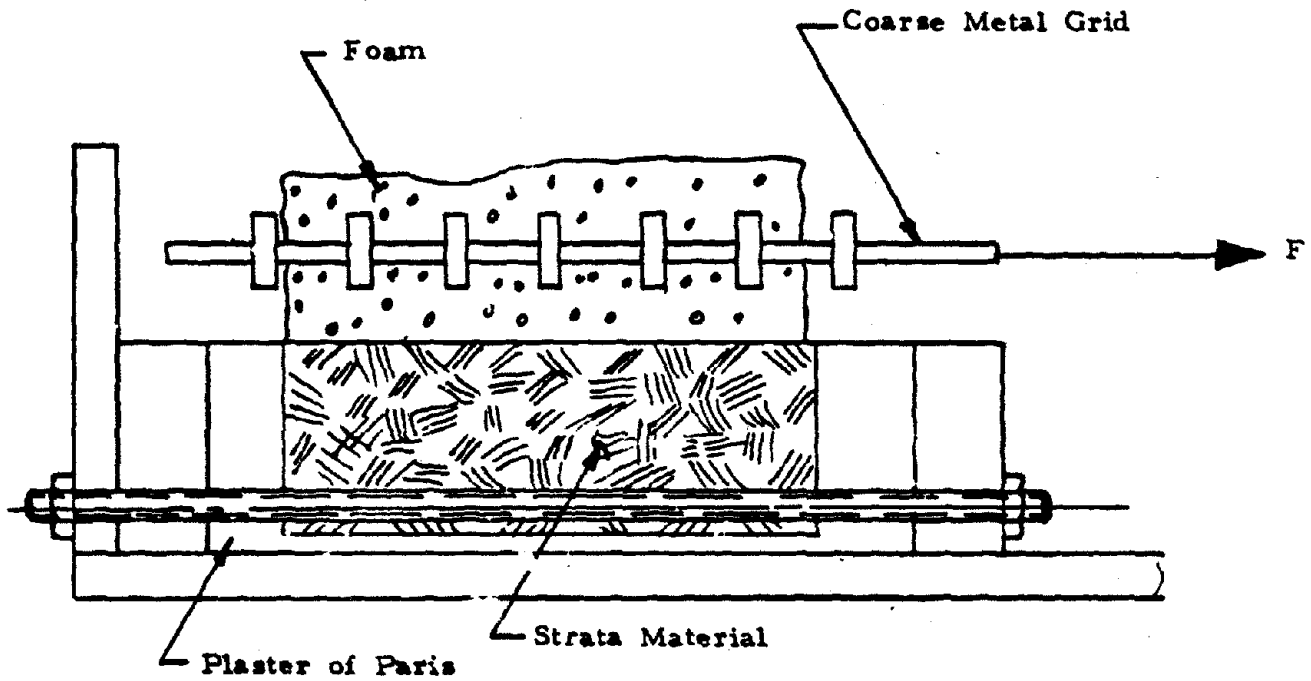
The preliminary tests provided verification of two crucial design assumptions:

1. That shear strength of foam-strata bonds is sufficient to transfer the pressure load from the foam to passage strata.



Test No. 1 - Adhesive Shear Strength Between Flashing and Foam

Figure 49



Test No. 2 - Adhesive Shear Strength Between Passage Strata Materials and Foam

Figure 50

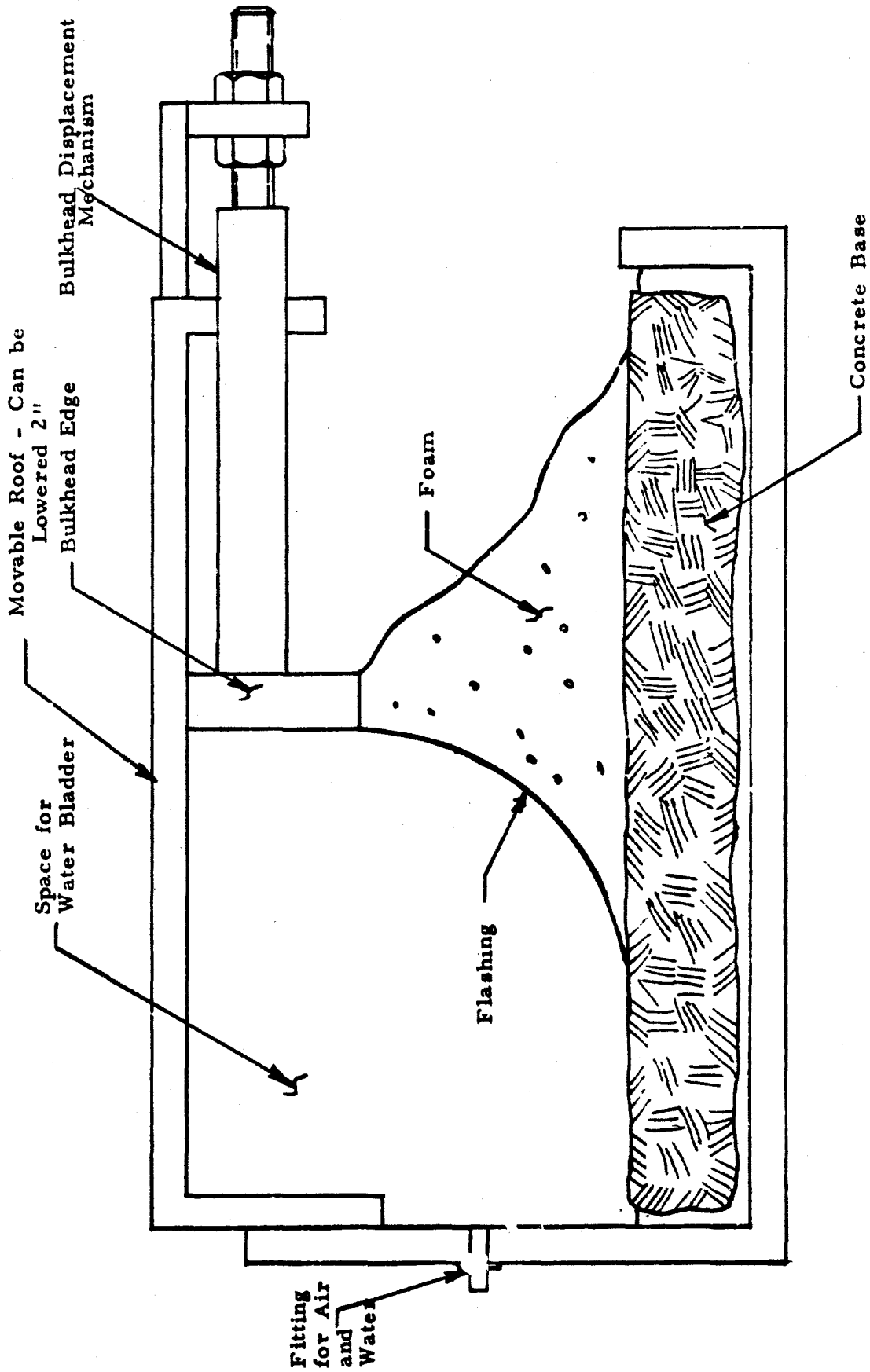
2. That the flashing-foam bond shear strength is sufficient to provide a firm anchorage for the strata edge of the flashing.

Final verification testing was carried out in a pressurized test chamber shown schematically in Figure 51. In this chamber, the explosion pressure was simulated using an expandable water bladder which was pressurized to 40 psi. Water was preferred for safety reasons, and the bladder was used to avoid excessive wetting of the test materials. The chamber is equipped with an adjustable roof, which could simulate convergence by squeezing down up to two inches on an emplaced seal. The portion of the apparatus which simulated the edge of the bulkhead was connected to a threaded shaft, which could displace that edge a distance of up to 6 in.

The apparatus was used to test a "section" of seal, 18" in length. This implies that we are assuming each section to act independently, which is somewhat conservative since each section will receive support from adjacent sections. This sectional simulation requires that the sides of the simulated seal be free to slide along the chamber walls without resistance. A low pressure air flow system was provided to quantitatively measure seal leakage.

This apparatus does not simulate the flexural action which a seal will experience along its length due to structural deflections. The strains involved, however, are well below failure strains for urethane foam, and hence this effect has not been considered important.

The following three sets of tests were carried out in the pressure test chamber.



Bulkhead Test Chamber
Figure 51

Test 3 - The first set tested a number of different seal configurations to failure. In this manner we were identifying foam and flashing geometries which performed most favorably. Candidate foam-flashing seals were constructed in the test chamber. Then the following test procedure was followed.

1. The water bladder was filled and pressurized to 40 psi.
2. With the water pressure held at 40 psi, the bulkhead section was displaced to the point of mechanical failure of the seal.

Tests 4 and 5 - These tests provided final verification testing of the selected seal configurations. Test 4 was conducted without convergence, and test 5 was conducted with convergence.

Air leakage measurements were used to assess the quality of the deformed seal. One problem which arose was the treatment of the sides. These are artificial boundaries created because of the size limitations of the test facility. Therefore, they had to be sealed artificially, yet in such a way that there was no resistance to the motion of the seal.

Our approach was to seal the side boundaries with a silicone sealant before each leakage measurement was made. Subsequent loadings and deformations destroyed this seal, and it was redone for subsequent leakage measurements. The method was tedious, but necessary for an accurate quantitative assessment of the seal performance. The following test procedure was followed for each seal.

1. The air leakage rate of the undisturbed seal was measured.

- 1a. For tests with convergence, the seal was compressed and the leakage rate remeasured.
2. The water bladder was filled and pressurized to 40 psi.
3. With the pressure held at 40 psi, the bulkhead section was displaced the sum of the estimated anchorage displacement and the structural displacement expected for a given bulkhead design (2 to 3 inches - see Appendix C).
4. The water bladder was depressurized and removed, and the seal was inspected for mechanical damage.
5. The air leakage rate was measured.
6. The bulkhead was displaced back toward its original position by the distance of the estimated elastic structural displacement. (~ 1 in. - see Appendix C).
7. The air leakage rate was measured.

7.4 Test Program Results

The results of the test program are presented below.

7.4.1 Preliminary Tests

The preliminary tests served the purpose of verifying basic design assumptions and providing a basis for final selection of a flashing material.

Test No. 1 and Final Selection of Flashing Material

Test no. 1 measured the shear strength of the bond between the polyurethane foam and the flashing materials considered. It is a critical property because the foam emplaced between the flashing and the strata provides the anchorage of the strata end of the flashing to the strata. The results are tabulated in Table 19. Aluminum and galvanized steel sheet as supplied both exhibited excellent bonding to the foam. As shown in Figure 52, shear failures tended to occur in the foam near the bonding surface instead of at the bonding surface itself. Lead, Fiberglass cloth and rubber coated fiberglass cloth as supplied all developed very weak bonds with the foam. Initially there were similar problems with lead, but after being degreased with the methylene chloride (urethane solvent), the lead flashing bonded adequately to the foam.

Aluminum was selected as the final flashing material for the following reasons:

1. The aluminum bonded excellently to the foam.
2. The 30 gage aluminum flashing has ample strength, yet is easily bent and formed to the shapes required for final seal design.
3. 30 gage aluminum flashing is inexpensive and obtainable in widths from 6" to

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Foam-Flashing Adhesion Tests. Typical Failure Surfaces
Figure 52

Table 19

Summary of Results of Flashing - Foam Adhesive Bond Strength Tests

Sample Tested	Bonding Area	Maximum Load Withstood	Maximum Bonding Surface Shear Stress	Displacement of Flashing at Rupture	Brief Description of Failure	
Aluminum #1 (0.0135")	65 in ²	850 lb	13.0 psi	1/4"	Both the aluminum and galvanized samples failed in similar fashion. Typically, the adhesive bond withstood the load. The rear section of the foam is loaded in tension to satisfy internal equilibrium requirements and fails under this tensile loading.	
Aluminum #2	50 in ²	520 lb	10.4 psi	1/8"		
Aluminum #3	30 in ²	325 lb	10.8 psi	1/8"		
Aluminum #4	30 in ²	410 lb	13.5 psi	1/8"		
Aluminum - Avg.	-	-	12.0 psi	3/16"		
Galvanized #1 (0.0179")	48 in ²	910 lb	18.7 psi	1/8"		
Galvanized #2	50 in ²	520 lb	10.4 psi	1/8"		
Galvanized #3	31 in ²	455 lb	14.5 psi	3/16"		
Galvanized #4	31 in ²	325 lb	10.5 psi	3/16"		
Galvanized - Avg.	-	-	13.5 psi	5/32"		
Lead #1 (1/32")	50 in ²	65 lb	1.0 psi	0		At Bond Surface
Lead #2	50 in ²	240 lb	4.8 psi	1/4"*		At Bond Surface
Lead #3 - Cleaned	35 in ²	325 lb	9.3 psi	5/16"*		At Bond Surface, but Started Rear Tensile Failure
CHR Fabric	50 in ²	65 lb	1.0 psi	0		At Bonding Surface Almost no Adhesion

* Includes about 20% flashing stretch.

24" from any roofing materials supplier. Steel thinner than 26 gage is not widely available. 26 gage steel is stiff enough to be inconvenient to work with.

Test No. 2

Test no. 2 verified the assumption that the foam bonds well to three normally encountered strata materials - coal, slate, and fire clay. This is an important assumption since the majority of the pressure load on the seal is transferred from the foam to the adjacent strata.

The results of the tests are tabulated in Table 20. Coal and foam form an excellent bond. Fireclay and slate each form good bonds with the foam after being cleaned with methylene chloride.

The bond strengths obtained here are conservative, since the small rock samples do not provide the degree of roughness that would be encountered in an actual strata surface. Consequently the favorable effects of mechanical bond are not exhibited.

7.4.2 Final Test and Verification

Tests no. 3, 4, and 5 provided the means for final determination of seal design and subsequent verification testing of these designs.

7.4.2.1 Test No. 3

In Test no. 3, seal geometries capable of mechanically withstanding the explosion pressure and bulkhead displacements were demonstrated. Six seals were tested - three of the kind depicted in Figure 46, one with the flashing indented into the foam

Table 20

Results of Foam-Strata Adhesion Tests

<u>Strata Material</u>	<u>Failure Stress</u>	<u>Comment</u>
Coal	10 psi	Failure occurred in coal
Fireclay	2 psi	Failure occurred at interface
Fireclay, cleaned with Methylene Chloride	9-1/4 psi	Failure occurred at interface
Slate	7-1/2 psi	Failure occurred at interface
Slate, cleaned with Methylene Chloride	9.9 psi	Failure occurred at interface

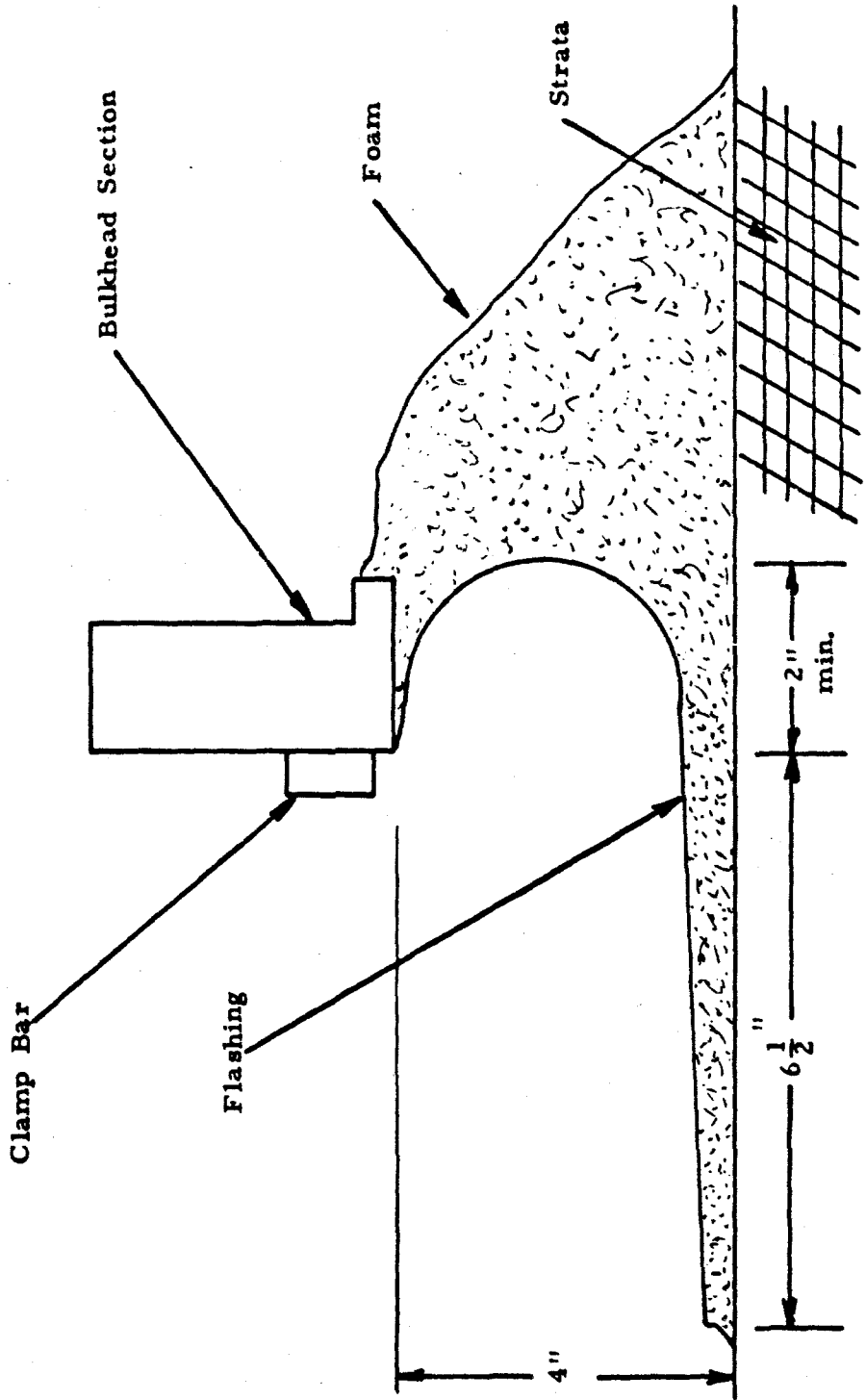
as shown in Figure 53, and two with the flashing bulged out from the bulkhead as shown in Figure 54. The three seals of the original design all proved to be capable of supporting the simulated explosion pressure, but bulkhead displacements in excess of approximately 1/2 inch resulted in seal failure along the strata in each case. When the bulkhead had been displaced to the failure point, the flashing was under much more tension than the design assumptions allowed for, resulting in debonding of the foam between the flashing and strata from the strata. This was immediately followed by the catastrophic failure of the entire foam-strata bond. It was concluded that the design assumptions had been conservative with respect to supporting the explosion pressure, but they had not adequately considered the displacement requirement. Since the failure resulted from insufficient slack in the flashing, the original design was modified to allow enough flashing slack to accommodate the displacements. Two approaches were used. In the indented configuration (Figure 53) the flashing is set up with a smaller radius curve, extending back several inches into the space between the bulkhead structure and strata. When the maximum bulkhead displacement is reached, the flashing geometry approaches that of the original seal design. In the bulged configuration (Figure 54) the flashing is bulged forward from the bulkhead as shown. As the bulkhead is displaced, the bulge in the flashing flattens out to accommodate the displacement, crushing the foam immediately underneath the flashing. The former has the advantage that quite large displacements can be accommodated, approximately twice the distance the flashing is indented behind the front face of the bulkhead. The disadvantage is that it is difficult to install, because it is between the bulkhead and the strata. The bulged configuration has greater ease of installation since the flashing is located entirely outside the space between the bulkhead and flashing, but it can accommodate maximum displacements of only 2 inches.

Figures 55 and 56 show the deformations of the three seal types. The seal samples have been removed from the chamber for observation. The test results for these seals are shown in Table 21.

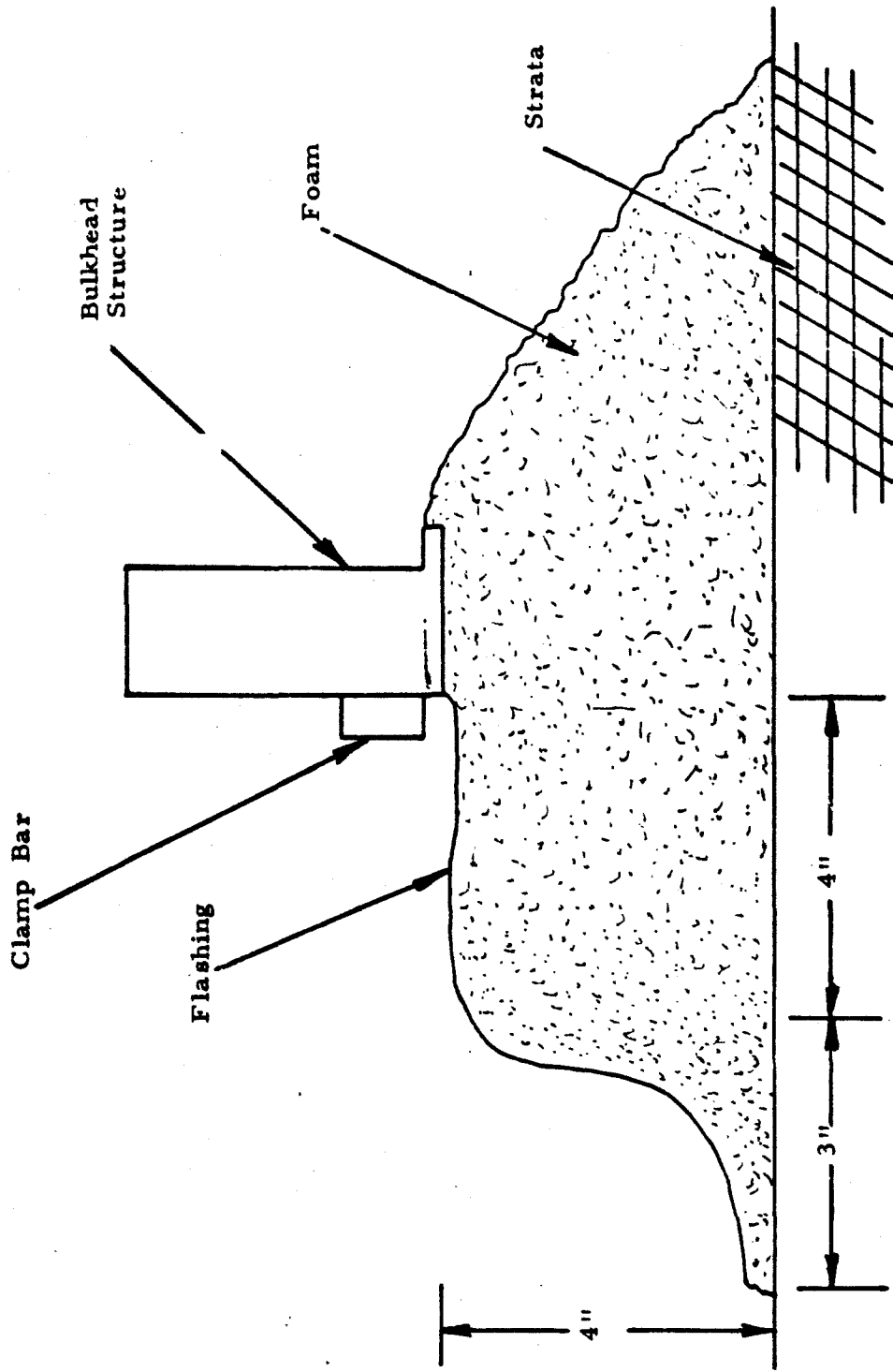
Table 21

Results of Test 3 Series

<u>No.</u>	<u>Seal Type</u>	<u>Gap Width</u>	<u>Displacement at Failure</u>	<u>Comments</u>
1	Original Design	6"	9/16"	
2	Original Design	6"	9/16"	
3	Original Design	6"	7/8"	
4	Indented	4"	3-1/8"	Indentation depth was 1-1/2
5	Bulged	4"	1-3/8"	Insufficiently bulged
6	Bulged	4"	2-1/8"	Bulged as shown in Figure



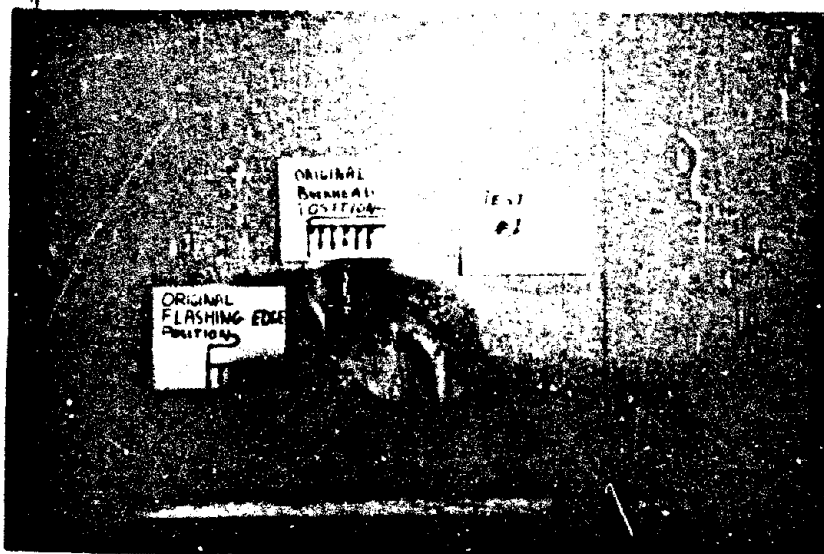
Indented Seal, as Set Up for Verification Testing
Figure 53



Bulged Seal, as Set Up for Verification Testing

Figure 54

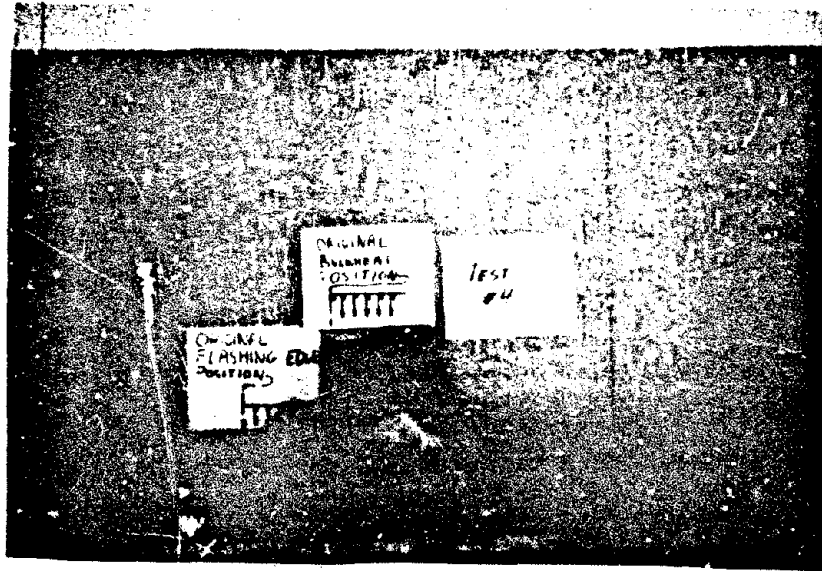
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Failure Geometry of Initial Seal Configuration

Figure 55

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(a) Indented Seal After 4" Displacement



(b) Bulged Seal After 2" Displacement

Deformation of "Indented" and "Bulged" Seals

Figure 56

7.4.2.2 Final Seal Designs

Final seal designs were determined on the basis of requirements imposed by the different bulkhead designs and on the experience gained in the test program. The requirements imposed by each bulkhead design are based on the closeness of fit between the bulkhead and the strata and on the anchorage and elastic structural displacements anticipated at the four sides of the bulkhead due to the explosion pressure loading.

In general we anticipate gaps no more than 6" in width, and more likely 1 - 3". For structural displacements we anticipate a maximum displacement of 1" due to anchorage movement, and a maximum of 2" due to the flexure of the bulkhead, or a total of 3" maximum. Based on the test program, we found the bulged configuration more suited to smaller gaps that will be subjected to displacements of 2" or less, while the indented configuration could handle larger gaps with displacements up to 3".

The particular seal details for each design are included in the design package of fabrication drawings, and are also indicated on the layout drawing in Appendix D.

7.4.2.3 Verification Tests

A typical design of the bulged seal for meeting the first set of requirements listed in the previous section and a typical design of the indented seal for meeting the second set of requirements was subjected to the verification testing provided by Test no. 4 and 5.

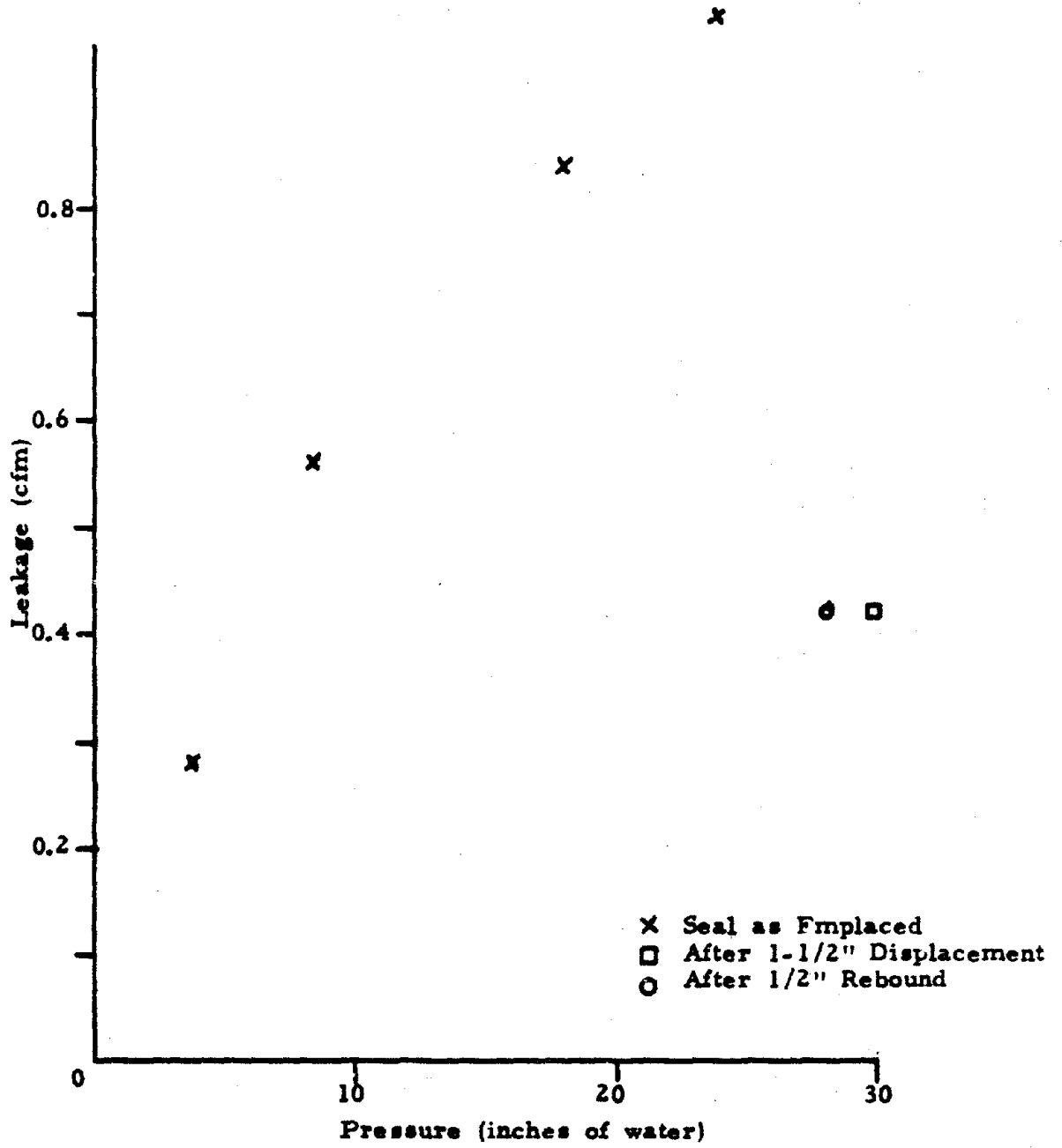
The test procedures described in Section 7.3.2 were followed in each case, using the displacements

listed in the previous section. In each of the four cases, the seals remained intact throughout the entire process of roof convergence, pressurization, anchorage and elastic structural displacement, depressurization, and rebound of the elastic displacement of the structure. The air leakage measurements for the four cases, are presented as plots of the air leakage rate through the seal versus the pressure difference across the seal, and are shown in Figures 57 through 60.

7.5 Application Trials

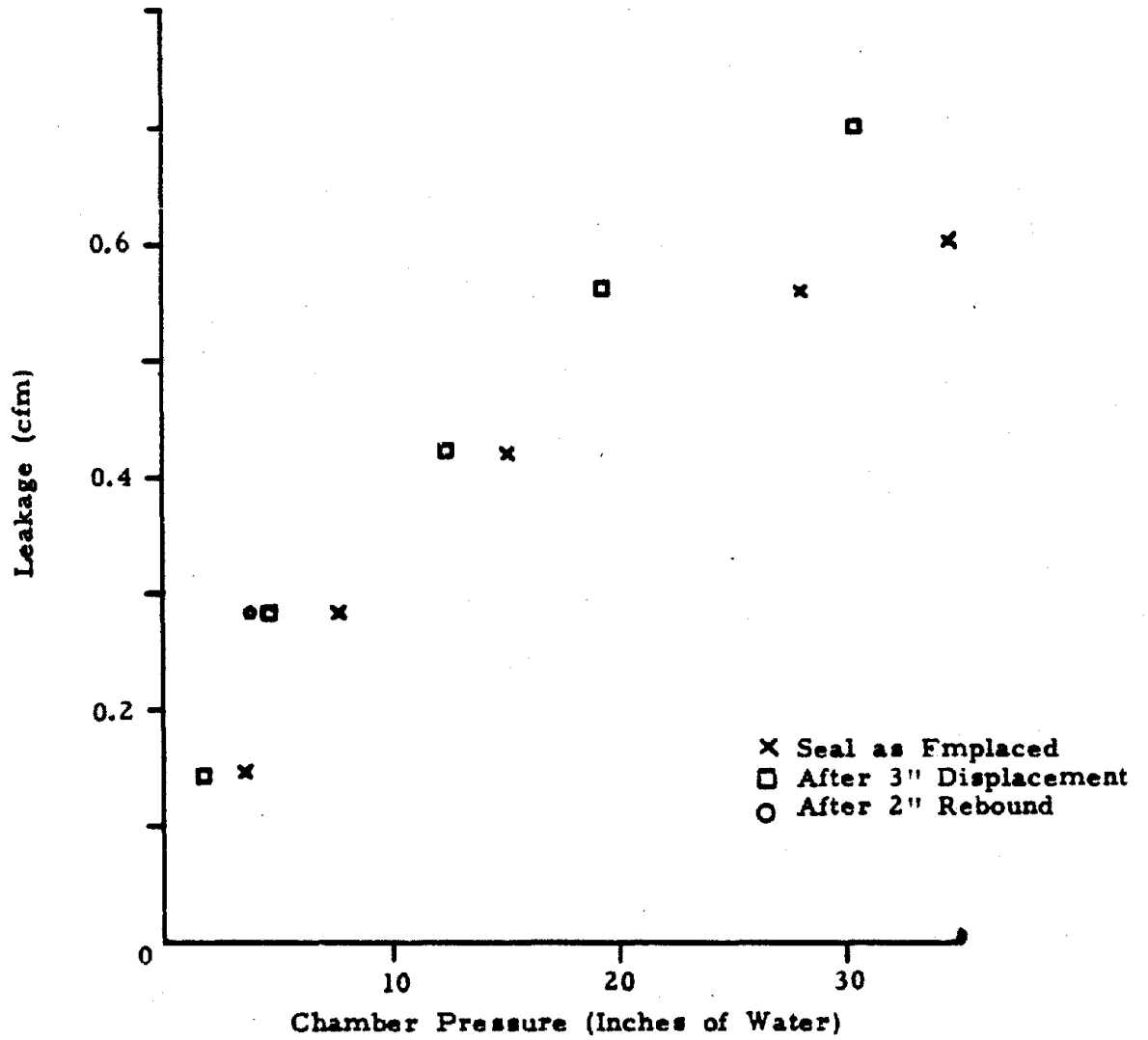
As a matter of practical concern, we investigated the practical feasibility of applying the froth foam in the manner we have prescribed. For this purpose, a frame was constructed within the Foster-Miller full scale mine ventilation facility. We then attached flashing from this frame to the walls, and then tested the froth foam application.

Figure 61 shows views of the emplaced seals. We anticipated potential problems in containing the foam while spraying a roof seal, but this concern proved to be unwarranted. A bit of practice enables one to develop some technique, after which the proper foam application becomes quite simple.



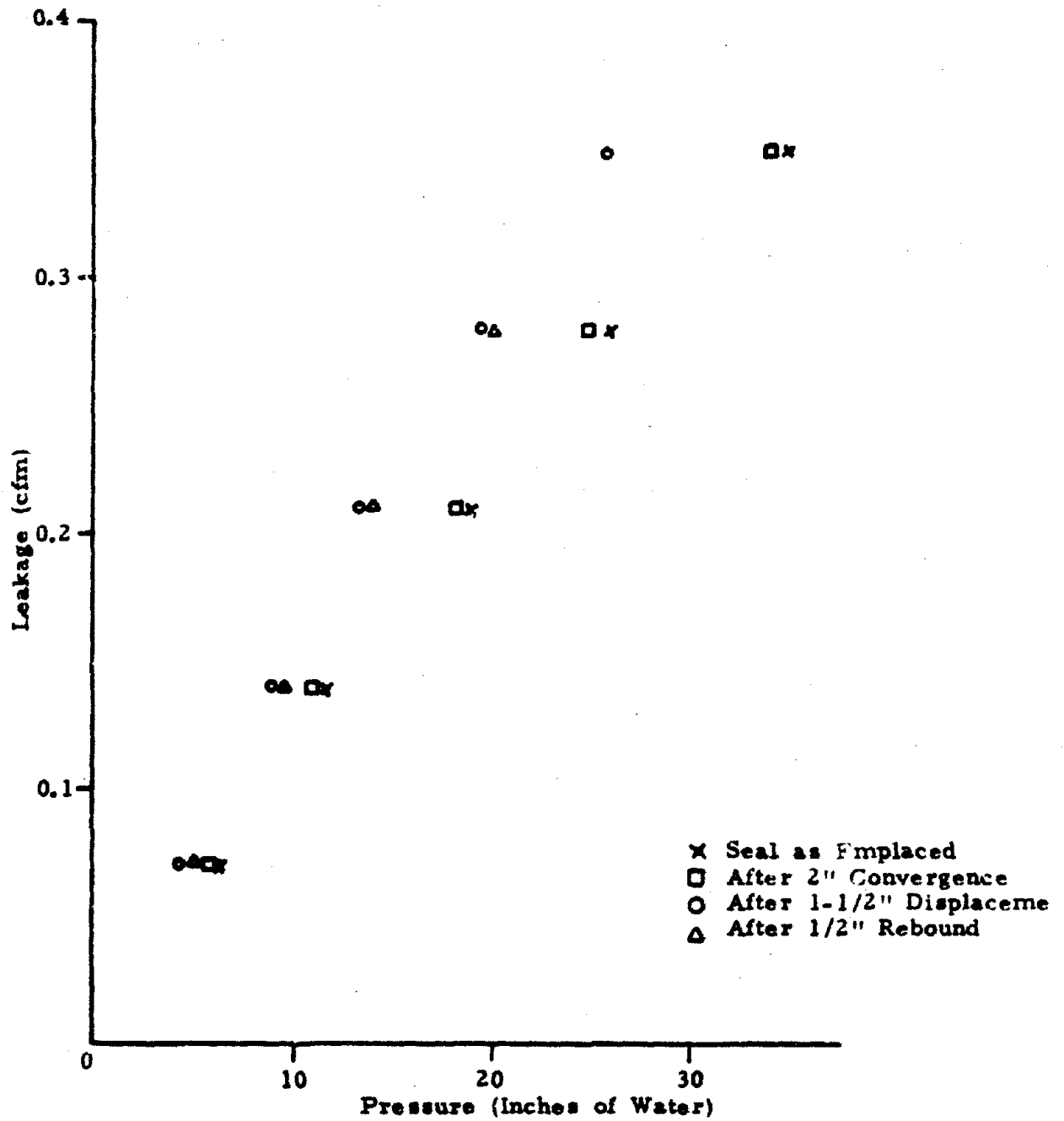
Pressure Leakage Results - Bulged Seal

Figure 57



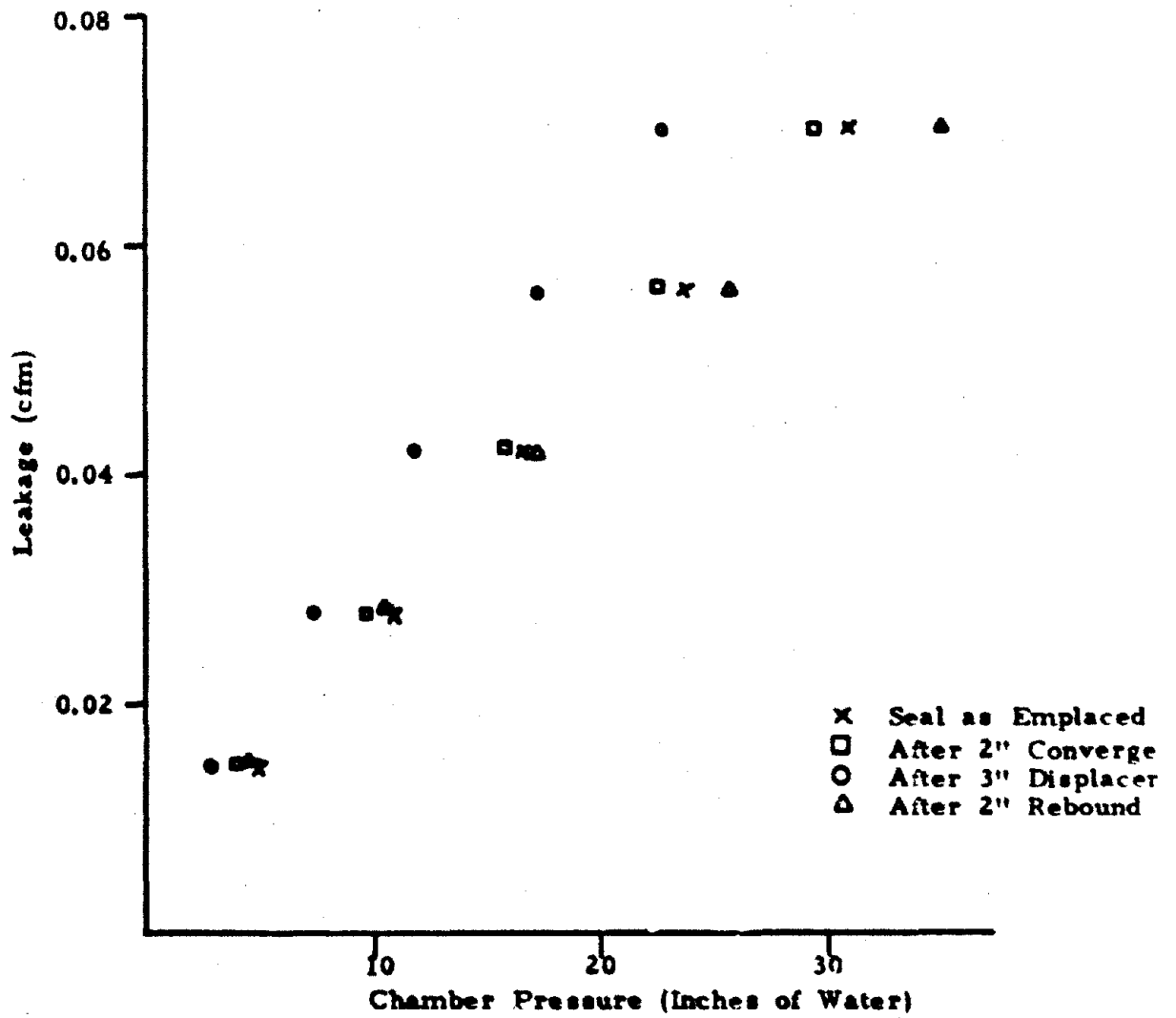
Pressure-Leakage Results - Indented Seal

Figure 58



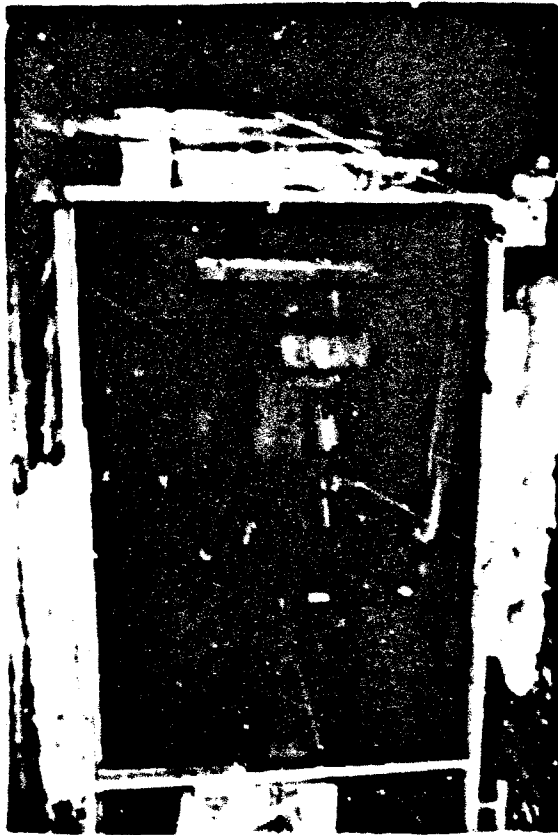
Pressure-Leakage Results - Bulged Seal Subjected to Convergence

Figure 59



Pressure-Leakage Results for Indented Seal Subjected to Convergence

Figure 60



(a) Mockup Bulkhead Frame



(b) Application of Foam



(c) "Bulged" and "Indented" Flashing Configurations



(d) Section of "Indented" Seal

Application Trials of Foam-Flashing Seal

Figure 61

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8. Conclusions

The preceding report has suggested the application of emergency shelters in a mining environment, and has reviewed, developed, selected and compared a variety of concepts for explosion proof bulkheads for a crosscut emergency shelter. The principal conclusions which were drawn from this effort are summarized below.

1. Location of Emergency Shelters

The optimum locations for emergency shelters are at the intersection of mains and developing entries, and midway along developing entries. The exact spacing requirements for emergency shelters will depend on the height of the coal seam, since this will dictate how far a miner can travel during an emergency situation.

2. Life of an Emergency Shelter

Shelters established along a main entry will be potentially functional for all further development operations along that main, and can be regarded as permanent. Their existence in conjunction with a borehole is economically justified and highly recommended.

Shelters established along developing entries may have a useful life of 6 months to 3 years, depending on the rate of development. These shelters may be regarded as temporary.

3. Type of Bulkheads

Explosion-proof, air tight structures for a crosscut emergency shelter can be either permanent or reusable. Permanent structures tend to be cheaper in materials costs, but generally involve more cost in labor, materials handling, and equipment. Reusable structures, which use more expensive materials, require less labor and are logistically simpler to work with. The number of reuses also reduces the material cost per installation.

4. Reusable Concepts

The principal problems associated with the design of a reusable concept are:

Anchorage - connection of an explosion proof structure to the unpredictable support material comprising the rib, roof, and floor of a mine entry.

Sealing - maintaining an air tight seal after the structure has experienced an explosive pressure wave.

5. Permanent Concepts

There has been much experimental work regarding the development of permanent explosion-proof bulkheads. Most of the concepts which have been developed are excessively massive and inappropriate for use as an emergency shelter bulkhead.

6. Costs

Three proposed permanent concepts and three proposed reusable concepts have been compared in terms of total installation cost. The reusable structures are cost competitive with the permanent structures.

7. Final Reusable Designs

Due to the expected variability in support conditions, no universal reusable bulkhead design can be proposed. To span the expected range of conditions, three designs have been proposed and worked out in detail.

8. Foam-Flashing Seal

A special sealing system, consisting of a membrane of aluminum flashing supported by sprayed-in-place frothed urethane foam has been developed to seal the potentially large, irregular gaps that will exist between the bulkhead structure and the walls of the entry. A testing program has shown this seal to be capable of withstanding a combination of 40 psi static pressure and 2" to 3" of relative structural displacement.

9. Recommendations

Our prime recommendation is that the three reusable designs be built and tested. A recommended test program is outlined below.

1. Component Tests

We suggest that the roof bolt, trench, and rib abutment support components be tested prior to full scale explosion trials. This can be carried out in a simple, cost effective manner by using a hydraulic ram and simulating explosion loads statically. The tests can be carried out in-situ for a number of representative mine conditions, to obtain a spread of anchorage capabilities.

2. Assembly Trials

The bulkheads should be fabricated and subjected to in-mine assembly trials. Construction feasibility and man hour projections can be verified, and modifications for construction simplicity can be implemented at this time.

3. Explosion Trials

The bulkheads should be installed and explosion tested at the U.S. Bureau of Mines explosion testing facility. This will be the final verification of the performance of the three bulkhead concepts.

This testing would be carried out for the high coal bulkheads whose designs have been presented herein. A complete verification of the bulkhead concept would involve low coal as well, particularly since low coal operations will require more temporary shelters. Consequently, a low coal program similar to that described above is recommended.

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APPENDIX A

EXPLOSIVE ENVIRONMENT DEFINITION

A.1 Introduction

The design of any structure requires a careful examination of the environment to which it will be subject. For the emergency shelter bulkhead, we are primarily concerned with the characteristics of the applied pressure wave; secondary attention will be given to the thermal environment. The nature of the applied pressure wave will dictate both what type of design configurations will be effective and what type of analysis will be most applicable for evaluating a given design. The thermal environment will dictate the type of materials used in the design.

In order to understand the nature of the blast environment, we begin by first assuming that the pressure pulse is being supplied by a methane-air explosion. Coal dust explosions are much more lethal; not only would there be fewer survivors remaining to use the rescue chamber, but also it would not be practical to design shelter bulkheads against such explosions. In addition, conscientious rock dusting should minimize the possibility of a coal dust explosion.

The pressure-time characteristics for a methane-air explosion vary considerably depending on the concentration of methane, the length of the gas zone, the location of the ignition source within the gas zone, and the location of the gas zone within the mine. The pressure felt by the bulkhead will depend on the location of bulkhead with respect to the gas zone. These factors will be discussed in the following sections. Although it will not be possible to generate a well defined design pressure pulse, the following sections will discuss orders of magnitude and basic physical occurrences. These along with actual in-situ experimental measurement, yield a fairly good picture of the pressure environment for design.

A.2 Physical Characteristics of a Methane-Air Explosion

A mixture of any where between 5 percent and 15 percent of methane with air can be ignited to produce an "explosion" in one of three different ways. These are outlined below.

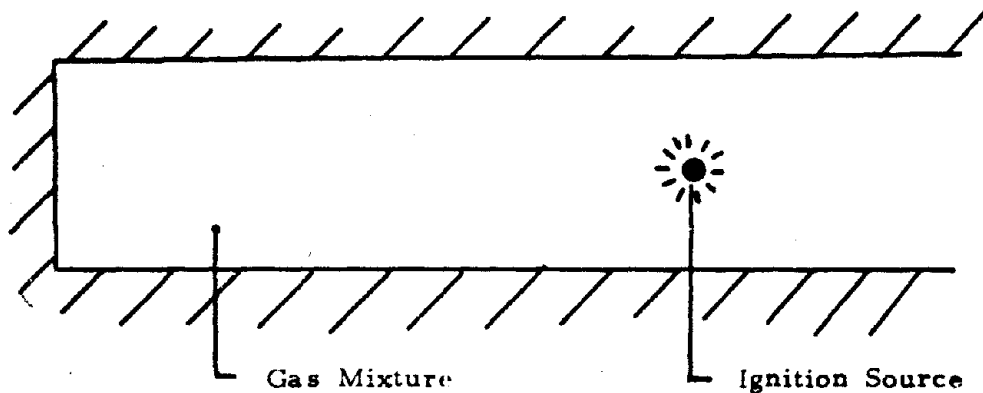
A.2.1 Simple Deflagration

In a simple deflagration the reaction zone travels away from the ignition source at constant velocity (which is much less than the speed of sound - about 1-5 ft/sec.). Figure A.1 shows a configuration where this will occur. The reaction zone will travel toward the face at constant velocity. The high temperature burnt gasses will produce a pressure which will propagate out of the passage. This pressure will be similar to that produced by a piston moving at a constant velocity down the passage, accelerating the gasses ahead of it. Here, the piston is replaced by the high pressure burnt gasses produced by the constant velocity reaction zone. Figure A.2 shows a typical pressure pulse produced in this case.

A.2.2 Accelerating Deflagration and Resulting Shock Wave

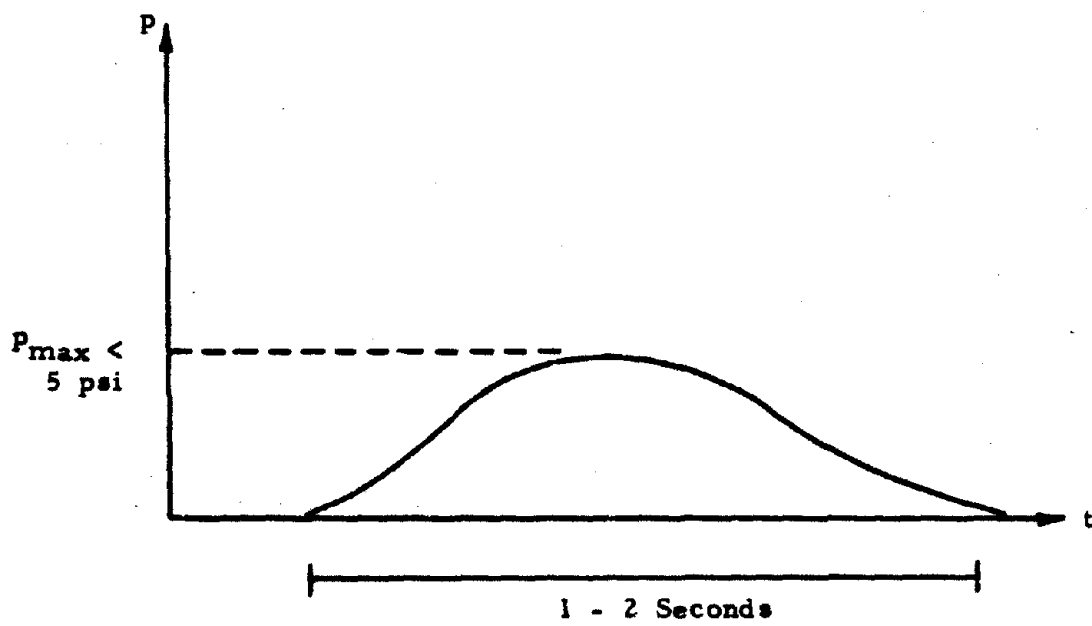
When the burnt gasses cause pressures and accelerations in the unburnt flammable gasses, the reaction zone accelerates. Figure A.3 shows a condition where this would occur. Here the burnt gasses expand into the unburnt gas, causing turbulence and local distortions of the flame front. The distorted flame front has greater surface area than a planar front, and this increases the reaction rate. This process is self accelerating. The higher the reaction rate, the greater the gas velocity and pressure, the greater the turbulence, the greater the reaction rate, etc. Since the observed peak pressures are proportional to the reaction rate, this condition will produce higher peak pressures.

The accelerating reaction zone produces a series of pressure pulses. Each pressure pulse travels at the speed of sound



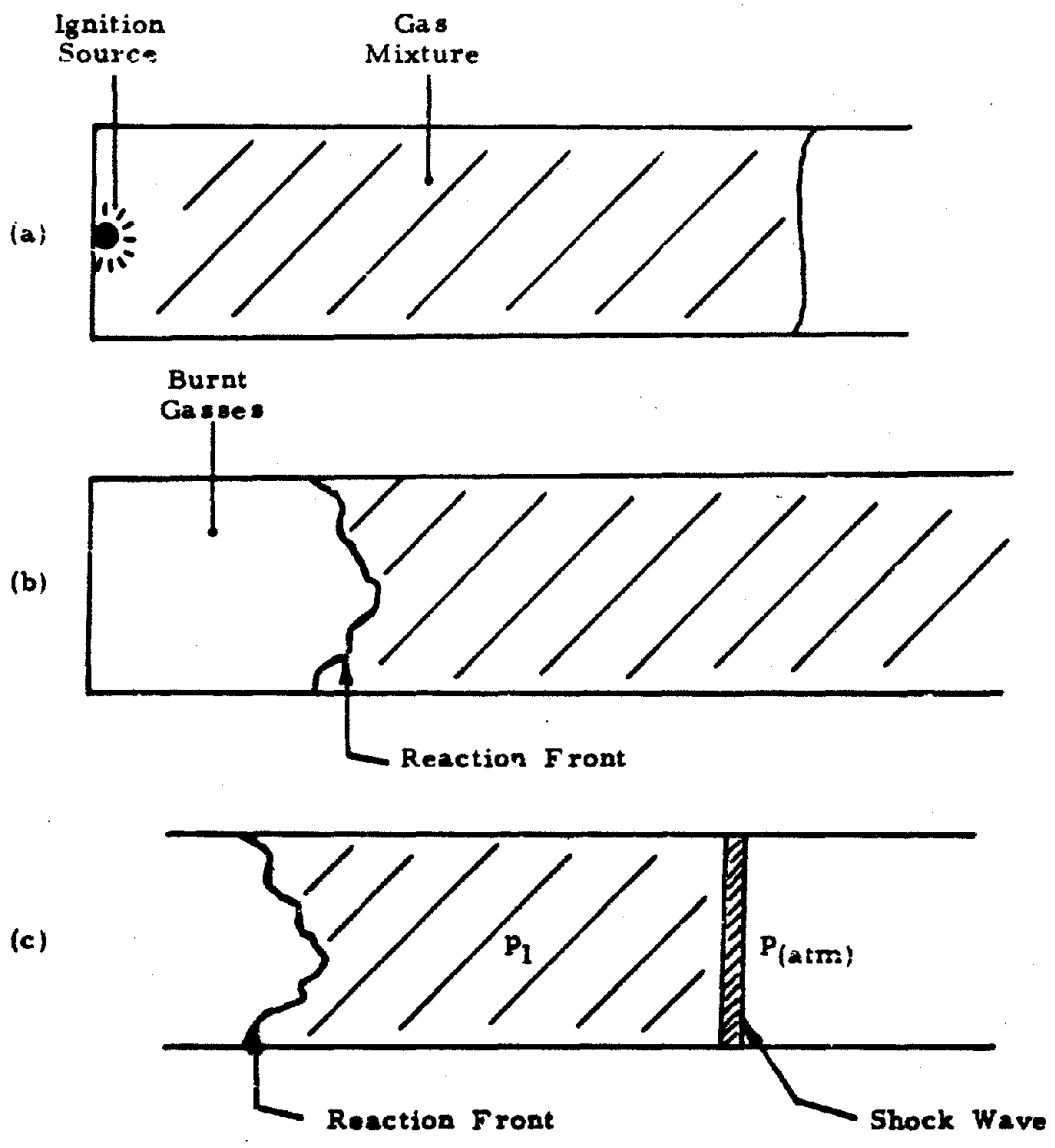
Conditions for a Simple Deflagration

Figure A.1



Typical Pressure Pulse Resulting from a Simple Deflagration

Figure A.2



Conditions for Accelerating Deflagration and Resulting Shock Front

Figure A.3

of the pressurized medium of the pressure pulse which occurred before it. Since the velocity of sound increases with increasing pressure, the pressure waves tend to coalesce. If the gas zone is long enough, these pressure waves will coalesce to form a well defined shock front as depicted in Figure A.4. This model of shock wave formation is discussed in Reference 36, 37 and 38. The principal feature of the shock wave is its instantaneous rise time, and its pressure peak which is greater than that for simple deflagration. The shock wave moves at the speed of sound in the ambient air.

A.2.3 Detonation

Detonation is basically the joint movement of the shock wave and the reaction zone. Once it occurs, the phenomenon is self sustaining, and the detonation wave moves at a speed greater than the speed of sound of the ambient air. Figure A.5 shows a typical detonation pressure pulse. It is characterized by a high peak (about 10 atmospheres) and a very short duration. The causes of detonation require a more lengthy discussion than is appropriate here (see Reference 37 for example). It will be adequate to say that detonation is likely to occur in a very long gas zone (>60 x passage diameter) or in a confined gas zone (bulkheads at both ends). Both of these conditions are unlikely to occur in the working areas around the emergency shelter.

One can conclude from the above that the most severe pressure wave resulting from a methane explosion in a working mine would be a shock wave due to an accelerating deflagration. The maximum possible amplitude of such a wave is about 5 atmospheres, i.e., that due to the completely confined expansion of the burnt gasses. The actual pressure is less for an open ended region. The duration of the pulse will be proportional to the initial gas zone and the peak pressure, as computed from the expanding volume of gas. This duration is on the order of fraction of a second.

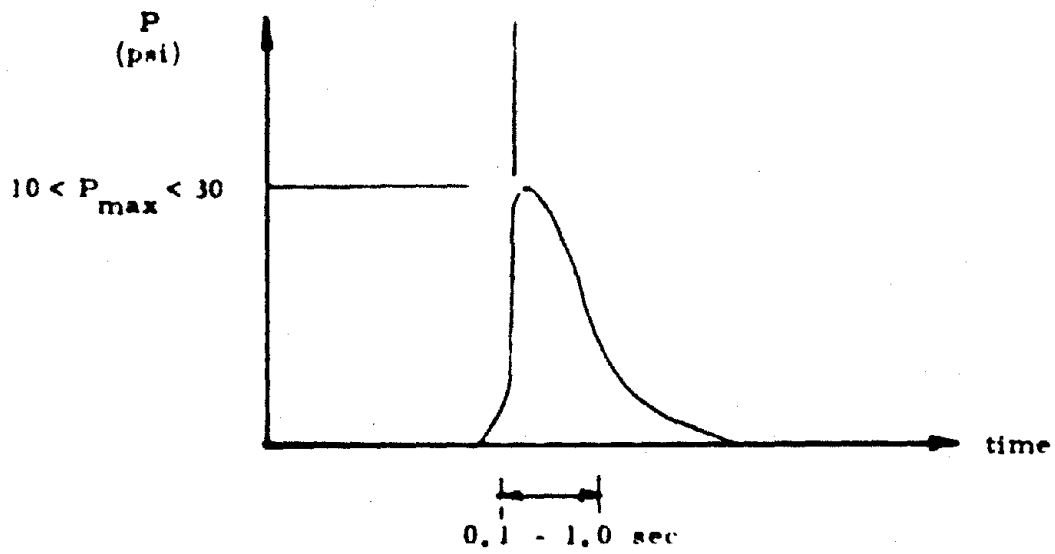


Figure A.4 - Typical Pressure Pulse From a Shock Wave Induced by an Accelerating Deflagration

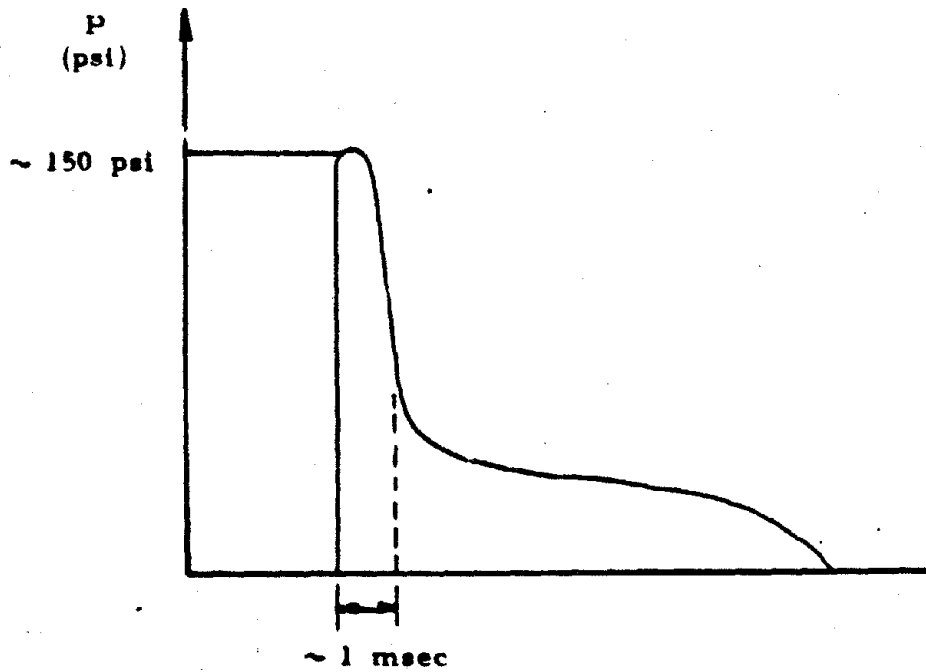


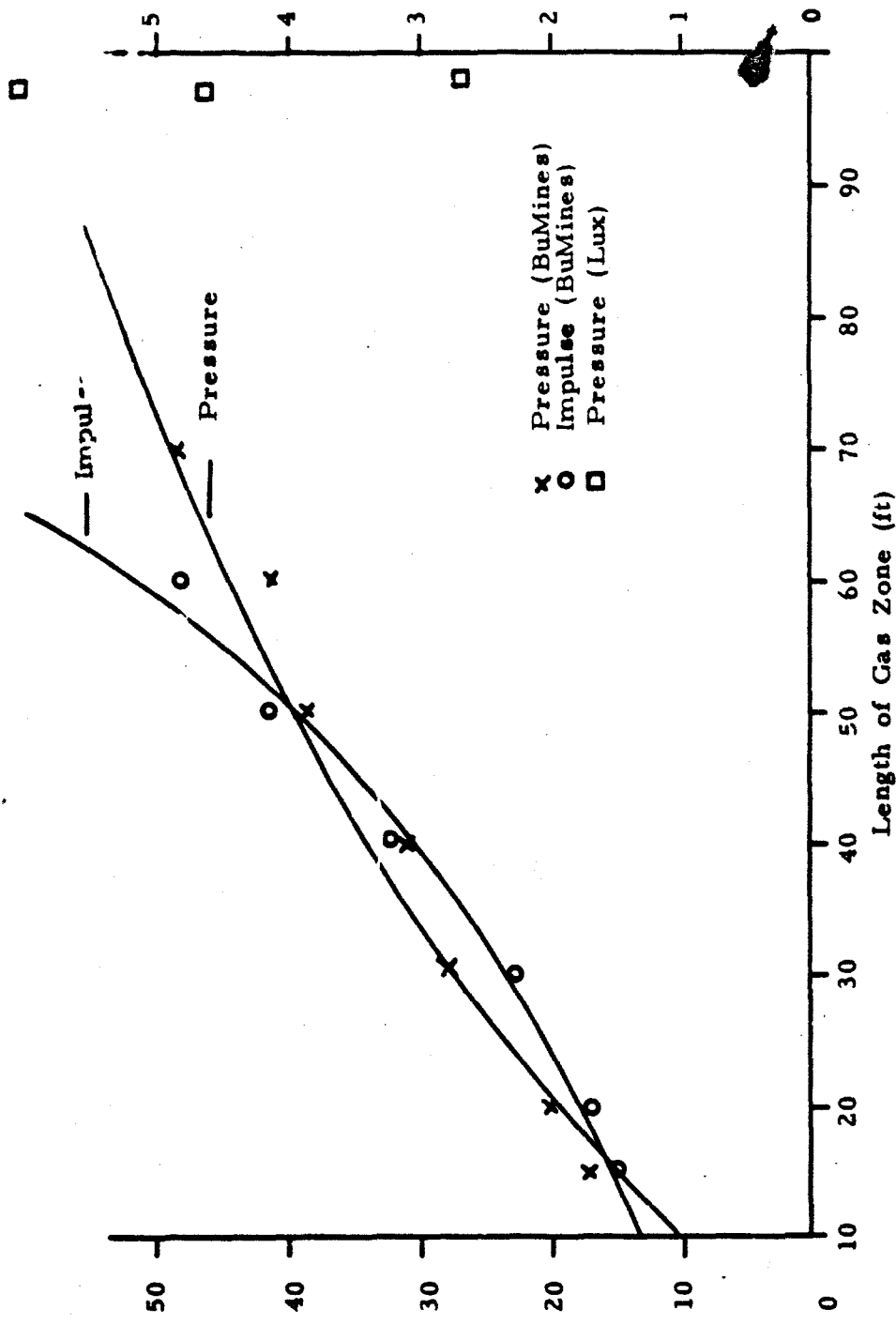
Figure A.5 - Typical Detonation Pressure Wave

A.3 Experimental Measurements of Pressure Waves in Mine Passages

The two principle sources of experimental data are the U. S. Bureau of Mines and European Community for Coal and Steel. Both of these organizations have carried out experimental programs in which methane-air explosions were artificially generated in a mine passage. These experiments typically involve filling a portion of a mine passage, sealed with a plastic diaphragm, with a mixture of methane and air. The mixture is then ignited with a spark or small powder explosion, and resulting pressure pulses are measured at points along the passage.

The only available pressure records are presented by Mitchell and Nagy.⁽³⁹⁾ These records show pressure pulses with peaks from 1 psig to 15 psig, and durations of about one second. The results are for a 25' gas zone. These investigators measured pressures up to 35 psi for a 50' gas zone. In another study by the Bureau of Mines, Kawenski and Bercik⁽⁴⁰⁾ demonstrated that a gas zone confined at a face produces both the highest and lowest explosion pressures. When the source of ignition is outby the gas zone (see Figure A.1) the pressure developed at the face was less than 1 psi (simple deflagration). When the source of ignition was at the face (see Figure A.2) the pressure developed at the face was 39 psi (accelerating deflagration). They also show that gas bodies located away from the face produce lower pressure extremes and are less sensitive to the location of the ignition source.

Tests have been conducted by the U. S. Bureau of Mines and the European Community for Coal and Steel⁽⁴¹⁾ to measure the pressures on bulkheads due to controlled methane-air explosions. These results are shown in Figure A.6. These are reflected pressures, and hence are twice the magnitude of the pressure pulse travelling down the passage. Hence the maximum pressure peak observed is 30 psig. If the pressure pulse is assumed to be a shock with a linear decay, the impulse values imply a pulse duration of approximately 1/4 of a second.



Pressure and Impulse on Bulkhead versus Length of Gas Zone
 Figure A.6

A.4 Effects of Mine Passage Geometry

The influence of intersections and turns on a shock wave propagating in a continued tunnel has been studied by Taylor.⁽⁴³⁾ Two significant observations can be gleaned from this study. First, a shock wave is attenuated by a factor of about 0.8 for every intersection or cross-cut that it passes. Thus, if an explosive shock passes three cross-cuts before reaching the rescue chamber, its peak pressure has been halved in the process. A second observation is that in the neighborhood of a cross-cut, there are high local reflected pressures at short distances down the cross-cut. Consequently, it would be desirable to build a bulkhead flush against the cross-cut.

A.5 Influence of Pressure Pulse Characteristics on the Structural Design of the Bulkhead

The available information discussed here indicates that a 20 psi peak over pressure design should be sufficient to resist the side-on pressure shock produced by a methane-air explosion. This limit will be strained somewhat if the bulkhead is directly in, or in the neighborhood of an explosion. Pressures up to 30 psi have been measured for these conditions.

The duration of the pressure pulse is on the order of seconds or fractions of a second. This duration will be much greater than the response time of a bulkhead structure. Consequently, the bulkhead will respond to the pressure pulse as if it were a step loading.

APPENDIX B

SUMMARY OF EXPLOSION TEST RESULTS

B-i

TEST SUMMARY

Plain Concrete (1:2:4)

Material	Agency	Date	Passage Cross Section	Stopping Thickness	Recesses	Appurtenances	Type of Explosion	Maximum Pressure on Stopping (psi)	Test Results
	USBM (Bureau of Standards)	1926	4' x 4'	8 in	none	none	static pressure	10	failed
				8 in	restrained at ribs	none	black powder enclosed	107, 146	withstood 107, failed at 146
				12 in	unrestrained	none	static pressure	23	failed
$\sigma_c = 2500$ psi	USBM (Experimental mine) Stopping #2	1928-1930	8' x 7'	19.2 in	recessed 18" in each rib	none	black powder enclosed	up to 148	remained intact
				12 in	recessed 18" in each rib	none	black powder enclosed	up to 125 psi	remained intact
$\sigma_c = 3300$	USBM (Experimental mine) Stopping #3A	1928-1930	8' x 7'	12 in	recessed 4" in each rib	none	black powder enclosed	60	cracks developed
							69	cracks grew	
								86	stopping disintegrated
$\sigma_c = 3300$	USBM (Experimental mine) Stopping #4	1928-1930	8' x 7' 2"	12 in	recessed 6" in each rib	none	black powder enclosed	42, 98, 100	at 100 psi, spalling occurred, leakage occurred at the ribs, and a large vertical crack developed
				12 in	recessed 12" in each rib	none	black powder enclosed	175, 26.5, 35.5, 41, 55, 45	at 55 psi, leakage occurred at roof; and cracks at the edge of the inby face and the center of the outby face indicated that strength had been exceeded. In the next test (45 psi), bulkhead failed completely
$\sigma_c = 2800$	USBM (Experimental mine) Stopping #6	1928-1930	12' 8" x 6' 10"	9.5"	recessed 6" at each side	none	black powder enclosed	22	bulkhead failed, forming central crack
$\sigma_c = 3200$				19.5"	recessed 7.7" at each rib	none	black powder enclosed	20, 36, 50, 55	failed at 55 psi

TEST SUMMARY

Masonry and Composite Bulkheads

Material	Agency	Date	Passage Cross Section	Stopping Thickness	Recesses	Appurtenances	Type of Explosion	Maximum Pressure on Stopping (psi)	Test Results
Masonry	FCCS (King Ludwig Mine)	1964-1965	43 ft ²	2.5 ft	none	none	coal dust	27.4	both bulkheads withstood all explosives
			135 ft ²	2.5 ft				19.2	
	ECCS (Hagenbeck Mine)	1965-1968	134.5 ft ²	2.5 ft (3 layer stone)	none	none	coal dust side on	>24	survived 19.5 psi, failed at >24 psi
			134.5 ft ²	1.7 ft (2 layer stone)	none	none	coal dust	25	survived 15 tests up to 25 psi; failed No. 17 at 18 psi
			61.5 ft ²	1.7 ft (2 layer stone)	none	3.3 ft x 6.6 ft steel door	coal dust	22.5	survived one test
	ECCS ⁽⁵⁾ (Kaiserstuhl Mine)	1965-1968	107.6 ft ²	2.5 ft (3 layer stone)	none	2.94' x 5.24' steel door	coal dust	30.5	shifted slightly
				1.7 ft (2 layer stone)	none	steel door	coal dust	≤23	withstood 11.6, failed at 24
				1.3 ft	recessed 0.65' into ribs; 1.0' into floor	none	methane-air from 250' side-on	≤48.5 psi 7.1 psi-sec	withstood 9 blasts up to 48.5 psi
Masonry (Concrete blocks)	USBM ⁽³⁾ (Experimental Mine)	1970-1971	14.5 ft x 6.5 ft	1.3 ft		none	methane-air from 250' side-on	≤48.5 psi 7.1 psi-sec	withstood 9 blasts up to 48.5 psi
					none	none	coal dust	≤11.6	outby wood brattice destroyed at 11.6 psi
					none	none	coal dust	≤62	up to 41 psi, inby wall had slight deformation destroyed at 62 psi
Composite Wall	ECCS ⁽⁵⁾ (Tremonia mine) wood-rock dust-masonry masonry-rock dust-masonry	1965-1968	9 ft x 9.5 ft	masonry 0.8 ft rock dust 18 ft	none	none	coal dust	≤11.6	outby wood brattice destroyed at 11.6 psi
				masonry 0.8 ft rock dust 18 ft	none	none	coal dust	≤62	up to 41 psi, inby wall had slight deformation destroyed at 62 psi
				masonry 1.7 ft rock dust 18 ft	none	none	coal dust	≤319	60.7, 151 psi without damage; slight deflection of inby wall at 100 psi; 0.65 ft deflection of inby wall at 119 psi
	ECCS ⁽⁵⁾ (Dorstfeld Mine) masonry-rock dust-masonry	1964-1965	240 ft ²	masonry - 2.5 ft rock dust - 4 feet masonry - 2 feet	none	27" dia. pipe	methane-air at 250 ft	4, 36, 19, 49	bulged 8' in center at 36 psi - moved an additional 2' at 49 psi

TEST SUMMARY

Gypsum

Material	Agency	Date	Passage Cross Section	Stopping Thickness	Recesses	Appurtenances	Type of Explosion	Maximum Pressure on stopping (psi)	Test Results				
Plaster of Paris w/s = .5	ECCS (Tremonia) ⁽¹⁾	1961	86 ft ²	4.9 ft	none	none	methane-air	55	bulkhead undamaged				
								71	bulkhead undamaged				
w/s = .75	ECCS ⁽²⁾ (Tremonia)	1965-1968	9 ft x 9.5 ft	10.8 ft	none	none	methane-air	29, 54 23	bulkhead undamaged				
w/s = .5	ECCS ⁽³⁾ (Dorstfeld)	1964-1965	237 ft ²	8.2 ft	none	none	methane-air	4	bulkhead undamaged				
								47	failed after 0.7 sec				
w/s = .75	ECCS ⁽⁵⁾ (Dorstfeld)	1964-1965	237 ft ²	13.1 ft	none	none	methane-air	30	shattered at top, then repaired				
w/s = .68	ECCS ⁽⁵⁾ (Kaiserstuhl)	1965-1968	151 ft ²	12.5 ft	none	27" dia. ventilation tube		64	completely destroyed				
								18, 8	no damage				
w/s = .35 (with retarder)	USBM ⁽³⁾ (Experimental Mine)	1970-1971	9 ft x 6.5 ft	2 ft	none		methane-air 250' from bulkhead side-on	9 explosions maximum 48.5 psi 7.1 psi-sec	no visible damage. A.F. leakage not affected				
				3 ft	none								
				4.6 ft	none								
Hardstem	NCB ^(11, 14) (Weiden Mine)	1965	10 ft x 8 ft	10 ft	none	27" dia. vent tube - small instrument tubes	black powder and coal dust 30' contained chamber	76 psi	no damage to bulkhead ventilation tube hatch developed leaks				
										3 ft	none		
Saarflit	ECCS (Tremonia) ⁽⁵⁾	1965-1968	9 ft x 9.5 ft	3 ft	none	none	methane-air	16, 58, 217, 275	damaged at 275 psi				
								220 ft ²	5 ft	none	methane-air	70	no damage
Anhydrite w/s = .36	EMC ⁽¹²⁾	1968	108 ft ²	3.3 ft	yes (22 ft ²)	none	methane-air 41' chamber in front of bulkhead	6 explosions from 21.6 to 261.5	seal remained intact				
										194 ft ²	3 ft	no	methane-air coal dust

TEST SUMMARY

Cementitious and Miscellaneous Materials

Material	Agency	Date	Passage Cross Section	Stopping Thickness	Recesses	Appurtenances	Type of Explosion	Maximum Pressure on stopping (psi)	Test Results
<u>Fly Ash-Cement</u> 62% fly ash 25% cement 31% water	(3) USBM (Experimental Mine)	1970- 1971	4.5 ft x 6.5 ft	2.9 ft, 4.6 ft	none	none	methane-air 260 ft from bulkhead side-on	9 explosions up to 48.5 psi 7.1 psi-sec	vertical crack occurred at 45 psi otherwise no damage
<u>Rock Dust-Cement</u> 48.5 tons Fly Ash 7.5 tons Cement	ECCS(7)	1961	86 ft ²	19.7 ft	none	none	methane-air 260 ft from bulkhead	28, 33, 57.	failed tested a few hours after poured undamaged, tested after a few days undamaged - tested after 3 weeks
<u>Reinforced Concrete</u> 1600 c c' 4320 psi	USBM(16) (Experimental Mine) Stopping 4'	1923	4 ft x 7 ft	8.5 in.	1 ft in each rib	none	black powder in closed chamber	34,70, 297	full length vertical crack
	USBM(16)	1926	4 ft x 4 ft	8 in.	entire perimeter recessed 1/2 ft stumply supported	none	black-powder closed chamber	46 76	average maximum pressure resisted average pressure causing failure
<u>Sandbags</u> (late sacks)	ECCS(7)	1961	65 ft ²	8.2 ft	none	none		20	maximum safe pressure determined from earlier tests
	ECCS(7) (Tremonia Exp. Mine)	1961	86 ft ²	20 ft	none	27" dia. vent tube	methane air from 260 ft	43	seal moved 8" and leakage occurred at the top
<u>Brattice and Rock Dust</u>	ECCS(7)	1961	86 ft ²	11 ft	none	none	methane air from 260 ft	14 and 28	bulkhead disintegrated at both pressures

*Number in parentheses indicates reference; name in parentheses indicates mine.

APPENDIX C

DESIGN CALCULATIONS

C.1 Truss Bulkhead (See Figure D.2)

The truss has been designed to operate entirely within the elastic range. Each bay is assumed to be loaded instantaneously by the side-on pressure wave. Consequently a 40 psi static load is the principal design requirement. From a dynamic point of view, the truss, with corrugated sheet, ribs, posts, and struts, is a multi-degree-of-freedom system. To simplify the analysis, however, we can take advantage of the fact that the natural periods of the individual components are different enough to permit the uncoupling of their dynamic responses, and hence each component can be designed statically based on 40 psi.

C.1.1 Overall Equilibrium (See Figure C.1)

$$\text{Bay width} = 27''$$

$$\text{Load/truss} = 27'' \times 78'' \times 40 \text{ psi} = 84,300 \text{ lbs.}$$

Since it is indeterminate as to how much of the horizontal load is carried by R_1 and R_3 , we shall assume two extreme cases:

Case I - Horizontal shear failure of rear footing

$$(R_1 = 0, R_3 = 84,300, R_4 = 45,600)$$

Moments about C (Figure C.1(c))

$$84,300(39-25) + P_{AB}(49.5) = 84,300 \times 55$$

$$P_{AB} = \frac{84,300 \times 41}{49.5} = 69,824 \text{ lbs. tension}$$

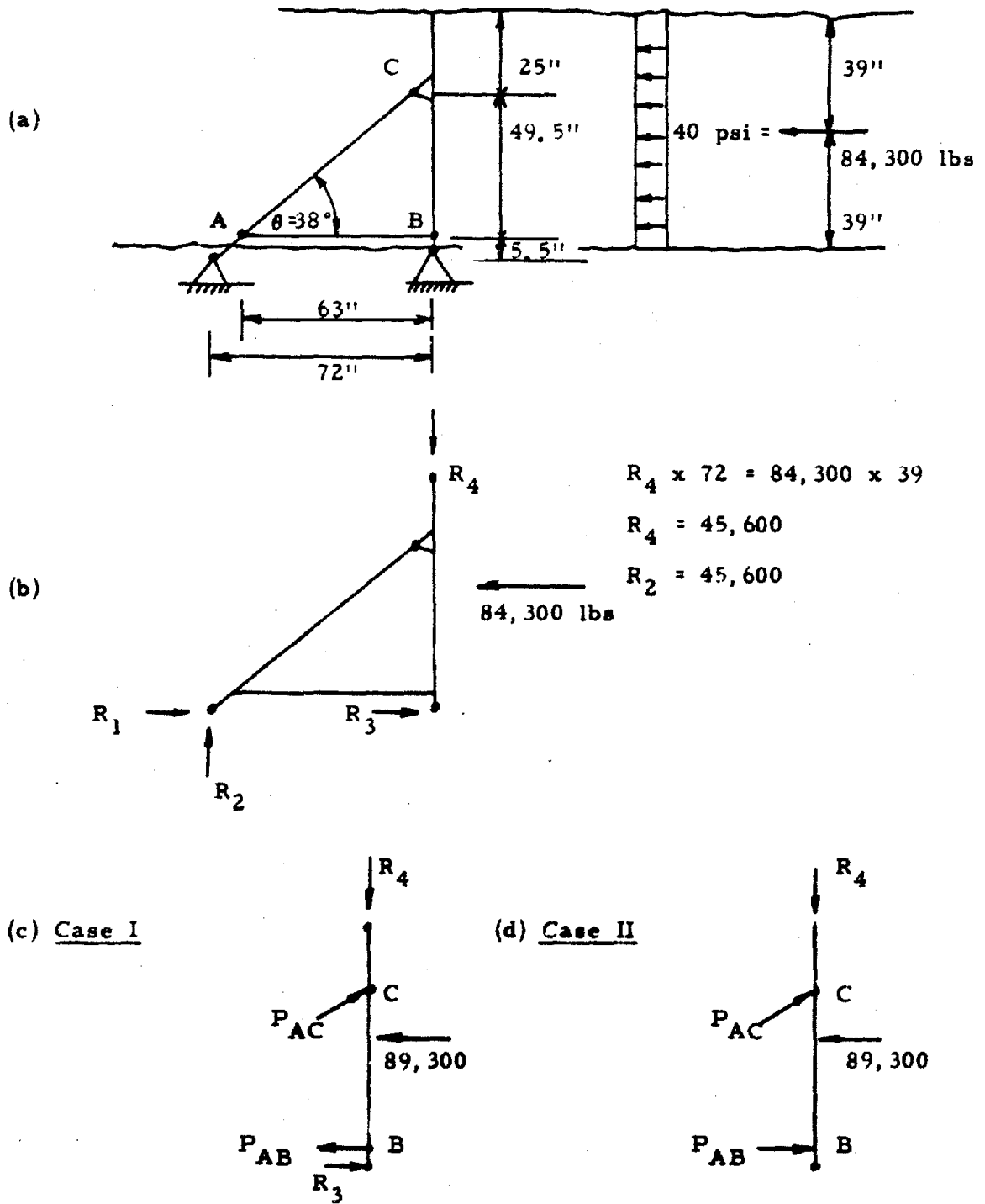


Figure C.1 - Truss Static Analysis - Overall Equilibrium

Vertical Equilibrium

$$P_{AC} \sin \theta = 45,600$$

$$P_{AC} = 73,807 \text{ lbs compression}$$

Case II - Horizontal shear failure of front support

$$(R_3 = 0, R_1 = 89,300, R_4 = 45,600)$$

Moments about C (Figure C1(d))

$$84,300 \times 14 = P_{AB} \times 49.5$$

$$P_{AB} = 23,842 \text{ lbs compression}$$

$$P_{AC} = 73,807 \text{ lbs compression}$$

(same as case I)

Maximum resultant load at rear support

$$\begin{aligned} &= \sqrt{R_1^2 + R_2^2} \\ &= \sqrt{(84,300)^2 + (45,600)^2} \\ &= 95,800 \text{ lbs} \end{aligned}$$

C.1.2 Bolt Sizes

Front and rear supports

Use 1-1/2" diameter Grade 5 bolt minimum yield stress
in tension = 74,000 psi (37,000 psi in shear)

Total load carried in double shear

$$F = \frac{\pi d^2}{4} \times 2 \times 37,000 = 130,768 \text{ lbs (vs. 95,800 reqd.)}$$

hence, factor of safety

$$F.S. = \frac{130,768}{95,800} = 1.37$$

At A and B

Use 1-1/4" diameter Grade 5 bolts. Total load carried in double shear

$$F = \frac{\pi d^2}{4} \times 2 \times 37,000 = 90,811 \text{ lbs}$$

(vs. maximum P_{AB} of 69,824)

$$F.S. = \frac{90,811}{69,824} = 1.30$$

At C

Use 1-1/4" diameter Grade 5 bolt. Load capacity (see above)
= 90,811 lbs (vs. $P_{AC} = 73,807$)

$$F.S. = \frac{90,811}{73,807} = 1.23$$

Pins for Post Slide (2)

Use 1" diameter Grade 5 bolt.

$$F = \pi d^2/4 \times 2 \times 37,000 \times 2 = 116,000 \text{ lbs}$$

(vs. $R_4 = 45,600$ required)

$$F.S. = \frac{116,000}{45,600} = 2.54$$

C.1.3 Bearing Areas

The required bearing thickness at bolted connections in double shear is computed as;

$$t = \frac{P}{\sigma_y \times d \times 2}$$

where

- P = bolt load in lbs
- σ_y = yield stress of bearing material
- d = bolt diameter
- t = required bearing thickness

A yield failure in bearing is not critical, since the load capacity increases when such a failure occurs. Consequently, low factors of safety will be acceptable.

Footing box (A36 steel, $\sigma_y = 36,000$ psi)

$$t = \frac{95,800}{36,000 \times 1.5 \times 2} = .89''$$

(1.00" provided)

Front Post ($\sigma_y = 46,000$ psi)

at footing box

$$t = \frac{84,300}{46,000 \times 1.5 \times 2} = .61''$$

(vs. .75" provided)

at B

$$t = \frac{69,824}{46,000 \times 1.25 \times 2} = .61''$$

(vs. .625 provided)

at C (Mounting Plates - AISI 1018 cold finished, $\sigma_y = 54,000$)

$$t = \frac{73,807}{54,000 \times 1.25 \times 2} = .55''$$

(.75" provided)

at pin for vertical slide (2 pins)

$$t = \frac{46,500}{46,000 \times 1.00 \times 2} = .505'' \text{ (.25 per pin)}$$

(.625" available per pin)

Vertical Slide

at pins

$$t \text{ (required)} = .25'' \text{ (see above)}$$

(.44" available)

Horizontal Ties

at A and B (2 ties)

$$t = \frac{69,824}{46,000 \times 1.25 \times 2 \times 2} = .30''$$

(.44" available)

Diagonal Strut (bearing pads, A36 steel, $\sigma_y = 36,000$ psi)

at rear support

$$t = \frac{95,800}{36,000 \times 1.5 \times 2} = .89''$$

(1.25" available)

at horizontal tie

$$t = \frac{69,824}{36,000 \times 1.25 \times 2} = .77''$$

(1.0" available)

C.1.4 Horizontal Ties (A500 steel)

3 x 2 x 3/16 rectangular tubing

Area = 1.64 sq. in

Minimum Moment of Inertia = .976 in⁴

Tensile Load

$$\sigma_t = \frac{69,824}{1.64 \times 2} = 21,287$$

$$F.S. = \frac{46,000}{21,287} = 2.15$$

Area at bolted connection

$$\begin{aligned} &= \text{tube section} - \text{bolt holes} + \text{bearing plates} \\ &= 1.64 - 2 \times 1.25 \times 3/16 + 2 \times (3.0 - 1.25) \times .15 \\ &= 1.64 - .47 + .875 = 2.04 \text{ in}^2 \end{aligned}$$

Therefore, the tensile stress at the bolted connection is less than the tensile stress throughout.

Compressive Load

Consider buckling load, F_b , per tie

$$F_b = \frac{\pi^2 EI}{L^2} = \frac{\pi^2 \times 30 \times 10^6 \times .976}{632} = 72,772 \text{ lbs}$$

The actual load on both ties is 23,842 lbs, or 11,921 lbs per tie.

C.1.5 Front Rails (2 x 2 x 3/16 tubing, A500 steel)

$$w = \text{loading} = 40 \text{ psi} \times 13.5'' = 540 \text{ lbs/in}$$

$$\text{Section modulus, } S, = .75 \text{ in}^3$$

$$\text{Moment} = \frac{wl^2}{8} = \frac{540 \times 23^2}{8} = 35,707 \text{ in lbs}$$

Maximum stress

$$\sigma = \frac{M}{S} = \frac{35,707}{.75} = 47,610$$

slight yielding will occur.

(σ_y = yield stress = 46,000)

Ultimate plastic moment

$$\begin{aligned}M_u &= \sigma_y \times 3/16 \times 2 \times (11 - 3/32) \\ &+ \sigma_y \times 3/16 \times (2-3/8)^2 \times 1/4 \times 2 \\ &= 46,000 (.68 + .24) = 42,668 \text{ in lbs}\end{aligned}$$

Since the actual moment is well below the ultimate moment, the amount of yielding which will occur is insignificant

$$\text{F.S.} = \frac{42,668}{35,707} = 1.2$$

Rail Rebound Capacity

weldment of decking

assume weld thickness = thickness of decking = 0.0395";

weld strength = 11,000 psi

Capacity of Welded Connection = 0.875" x 2 x 0.0395 x
11,000 = 760 lb.

Loading of Welded Connection = 4.5" x 5 psi x 13.5" =
303.75 lb.

Rebound Capacity = $\frac{760}{303.75} \times 5 = 12.43$ psi

Bolting of Rib to Post

use 3/8" ϕ bolts, grade 1 (33,000 psi proof)

Bolt Load Capacity = 0.11 x 33,000 = 3,644 lbs

Loading = 27 x 13.5 x 5 = 1,822 lbs

Rebound Capacity = $2 \times \frac{3,644}{1,822} \times 5 = 20$ psi

C.1.6 Corrugated Decking

$$M = \frac{pl^2}{8} = \frac{40 \times 12.5^2}{8} = 911 \text{ in-lbs/in.}$$

$$S = 0.017 \text{ in}^3/\text{in} \text{ (Manufacturers data)}$$

$$\text{Yield Stress} = 80,000 \text{ psi (Manufacturers data)}$$

$$\sigma = \frac{M}{S} = \frac{911}{0.017} = 53,600 \text{ psi}$$

$$\text{Pressure Capacity} = \frac{80,000}{53,600} \times 40 = 60 \text{ psi}$$

C.1.7 Trench Support

The principles of trench support capacity have been discussed in detail in Section 3.2.2. Using these principles, the required shear strength of the footing rock material is

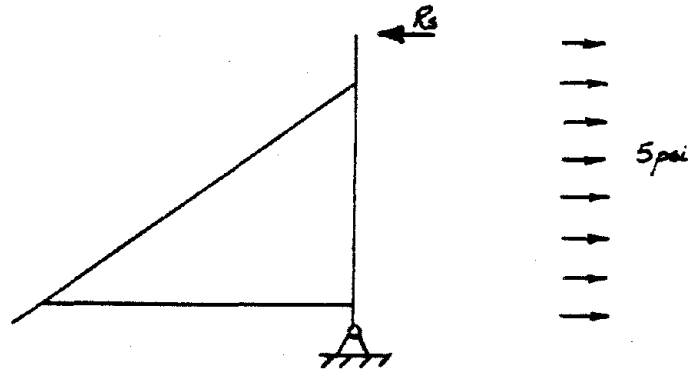
$$\tau = \frac{F}{\pi d}$$

where F is the horizontal reaction force per inch and d is the depth of the trench. For the worst case loading, $F = 84,300/24 = 3,512 \text{ lbs/in.}$ We have specified a minimum 7" deep trench. Hence the required shear strength is

$$\tau \geq \frac{3,512}{7\pi} = 160 \text{ psi}$$

(or $p \geq 7.4 \tau = 1184 \text{ psi}$)

C.1.8 Roof Stud



Assuming that the footing box has no tensile resistance, then the roof stud carries a horizontal reaction due to rebound.

$$R_s = 27 \times \frac{78}{2} \times 5 = 5,265 \text{ lb.}$$

The section of the stud in the roof above the roof bearing plate acts as a roof bolt in shear. A 1-1/4" stud has an approximate inside diameter of 1-1/8" bolt, the capacity is a minimum of 15,000 lb. (See Section 3.2.2.)

The section of the stud below the roof bearing plate tightening nut (maximum 1.6") is subjected to a bending moment

$$M = \frac{R_s l}{2} = 5,265 \times \frac{1.6}{2} = 4,212 \text{ in. lbs.}$$

Hence

$$\sigma = \frac{M}{S} = \frac{4,212}{0.098 d^3} = \frac{4,212}{0.098 \times 1.125^3} = 30,185$$

stud proof stress = 75,000 psi

$$\text{rebound capacity} = \frac{75,000}{30,185} \times 5 = 12.42 \text{ psi}$$

C.1.9 Front Post (4" x 8" x 1/4" rectangular tubing,
A500 steel)

Assume rail load is uniform

$$w = 40 \times 27 = 1,080 \text{ lbs/in.}$$

$$M = \frac{wl^2}{8} = \frac{1,080 \times 54^2}{8} = 393,660 \text{ in. lbs.}$$

$$\sigma = \frac{M}{S} = \frac{393,660}{11.06} = 35,600 \text{ psi}$$

The post has 46,000 psi yield, hence

$$\text{Pressure Capacity} = \frac{46,000}{35,600} \times 40 = 52 \text{ psi}$$

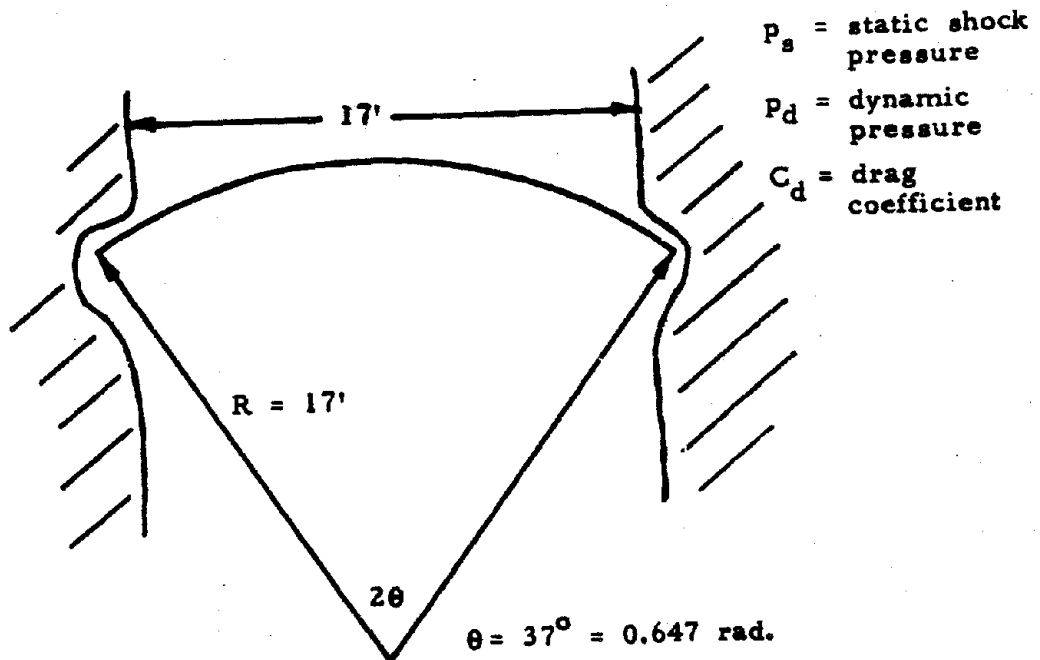
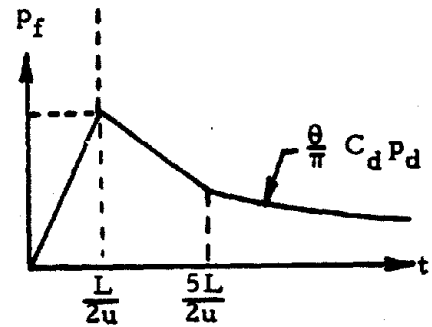
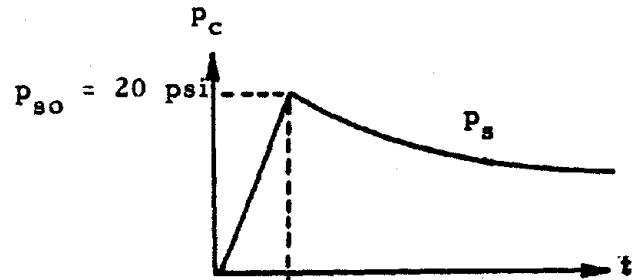
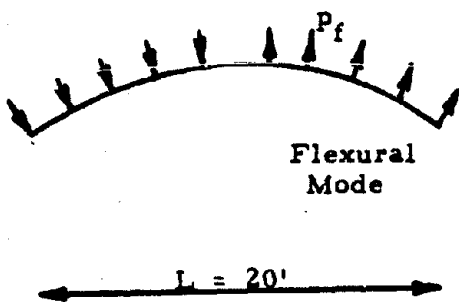
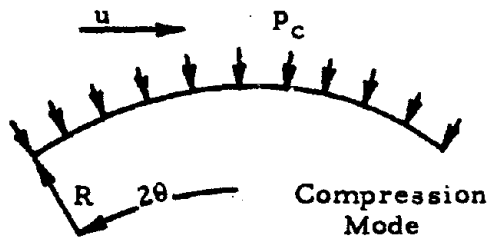
$$\text{Deflection } \delta = \frac{5}{384} \frac{Wl^4}{EI} = \frac{5}{384} \times \frac{40 \times 54^4 \times 27}{30 \times 10^6 \times 45} = 0.1''$$

C.2 Liner Plate Arch (see Figure D.3)

The passage time of the pressure wave across the face of the arch is a principal consideration in its design. The lengthened full pressure rise time lowers the equivalent static pressure load for stability consideration. On the other hand, the unsymmetric loading nature caused considerable bending, which ultimately governs the design. A standard design procedure for arches subjected to this type of loading has been employed (see Reference 2).

Main Arch Components

24" wide, 5 ga Liner Plates
(Commercial Shearing)
6WF20 Ribs
6M 16.3 Channel Endpost



Liner Plate Arch - Definition of Loadings⁽²⁾

Figure C.2

C.2.1 Computation of Design Pulses

$$P_{fm} = \left(0.5 + \frac{\theta}{\pi}\right) P_{so} = \left(0.5 + \frac{0.647}{\pi}\right) \times 20 = 14.1 \text{ psi}$$

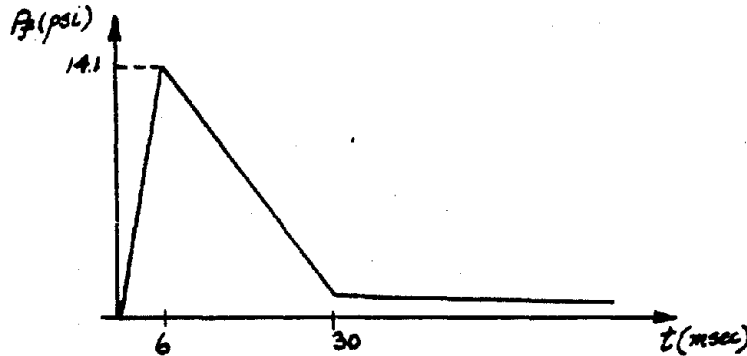
$$u = 1,120 \text{ ft/sec} \times \left(1 + \frac{6 F_{so}}{103}\right)^{1/2} = 1,650 \text{ ft/sec.}$$

$$\frac{L}{2u} = \frac{20}{3,300} = 0.006 \text{ sec} = 6 \text{ msec}$$

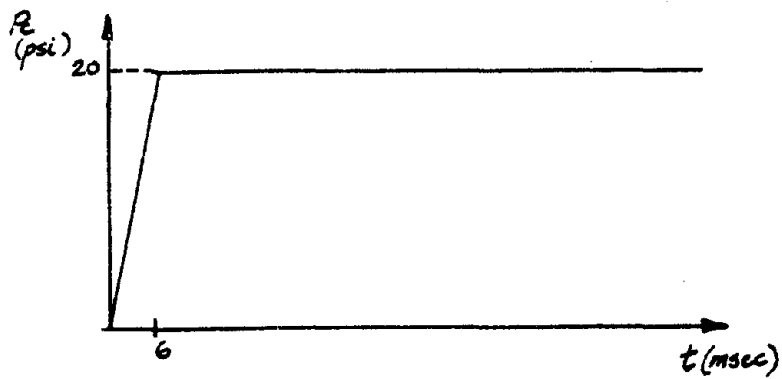
The peak dynamic pressure is strictly a function of p_{so} . For $p_{so} = 20 \text{ psi}$, $(\rho_d)_{\max} = 8$ (see Figure 7.2, reference 2). The value of p_f at $t = \frac{5L}{2u}$ should be something less than

$$\begin{aligned} (\rho_d)_{\max} \times \frac{\theta}{\pi} \times C_d & \quad C_d = \text{drag coefficient} \approx 0.4 \\ & = 8 \times \frac{0.6466}{\pi} \times 0.4 = 0.66 \end{aligned}$$

hence, p_f looks like



Since we know little about the pressure time pulse, we will conservatively assume the following for p_c .



C.2.2 Response to P_{fm}

Consider 1/2 of the arch as a simple supported beam

$$\text{Length} = R\theta = 17 \times 0.6466 = 11' = 132''$$

correction for crown displacement.

$$C = \frac{\left(\frac{\pi}{0.1466}\right)^2 + 1.5}{\left(\frac{\pi}{0.6466}\right)^2 - 1.0} = 1.11$$

Bending Section Properties

4 - 6WF20 ribs + 3 5 ga liner plates

$$I = 4 \times 41.7 + 3 \times 1.668 = 171.8 \text{ in.}^4$$

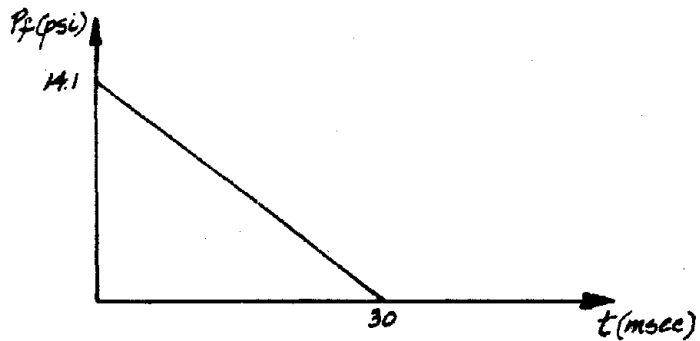
$$A = 4 \times 5.90 + 3 \times 6.10 = 41.9 \text{ in.}^3$$

$$S \approx \frac{171.8}{3.125} = 55 \text{ in.}^3$$

Natural Period (Beam Bending)

$$\begin{aligned} T_f &= 1.11 \times 2 \times \frac{l^2}{\pi} \times \sqrt{\frac{\rho A}{EI}} \\ &= 1.11 \times 2 \times \frac{(132)^2}{\pi} \times \sqrt{\frac{0.29 \times 41.9}{30 \times 10^6 \times 171.8 \times 386}} \\ &= 0.0303 \text{ sec} = 30.3 \text{ msec} \end{aligned}$$

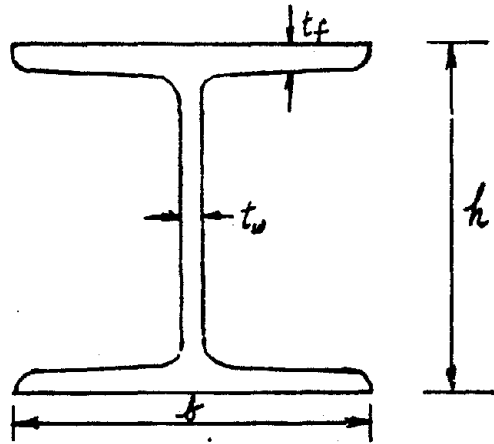
Assume the idealized load time curve



Referring to Figure 43

$$\frac{t_d}{T_f} = \frac{30.3}{30} \sim 1$$

The maximum resisting moment, M_p , available in bending is computed from the section properties of the ribs



$$M_p = \left(bt_f h + \frac{(h - 2t_f)^2}{4} t_w \right) \sigma_y$$

for each 6WF20 rib

$$t_f = 3/8''$$

$$h = 6-1/4''$$

$$t_w = 1/4''$$

$$w = 6''$$

$$\sigma_y \text{ (dynamic)} = 36,000 \times 1.2 = 43,200 \text{ psi}$$

$$M_p = 689,000 \text{ in. lbs.}$$

The uniform pressure equivalent to this moment is

$$p_m = \frac{4 \times M \times 8}{HL^2} = \frac{4 \times 689,000 \times 8}{77 \times (132)^2} = 16.4 \text{ psi}$$

Referring again to Figure 43

$$\frac{P_m}{P_f} \left(\text{equivalent to } \frac{R_M}{F} \right) = \frac{16.4}{14.1} = 1.16$$

Hence

$$\frac{y_m}{y_{el}} = 1.5$$

$$y_{el} = \frac{5}{384} \frac{P_f L^4}{EI} = \frac{5}{384} \times \frac{14.1 \times 132^4 \times 77}{30 \times 10^6 \times 171.8} = 0.83''$$

Total maximum arch deflection = $1.5 \times 0.83 = 1.24''$

Maximum seal deflection capacity = $3.00''$

Maximum estimated abutment deflection = $0.25''$

Maximum allowable arch deflection = $2.75''$, or $\frac{y_m}{y_{el}} = 3.31$

From Figure 43 $\frac{P_m}{P_f} = 0.8$

or

flexural pressure capacity = $\frac{16.4}{0.8} = 20.5 \text{ psi}$

C.2.3 Response to p_c

Natural Period in Compression

$$T_c = 2\pi \sqrt{\frac{R^2}{gE}} = 2\pi \sqrt{\frac{0.28 (17 \times 12)^2}{386 \times 30 \times 10^6}} = 6.3 \text{ msec}$$

referring to Figure 43

$$\frac{t_r}{T} = \frac{6}{6.3} = 0.95$$

for elastic design, $y_m/y_{el} = 1$

$$\frac{R_M}{F_1} = 1.1$$

hence we design for a static pressure of $20 \times 1.1 = 22$ psi

Stability (See Reference 13)

$$\begin{aligned} p_{cr} &= \frac{EI}{(1 - \nu^2) R^3 h} \left(\frac{\pi^2}{\theta^2} - 1 \right) \\ &= \frac{30 \times 10^6 \times 171.8}{(1 - 0.3^2) \times (17 \times 12)^3 \times 77} \left(\frac{\pi^2}{0.647^2} - 1 \right) = 195 \text{ psi} \end{aligned}$$

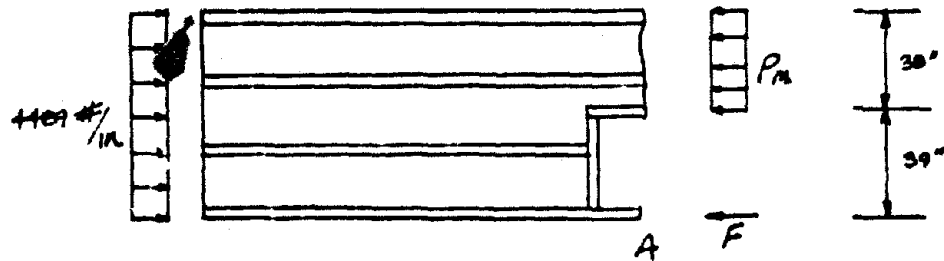
Axial Loads - At the wall

$$\sigma = \frac{p_c R h}{A} = \frac{22 \times 17 \times 12 \times 77}{41.9} = 8,250 \text{ psi}$$

$$\text{pressure capacity} = \frac{36,000}{8,250} \times 22 = 96 \text{ psi}$$

$$\text{Reaction Load} = 8,250 \times \frac{41.9}{77} = 4,489 \text{ lb/in.}$$

Axial Loads - Center Section



Moments about A

$$4489 \times \frac{77^2}{2} = P_m \times 38 \times (59 + 19)$$

$$P_m = 6,037 \text{ lbs/in.}$$

Force Balance

$$F + P_m \times 38 = 4,489 \times 77$$

$$F_A = 116,250$$

$$\sigma_A = \frac{116,250}{5.90} = 19,700 \text{ psi}$$

$$\text{Pressure Capacity} = \frac{36,000}{19,700} \times 22 = 40.2 \text{ psi}$$

C.2.4 Anchorage

Axial Load 4,489 lb/in.

width of abutment base - 20" min.

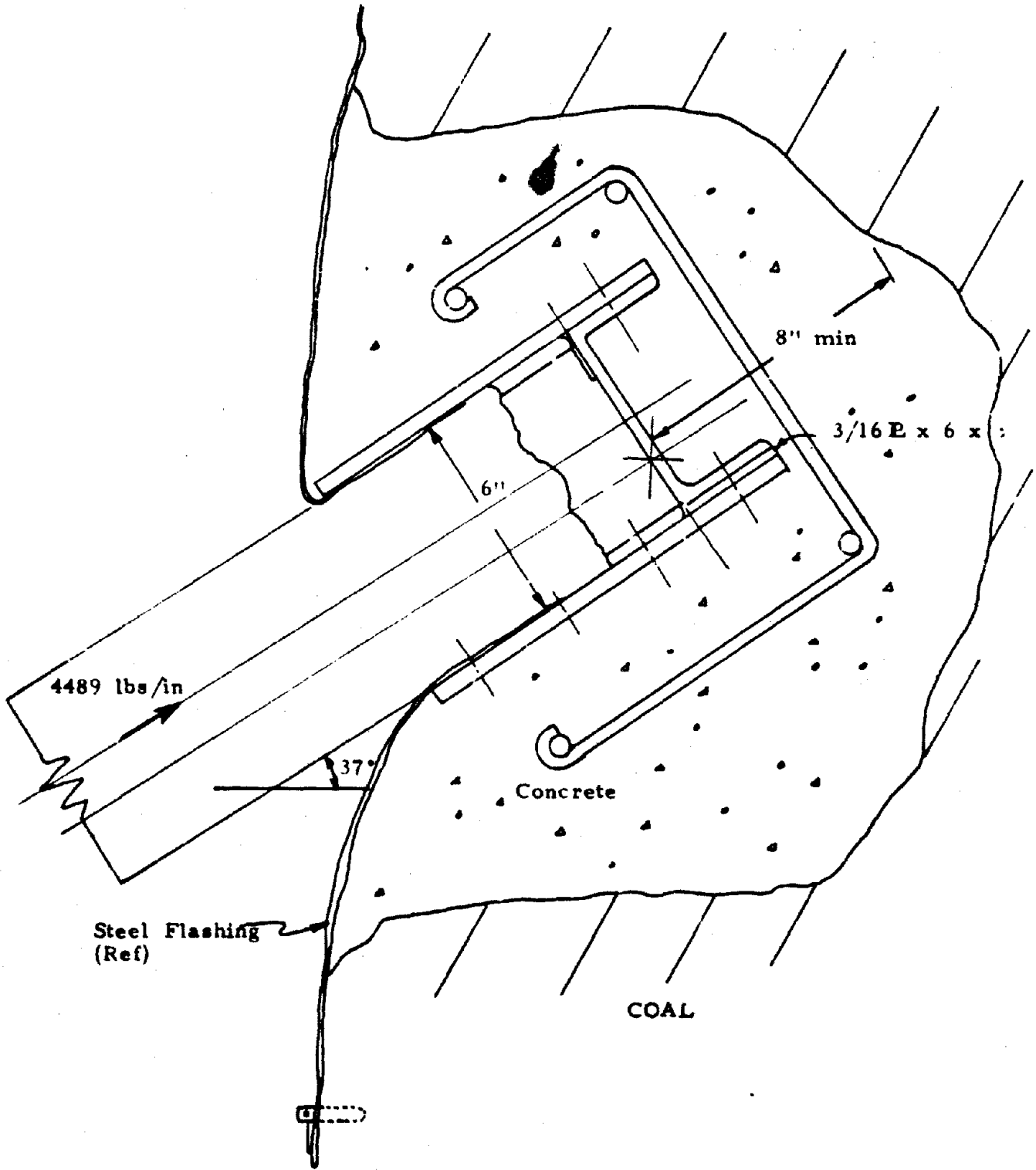


Figure C.3 - Arch Abutment

From the analysis presented in Section 3.2.2.3, with

$$\varphi = 37^\circ \quad \alpha = \pi/2 - 37^\circ = 53^\circ$$

$$P = 4489 \text{ lbs/n}$$

$$r = 8'' + d \quad (\text{see Figure C.3})$$

$$w = 6''$$

$$d = \frac{w}{\alpha + 1/2 \sin 2\alpha} = \frac{6}{.93 + 1/2 \times .96} = 4.25''$$

$$\begin{aligned} \sigma_{\max} &= \frac{P}{\alpha + 1/2 \sin 2\alpha} \times \frac{1}{r} \\ &= \frac{4,489}{1.41} \times \frac{1}{8 + 4.25} = 259 \text{ psi} \end{aligned}$$

(This compares to a minimum expected coal compressive strength of 500 psi)

$$F.S. = \frac{500}{259} = 1.92$$

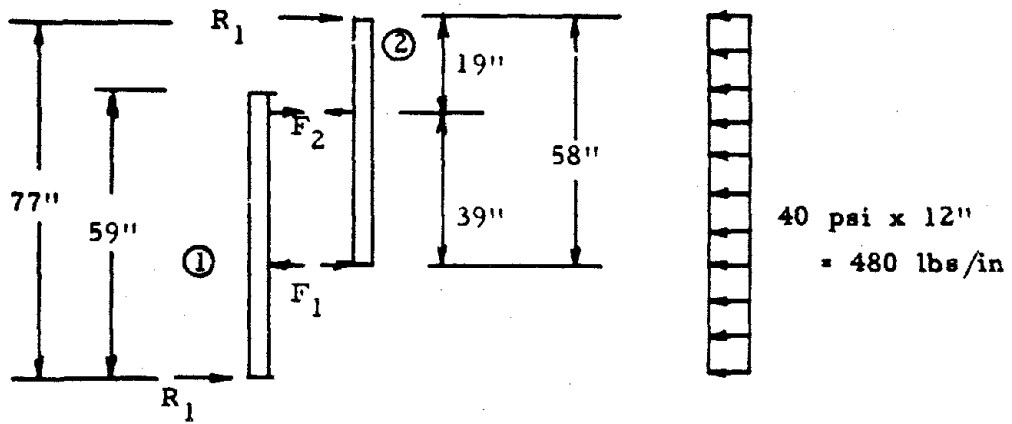
C.3 Channel Turnbuckle (see Figure D.4)

C.3.1 Channel Sections (see Figure C.4a)

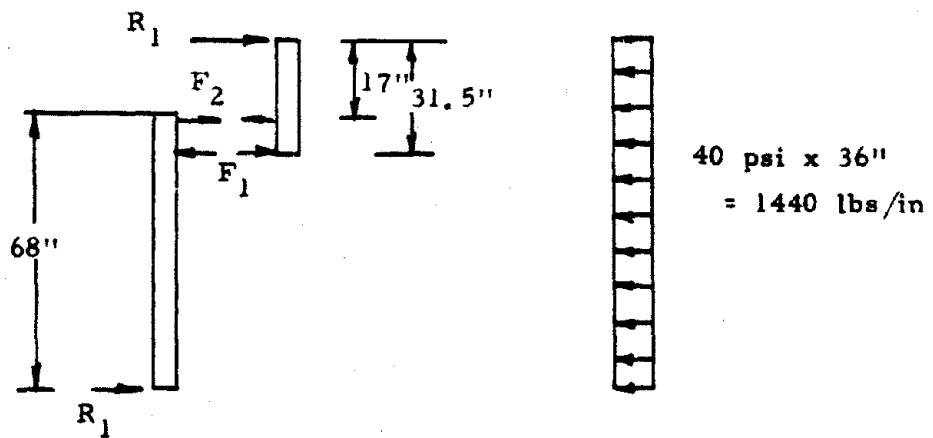
Top and bottom reactions

$$2R_1 = 480 \times 77$$

$$R_1 = 18,480 \text{ lbs}$$



(a) 12" Channel Sections



(b) 36" Door Section

Figure C.4 - Bending Moments and Bolt Loads in Channel-Turnbuckle Sections

Bolt Load (F_2)

$$39 F_2 + 480 \times (58)^2 / 2 - 58R_1 = 0$$

$$F_2 = 6782 \text{ lbs}$$

Contact Force (F_1)

$$F_1 + R_1 - F_2 - 480 \times 58 = 0$$

$$F_1 = 16,142 \text{ lbs}$$

Bending Moments

Upper channel at top bolt ②

$$M = -480 \times (39)^2 / 2 + 39 F_1 = 264,500 \text{ in lbs}$$

Lower channel at bearing point of upper channel ①

$$M = 39 F_2 = 39 \times 6782 = 264,500 \text{ in lbs}$$

Maximum Bending Stress (Channel Material is AL 6061-T6

$$\sigma_y = 35,000 \text{ psi})$$

Channel Properties

12" x 5" x 11.82 lbs/ft

$$\text{Section modulus} = 7.55 \text{ in}^3 = S$$

$$\text{Stress} = \frac{M}{S} = \frac{264,500}{7.55} = 35,033 \text{ psi (vs. 35,000 psi)}$$

The channels, however, have a large degree of reserve strength in plastic bending. The ultimate capacity is computed from the fully plastic moment, as follows:

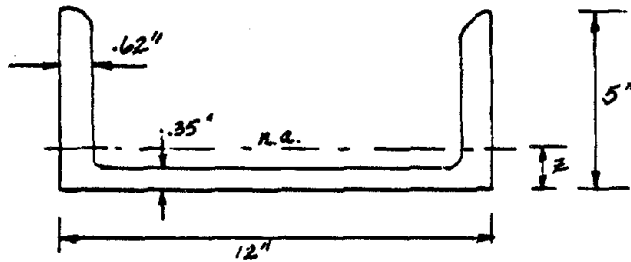


Figure C.5 - Channel Section

Equilibrium (see Figure C.5)

$$12 \times 0.35 + 2 (z - 0.35) \times 0.62 = 2 (5 - z) \times 0.62$$

$$z \approx 1.0$$

Ultimate Moment

$$\frac{M}{\sigma_y} = 12 \times 0.35 \times 1 + \frac{0.65^2}{2} \times 2 + 2 \times \frac{4^2}{2} \times 0.62$$

$$M = 509,000$$

Ultimate Pressure Capacity

$$= \frac{509,000}{264,500} \times 40 = 77 \text{ psi}$$

Elastic Deflection - upper channel

$$\delta = \frac{w x}{24 EI} (l^3 - 2l x^2 + x^3) + \frac{F_2 \text{ by}}{6 EI} (l^2 - b^2 - y^2)$$

where

$$\begin{aligned}w &= 480 \text{ lbs/in} \\b &= 19'' \\t &= 58'' \\I &= 25.53 \text{ in}^4 \\E &= 10 \times 10^6 \text{ psi} \\y &= 58 - x\end{aligned}$$

assume maximum deflection at mid height of the entry

$$x = 38.5'' \quad (y = 19.5)$$

$$\delta_{\max} = .24 + .07 = .31''$$

C.3.2 Door Section (see Figure C.4b)

$$R_1 = 3 \times 18,480 = 55,440 \text{ lbs}$$

$$R_1 \times 31.5 - F_2 \times 14.5 - 1440 \times (31.5)^2/2 = 0$$

$$F_2 = 71,168 \text{ lbs}$$

$$R_1 + F_1 - F_2 - 31.5 \times 1440 = 0$$

$$F_1 = 61,088 \text{ lbs}$$

Critical Bending Moments

$$M(x) = R_1 x - 1440 x^2/2$$

x measured up from the floor

mid door (x = 16'')

$$M = 55,440 \times 16 - 1440 \times (16)^2/2 = 702,720 \text{ in-lbs}$$

top of door (x = 32")

$$M = 55,440 \times 32 - 1440 \times (32)^2/2 = 1,036,800 \text{ in-lbs}$$

bottom of upper channels (x = 42)

$$M = 1,058,400 \text{ in-lbs}$$

Critical Section Properties (see Figure C.6)

Properties of stiffening channel

$$A = .38 \times 3.50 \times 2 + (7 - .35) .21 = 4.06 \text{ in}^2$$

$$I = 1/12 \times .21 \times 6.65^3 + 2 \times 3.5 \times .38 \times (3.5 - .19)^2 \\ = 34.29 \text{ in}^4$$

$$S = I/3.5 = 9.80$$

Section (a)

$$2 \times 4.06 \times d = 3/8 \times 36 (3.5 + .19 - d)$$

$$8.12 d = - 13.50 d + 49.82$$

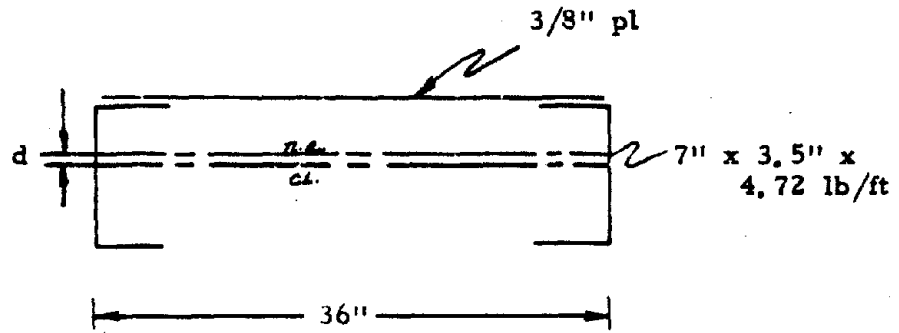
$$d = 2.30''$$

$$I = 2 \times 34.29 + 2 \times 4.06 \times (2.30)^2 + 3/8 \times 36 \times (3.69 - 2.3)^2$$

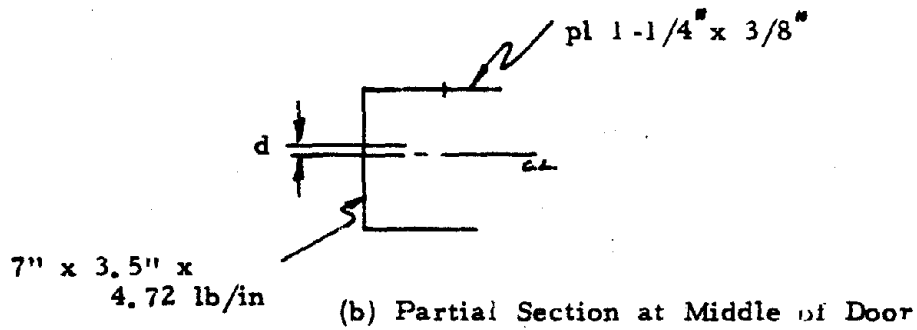
$$68.58 + 42.95 + 77.11 = 188.65 \text{ in}^4$$

$$S_a = \frac{188.65}{2.3+3.69} = 31.49 \text{ in}^3$$

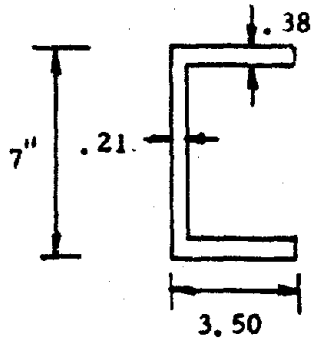
Composite Door Sections



(a) Full Section



(b) Partial Section at Middle of Door



(c) Properties of Channel

Figure C.6 - Composite Door Sections

Section (b)

$$4.06 d = 3/8 \times 1-1/4 \times (3.38 - d)$$

$$4.53 d = 1.58$$

$$d = .35$$

$$\begin{aligned} I &= 2 \times 34.29 + 2 \times 4.06 \times .35^2 + 2 \times 3/8 \times 1-1/4 \times (3.38-.35)^2 \\ &= 68.58 + .99 + 8.60 = 78.17 \text{ in}^4 \end{aligned}$$

$$S_b = \frac{78.17}{3.50 + .35} = 20.30 \text{ in}^3$$

Maximum bending stresses ($\sigma_y = 35,000$ psi)

at center of door

$$\sigma = \frac{M}{S_b} = \frac{702,720}{20.30} = 34,616 \text{ psi}$$

at top of door

$$\sigma = \frac{M}{S_a} = \frac{1,036,800}{31.49} = 32,925 \text{ psi}$$

at bottom of upper channel

$$\sigma = \frac{M}{S_a} = \frac{1,058,400}{31.49} = 33,610 \text{ psi}$$

Since each of these sections has considerable reserve moment capacity in the plastic range, these elastic stress levels are considered to be adequate.

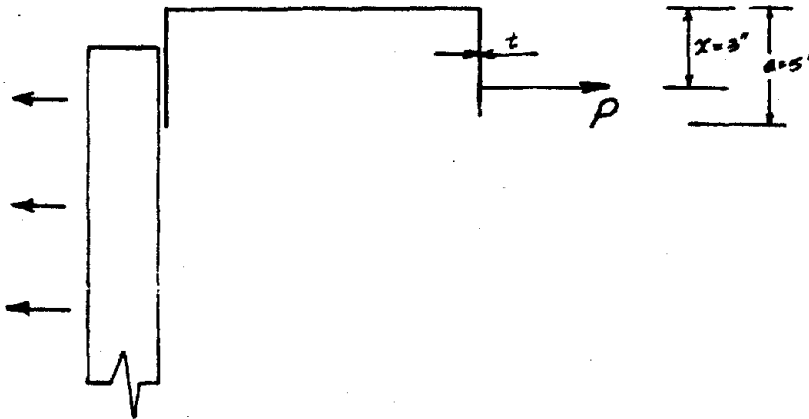
C. 3. 3 Top Channel

Maximum load occurs during rebound, when the flange is subjected to bending by the restraining rear turnbuckles.

$$\begin{aligned} P &= \text{maximum rebound force/turnbuckle} \\ &= \text{maximum force per channel} \times 3/8 \end{aligned}$$

(5 psi rebound vs. 40 psi overpressure, 3 channels per turnbuckle)

$$\text{hence } P = 3/8 \times 18,480 = 6,930 \text{ lbs}$$



Maximum bending moment in the flange (10, pg. 337)

$$\frac{M}{P} \quad (x/a = .6) = .4$$

Bending stress

$$\sigma = \frac{6 M}{t^2} = \frac{6 \times .4 \times 6,930}{(.62)^2} = 43,267 \text{ psi}$$

(vs. 35,000)

i. e., plastic yielding will occur.

Consider ultimate plastic moment (M_u) due to ultimate load P_u (distributed over 36")

$$M_u = P_u \times 3'' = \frac{\sigma_y t^2}{4} \times 36''$$

$$P_u = \frac{35,000 \times .62^2 \times 36}{3 \times 4} = 40,362 \text{ lbs}$$

(vs. 6,930 lbs)

Although localized plastic yielding occurs near the point of load application, the ultimate load capacity far exceeds the applied load.

C. 3.4 Bolts

Channel sections

2 - 5/8" ϕ Grade 5 bolts

Load capacity = 17,170 lbs each

Total load capacity = 34,340 lbs
(vs F_2 = 6,782 lbs)

$$F.S. = \frac{34,340}{6,782} = 5$$

Door Section

6 - 7/8" ϕ grade 5 bolts

Load capacity = 35,615 lbs/bolt

Total Load capacity = 34,615 x 6 = 213,690 lbs
(vs. F_2 = 71,168)

$$F.S. = \frac{213,690}{71,168} = 3.0$$

C.3.5 Turnbuckles and Roof Bolts

Turnbuckles have a 20,000 lb proof (yield) load, and a 50,000 lb ultimate load. Based on the proof load.

$$\text{Pressure capacity} = \frac{20,000}{18,480} \times 40 = 42 \text{ psi}$$

Roof Bolts - extrapolation of the following data.

5/8" full column resin bolt in Limestone yield at about 14,000 lbs (Figure 16).

1-1/8" Wedjet bolt in 3,750 psi concrete yield at 34,000 lbs. (Table 1).

Suggests that 34,000 lbs is a reasonable figure for a 1-1/8" full column resin bolt.

$$\text{Pressure Capacity} = \frac{34,000}{18,480} = 72 \text{ psi}$$

Deformation

From the USBM data (Figure 16), extrapolating to our load

$$\delta_{\max} \approx 0.25'' \times \frac{18,720}{14,000} = 0.33 \text{ in.}$$

C.4 Reinforced Concrete Design (See Reference 11)

Consider the design to be governed by bending of the short span, and neglect the effect of compression steel on the ultimate moment (M_u). Then,

$$M_u = \phi A_s f_y d \left(1 - \frac{0.59 p f_y}{f_c'} \right)$$

where

$$A_s = \text{area of steel} = 0.44 \text{ in.}^2/\text{ft (no. 6 bars)}$$

$$d = \text{depth to steel} = 15''$$

$$f_y = \text{steel yield} = 40,000 \text{ psi}$$

$$f_c' = \text{concrete compressive strength} = 4,000 \text{ psi}$$

$$\phi = \text{safety factor} = 0.9$$

$$p = \text{steel ratio} = \frac{0.44}{12 \times 15} = 0.0024$$

$$\begin{aligned} M_u &= 0.9 \times 0.44 \times 40,000 \times 15 \left(1 - \frac{0.59 \times 0.0024 \times 40,000}{4,000} \right) \\ &= 234,000 \text{ in.lbs.} \end{aligned}$$

$$\text{Pressure Capacity} = \frac{2}{3} \frac{M}{l^2} = \frac{2}{3} \times \frac{234,000}{(72)^2} = 30 \text{ psi}$$

from Figure 43

$$\frac{R_m}{F_1} = 1.5$$

hence

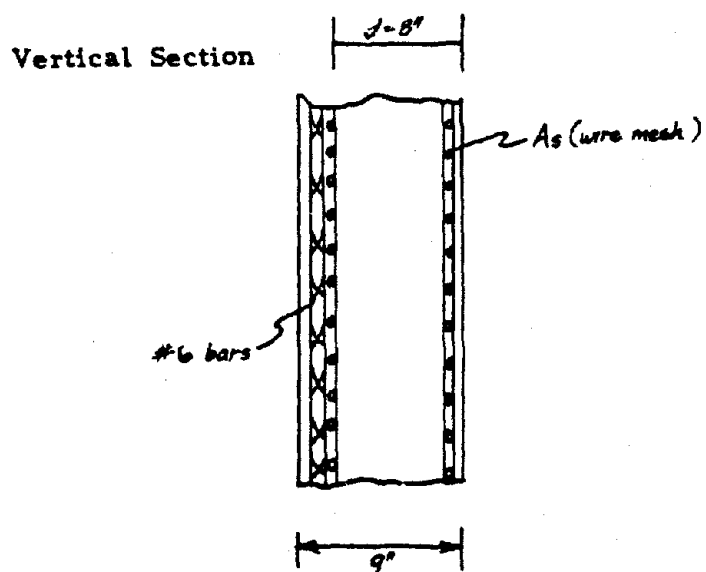
$$\frac{y_m}{y_{el}} = 1.5$$

The midspan deflection, results from plastic as well as elastic deformation, an amount equal to 1.5 times the static elastic deflection due to 20 psi.

C.5 Shotcrete Arch

See Section C.2 for arch design analysis technique and dimensions.

Bending



$$M_u = \phi A_s f_y d \left(1 - \frac{0.59 p f_y}{f_c'} \right)$$

$$A_s = 0.5 \text{ in.}^2/\text{ft}$$

$$f_y = 100,000 \text{ psi (high strength mesh)}$$

$$p = \frac{0.5}{8 \times 12} = .052$$

$$f_c' = 5,000 \text{ psi}$$

$$\phi = 0.9$$

$$M_u = 338,400 \text{ in. lbs/ft}$$

$$\underline{\text{Pressure Capacity}} = \frac{2}{3} \cdot \frac{M}{l^2} = \frac{2}{3} \times \frac{338,400}{(132)^2} = 13 \text{ psi}$$

from C.2

$$\frac{R_m}{F_1} = \frac{13}{14.1} = 0.92$$

Assume that due to its mass the shotcrete structure has a natural period $1/2$ of that of the liner plate arch. Then, referring to Figure 43 and C.2.2

$$\frac{t_d}{T} = \frac{1}{2}$$

hence

$$\frac{y_m}{y_{el}} = 1.4$$

The structure will deform a small amount into the plastic range.

Compression

As indicated in Section C.2, bending considerations result in a design which is more than adequate in compression.

APPENDIX D

ASSEMBLY PROCEDURES AND DESIGN LAYOUTS

Assembly Procedure

Following is a step by step procedure for the erection of each type of bulkhead.

1. Channel-Turnbuckle Bulkhead

Step 1 - Select a location in the passageway where there is a dry structurally sound roof and floor and where the passage width is 18 ft or less and the height is between 5-1/2 and 6-1/2 feet.

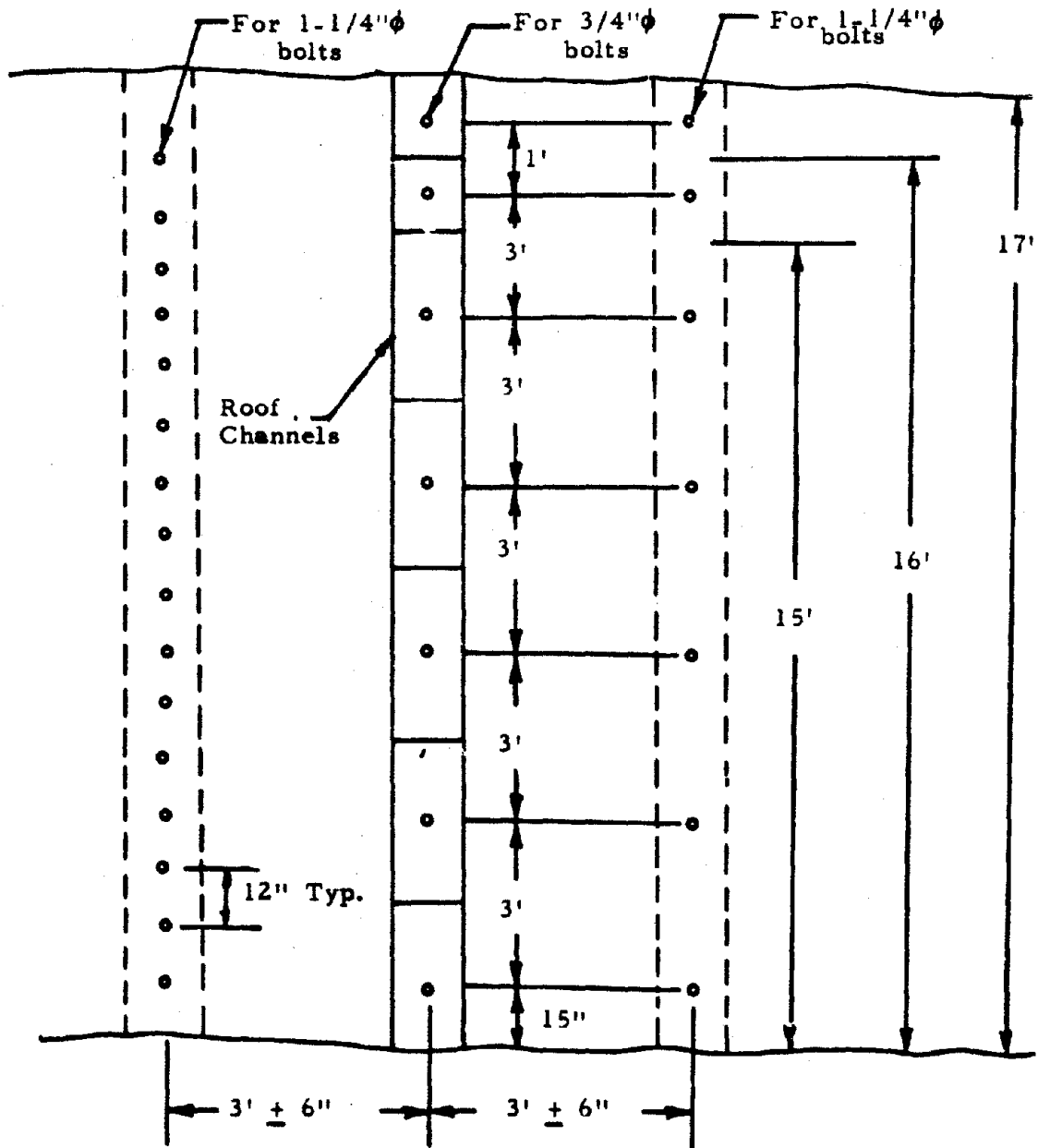
Step 2 - Select the appropriate number of roof channels to correspond to the gross passage width as follows:

18 ft wide	6 x 3 ft channels
17 ft wide	5 x 3 ft channels and 2 x 1 ft channels
16 ft wide	5 x 3 ft channels and 1 x 1 ft channel
15 ft wide	5 x 3 ft channels etc.

Step 3 - At the selected passage location drill the hole pattern in the roof appropriate to the passage width (Figure D.1).

Step 4 - Cut a trench in the passage floor centered about the selected location for the full passage width 7 - 9 inches deep x 15 - 18 inches wide.

Step 5 - Cut back the passage walls at the trench location equally on both sides to give a passage width at this point that is a multiple of a foot plus 1 to 9 inches, i.e. 17' 6" ± 3" etc.



Roof Bolt Hole Pattern for Channel Turnbuckle Design

Figure D.1

Step 6 - Loosely attach the appropriate number of roof channels at the selected location across the roof using 3/4 diameter roof bolts and mount plates.

Step 7 - Install the forward and aft 1-1/2 diameter roof bolts, turnbuckle mounts and turnbuckles into the roof. Assemble eyebolts to rear flange of roof channels and loosely assemble rear turnbuckles to eyebolts.

Step 8 - Loosely assemble splice plates to the rear flanges of the roof channels using 7/8 diameter bolts.

Step 9 - Install the appropriate number of 3 ft and 1 ft footing boxes in the floor trench. Vertically align the footing box mount face to correspond with the forward face of the roof channels.

Step 10 - Loosely assemble the appropriate number (passage width less 3) of upper facing channels to lower facing channels using 5/8 diameter bolts and bearing plates.

Step 11 - Loosely install each facing channel assembly to the footing boxes and the roof channel using 1/2 diameter bolts and eyebolts. Leave a 3 ft gap in the bulkhead centre for the door assembly.

Step 12 - Install the door mount plate to the footing boxes and the adjacent facing channels using 1/2 diameter bolts.

Step 13 - Loosely install the three door facing channels to the door mount plate using 7/8 diameter bolts and bearing plates and to the roof channel using eyebolts.

Step 14 - Lock all lower facing channel assemblies together at the battery flanges with 1/2 diameter bolts.

Step 15 - Tuck aluminum flashing between the roof channel and the facing channel and cut the flashing to clear the eyebolts as required. Tuck the flashing back into the space between the roof channel and the roof by a minimum of 2 inches then route the remaining flashing forward.

Step 16 - Fully tighten the 1/2 diameter bolts and the eyelets at the top and bottom of each facing channel.

Step 17 - Fully tighten the roof channel splice plate bolts.

Step 18 - Tighten the forward and rear turnbuckles until the roof channel is in firm contact with the roof. Tighten the roof channel roof bolts.

Step 19 - Insert flashing between the upper and lower facing channels at each end. Above and below the overlap region attach the flashing to the channel using 1/2 diameter bolts and the bearing strips.

Step 20 - Fully tighten the 1/2 diameter bolts at the upper facing channel butting flanges and also the 7/8 and 5/8 diameter bolts and bearing plates between the upper and lower facing channels.

Step 21 - Install the door using 1/2 diameter bolts.

Step 22 - Seal minor leakage paths in bulkhead structure with Throkol Sealant.

Step 23 - Attach flashing at sides and roof forward of the bulkhead to the passageway with suitable spads.

Step 24 - Apply frothed urethane foam to fill the full length space by the passage wall the flashing and the bulkhead edge on both sides of the bulkhead. Apply frothed foam to the rear portion of the roof channel so that all the voids between the roof and the channel are filled. Apply sufficient foam in this region to travel through to the flashing wherever possible.

Step 25 - Fill the spaces between the footing boxes and the trench with concrete.

2. Truss Bulkhead

Step 1 - Select a location in the passageway that has a dry firm floor where the passage width is 16 ft or less and the height is between 5-1/2 and 6-1/2 ft.

Step 2 - At the selected position of the bulkhead dig a trench 7 - 9 in. deep x 18 - 24 in. wide across the full width of the passageway. Dig a second identical trench 5'6" behind the first measured centre to centre.

Step 3 - Cut back the passage walls evenly at the front trench location to give a passage width of 17' ± 2" (or 14'9" ± 2 etc.)

Step 4 - Assemble an end post slide into each end post and position each end post in each wall recess such that they are 16-1/2 ft apart (or 14-1/4 ft etc.) centre to centre. Spot the position of one end post roof stud and drill a 1-1/2 diameter hole to a minimum depth of 6 in. in the roof. Assemble a bearing plate onto the stud, extend the slide until the bearing plate contacts the roof and pin the slide in the highest possible position with two 1 diameter bolts. Jack the bearing plate into firm contact with the roof using the jacking nut.

Step 5 - Place the appropriate number of footing boxes, spaced 27 in. on centres in both trenches.

Step 6 - Assemble a post slide into a post and attach this post assembly base to the footing box next to the wall post using a 1-1/4 diameter bolt. Holding the post assembly in the upright position locate its exact lateral position relative to the wall post using an 18 in. wide decking assembly. Extend the post slide and locate the position where the stud contacts the roof. Drill a hole 1-1/2 diameter x 6 in. min. deep at this position. Extend the post slide, inserting the stud into the hole until the bearing plate contacts the roof. Pin the slide in the highest possible position using two 1 diameter bolts. Jack the bearing plate into firm contact with the roof using the jacking nut.

Step 7 - Assemble the long and short 18 in. decking assemblies to the two posts using 3/8 diameter bolts.

Step 8 - Repeat steps 6 and 7 across the full passage width using 27 in. decking assemblies. Use one central bay for the door assembly using door posts, the door frame and 15 and 19-1/2 in. high decking assemblies.

Step 9 - Repeat step 4 at the second wall.

Step 10 - Install a truss beam to each passageway post and to a rear trench footing box using 1-1/4 diameter bolts. Add two horizontal ties to each post and beam assembly using 1 diameter bolts adjusting the height of the rear footing box if necessary.

Step 11 - Attach the upper and lower decking sheets to each other using self-tapping screws.

Step 12 - Attach the decking sheets to the door frame using the neoprene seal and self-tapping screws.

Step 13 - Assemble the door to the door frame using 1/2 diameter bolts.

Step 14 - Fill both trenches, but not the footing boxes with concrete.

Step 15 - Attach aluminum flashing to the top and bottom of the bulkhead using suitable lengths of bearing strip and neoprene seal and self-tapping screws. Position the flashing to extend towards the bulkhead centre. Bend the flashing to curve back on itself and attach the other end to the roof/floor using spads.

Step 16 - Attach aluminum flashing to the bulkhead ends using bearing strips and self-tapping screws. Dog-leg the flashing forward and attach it to the walls using suitable spads.

Step 17 - Fill the entire region behind the flashing on all four sides with frothed urethane foam.

3. Arch Bulkhead Assembly Procedure

Step 1 - Select a location in the passageway where the walls are structurally sound, where the floor is dry, where the passage height is between 5-1/2 and 7 ft and the passage width is less than 18 ft.

Step 2 - Cut back each wall equally both sides to form radiused backed cut-outs approximately 2'6" wide such that the maximum passage width is 23 ft.

Step 3 - Dig a curved trench in the passage floor to make the passage height in this region between 6'9" and 7 ft. The curved trench should be a 17 ft radius terminating at the two wall cutouts.

Step 4 - Assemble the floor rib assembly in the following manner. Place the 72 in. long door rib at the passage centre and stand it on temporary 2 in. high blocks. Loosely attach 54 in. ribs to each end using two splice plates each end and 7/8 diameter bolts. Stand these pieces on 2 in. blocks also. Similarly add two 36 in. ribs to the ends of this assembly. To the ends of this assembly add the two wall channels using 8 hole splice plates.

Step 5 - Loosely assemble the first course of liner plates in the following order from each wall; 1/2 liner plate, 2 full liner plates and a 1/2 liner plate. Attach plates to each other to the rib and the wall channel using 5/8 diameter bolts.

Step 6 - Loosely assemble the two door post channels to the edges of the central space using 5/8 diameter bolts.

Step 7 - Loosely assemble the door frame to the two door posts and the floor rib using 1/2 diameter bolts.

Step 8 - Assemble the lower rear splice plates to the door posts and the floor rib using 7/8 diameter bolts.

Step 9 - Assemble the second rib course comprising four 54 in. ribs to the top of the liner plates using 5/8 diameter bolts. Splice these ribs at the wall using 8-hole splice plates front and rear, at their junction using 12-hole splice plates and at the door posts using the door frame at the front and the upper door splice plates at the rear. Use 7/8 diameter bolts.

Step 10 - Loosely assemble the door rib to the door frame using 1/2 diameter bolts and to the rear splice plate using 7/8 diameter bolts.

Step 11 - Assemble the second course of liner plates comprising six full liner plates and one special liner plate above the door using 5/8 diameter bolts.

Step 12 - Loosely assemble the third rib comprising four 54 in. ribs and one 36 in. rib and splice together using splice plates and 7/8 diameter bolts.

Step 13 - Loosely assemble the third liner plate course comprising six full liner plates and two half liner plates using 5/8 diameter bolts.

Step 14 - Loosely assemble the top rib comprising four 54 ribs and one 36 in. rib using splice plates and 7/8 diameter bolts. Attach to the liner plates and the wall channels with 5/8 diameter bolts.

Step 15 - Draw the full bulkhead together horizontally by fully tightening all the 5/8 diameter bolts at vertical post junction except those at the wall posts.

Step 16 - Fully tighten all the remaining 5/8 diameter bolts at the liner plates/rib interfaces.

Step 17 - Trap the 24 in. wide aluminum flashing between the forward splice plates and the top rib and between the forward splice plates and

the door frame and the rib at the bottom. Between the splice plates bend the flashing around the rib flange.

Step 18 - Fully tighten all the 1/2 diameter bolts around the door frame.

Step 19 - Fully tighten all the 7/8 diameter bolts on all the splice plates except those at the wall posts.

Step 20 - Form the steel rod cages around each wall channel using 13 loops and 5 vertical rods per side using wire.

Step 21 - Insert steel flashing between the wall channels and the liner plates and the ribs and the wall splice plates on both sides of both wall posts.

Step 22 - Tighten all remaining bolts at the wall posts.

Step 23 - Attach the steel flashing to the passage wall forward and aft of the wall cutout and incrementally fill the space defined by the steel flashing and the wall with concrete. Continue this process until the entire wall cutout is filled with concrete.

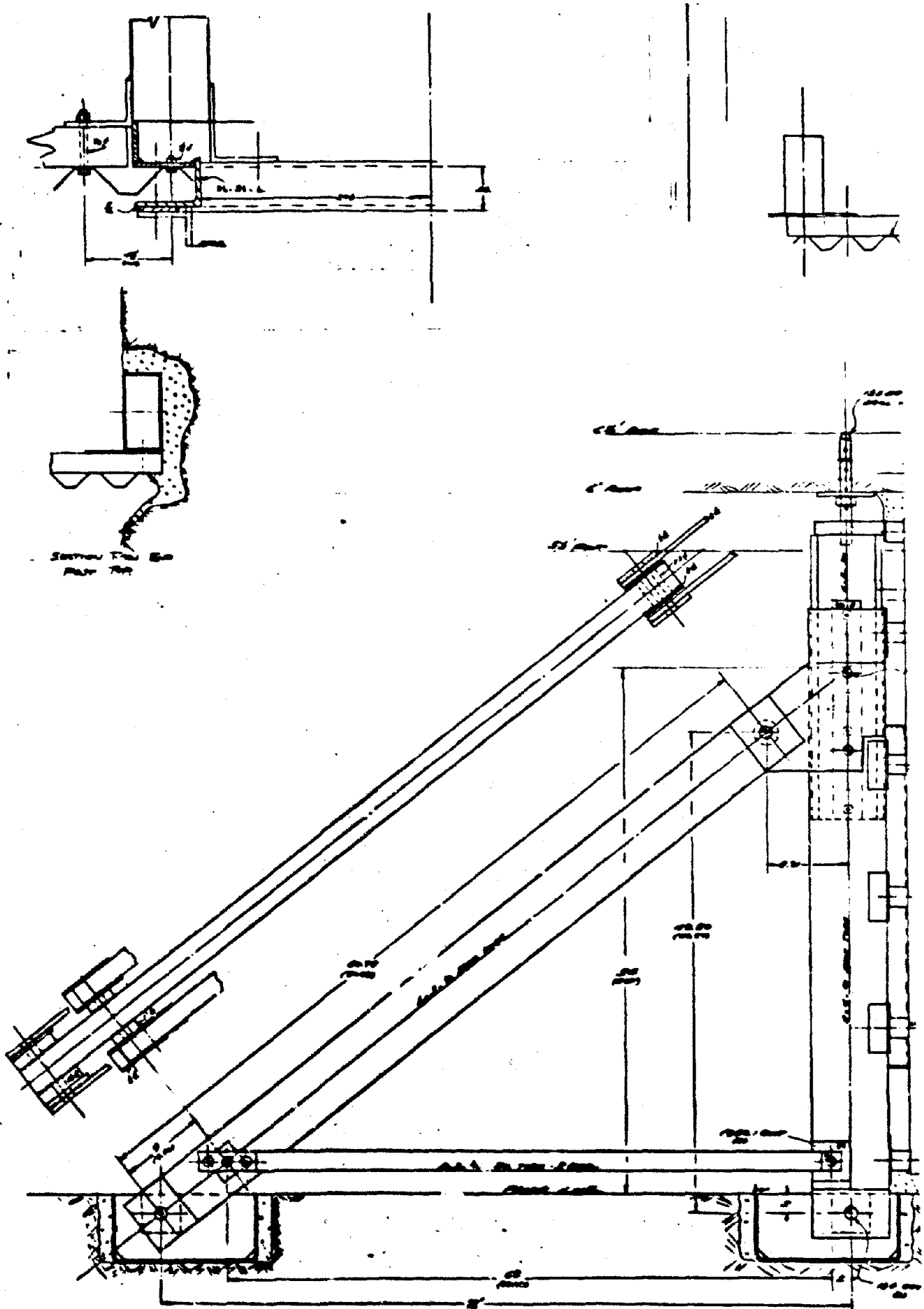
Step 24 - When concrete is set remove the spacing blocks from beneath bulkhead.

Step 25 - Push aluminum flashing into area between bulkhead and roof/floor and then route it forward and attach it to the roof/floor using suitable spads.

Step 26 - Fill areas defined by aluminum flashing, roof/floor and rib with frothed urethane foam.

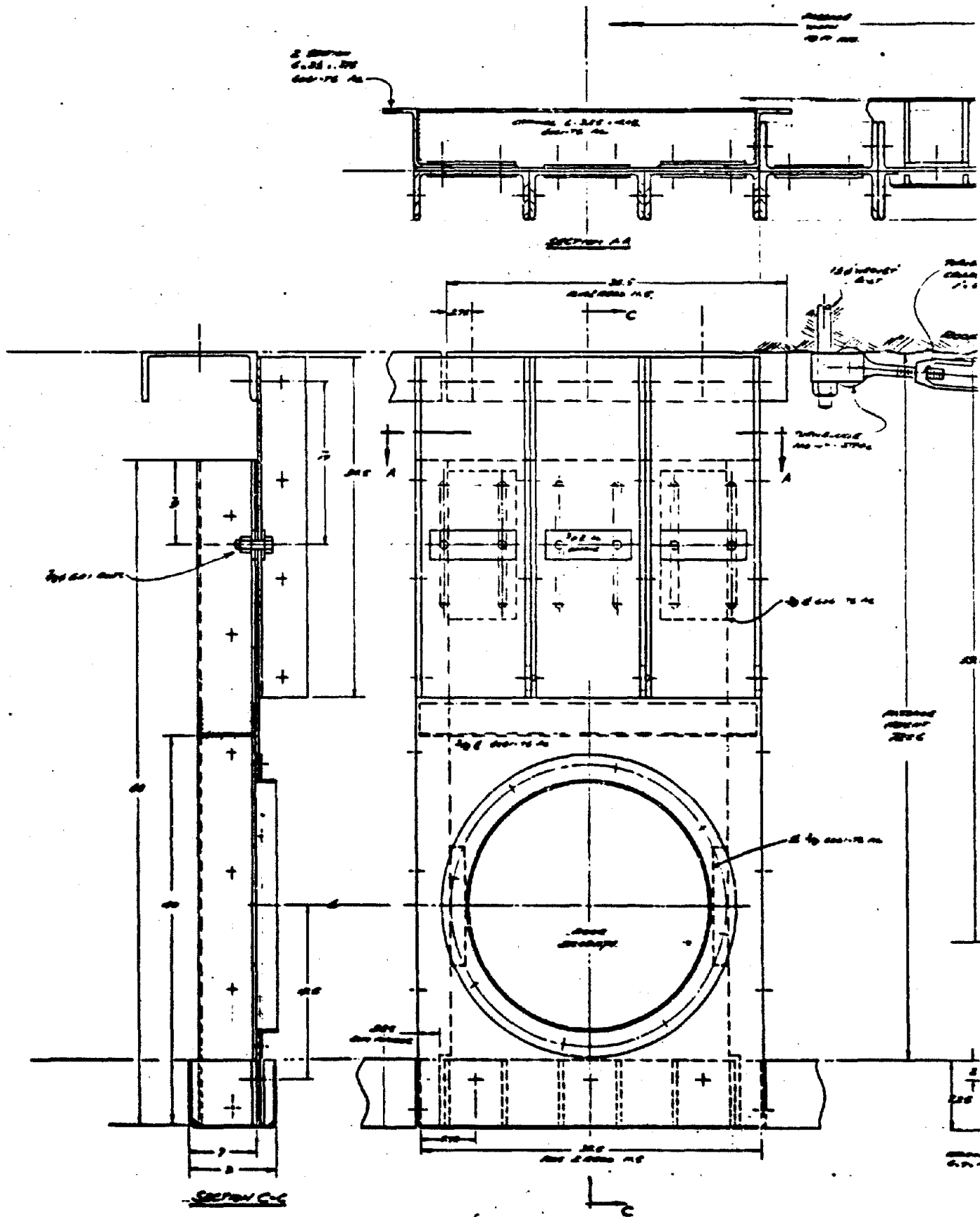
Step 27 - Assemble door using 1/2 diameter bolts. Dig away area in front of the door as required to ensure free door swing.

Step 28 - Seal minor leakage paths with Thiokol Sealant.



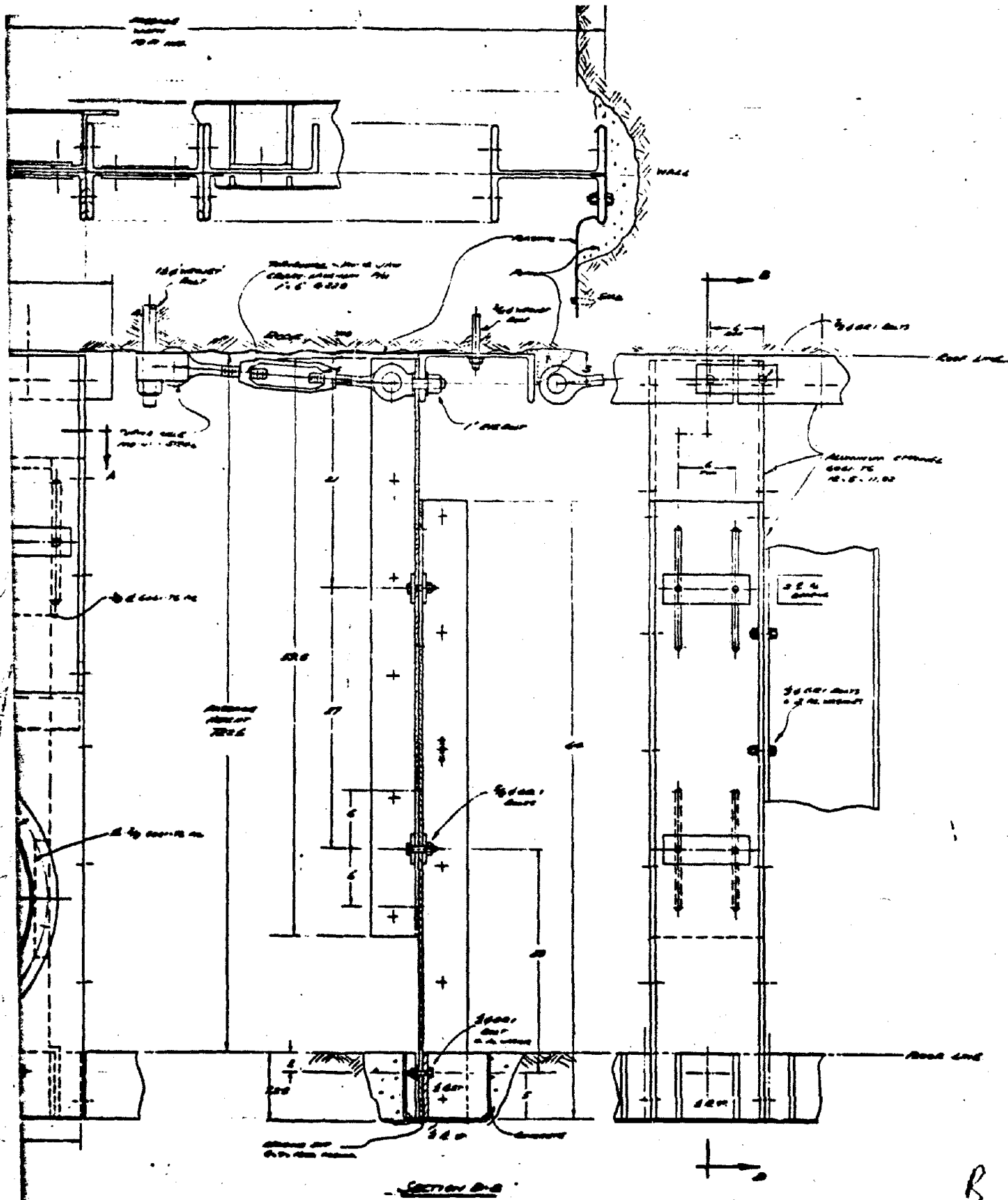
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Figure D, 2 - Truss Bulkhead Layout



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Figure D.4 - Channel Turnbuckle Bulkhead



Channel Turnbuckle Bulkhead Layout

