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Arch Canopy Design Procedure for Rehabilitation of High-Roof-Fall Areas

By Richard A. Allwes, C. P. Mangelsdorf, and Deno M. Pappas

UNITED STATES DEPARTMENT OF THE INTERIOR

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UNIT OF MEASURE ABBREVIATIONS USED IN THIS REPORT

ft	foot
ft^3	cubic foot
ft•kip/ft	foot kip per foot
ft•lbf	foot pound (force)
ft•lbf/ft	foot pound (force) per foot
ft/s	foot per second
ft/s^2	foot per square second
g	gram
ga	gauge
gal/min	gallon per minute
Hz	hertz
in	inch
in^2/lin ft	square inch per linear foot
in^3/lin in	cubic inch per linear inch
kip/ft	kilopound per foot
kip/ft^2	kilopound per square foot
ksi	kip per square inch
lbf	pound (force)
lbf/ft/ft	pound (force) per foot per foot
lbf/ft^3	pound (force) per cubic foot
lbf/in	pound (force) per inch
lbf/kip	pound (force) per kilopound
mm	millimeter
pct	percent
psi	pound (force) per square inch
rad	radian
rad/s	radian per second
s	second
slug/ft	slug per foot
V	volt

ARCH CANOPY DESIGN PROCEDURE FOR REHABILITATION OF HIGH-ROOF-FALL AREAS

By Richard A. Allwes,[1] C. P. Mangelsdorf,[2] and Deno M. Pappas[3]

ABSTRACT

This Bureau of Mines report presents a procedure for the design of an arch canopy for use in rehabilitating a high-roof-fall area. Only dynamic line loading of an unbackfilled arch canopy at its crown is considered, and the procedure does not account for an asymmetrical loading condition. The evaluation of whether an arch canopy is suitable for a particular installation depends on many variables, including in-mine conditions and the engineering properties of the structure. However, a general evaluation can be based on a comparison of the arch's maximum crown deflection and a prescribed allowable crown deflection. The design procedure is based on the concept that when an arch canopy is subjected to impact loading at its crown and deflects from its unloaded state to maximum crown deflection, the structure absorbs strain energy, both elastic and plastic. As a result, this strain energy can be calculated from a static load-displacement diagram for the structure. The significance of this design procedure is that it gives mine personnel an analytical tool to select an arch canopy to meet the dimensional and functional requirements of a mine entry and a prescribed allowable crown deflection.

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INTRODUCTION

The rehabilitation of a mine entry following a high-roof fall is an extremely hazardous job in underground coal mines and poses a time-consuming and expensive problem to mine management. Caved entries of vital aircourses, haulageways, and travel routes need to be restored by the safest and most cost effective methods (fig. 1).

Two main methods of resupport are currently practiced and pertain to the sequence in which the roof-fall material is removed and the permanent supports are installed (1).[4] Either of these conventional methods of resupport offers one or more of the following safety hazards: (1) Mine personnel and equipment are subjected to unsupported roof for extended periods of time, (2) mine personnel are usually required to work or operate equipment on top of potentially unstable platforms or roof-fall material, (3) temporary supports are usually placed on the roof-fall material (fig. 2), (4) long drill steel can easily break at the joints or buckle, and (5) stoper drills may produce noise levels in excess of regulatory limits (1).

Rehabilitating high-roof-fall areas using the conventional methods of resupport caused 56 fatalities and 13 injuries in underground coal mines between 1966 and 1982 (1-5), whereas only 3 fatalities and 3 injuries were attributed to the installation of steel sets. In addition to the safety hazards associated with resupport, experience has shown that the conventional methods of resupport do not provide adequate protection against roof and rib spalling and present long-term maintenance problems as shown in figures 3, 4, and 5 (6-7). Rocks falling from between roof bolts and cribs and from considerable roof-to-floor heights have the potential to seriously injure mine personnel. Furthermore, the use of cribbing for long-term entry stability is not ideal because of its susceptibility to shrinkage and deterioration, and to collapse if excessive side loading is applied by the rubble of sloughing cavity walls (fig. 6).

The use of arch canopies and arch canopy-backfill systems to rehabilitate high-roof-fall areas is receiving considerable interest as a method for improving safety and efficiency during restoration of caved entries. Arch canopies have gained the reputation of being significantly safer, more economical, and faster to install than the conventional methods of resupport. However, a potential safety hazard arises with the use of an arch canopy for protection against impending roof falls--the structure may collapse when subjected to the impact loading of a roof fall. An arch canopy is designed for static loads, such as stabilizing soft ground and hard rock tunnels. However, when installed in a mine entry to rehabilitate a high-roof-fall area, the arch canopy is basically a free-standing structure, making no contact with the mine roof or ribs. Therefore, the dynamic and static loading of moderately sized roof falls, occurring during or after installation of the arch canopy, must be withstood by the structure without injury to mine personnel within the protected entry.

The Labor Department's Mine Safety and Health Administration (MSHA) conducted limited tests to verify the strength of an arch canopy at the leading edge of construction (6). The 6-ft-long arch canopy selected for both static and dynamic tests was composed of four rings; each ring was constructed of eight liner plates to form a semielliptical arch canopy. The results of those tests suggested that a correctly sized, properly installed arch canopy can safely withstand reasonable static and dynamic loads while under construction. The conclusion drawn was that future work was required to establish design criteria and to determine the effects of dynamic loading, liner plate thickness, liner plate configuration of a ring, geometric shape, steel set cross section, joint connections, and backfill on the energy absorption capacity of arch canopies.

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[4]Underlined numbers in parentheses refer to items in the list of references preceding the appendixes.

FIGURE 1.—Massive roof fall along a haulageway.

FIGURE 2.—Cribbing and posts used as temporary supports.

FIGURE 3.—Failure of cribbing, posts, and roof bolts to stabilize resupported roof.

FIGURE 4.—Failure of roof bolts, headers, and straps to stabilize resupported roof.

FIGURE 5.—Jointed and fragmented mine roof resupported with cribbing and roof bolts.

FIGURE 6.—Side loading of cribbing due to sloughing of the cavity walls.

Prior to this research, no guidelines existed for the use of arch canopies to rehabilitate high-roof-fall areas. Because of the interest in and the safety of this approach, MSHA therefore requested the Bureau of Mines to develop arch canopy design procedures for the restoration of caved entries.

ROOF-FALL REHABILITATION PRACTICES

Many methods of rehabilitation have been experimented with in an effort to improve safety and reduce cleanup and permanent support installation costs. Some of these methods have met with disastrous results, as evidenced by the 59 fatalities and 16 injuries cited earlier. All of the methods can be classified into one of two rehabilitation approaches. The first approach, which is characteristic of all except one of the conventional methods of resupport, is to stabilize the caved entry by installing active and/or passive supports--cribs, roof bolts, wire mesh, straps, crossbars, steel rails, and rectangular steel sets. The atypical method of resupport requires tunneling through the roof-fall material using forepoling techniques; this method is rarely practiced. The second approach protects the mine entry from recurring roof falls with the construction of a structure (e.g., an arch canopy) that insulates the mine entry from recurring roof falls and sloughing ribs.

CONVENTIONAL METHODS OF RESUPPORT

A variety of traditional methods of resupport are utilized by the mining industry and are characterized not only by their approach to the removal of fallen roof and installation of permanent supports, but also by the types of permanent supports used. The extent (roof-to-floor height) of the roof fall and current mining practices are usually the controlling factors in determining the order in which the roof-fall material is removed and permanent supports are installed. One procedure used in resupport requires a linear footage of the fallen roof to be cleared away (down to the mine floor) before permanent supports are installed. In general, a mine entry that has experienced a roof fall extending to a height of up to 15 ft above the mine floor may be rebolted with a roof-bolting machine, provided that long drill steel is used. Angle iron is sometimes welded to the drill steel to increase its stiffness and prevent buckling. For roof-to-floor heights exceeding 15 ft, platforms are usually constructed from which to roof bolt; however, this is not a recommended practice. Cribbing, crossbars, and rectangular steel sets may be used as permanent support in place of roof bolts or as supplementary support for precautionary measures. The other procedure used in resupport is to install the permanent supports before the roof-fall material is removed. This procedure is preferable to the aforementioned procedure because the working clearances between the fall material and mine roof and ribs are usually less than 8 ft, making the roof and ribs more easily accessible for testing, scaling, and temporarily and permanently supporting (1). However, this procedure requires mine personnel to work and operate equipment on top of the unstable roof-fall material.

Figure 7 shows a conventional method of resupport utilizing cribbing, roof bolts, and steel straps, and reveals the procedure in which the roof-fall area was resupported and the roof-fall material was removed. The striations on the mine roof and ribs indicate that a continuous miner was trammed onto the roof-fall material to remove all of the loose rock from the roof. The rock removal was conducted in incremental steps since the continuous

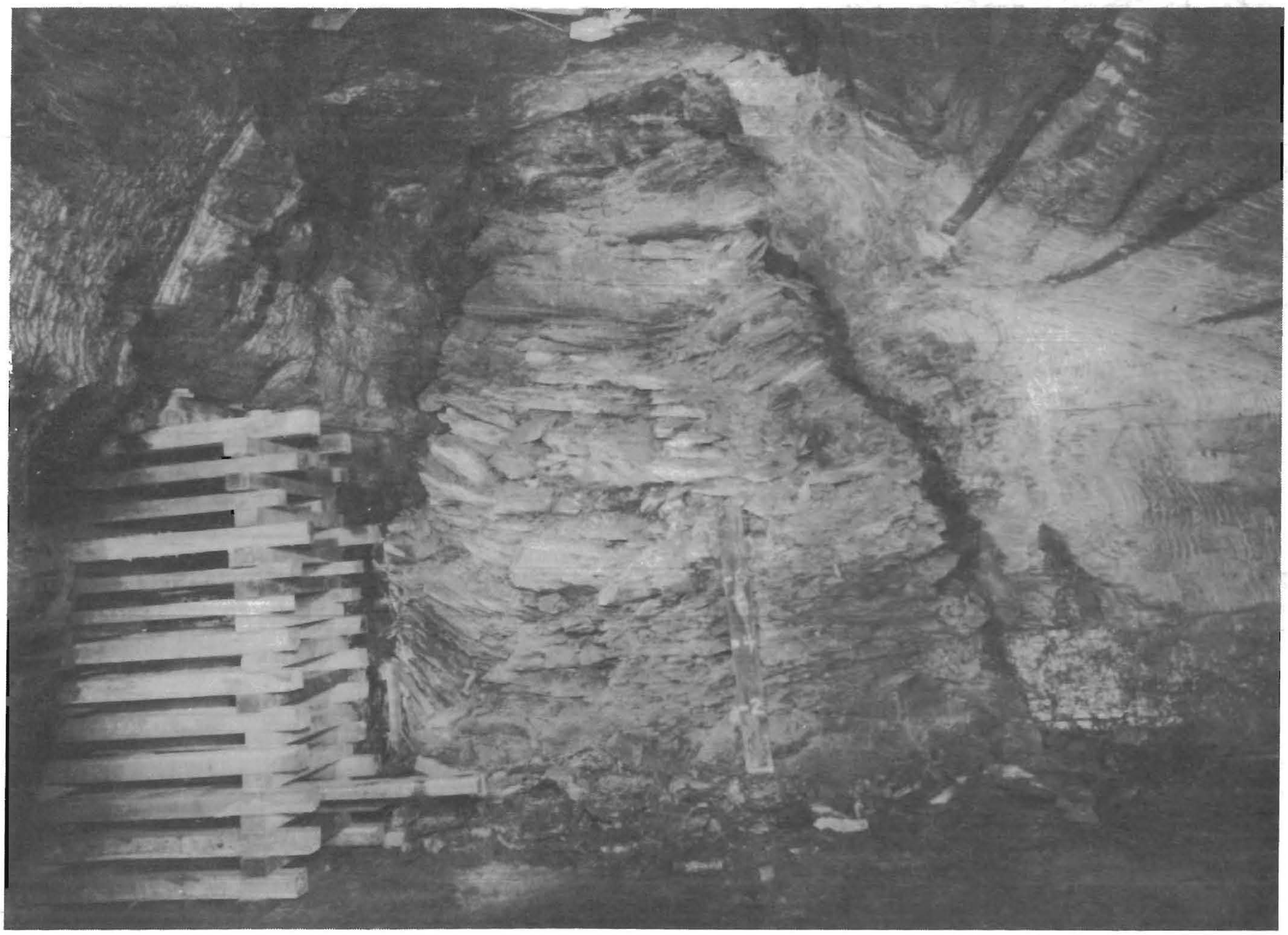

FIGURE 7.—Roof bolts, cribs, and straps used as resupport in an aircourse (crosscut view).

miner operator could not proceed beyond permanent supports. Upon removal of the loose rock, the area was temporarily supported and permanent supports were installed with a roof-bolting machine, which was also trammed on top of the roof-fall material. After the caved area was totally resupported, the roof-fall material was removed.

Rebolting

Rebolting, the most prevalent method of resupport, requires miners to work under unsupported roof until they install some type of temporary support. (Recently developed automated temporary roof support (ATRS) systems were not designed to reach such high places--sometimes more than three times the original height of the entry--and therefore should not be used as a temporary support.) Mine personnel are required to climb onto the fallen material to install roof bolts with a stoper drill or, if available, a roof-bolting machine with special drill extensions, to resupport the mine roof. Wire mesh or steel straps are sometimes used in conjunction with roof bolts to prevent rocks from falling out from between the roof bolts.

Cribbing

Cribbing is often used as either a temporary or permanent support subsequent to a massive roof fall. The crib supports the roof, using the fallen material as a base when the crib is used as a temporary support (fig. 2). This is not always effective owing to shifting or settling of the fallen material during removal. Also, the length of time required for construction exposes mine personnel to the unsupported roof for prolonged periods. When cribs are used as a permanent support, as shown in figures 8 and 9, they are susceptible to shrinkage and deterioration, and to collapse if extensive rib sloughing occurs (fig. 6).

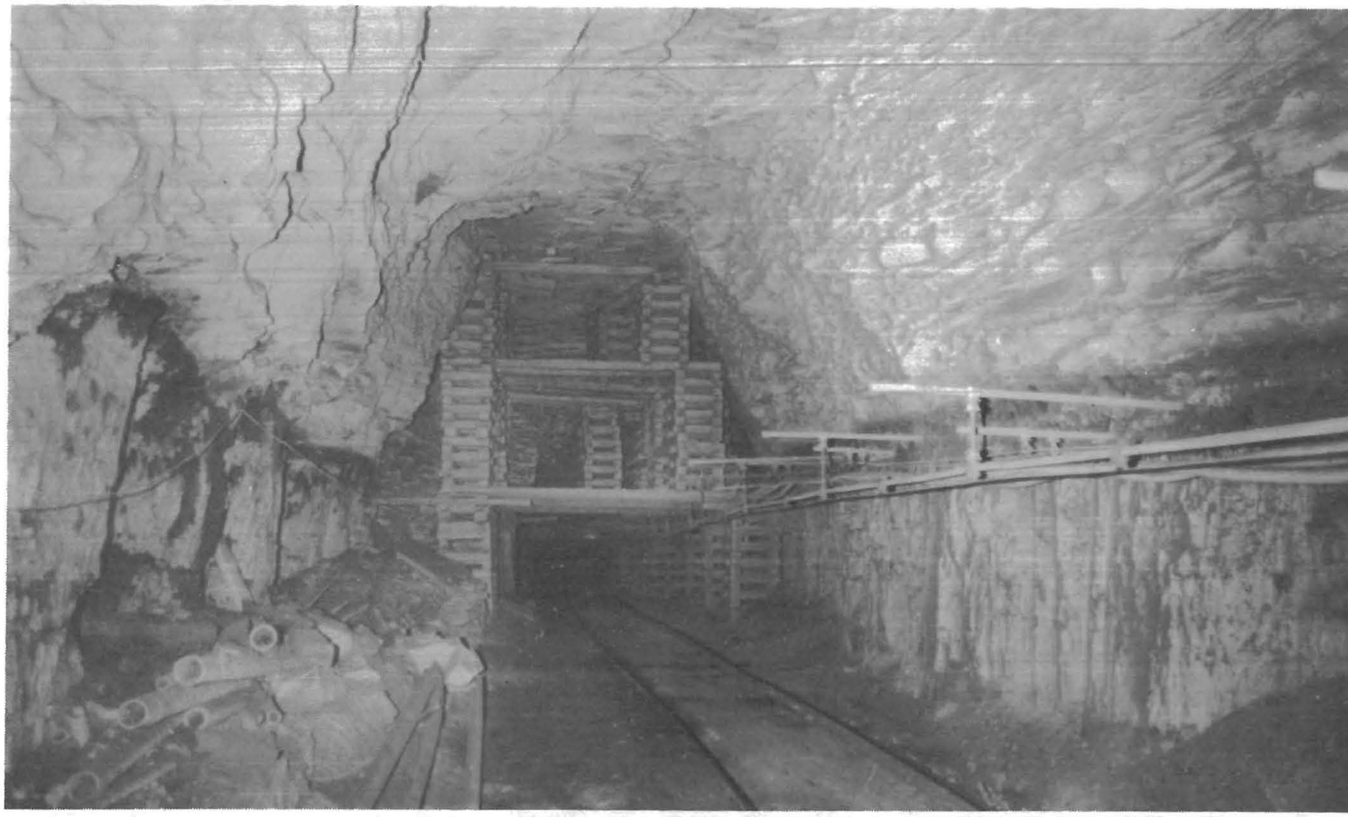

FIGURE 8.—Multiple stories of cribbing.

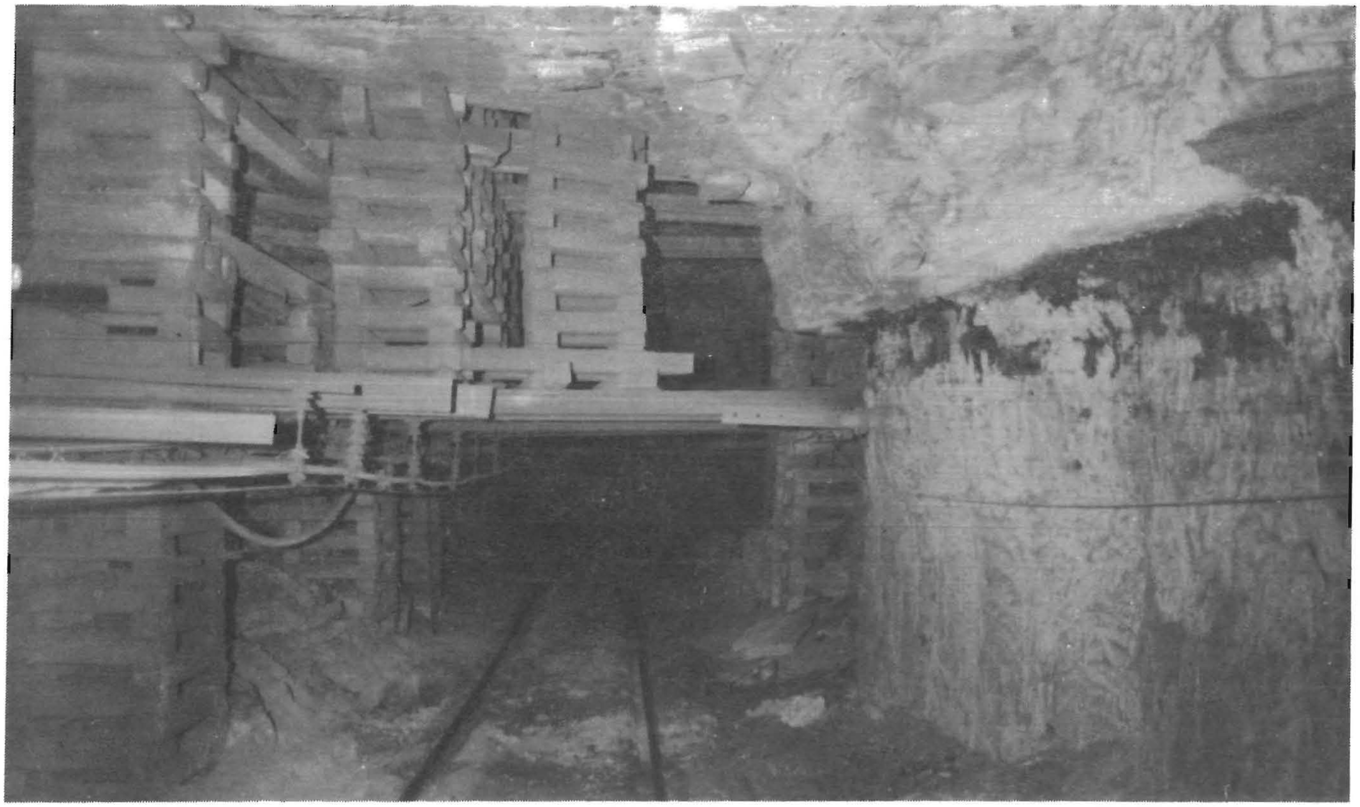

FIGURE 9.—Cribbing and crossbar resupport system.

Multiple-Story Steel Sets

Multiple-story steel sets are used in place of multiple tiers of cribbing as a permanent support (8-9). Rectangular steel sets are bolted together on top of one another to form multiple stories of steel sets, and linearly along the length of the caved entry using spacers to form a stable structure. Cribbing is used to fill the voids between the mine roof and ribs to prevent roof falls and rib sloughing.

TUNNELING

Tunneling is rarely used and is an alternative to completely cleaning up the roof fall (8). Forepoling is the method used to support the fall material in advance of the roof supports. The procedure is to drive steel sections in advance of the last erected support to prevent the rubble from falling into the excavation. Rock is then loaded out and another support is installed to support the excavated opening. This procedure is repeated until the roof-fall material is completely tunneled through.

ARCH CANOPIES

Arch canopies were first used to rehabilitate high-roof-fall areas in 1977 at the Urling No. 3 Mine, Rochester and Pittsburgh Coal Co. (9). Although arch canopies have been used in underground coal mines to support ground under adverse geologic and mining conditions, this is the first known use of these structures for restoration of caved entries. The need by mine management for a rehabilitation method that would virtually eliminate exposure of mine personnel to unsupported roof, increase the rates of advance through the roof fall per day, and reduce the cleanup and construction costs per linear foot of advance precipitated the use of arch canopies.

Two types of arch canopies are currently used for rehabilitation: liner plate (fig. 10) and steel set (fig. 11). Arch canopies can be manufactured to form a variety of profiles; the prevalent shapes are semicircular, semielliptical, horseshoe, and gothic (fig. 12). To achieve the desired shape of a steel set arch or a liner plate arch, members of a steel set are cold-formed and various curved liner plates are selected. The installation procedures for both types of arch canopies are similar except that a protective shield should be used with steel sets to protect mine personnel from roof falls during the erection of a steel set and the installation of the lagging.

Liner Plate Arch

A liner plate arch is an assemblage of rings, each ring being composed of many contoured liner plates bolted together. The number and degree of curvature of the plates determine the size and shape (figs. 12*B*-12*D*) of the arch. Two types of liner plates are currently manufactured, two-flange and four-flange. A two-flange liner plate (fig. 13*A*) is a fully and deeply corrugated plate with an offset lapped longitudinal joint (10). A four-flange liner plate (fig. 13*B*) is a rectangular steel plate, flanged on all four sides, and longitudinally curved and corrugated (11). The liner plates are corrugated to increase their resistance to bending. For added strength, steel sets, called inner arch supports for this type of application, are frequently used and are spaced along the interior of the liner plate arch, as shown in figure 14.

Steel Set Arch

A steel set arch is an assemblage of steel sets, lagging, spacers, and tie rods. Steel sets are usually placed on 3-, 4-or 5-ft centers, and spacers are used to space the steel sets and to align them at right angles to the centerline of the entry, both vertically and horizontally (fig. 11). Tie rods are used to

FIGURE 10.—Erection of liner plate arch. (Courtesy Camber Corp.)

FIGURE 11.—Steel set arch. (Courtesy Commercial Shearing, Inc.)

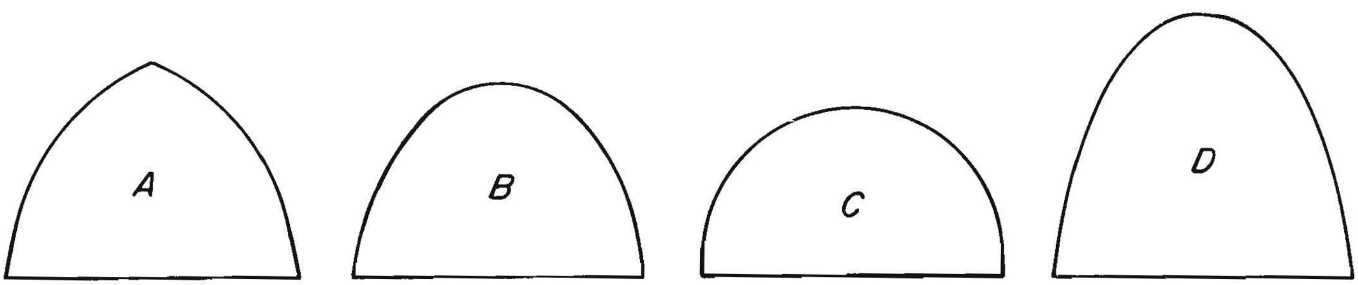

FIGURE 12.—Arch canopy shapes. *A*, Gothic; *B*, horseshoe; C, semicircular; *D*, semielliptical.

pull the steel sets against the spacers and provide stability. Lagging is composed of wood or steel and is normally installed between the flanges of the steel sets to enclose the area between the steel sets and to protect the entry from roof falls. Wood lagging may be used but is not recommended if more than 10 years of service is required (7).

The steel sets currently being used for rehabilitation are manufactured with a variety of shapes, configurations, and cross sections, both constant and variable. The shape and size of the steel set selected for a particular site depend upon the anticipated use of the caved entry and the required working clearances. Common shapes are horseshoe, semicircular, and gothic; each shape has a different effective entry width for specific heights (figs. 12*A*-12*C*). Steel sets can be classified by the number of hinges they possess; the term "hinge" is used in this context to mean either a pin-type connection (fig. 15*B*) or merely a non-moment-resisting one. (Although a

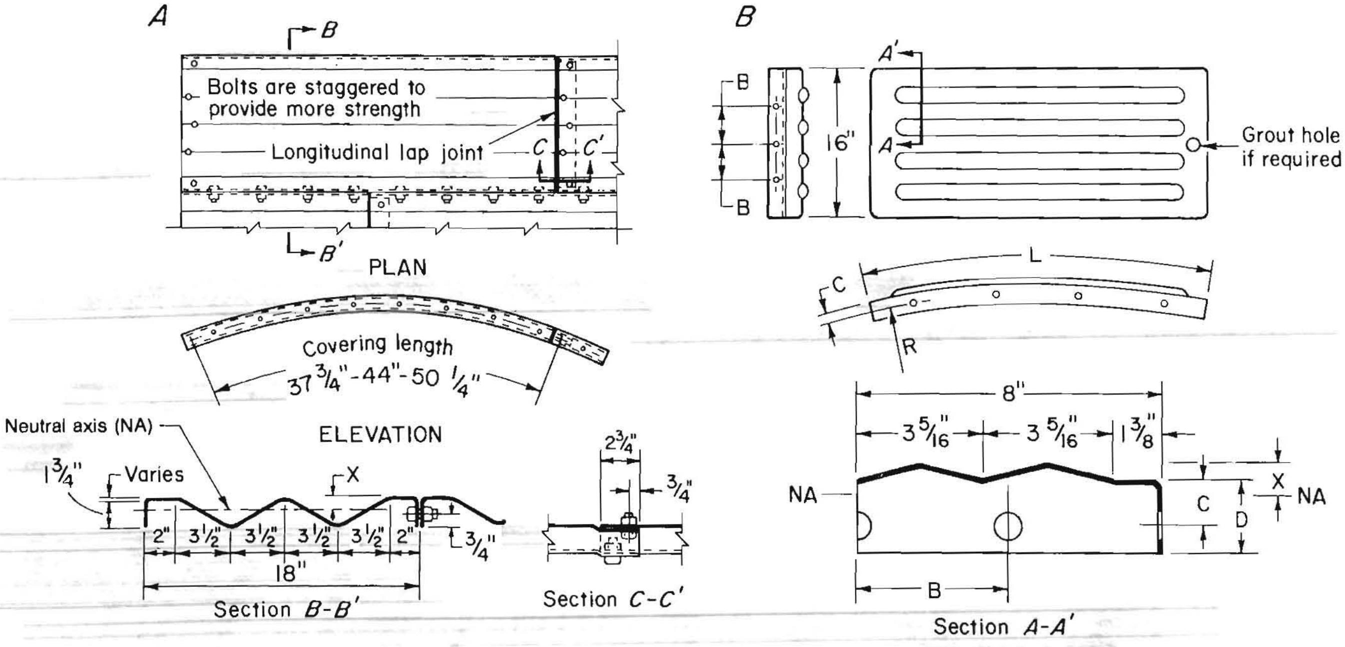

FIGURE 13.—Liner plate. *A*, Two-flange; *B*, four-flange.

FIGURE 14.—Liner plate arch with inner arch supports. (Courtesy Camber Corp.)

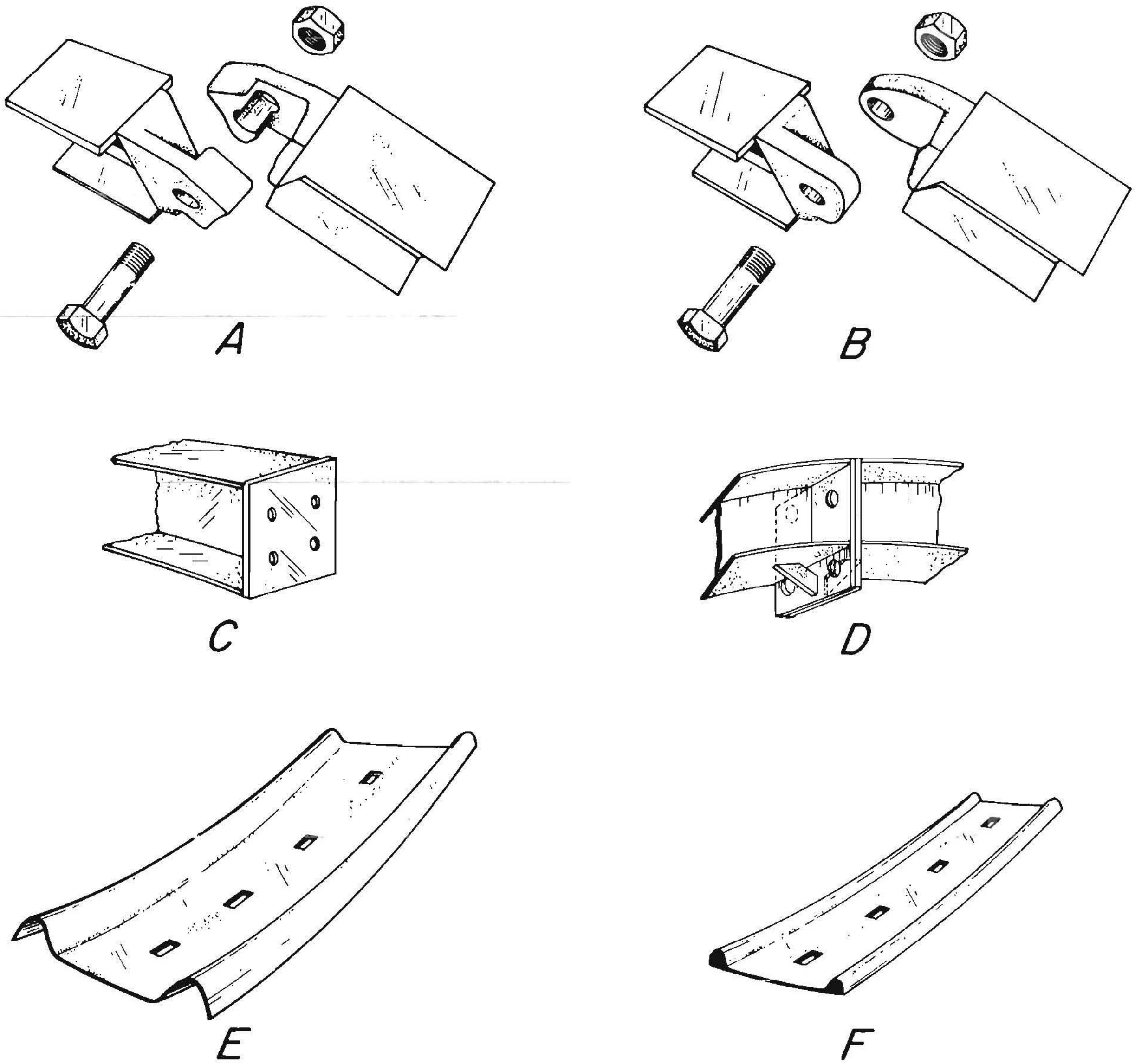

FIGURE 15.—Types of bolted joints. *A*, **Rigid;** *B*, **flexible;** *C*, **butt plate;** *D*, **gusseted butt plate;** *E*, **wraparound fish plate;** *F*, **in-flange fish plate.**

pin connection is not present at the base of an arch, the base can be considered as a hinge if it is restrained from translation but is free to rotate.) Two-hinge, three-hinge, and four-hinge steel sets are the main types of steel sets used in underground coal mines. Four-hinge steel sets are structurally unstable and, since they cannot be blocked to constrain their lateral movement without subjecting mine personnel to long exposure to unsupported roof, these structures are not recommended for rehabilitative purposes. The cross sections available for steel sets are the M-section, W-section, S-section, RSJ-section (rolled steel joist section), and variable-depth fabricated section (fig. 16).

Steel sets are composed of an assemblage of curved and possibly straight steel flexural members. Hinges and rigid joints (fig. 15) are the two types of

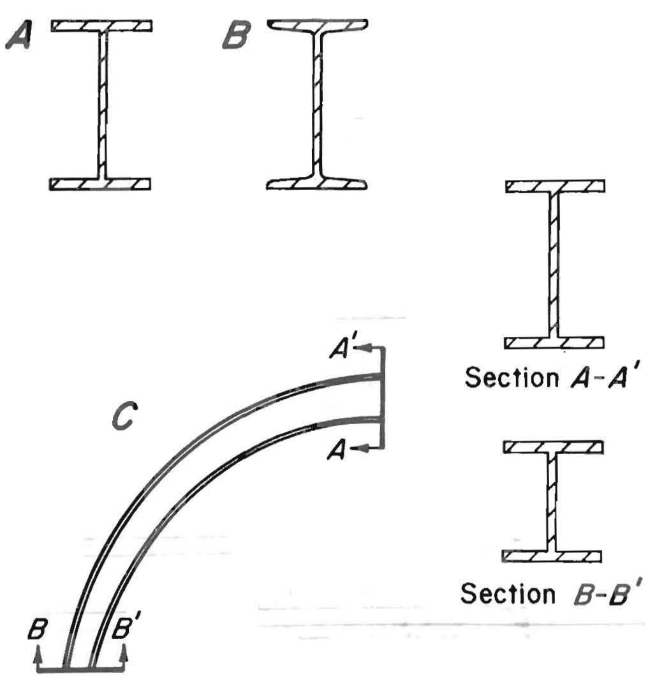

FIGURE 16.—Arch canopy member cross sections. *A*, W-section (wide-flange beam) and M-section; *B*, S-section (American Standard beam) and RSJ-section (rolled steel joist); *C*, variable-depth fabricated section.

connections used to join the structural members of the steel sets together. Hinges allow thrusts and shear forces to develop between adjoining structural members; moments cannot be transmitted since the members are free to rotate. Rigid joints prevent rotation of adjoining structural members so that thrusts, shear forces, and bending moments can be transmitted from one member to another.

Backfill

An arch canopy can be backfilled for added strength. Backfill is usually hand or pneumatically stowed, but there is no reason why hydraulic stowing could not be used. Common materials considered for backfilling are slag, crushed waste rock, and fly ash. Backfill material resists outward displacements of the arch sides and discourages buckling, thus increasing the stiffness of the arch members to loading and the overall strength of the arch canopy. A void filler such as AQUALIGHT[5] (a quick-setting aerated cementitious composition that forms a thixotropic foam with an expansion factor of 10 to 15) was recently used as a backfill material in this country. Such a void filler with a reasonable compressive strength could potentially reduce the amount and cost of backfilling and provide greater support resistance to outward motion of arch members than the traditional types of backfill currently used. Roof falls and sloughing ribs that occur after the installation of the arch canopy will act as natural backfill.

Installation Considerations

Prior to the rehabilitation of a roof-fall area, mine personnel should be instructed in the basic safety procedures for handling and lifting steel members, avoiding roof-fall hazards, and operating special tools used for the construction of the arch canopy (12). Mine personnel should also become familiar with the proper procedure for cleaning up the roof-fall material and assembling the arch canopy.

Before the erection of the arch canopy is started, a company official should examine the area. The lips of the fall should be reinforced with rectangular steel sets since the roof strata have been disturbed and may fail. The first two rings or courses of an arch canopy should be secured to the rectangular steel sets or roof-bolted to the mine roof or ribs to prevent the structure from tipping over in the event of a recurring roof fall (figures 17 and 18). The base of the arch canopy should be secured to prevent translation. Mine personnel should not proceed into an unsupported area while installing liner plate or removing and loading the fallen roof material (figures 19 and 20). As a worst case, only the arm of a worker should be exposed when aligning the liner plate for bolting. When steel sets are being installed, a shield should be present to protect mine personnel from roof falls. The steel sets should be assembled under the protection of the arch canopy and

[5]Reference to specific products does not imply endorsement by the Bureau of Mines.

FIGURE 17.—First ring of liner plate chained to a steel beam. (Courtesy Camber Corp.)

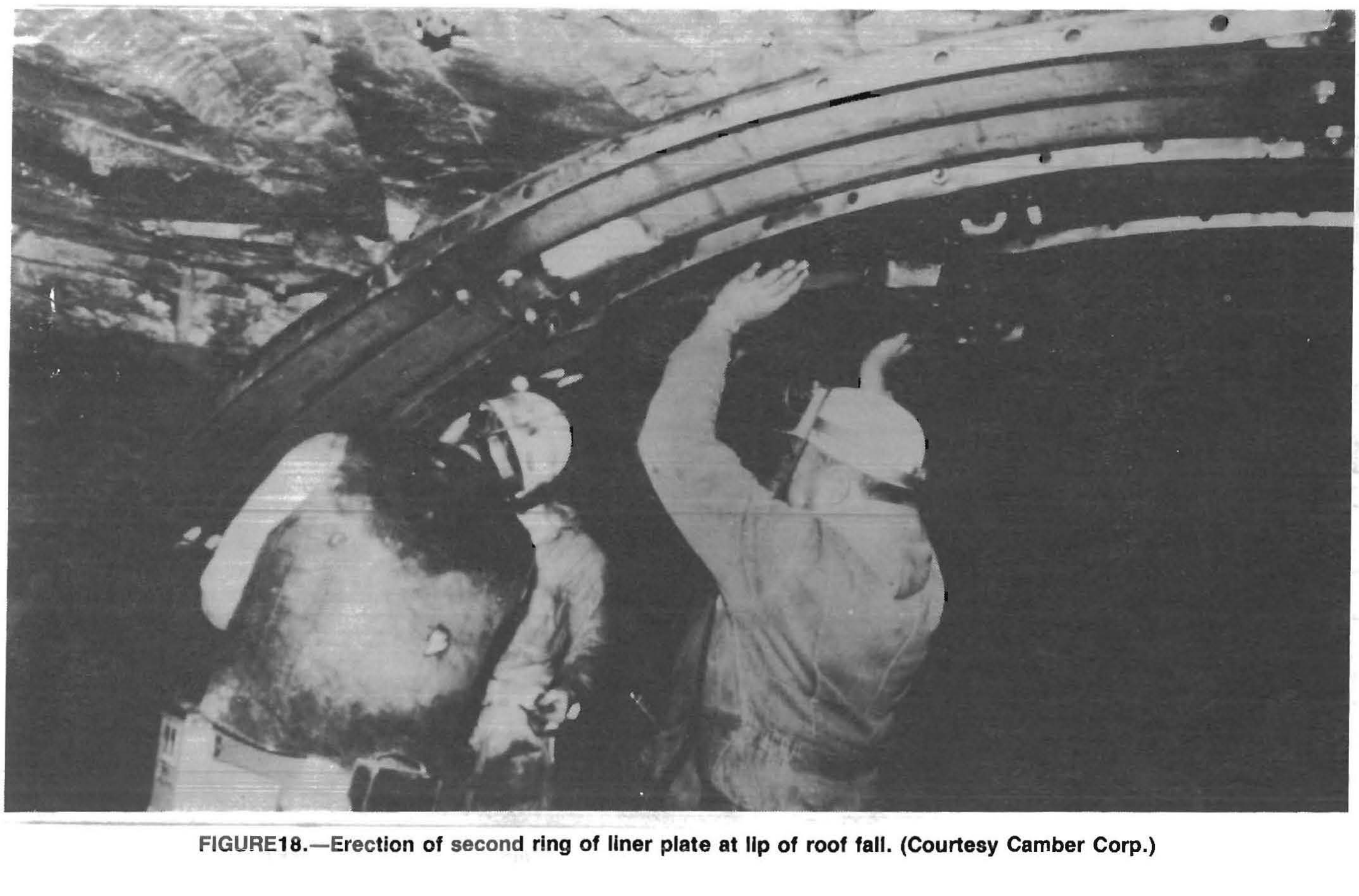

FIGURE 18.—Erection of second ring of liner plate at lip of roof fall. (Courtesy Camber Corp.)

FIGURE 19.—Continuous miner removing roof rock at leading edge of installation. (Courtesy Camber Corp.)

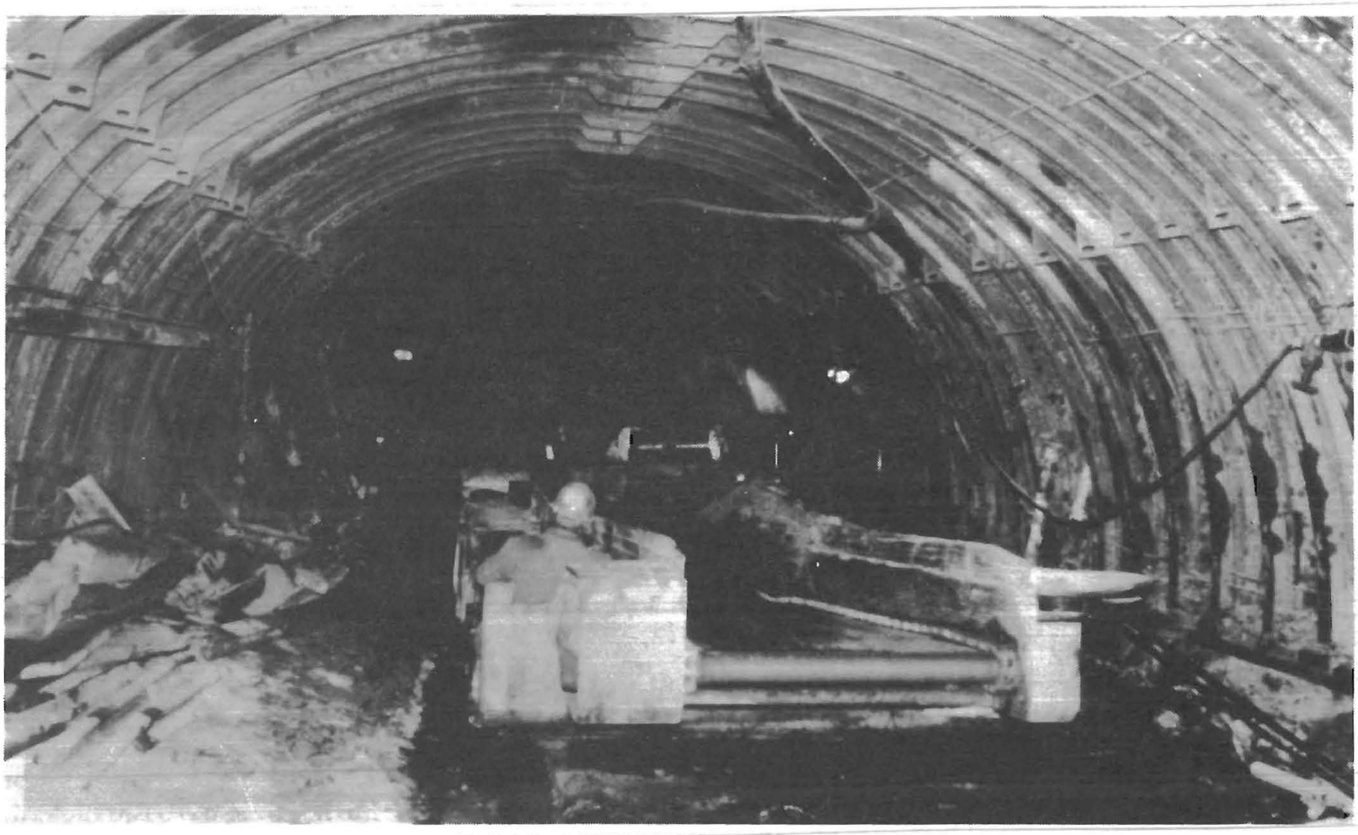

FIGURE 20.—Shuttle car being loaded with rock by continuous miner. (Courtesy Camber Corp.)

raised into position by mechanical means (e.g., cutter head of a continuous miner) under the protection of a shield. When trolley wire is to be installed in an entry protected by an arch canopy, the canopy should be adequately grounded to the track and/or return feeder cable at frequent intervals (6).

SAFETY AND COST COMPARISON: ARCH CANOPY VERSUS CONVENTIONAL METHODS OF RESUPPORT

The use of an arch canopy provides many advantages not associated with the conventional methods of resupporting high-roof-fall areas. The installation of an arch canopy reduces most of the hazards associated with resupport. Mine personnel are not required to work on top of the unstable roof-fall material. The installation of temporary supports is eliminated, and the exposure time of mine personnel to unsupported roof is significantly reduced. An arch canopy requires no intermittent maintenance, whereas cribbing is susceptible to shrinkage and deterioration. Moreover, an arch canopy completely encloses an entry and prevents blockage of the entry when a roof fall or sloughing of the cavity walls occurs (fig. 21); this is impossible with a conventional method of resupport.

Labor and material cost figures for the conventional methods of resupport and arch canopies are difficult to obtain. Costs per foot of advance and rates of advance have been provided by two mining companies for both types of rehabilitation methods. One study has reported that labor and material costs to resupport a high-roof-fall area using multiple-story steel sets have ranged from $1,000 to $2,000 per foot of advance, depending on the height of the roof fall (8). Furthermore, rates of advance have averaged 5 to 15 ft per week for this particular method of resupport. Costs for rehabilitation, using a liner plate arch-backfill system, have ranged from $700 to $1,000 per foot of advance, and

FIGURE 21.—Downward view of arch canopy. (Courtesy Commercial Shearing, Inc.)

advance rates of 10 to 25 ft per week have been achieved. Another study has also presented cost figures for both types of rehabilitation methods (12). Labor and material costs for resupporting a high-roof-fall area using roof bolts and cribbing were $374 per foot of advance; advance rates ranged from 5 to 30 ft per shift. Costs for rehabilitation using steel sets and wood lagging were $183 per foot of advance; advance rates averaged 30 to 35 ft per shift.

The reported rates of advance and costs per foot of advance are drastically different for both studies. However, reasonable explanations for these differences in costs per foot and rates of advance can be given. The roof falls for the first study ranged from 15 to 70 ft, while those for the second study only ranged from 10 to 30 ft (roof-to-floor height). Furthermore, the costs of labor and material reported in the first study are based upon erections of more elaborate structures than were used in the second study. Despite differences in roof-fall heights and types of structures used for rehabilitation, both studies show that arch canopies are an effective method of reducing labor and material costs and increasing rates of advance through the roof falls. Another important aspect of arch canopies is that the amount of arch canopy materials used for rehabilitation is independent of the height of the roof-fall area, which is not the case for the conventional methods of resupport. Thus, the greater the roof-fall height, the more cost-effective the arch canopy becomes.

ROOF-FALL REHABILITATION ACCIDENT STATISTICS

The term "roof-fall rehabilitation" is used to describe the process or state of a caved mine entry being restored to its original useful purpose. Roof-fall rehabilitation encompasses the use of the conventional methods of resupport and arch canopies. Therefore, any roof-fall accident that occurs during resupport or the installation of an arch canopy will be classified as a roof-fall rehabilitation accident.

An integral part of the design of a structure is establishing the type and magnitude of loading to which it will be subected. Once the loading criteria have been established, the structure can be designed to support a static load and/or absorb a dynamic load to desired specifications of stresses or deflections.

An arch canopy selected for the restoration of a caved entry will be subjected to roof falls, which constitute dynamic loading situations, or more specifically, impact loads. For convenience, the energy of a roof fall will be expressed in terms of kinetic energy gained or potential energy lost, each being equivalent. An in-depth explanation for this approach is given in the section entitled "Arch Canopy Design Considerations and Procedure."

To identify possible loading criteria for arch canopies and compile injury statistics, a study of fatal roof-fall accidents was conducted. The information sought from this study was number and type of injuries; length, width, and thickness of each roof fall; and the distance the roof fall traveled (void height) for each accident. This information was compiled from Bureau IC 8723 (1), which was a study of roof-fall resupport accidents that occurred from 1966 to 1974, and from 12 roof-fall fatality reports for 1975-82 (5).[6] Table 1 is a compilation of these roof-fall rehabilitation accidents and presents the dates of the accidents, the dimensions of the roof falls, and their respective voids heights and energy levels.

INJURIES

A majority of the roof-fall rehabilitation accidents (56 out of 59) are associated with the conventional methods of resupport. The conventional methods of resupport accounted for 56 fatalities and

[6] It should be emphasized that the rehabilitation accident statistics generated for 1975-82 were compiled only from roof-fall fatality reports and that an undisclosed number of rehabilitation accidents could have been compiled from nonfatal accident reports.

Table 1. - Roof-fall rehabilitation accidents

Date	Injury		Void	Roof-fall dimensions				Energy	
	Fatal	Nonfatal	height, ft	Length, ft	Width, ft	Thickness, ft	Volume, ft³	ft·lbf	ft·lbf/ft
1966:									
Jan. 28..........	1	0	13.33	14.75	19.75	0.29	84.48	176,801	11,986
Mar. 19..........	1	0	4.00	23.50	6.60	.25	38.77	24,348	1,036
May 27..........	1	0	4.50	10.00	6.00	2.50	150.00	105,975	10,598
June 3..........	1	0	6.00	9.50	14.00	1.25	166.25	156,608	16,485
Aug. 20..........	1	0	7.33	4.00	14.00	.33	18.48	21,267	5,317
Dec. 9..........	1	0	13.00	2.67	1.50	1.50	6.01	12,266	4,594
Dec. 20..........	1	0	10.67	13.50	4.75	.25	16.03	26,853	1,989
1967:									
Apr. 17..........	1	0	16.25	2.50	1.33	.50	1.66	4,235	1,694
Dec. 28..........	1	0	15.00	5.50	3.00	1.50	24.75	58,286	10,598
Dec. 28..........	1	0	15.00	9.67	3.00	2.50	72.53	170,808	17,664
1968:									
Feb. 5..........	1	0	16.50	5.00	4.50	1.00	22.50	58,286	11,657
Aug. 14..........	1	0	12.00	13.00	8.00	1.75	182.00	342,888	26,376
Sep. 8..........	1	0	6.00	6.50	5.75	1.75	65.41	61,616	9,479
1969:									
Mar. 24..........	1	0	6.83	6.83	4.00	.25	6.83	7,324	1,072
June 5..........	1	0	19.00	8.00	3.50	4.00	112.00	334,096	41,762
July 31..........	1	0	20.50	8.00	6.00	.50	24.00	77,244	9,656
Aug. 13..........	1	0	14.00	65.00	8.00	.83	431.60	948,657	14,595
Aug. 27..........	2	0	8.67	10.00	20.00	.38	76.00	103,450	10,345
Sep. 10[1]..........	1	0	12.00	12.50	4.50	4.00	225.00	423,900	33,912
Sep. 10[1]..........	1	0	12.00	8.00	3.50	1.50	42.00	79,128	9,891
Nov. 12..........	1	0	6.09	31.00	12.00	.66	245.52	234,749	7,573
Dec. 9..........	1	0	15.83	3.33	7.33	3.33	81.28	202,006	60,662
1970:									
Apr. 15..........	1	0	10.00	7.00	6.00	.25	10.50	16,485	2,355
May 27..........	1	0	17.00	5.00	10.00	2.00	100.00	266,900	53,380
July 18..........	1	0	15.00	5.75	3.50	.30	6.04	14,224	2,474
Aug. 12..........	1	0	12.00	24.00	21.00	.25	126.00	237,384	9,891
Sep. 3[1,2].......	1	1	17.00	6.00	6.00	.58	20.88	55,729	9,288
Sep. 3[1,2].......	1	0	17.00	6.00	5.00	.67	20.10	53,647	8,941
Sep. 5[1]..........	1	1	20.00	4.00	2.00	.75	6.00	18,840	4,710
Sep. 5[1]..........	0	1	20.00	3.00	1.50	.83	3.74	11,744	3,915
Sep. 9..........	1	0	8.17	15.00	2.00	1.00	30.00	38,481	2,565
Sep. 21..........	1	1	35.00	4.00	3.00	1.00	12.00	65,940	16,485
Oct. 15..........	1	0	5.00	6.00	3.00	.42	7.56	5,935	989

[1]2 separate roof falls occurred. [2]An arch canopy constructed of steel sets and lagging was being installed.

Table 1. - Roof-fall rehabilitation accidents--Continued

Date	Injury		Void height, ft	Roof-fall dimensions				Energy	
	Fatal	Nonfatal		Length, ft	Width, ft	Thickness, ft	Volume, ft^3	ft·lbf	ft·lbf/ft
1971:									
Apr. 5	1	0	6.00	19.00	2.00	1.08	41.04	38,660	2,035
May 26	1	0	10.00	4.00	16.00	.33	21.12	33,158	8,290
Sep. 9	1	0	5.00	3.00	1.67	.55	2.76	2,167	722
Nov. 25	0	4	9.42	6.00	5.00	.25	7.50	11,092	1,849
1972:									
Sep. 28	0	1	10.00	9.00	3.50	.84	26.46	41,542	4,616
1973:									
June 1	1	0	22.70	9.00	5.00	.60	27.00	96,225	10,692
July 14	1	0	12.00	13.00	6.50	1.33	112.39	211,743	16,288
Aug. 7	1	0	27.00	3.00	2.00	1.00	6.00	25,434	8,478
Aug. 25	1	0	5.08	20.00	14.00	.83	232.40	185,353	9,268
Sep. 1	1	0	10.00	30.00	8.00	.75	180.00	282,600	9,420
Sep. 8	1	0	13.75	5.00	5.00	.33	8.25	17,810	3,562
Dec. 19	0	1	8.00	6.25	4.83	.50	15.09	18,953	3,032
1974:									
Jan. 8	1	0	30.00	3.00	3.00	1.00	9.00	42,390	14,130
Apr. 11	1	3	9.00	10.00	5.00	3.00	150.00	211,950	21,195
1975:									
Sep. 25	1	0	6.50	17.00	3.50	.83	49.39	50,402	2,965
May 12	2	1	16.50	10.00	6.00	2.50	150.00	388,575	38,858
1976:									
July 31	1	0	14.00	11.00	10.00	2.00	220.00	483,560	43,960
1977:									
Oct. 15	1	0	14.00	5.00	1.50	1.00	7.50	16,485	3,297
1978:									
Aug. 14	2	0	8.00	3.00	2.00	.79	4.74	5,953	1,984
1979:									
Aug. 30	1	0	7.00	8.00	6.00	1.17	56.16	61,720	7,715
1980:									
Sep. 2	1	0	7.00	7.50	7.00	1.17	61.43	67,512	9,002
Sep. 29	1	0	13.50	2.33	3.00	.75	5.24	11,106	4,767
1981:									
Sep. 16	1	0	5.67	11.00	22.00	.67	162.14	144,335	13,121
1982:									
Apr. 26	2	0	5.17	9.00	11.00	.75	74.25	60,268	6,696
Sep. 16	1	0	6.33	9.00	5.00	.50	22.50	22,361	2,485
Dec. 22[2]	1	2	26.50	7.00	2.00	.33	4.62	19,222	2,746

[1]2 separate roof falls occurred. [2]An arch canopy constructed of steel sets and lagging was being installed.

13 injuries as shown in columns 2 and 3 of table 1 for 1966-82. These injury statistics support the view that safer methods of roof-fall rehabilitation are required to protect mine personnel from impending roof falls. Three fatalities and three injuries occurred while mine personnel were installing arch canopies constructed of steel sets and lagging. These three accidents strongly confirm the notion that if steel set arches are to be used for roof-fall rehabilitation, a shield should be used in conjunction with their construction to protect mine personnel from subsequent roof falls while steel sets are being placed in position and lagging is being installed. Furthermore, if shields are required, as these three accidents suggest, they should be required to withstand the same dynamic and static loads as the arch canopies.

ENERGY ABSORPTION CRITERIA FOR ARCH CANOPIES

Columns 4 through 10 of table 1 present the dimensions of the roof falls and their respective void heights and energy levels. Table 2 summarizes the pertinent statistics of the roof falls and reveals that the roof-fall dimensions, void heights, and energy levels are not normally distributed, as evidenced by a comparison of the mean and standard deviation of each column. During the compilation of the roof-fall data, the dimensions of the roof falls and void heights provided in the roof-fall fatality reports were suspected at times to be inaccurate. These inaccuracies are attributed to the following: (1) A secondary roof fall that breaks apart is difficult, if not impossible, to measure if portions of it appear to be part of the original or primary roof fall, (2) the void height may be too high to measure and must be estimated, and (3) unsafe conditions may prevail at the accident site and the roof-fall dimensions must be visually estimated. Also, in the event that a range was given for a certain dimension of a roof fall, an average of the range was used for that dimension. Furthermore, as given in the fatality reports, information on the shapes of the roof falls was usually inadequate for a proper assessment of the volumes of the falls, so all volumes were calculated from the product of the roof-fall dimensions. This procedure calculating volume is not entirely precise because it forces all roof falls to have the shape of a rectangular parallelepipeds, which in reality they are not. A shape factor could have been incorporated into the volumetric calculation to account for a more realistic roof-fall shape; however, this would only add to the uncertainties already involved.

The energy of each roof fall shown in column 9 of table 1 is calculated from the equation

$$E_p = \gamma H \times \text{vol}, \tag{1}$$

TABLE 2. - Roof-fall rehabilitation statistics

(59 accidents, 1966-82, as reported in table 1)

	Void height, ft	Roof-fall dimensions				Energy	
		Length, ft	Width, ft	Thickness, ft	Volume, ft^3	ft·lbf	ft·lbf/ft
Sum..........	738.7	598.5	384.5	63.7	4,092.9	6,966,676.0	685,107.0
Mean..........	12.5	10.1	6.5	1.0	69.3	118,079.2	11,611.9
Median........	12.0	8.0	5.0	.8	27.0	58,286.0	8,941.0
Standard deviation....	6.6	9.6	5.1	.9	84.0	160,630.5	13,009.1
Maximum.......	35.0	65.0	22.0	4.0	431.6	948,657.0	60,662.0
Minimum.......	4.0	2.3	1.3	.2	1.6	2,167.0	722.0
Range.........	31.0	62.6	20.6	3.7	429.9	946,490.0	59,940.0
Skewness......	1.1	3.6	1.5	1.7	1.9	2.9	2.1
Kurtosis......	1.6	17.5	1.9	2.7	4.8	11.6	4.7

where E_p = potential energy, ft•lbf,

γ = weight density, lbf/ft^3,

H = void height, ft,

and vol = volume, ft^3.

The weight density of all of the roof falls was assumed to be 157 lbf/ft^3. The energy calculated by equation 1 represents the potential energy lost owing to a change in position of the rock from the mine roof to the mine floor; stated in another way, this energy is the kinetic energy of the rock on impact with the mine floor. The energy levels shown in column 10 of table 1 represent the kinetic energy of a roof fall per unit length and are computed by dividing column 9 by column 5. This is the most useful form of energy for design purposes because the energy absorption capacity of an arch canopy per linear foot can be compared to the energy of a roof fall per linear foot. An in-depth explanation for this approach is given in the following section.

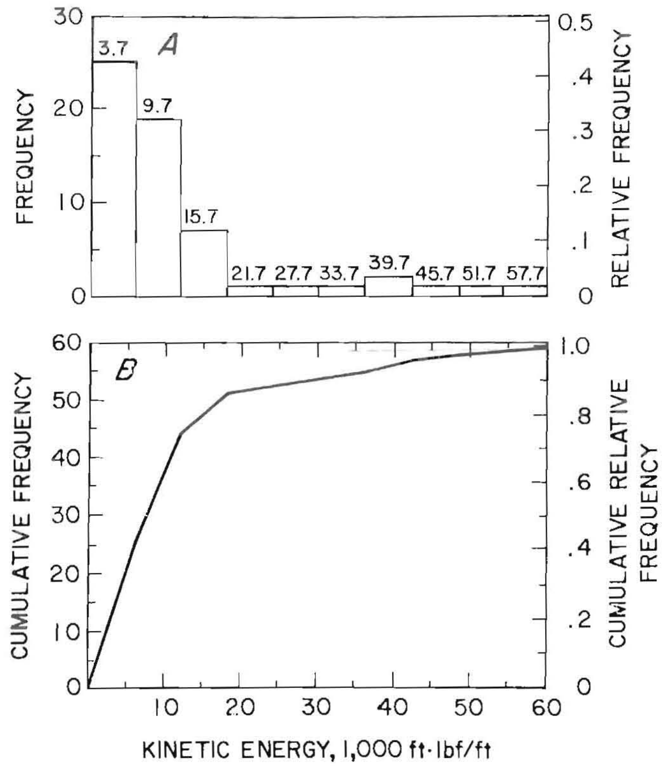

FIGURE 22.—Kinetic energy of roof falls at mine floor. *A*, Histogram; *B*, cumulative frequency.

The kinetic energies of the roof-fall rehabilitation accidents can be presented graphically to explain their distribution and frequency of occurrence. Figure 22*A* is a histogram that relates the kinetic energy of the roof falls on impact with the mine floor to the frequency of occurrence for all of the roof-fall rehabilitation accidents given in table 1. As can be seen from this figure, a majority of the roof falls had kinetic energy values less than 18,700 ft•lbf/ft. The kinetic energy values of the roof falls can also be presented as a cumulative frequency graph (fig. 22*B*) so that the number and/or fraction of the roof falls having a kinetic energy above or below a specific value can be ascertained quite easily.

Based upon the kinetic energy distribution of the roof falls, an energy absorption requirement of 20,000 ft•lbf/ft has been selected for purposes of discussion and displaying the use of the design procedure. As can be seen in figure 22*B*, this energy level represents the point at which the slope of the cumulative frequency graph radically changes. The implication of this energy level is that an arch capable of absorbing 20,000 ft•lbf/ft of energy would be expected to provide protection against at least 87 pct of the roof-fall rehabilitation accidents that occurred from 1966 to 1982. This energy absorption requirement is conservative when the following factors are considered:

1. Roof falls are usually not composed of a single large mass of rock but of many layers of rock. Generally, when a roof fall does occur, the rocks separate from the mine roof in layers along weak bedding planes.

2. Roof falls comprised of small thicknesses will more readily break up on impact with an arch canopy; therefore, not all of the energy of the roof fall will be transmitted to the arch canopy. A review of table 1 reveals that a significant portion of the roof falls were, in fact, small in thickness.

3. The energies given in table 1 are for roof falls impacting the mine floor and not a structure with an appreciable height.

ARCH CANOPY DESIGN CONSIDERATIONS AND PROCEDURE

The purpose of the proposed design procedure is to provide a reasonable and reasonably accurate means of selecting currently manufactured arch canopy components for entry rehabilitation. The requirement of safety dictates that an arch canopy should not deflect more than some maximum amount under the action of a subsequent roof fall. The following procedure is intended to control that maximum deflection.

Although the problem is a dynamic one, this procedure has been developed so that the selection of arch canopy components can be made on the basis of the energies involved, quantities that can be determined analytically or from static tests. These energies consist of the potential energy of an anticipated roof fall and the strain energy, both elastic and plastic, absorbed by the structure during deformation.

One difficulty recognized at the outset of this investigation was that roof falls could occur over small regions of a rehabilitated entry, thereby loading only a portion of the arch canopy. As a result, the resistance mobilized by the structure would be three-dimensional in nature and would be an interaction between the directly loaded and the unloaded portions. On the other hand, a roof fall of the same width and thickness but extending over the entire length of the arch canopy would, by definition, load all portions of an arch equally, with the result that the response of the structure would be the same throughout its length. Since each unit length of an arch must resist its load without any help from adjacent units, it follows that this is a more severe condition than the previous one. It is for this reason that the statistics of the energies of the roof falls in the previous section and the properties of the arch in the design procedure that follows are all expressed in terms of "per unit length" or "per foot."

DESIGN ENERGY

The preceding section has demonstrated that for the roof-fall data available, 87 pct of all rehabilitation accidents were due to roof falls involving less than 20,000 ft•lbf of energy per foot length of roof-fall. The value of 20,000 ft•lbf/ft will be used as the basis for discussion, and a curve (fig. 23) can be plotted of weight of rock per foot, W, versus void height, H, for all possible falls possessing that much energy. Thus a 20,000-lbf/ft rock falling 1 ft and a 20-lbf/ft rock falling 1,000 ft are represented by the same curve. Figure 24 illustrates the notation used in figure 23 and subsequent discussion.

The implication of figure 23 is that the greater the void height, the lighter will be the rock that falls to create an energy of 20,000 ft•lbf/ft. While this may in fact be true, the concern here is primarily energy, not weight of rock, although weight of rock will enter into the discussion. If the void height is observed to be a certain magnitude, H, then the weight of rock per foot of length for design purposes will be given by the curve as 20,000/H.

For the purposes of safety and design, a minimum clearance (h_p) at the crown at

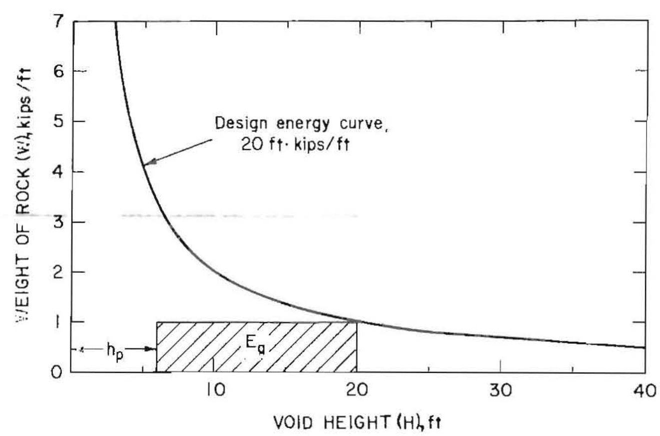

FIGURE 23.—Design energy curve.

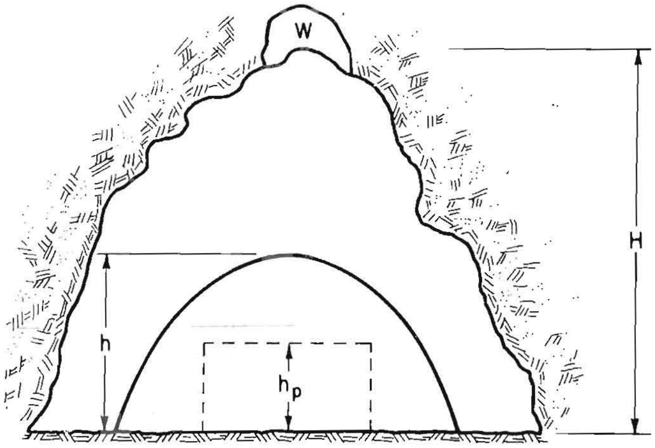

FIGURE 24.—Dimensions for design calculations.

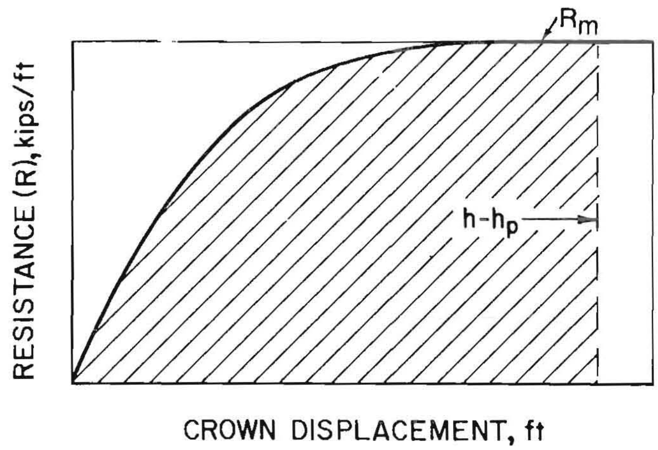

FIGURE 25.—Typical resistance function.

the time of maximum deflection needs to be established, and for discussion purposes will be equated to 6 ft. The gross energy available for deforming the arch is, therefore, the loss in potential energy of the rock, namely,

$$E_g = W(H-h_p), \qquad (2)$$

where W = weight of rock, lbf/ft,

H = void height, ft,

and h_p = protection height, ft.

This energy is not a constant but increases as H increases (subsequently W decreases) and approaches 20,000 as H approaches infinity. A typical value of E_g is shown as the shaded area in figure 23.

STRAIN ENERGY

The area under a load-displacement diagram represents the amount of strain energy a structure is capable of absorbing. For an arch canopy, the strain energy occurring during deformation will always be expressed as energy per linear foot (E_a) for this design procedure. This will allow a comparison to be made between the amount of energy that an arch is capable of absorbing and the energy of a roof fall (ft•lbf/ft). The availability of load-displacement diagrams, whether they are developed analytically or experimentally, should be the responsibility of the arch canopy manufacturers.

In conducting a pull test on an arch, both load and deflection would be recorded until the crown came to within h_p ft of the base. The load-deflection curve might look something like figure 25. The area under the curve represents the amount of strain energy, E_a, per foot that the structure is capable of absorbing during the deflection h-h_p, where h is the height of the arch prior to deformation (fig. 24).

For liner plate arches, the load of the pull test should be applied as a line load (uniformly distributed load) along the length of the crown. The total load is then divided by that length to obtain a load per foot of length of the arch. The area under the curve is the energy absorbed per foot of length of the arch canopy. Thus, it does not matter how long the arch in the pull test is, provided it is long enough to prevent buckling out of its plane. The problems of testing steel sets will be discussed later.

It should be noted that, once yielding begins, the load values determined by a pull test will probably be higher than would result if the arch were constructed of material whose yield strength was only the minimum guaranteed by the mill. Therefore, a more appropriate measure of energy absorption is one where the observed energy absorption is multiplied by the ratio of the specified minimum yield to the actual yield, as determined from tests on coupons taken from the same heat as the tested arch.

ENERGY LOSSES

At the instant of contact between the falling rock and the arch canopy at rest, there begins a time-dependent force that acts to slow down the rock and accelerate the arch canopy at its crown. The time interval over which the two have different velocities is quite small. Furthermore, the collision between the rock and the arch canopy crown is a fully plastic impact (completely inelastic collision). Hence, both the rock and the crown of the arch can be said to be moving at the same velocity after a negligibly small time, compared to the natural period of vibration of the structure. Thus, the arch appears to have an instantaneous velocity with no initial displacement at the instant of impact. From that time on, the rock and arch move together as a single-degree-of-freedom system until after the maximum deflection is reached.

This instantaneous velocity, V_o, for the arch may be found from equating the momentum of the rock just prior to impact with the momentum of the rock and the arch just after impact and is

$$M_r \sqrt{2G\ (H-h)} = (M_r + M_a)\ V_o, \qquad (3)$$

where M_r = mass of rock, slugs/ft,

G = acceleration of gravity, ft/s^2,

h = height of arch canopy, ft,

M_a = effective mass of the arch (i.e., the mass required to represent the arch as a single spring-mass system in simple harmonic motion), slugs/ft,

and V_o = instantaneous velocity of the arch at impact, ft/s.

The effective mass (M_a) of the arch can be determined experimentally or can be approximated by Rayleigh's method, and this information should be provided by the arch canopy manufacturers.

Although momentum at impact is preserved, kinetic energy does not appear to be. The transmission ratio (r_t) of kinetic energy after impact to the kinetic energy prior to impact is given by

$$r_t = \frac{1/2\ (M_r + M_a)\ V_o^2}{M_r\ G(H-h)}. \qquad (4)$$

Substitution of V_o from equation 3 into 4 leads to

$$r_t = \frac{M_r}{M_r + M_a}. \qquad (5)$$

The kinetic energy "lost," represented by $1-r_t$, goes into local deformations, heat, noise, and the excitation of various natural frequencies of the arch higher than the fundamental vertical one.

The transmission ratio, r_t, is strictly applicable only to the kinetic energy of the rock at the time of impact, i.e., the energy represented by W(H-h), not the gross energy available as given in equation 2. Thus, one would expect the ratio of energy absorbed by the arch to the gross energy available to be greater than r_t, if all energy not otherwise lost at impact was absorbed. As is shown in the next section, the absorption ratio for point-loaded arches appears to be less in some cases, depending upon the magnitude of r_t. It remains to be seen what the absorption ratio for line-loaded arches will be. It is tentatively proposed that the absorption ratio be assumed equal to r_t.

The assumptions of rigid body mechanics employed above do not take into account the strain energy absorbed by the rock during impact. If this is sufficient to cause fracture and shattering of the rock, which it frequently will, then even less energy is available for deforming the arch.

DESIGN CRITERIA

There are two design criteria that must be satisfied if any given arch is to be acceptable. The first is a dynamic criterion using the assumption of the previous section. Expressed in the form of an inequality for design purposes, this assumption requires that

$$\frac{E_a}{E_g} > r_t, \tag{6}$$

where E_a is the amount of energy that the arch canopy is capable of absorbing. The energy absorbed by an arch canopy can be calculated from the following equation, which is derived from an elasto-plastic resistance diagram (fig. 26):

$$E_a = R_m \, y_e/2 + R_m \, (h-h_p-y_e), \tag{7}$$

where R_m = maximum resistance, lbf/ft,

and y_e = yield limit, ft.

Equation 6 is strictly an energy relation and by itself does not actually guarantee that the arch will not collapse under the dead weight of the rock. Such a situation might occur if the safety zone, h_p, was only slightly less than the arch height, h, which in turn was only slightly less than the void height, H. In such a circumstance the gross energy available from figure 23 would be quite small, and the energy absorption required might be even less. We must therefore specify that the maximum resistance, R_m, that the arch can develop must be greater than the weight of rock, W.

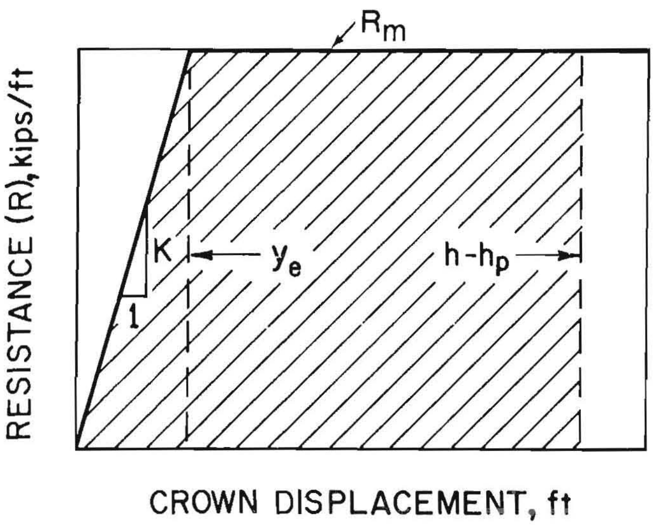

FIGURE 26.—Idealized elasto-plastic resistance function.

How much greater R_m should be than W can also be determined from energy considerations. Equations 2 and 7 can be substituted into equation 6 to obtain

$$r_t \, W(H-h_p) < R_m \, y_e/2 + R_m \, (h-h_p-y_e), \tag{8}$$

where R_m and y_e are depicted in figure 26 as the maximum resistance and the maximum elastic displacement, respectively. Rearranging terms in equation 8 leads to

$$\frac{W}{R_m} < \frac{h-h_p-y_e/2}{r_t \, (H-h_p)}. \tag{9}$$

For the situation where $h-h_p$ is small (it can never be zero because there would be no allowance for crown displacement) and H = h (when H-h is zero, the initial velocity is zero; the transmission ratio is undefined but can be taken as equal to unity), equation 9 can be written as

$$\frac{W}{R_m} < \frac{h-h_p-y_e/2}{h-h_p}. \tag{10}$$

To estimate y_e from a resistance function, as in figure 25, the curved portion may be replaced by a straight line enclosing the same amount of area.

One factor that contributes to the conservativeness of this procedure is that the arch canopy will almost always be longer than the roof fall, so that the structure may resist the load in a three- instead of a two-dimensional manner. In such cases, the structure will always be stronger than when it is line loaded, for the same load per unit length. Even when a rock strikes the lip of the canopy during erection, it will be a less severe case than for a fully loaded structure.

Another extreme condition occurs, however, when the void height, H, is very great so that the weight of rock, W, is small. In this case the impact velocity becomes quite large. It may be possible for a small rock of short length and high velocity to puncture the canopy and even pass through it without otherwise permanently deforming it. The combination of parameters at which this would become the design criterion is unknown at this time.

DESIGN PROCEDURE

The design procedure is based upon a limiting design energy, such as the basis of figure 23, and a safety zone of height, h_p. With these in place the mine engineer proceeds as follows:

1. By observation determine the void height, H.
2. Using H and a design curve similar to that shown in figure 23, determine the weight of rock per foot, W.
3. Using W, H, and h_p, calculate the gross energy per foot available, E_g, from equation 2.
4. Using the reduced load-deflection curve or other energy desorption information based on minimum material properties that may be supplied by the arch canopy manufacturer, select an arch canopy that will satisfy equation 6.
5. Check that the maximum resistance R_m is greater than W in accordance with equation 9 if H > h or equation 10 if H = h.

In step 4 above it is assumed that the information supplied by an arch canopy manufacturer will also include a value for the effective mass, M_a, of the arch for use in equation 5 for finding r_t. It is quite possible that a manufacturer will want to incorporate steps 2 through 5 into design charts or tables that will cover all permissible values of H for its product.

Example

Select an arch canopy to rehabilitate a mine entry with a void height (H) of 17 ft. Use the energy curve of 20 ft·kips/ft and a protection height (h_p) of 6 ft. Three hypothetical products are available for selection; their engineering properties and dimensions are given in table 3.

The weight of the rock is obtained from the design energy curve as 1.18 kips/ft (W = 20 ft·kips/ft ÷ 17 ft). The gross energy available is given by equation 2 as E_g = 1.18 kips/ft × (17 ft − 6 ft) = 12.98 ft·kips/ft. The mass of the rock in units of slugs per foot is 36.5 (M_r = 1.18 kips/ft × 1,000 lbf/kip × (slug·ft/s^2)/lbf ÷ 32.2 ft/s^2). From equation 5, the transmission ratio for arch A is 0.92 [$r_t = 36.5/(36.5 + 3.2)$]. The energy absorption requirement for arch A is obtained from equation 6 and must be greater than or equal to 11.9 ft·kips/ft [$E_a \geqslant r_t \; E_g = 0.92 \times 12.98$)]. As can be seen from table 3, E_a for arch A is not greater than 11.9 ft·kips/ft. Therefore, arch A cannot be considered for this mine entry.

Next, consider arch B as a candidate to rehabilitate the mine entry. The transmission ratio (r_t) for arch B is 0.90. The required energy absorption for arch B must be greater than 11.68 ft·kips/ft. Since the energy absorption of arch B is greater than 11.68, the next step is to check the arch's static strength against the required static strength, which is obtained from equation 9. The required static strength (R_m) is 3.05 kips/ft. Since the value of R_m for arch B is greater than 3.05, arch B is satisfactory for rehabilitating this mine entry. Similar calculations will show that arch C is also satisfactory.

CROWN DEFLECTION CALCULATIONS

In the event that the actual crown deflection is desired, an estimate can be made from a statement of the equality of potential energies more exact than

TABLE 3. - Arch canopy design data

Data	Arch A	Arch B	Arch C
Height, h..................................ft..	9	10	11
Energy absorption, E_a[1]..............ft·kips/ft..	11	13.2	15.9
Effective mass, M_a.....................slugs/ft..	3.2	4.1	5.3
Maximum resistance, R_m.................kips/ft..	3.96	3.45	3.26
Maximum elastic displacement, y_e.............ft..	0.44	0.35	0.26

[1]Based on the crown deflection of an arch equal to $h-h_p$, where h_p = 6 ft.

equation 8. By applying the transmission ratio only to the free fall of the rock and equating energy available to energy absorbed by an elasto-plastic resistance function (fig. 26), the following equation is derived for the deflection of an arch into its plastic range:

$$W(H-h)\, r_t + (W + W_a)\, y_{max} = R_m\, (y_{max} - y_e/2), \tag{11}$$

where y_{max} is the maximum crown displacement. Dividing both sides of equation 11 by K, the arch stiffness per unit length, and recognizing that $R_m/K = y_e$, $W/K = y_s$ (the deflection the arch would experience if W was statically applied), and $W_a/K = y_a$ (the deflection the arch would experience if W_a was statically applied), equation 11 can be arranged to obtain

$$y_{max} = \frac{r_t\, y_s\, (H-h) + y_e^2/2}{y_e - y_s - y_a}. \tag{12}$$

It should be noted that the unit stiffness, K (lbf/ft/ft), used to determine y_e, y_a, and y_s above, is the average stiffness found by replacing the nonlinear load-displacement curve between no load and the fully plastic condition (fig. 25), R_m, by a straight line such that the area under the curve remains unchanged (fig. 26).

Figure 27 displays the use of equation 12 for determining the deflections of the crown of an arch canopy for various void heights (H). The properties of the arch canopy are given in the design data section of figure 27; also shown is the resistance function, which was obtained from a load-deflection curve similar to the one shown in figure 25. As has been

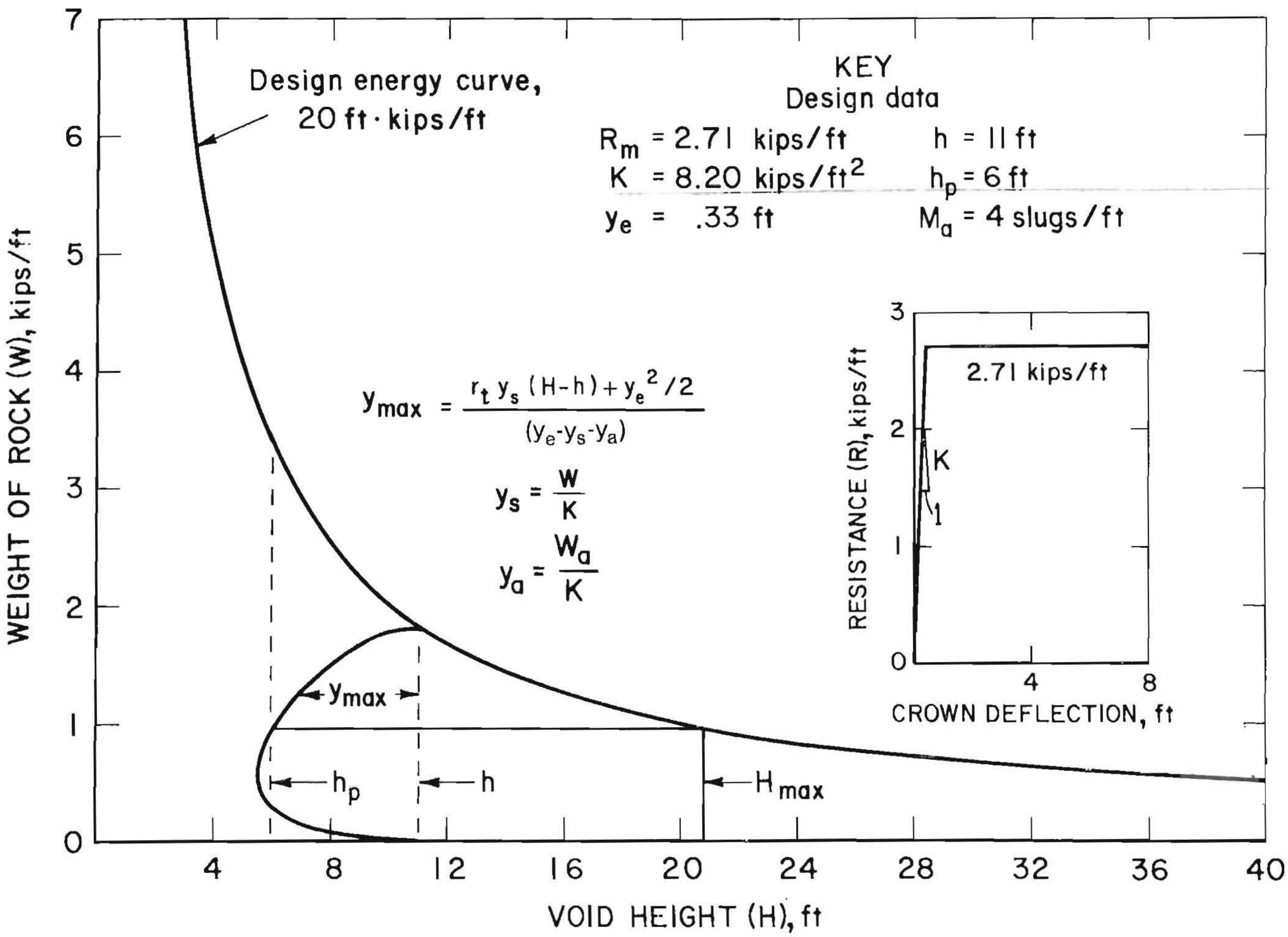

FIGURE 27.—Arch canopy deflections.

previously discussed, there are two design criteria that an arch canopy must satisfy for a particular void height: (1) The maximum resistance (R_m) of the arch canopy must be greater than the weight (W) of the rock, and (2) the quantity $h-y_{max}$ must be greater than the protection height (h_p). The height of the arch canopy crown (h) is given as 11 ft, and the weight of the rock at this void height (11 ft) is approximately 1.82 kips/ft. The significance of this calculation is that the maximum static weight of rock the arch canopy would be subjected to (for the design energy of 20 ft·kips/ft) is 1.82 kips/ft. Furthermore, since the maximum resistance of the arch canopy is greater than 1.82 kips/ft, the governing design criterion is the deflection of the arch canopy crown. By selecting various quantities for the void height and solving for y_{max} (equation 12), a curve for the crown deflections can be plotted as part of the design energy curve. At the first point at which y_{max} crosses the protection height boundary (i.e., $h-y_{max} = h_p$) the arch canopy can no longer be used, and a stiffer arch canopy must be selected.

For resistance functions other than the simple bilinear elasto-plastic case, the calculation of maximum crown deflection becomes more difficult but is still possible through the application of the principles discussed earlier. The right-hand side of equation 11 must be altered to accommodate the shape of the resistance function; this may result in a formulation whereby y_{max} (equation 12) will have to be determined by trial and error.

In any event, this approach is conservative and yields maximum deflections that may be in error by as much as 20 pct even when the falling rock does not break up on impact. This is due to variations in the extent of local deformation during the instant of impact. If the roof fall does not extend over the entire length of the canopy or if the rock breaks up on impact, these calculations will yield conservatively large values of deflection.

IMPACT TEST STRUCTURE

The impact test structure (ITS) was designed to provide a versatile testing frame for the static and dynamic testing of various arch canopy and arch canopy-backfill system configurations. Static tests are conducted with the ITS by using a hydraulic load ram that applies a downward load (pull force) to the crown of the arch canopy. The ITS will also allow impact testing of arch canopies by the use of a crane-mounted release hook assembly that drops a tup from various heights. Sidewalls and end walls permit the placement of backfill on the sides or on top of the arch canopies to accommodate testing of arch canopy-backfill systems.

Figures 28 and 29 illustrate the front elevation and plan view of the ITS, respectively. As shown in figure 29, six reaction beams and a centerline beam span the width of the test structure. The reaction beams are firmly anchored to the reinforced concrete foundation with tension rebar and shear bolts. These beams are used to transmit the base reaction loads of the arch canopies to the foundation of the ITS during the static and dynamic tests. The centerline beam provides an anchor for the hydraulic cylinder during the static tests (fig. 30). The centerline beam is permitted to bend and is only restrained at its ends by transfer beams which are bolted to the reaction beams. This was done to avoid

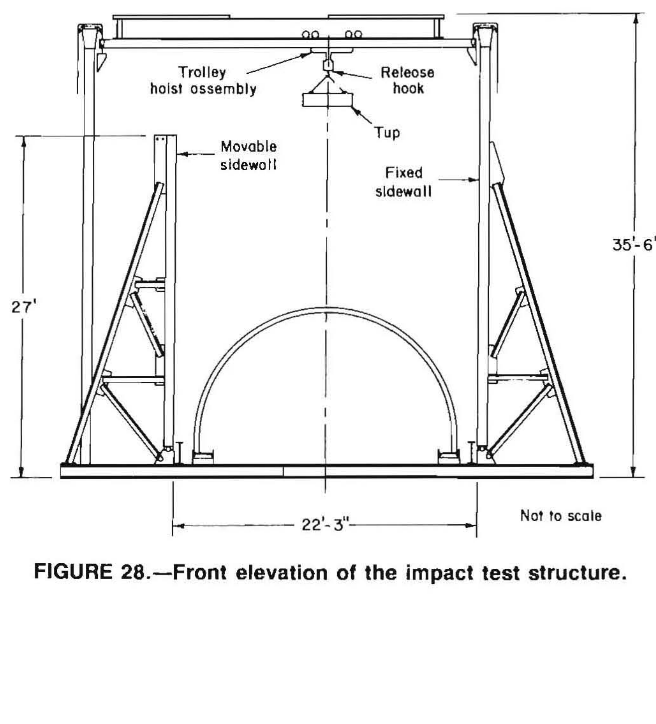

FIGURE 28.—Front elevation of the impact test structure.

installing numerous tension rebar bolts into the ITS foundation to anchor the centerline beam in place during the static pull test of a liner plate arch. Figure 31 shows the base reaction support for a liner plate arch in detail. As can be seen, the base reaction support allows rotation but prohibits translation of the arch canopy base. This base reaction support is not needed for the static

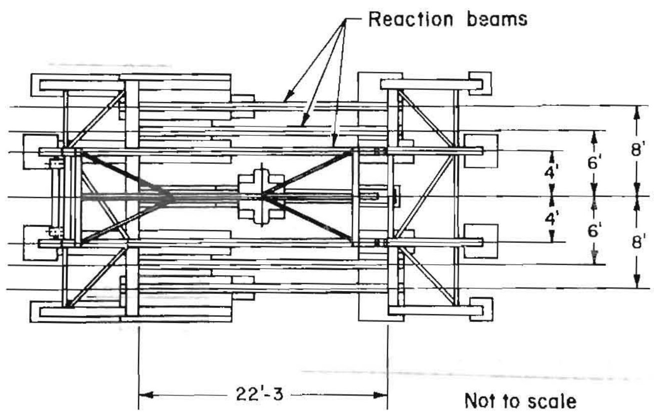

FIGURE 29.—Plan view of the impact test structure.

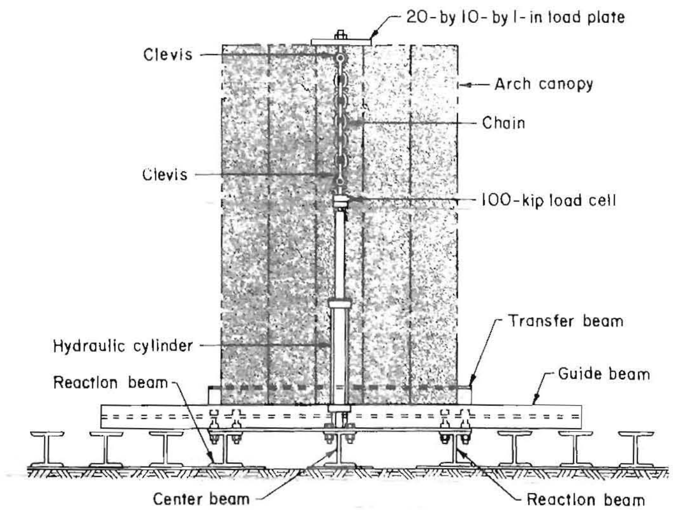

FIGURE 30.—Arch canopy installation for static test.

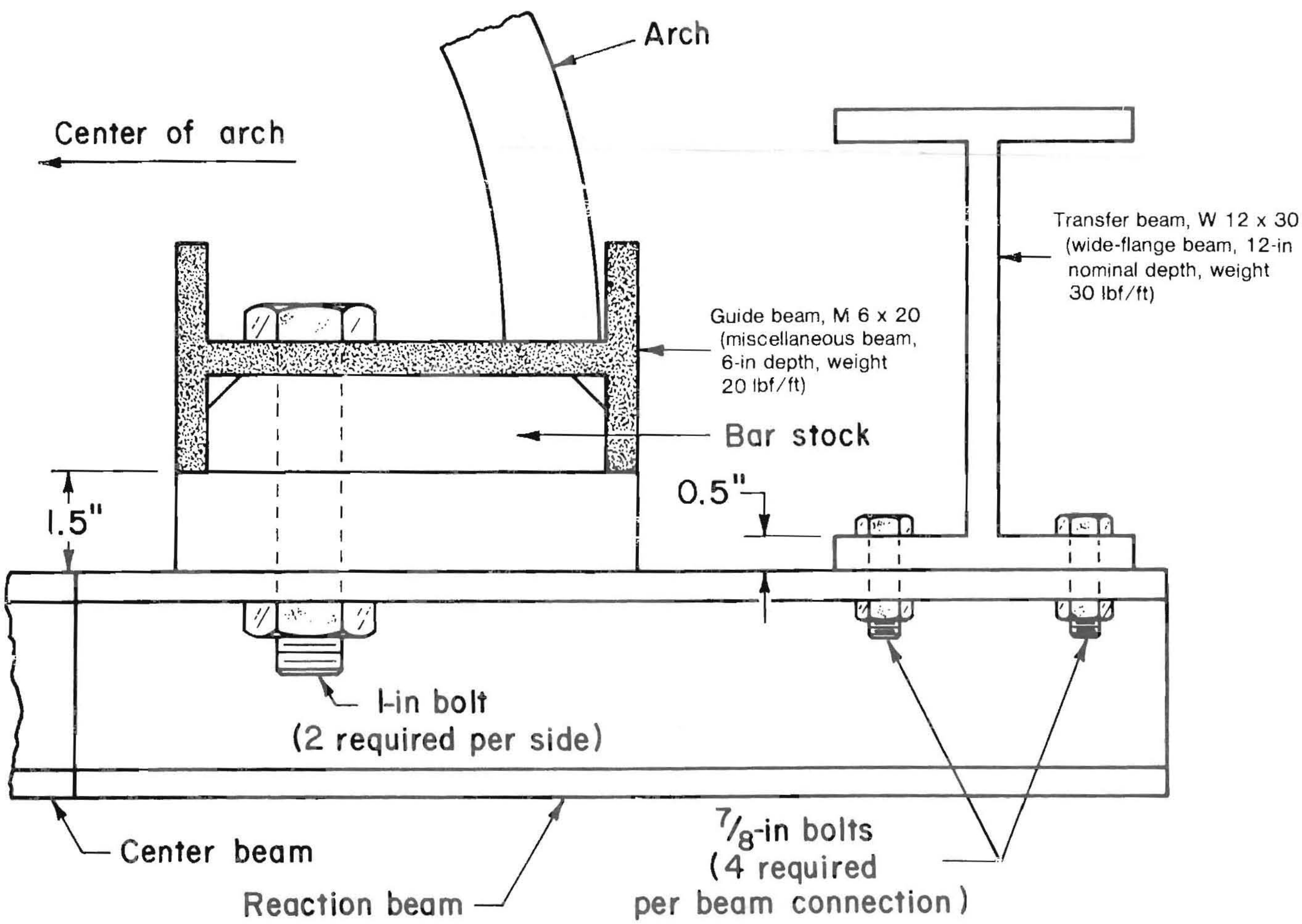

FIGURE 31.—Base reaction detail.

testing of a steel set arch because the leg members of the steel set serve as the restraining support for the centerline beam (fig. 32). However, additional steel sets can be installed and anchored (pin end condition) to the reaction beams to provide stability to the steel set arch during tests.

The sidewalls, shown in figure 28, provide a reaction frame for the backfill during the testing of arch canopy-backfill systems. One sidewall was designed to be movable, so that different widths of arch canopies can be accommodated, and also so that the amount of backfill installed between the arch canopies and sidewalls can be varied. End walls are also required to contain the backfill material. The end walls are constructed of steel beams and wood lagging; the steel beams are bolted to the vertical columns of the sidewalls, and the wood lagging is placed between the flanges of the steel beams to contain the backfill.

The tup support tower was designed to provide a maximum drop height of 30 ft, which is measured from the bottom of the release hook to the top of the centerline beam. The trolley-mounted hoist crane is used to hold and position the tup release hook. The trolley hoist, which has a working rate of 15,000 lbf and an ultimate rating of 60,000 lbf, is also used to position the movable sidewall. The tup is attached to the trolley-suspended, 6,000-lbf-rated (ultimate rating is 27,000 lbf) helicopter release hook. To drop the tup, a 24-V, 15-A signal is supplied to the solenoid of the release hook assembly.

The two tups fabricated for the impact tests are constructed of concrete and 1/4-in steel plate. The tups weigh 882 and 3,150 lbf[7] and have impact surfaces of 17 by 24 in and 36 by 25 in, respectively.

Figure 33 is a schematic of the hydraulic system used for the static tests. Figure 30 shows the hydraulic cylinder attached to the liner plate arch and centerline beam. This attachment is made via a load plate, load cell, chain, eyebolts, and clevices to obtain the desired 36-in cylinder extension before static

[7]The 3,150-lbf tup was used for the nondestructive impact tests; a 200-lbf chain was installed onto it for the destructive impact tests as a safety device.

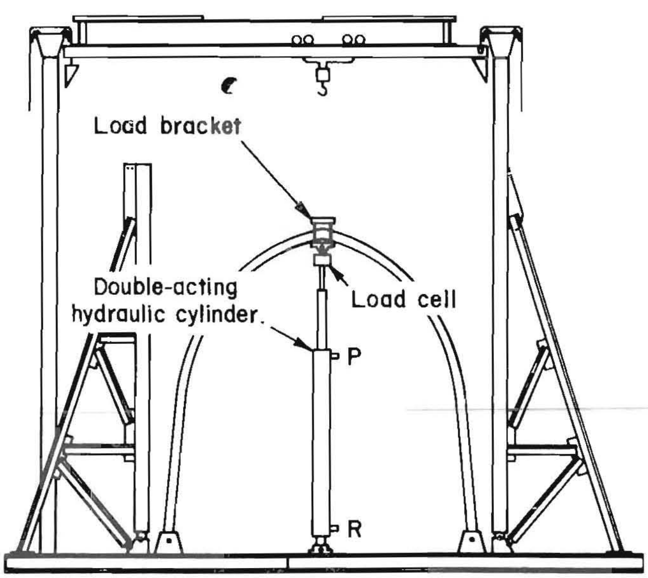

FIGURE 32.—Static-load test configuration for steel set

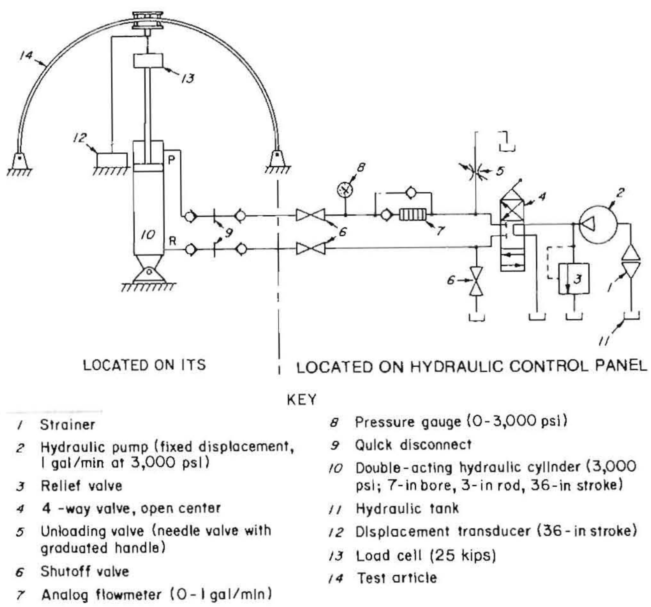

FIGURE 33.—Schematic of hydraulic system used for static-pull test.

testing begins. The power center has a relief pressure setting of 3,000 psi. The double-acting hydraulic cylinder is capable of generating 94,000 lbf of pull force at this relief setting.

TEST ARTICLE

The test articles selected for the static and dynamic tests were liner plate arches. Although liner plate arches were used to determine the dynamic behavior characteristics of arch canopies, steel set arches would have been equally suitable for experimentation and dynamic testing. The design procedure for arch canopies evolved from structural and dynamic analyses, and from dynamic and static physical testing.

Each liner plate arch was comprised of five rings to prevent it from buckling out of its plane and also to preserve its symmetry of behavior with respect to its midlength plane. Every ring was constructed of nine liner plates (four 12-Pi plates and five 16-Pi plates) to form a semielliptical arch (fig. 34) with a radius of 9 ft 10-9/16 in turning 194°, a span (width) of 19 ft 7-1/2 in, a rise (height) of 11 ft 5/8 in, and a length of 7 ft 6 in. All of the liner plates were constructed of a 5-ga material (0.2092 in thick). The dimensions of the liner plate are given in figure 35 and in the following tabulation, which also gives certain strength and weight data (for a single plate):

Area................ in^2/lin ft..	3.263
Section modulus...... in^3/lin in..	0.0928
Moment of inertia.... in^4/lin in..	0.1031
Radius of gyration........... in..	0.616
Approx weight, including bolts, lbf:	
12-Pi plate..................	61
16-Pi plate..................	79

PHYSICAL TESTING PROCEDURE

The ITS was used to conduct static and dynamic tests on liner plate arches and will be used to conduct tests on other arch canopies and arch canopy-backfill systems. Only full-scale physical tests are currently being considered for this research project because of the uncertainty of the results that model testing would produce owing to the problems of achieving structural similarity. Static tests are performed to establish the behavior of the arch canopies in their elastic and plastic ranges. Besides providing a detailed understanding of the failure processes that the arch canopies

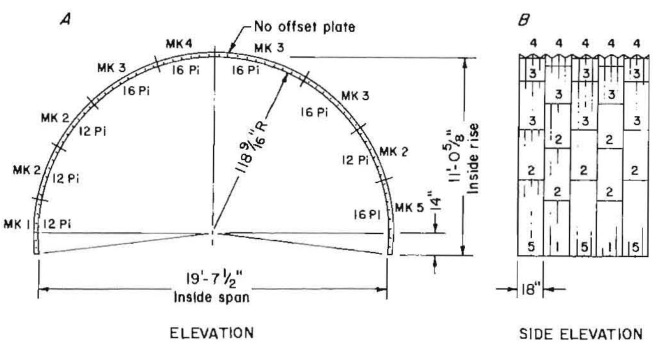

FIGURE 34.—Test article configuration. *A*, Elevation view; *B*, side view.

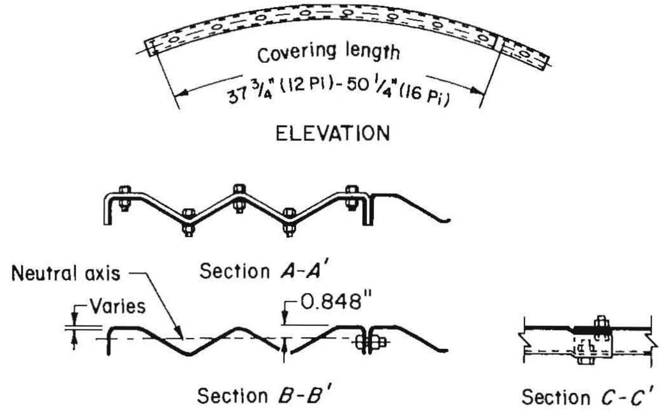

FIGURE 35.—Test article—liner plate.

undergo, these tests also allows the large amount of energy that each arch canopy is capable of absorbing as it is stressed beyond its elastic range to be determined. This information is critical in the design of the arch canopies because they will be dynamically loaded by recurring roof falls into their plastic ranges when they are used for rehabilitating high-roof-fall areas. The dynamic tests are used to determine the dynamic response of the arch canopies to impact loading and also to establish the total amount of energy that the arch canopies can absorb.

The static and dynamic test procedures outlined below were developed prior to any of the actual physical tests and were used for the full-scale physical tests described in this report. The experience and knowledge gained from conducting these tests have evolved into improved test procedures, which are described later in this report in the section entitled "Recommended Testing Procedures."

STATIC TEST

The liner plate arch was initially tested statically to establish its elastic and plastic behavior. A hydraulic cylinder was used to apply a point load to the crown of the test article. Although the pull force was applied to the middle ring of the assemblage, all of the rings were loaded and provided resistance since they were all bolted to one another. Equal increments of vertical deflection (crown deflection) rather than equal increments of vertical force were used to govern the incremental loads applied to the arch canopy. A tension load cell was used to accurately measure the applied point loads (figures 30 and 36). Displacement transducers were used to

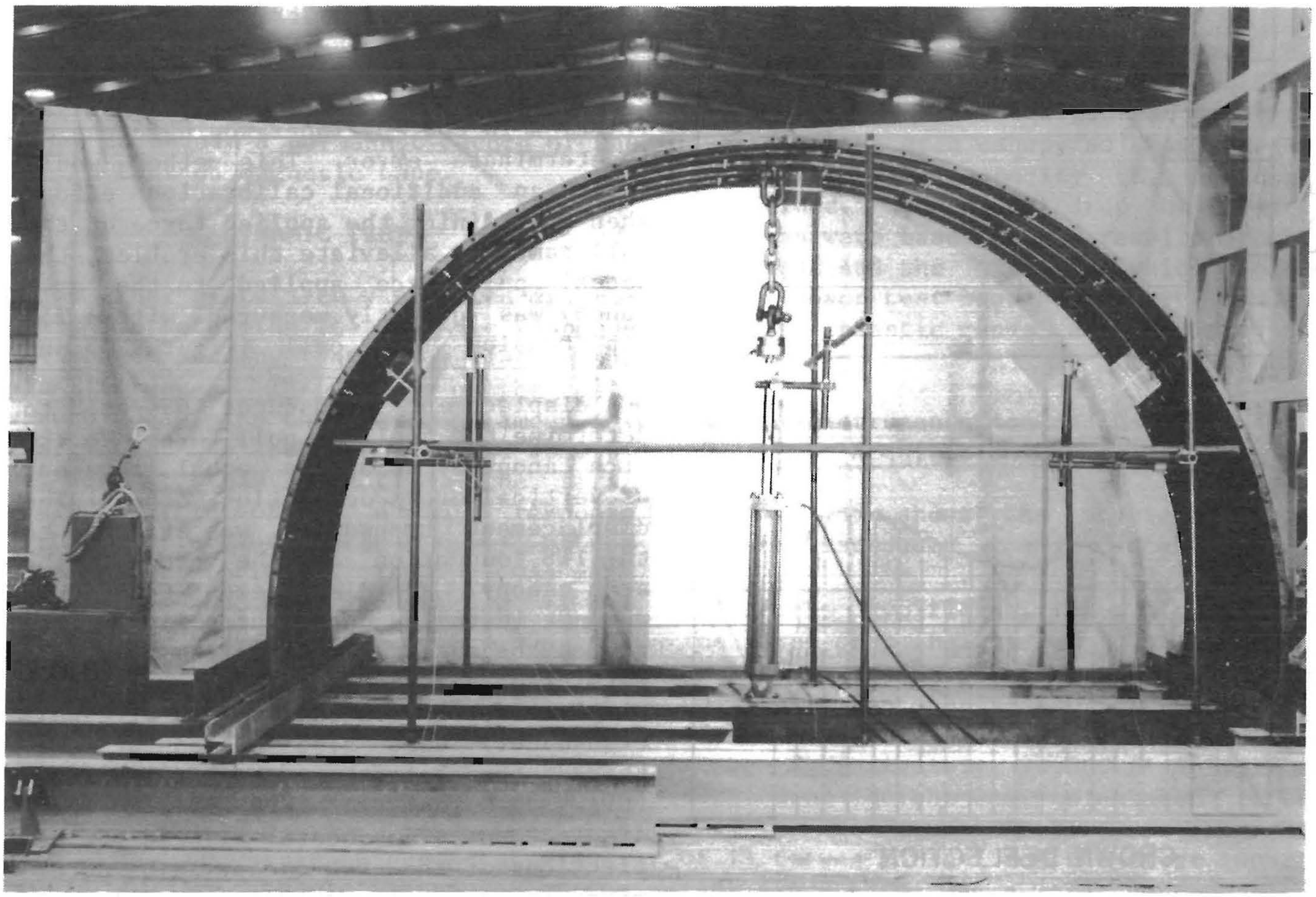

FIGURE 36.—Static test arrangement.

determine the deflections of the arch during the pull tests, and photographs were taken to obtain a permanent visual record of the deflection profiles.

Equal Increments of Vertical Deflections

The arch canopy was loaded with respect to equal increments of crown deflection because this procedure allowed the force-deflection diagram to be more accurately determined than it could be by the method of using equal increments of vertical force (fig. 37). The use of equal increments of deflection permits the crests and trough of a force-crown deflection diagram to be accurately determined. This is because small increments of deflection correspond to small changes in force when approaching the crest or trough of a force-deflection curve, whereas small increments of force can result in large changes in deflection for the same situation.

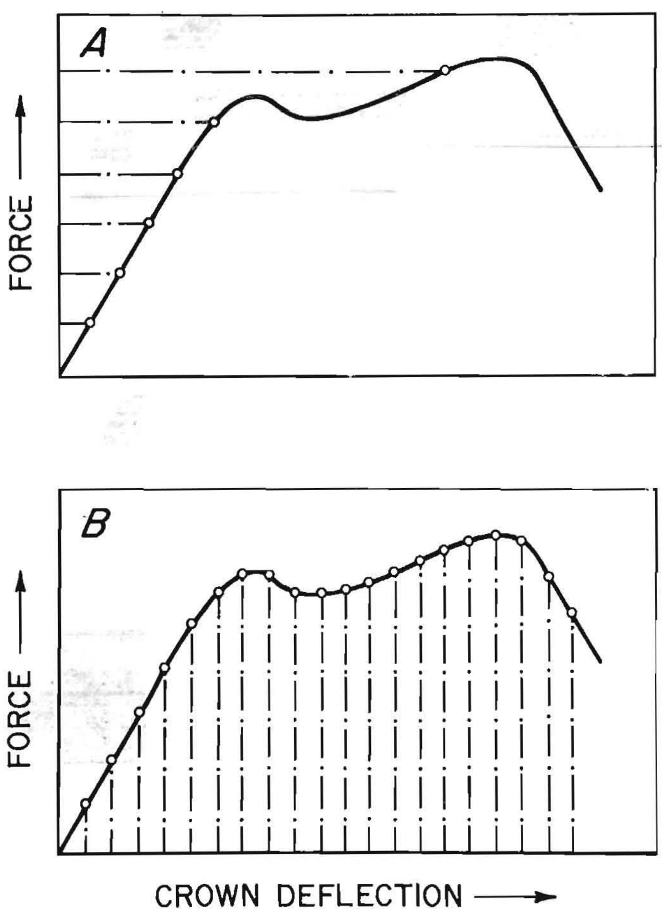

FIGURE 37.—Force-crown deflection diagram. *A*, Equal increments of force; *B*, equal increments of crown deflection.

Point Load

The arch canopy was point-loaded at the crown until failure of the structure occurred. Failure of an arch canopy was defined as the state of the structure when the crown was only 6 ft above the arch canopy base line. [For a steel set arch, the point load was to be applied across the width of the flange (fig. 32).] For the liner plate arch, the point load was evenly distributed across one ring of liner plate (fig. 30). A small hole was drilled through the crown of the liner plate arch to allow the installation of the load plate.

Instrumentation and Data Acquisition System

A pressure gauge installed in the hydraulic system of the pull ram could not be used to determine the point load applied to the arch canopy because internal friction in the cylinder would cause an indeterminate error. This method would create an additional calibration effort when determining the applied force of the pull ram. To alleviate this problem, the actual pull force applied to the arch canopy was directly measured with a 25-kip tension load cell (figures 33 and 36).

A displacement transducer was used to determine the applied pull force to the arch canopy by using the equal increment of vertical crown deflection method. The displacement transducer was attached to the ITS foundation and the crown of the arch canopy. The displacement transducer was not installed onto the pull ram base plates because the centerline beam was permitted to bend during a pull test. Displacement transducers (wire-pull transducer) were also placed at two-thirds the height of the arch canopy (fig. 38). The two sets of orthogonal displacement transducers allowed the deflections of the arch canopy to be determined in cartesian coordinates. The two extra monitoring locations were added

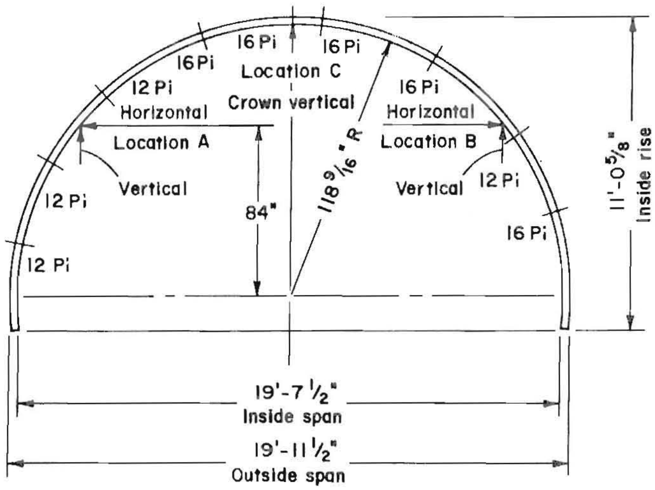

FIGURE 38.—Transducer locations.

since it was believed that the arch canopy would buckle during the static pull test.

An XY analog plotter was used to continuously plot the pull force versus crown deflection. This allowed the pull force to be applied as a function of crown deflection. An FM tape recorder was used to record the output of the displacement transducers and load cell in order to obtain a permanent record of the entire test on magnetic tape.

DYNAMIC TEST

The liner plate arch was tested dynamically to determine its dynamic response to impact loading into the elastic and plastic ranges. The purpose of the tests was to determine the maximum amount of energy that the arch canopy could absorb. To achieve this objective, the first quarter cycle of the dynamic response (crown deflection versus time) of the arch canopy to impact loading was measured. The tup weights utilized in the tests were 882, 3,150, and 3,350 lbf. The 882- and 3,150-lbf tups were used for the nondestructive tests, and the 3,350-lbf tup was used for the destructive tests. Instrumentation was used to measure the deflections of the arch canopies at three locations. Each impact test was also filmed to obtain a permanent visual record of the dynamic response of the arch canopy.

Drop Height and Tup Weight

The arch canopies were dynamically tested with respect to specified drop heights and a tup weight of 3,350 lbf. A series of four or five impact tests was desired so that the arch canopy could be incrementally brought within proximity to its failure state. The drop height (d_h), measured from the bottom of the tup to the top of the arch canopy crown, for the first destructive impact test was calculated by the equation

$$d_h = 0.25\ E_a/(r_t W_t), \qquad (13)$$

where d_h = drop height, ft,

E_a = total energy absorbed by arch canopy, ft•lbf,

r_t = transmission ratio,

and W_t = tup weight, lbf.

The use of equation 13 to calculate the drop height for an impact test required a force versus crown deflection curve to exist for the arch canopy so that the energy absorption capacity (E_a) could be determined.[8] Subsequent drop height selections were based on the results of the pull test and the extent to which a previous impact test brought the arch canopy into its plastic range and proximity to failure.

Instrumentation and Data Acquisition System

Figure 39 shows the typical wire-pull transducer and accelerometer

[8]For the two-flange liner plate arch, the first drop height was determined to be 11 ft. Based on the results of the pull test, the energy absorption capacity of the arch canopy (E_a) was determined to be approximately 98,000 ft•lbf. The first drop height was calculated to be 11.7 ft from equation 13. A drop height of 11 ft was chosen for the first impact test since this was a more conservative value than 12 ft.

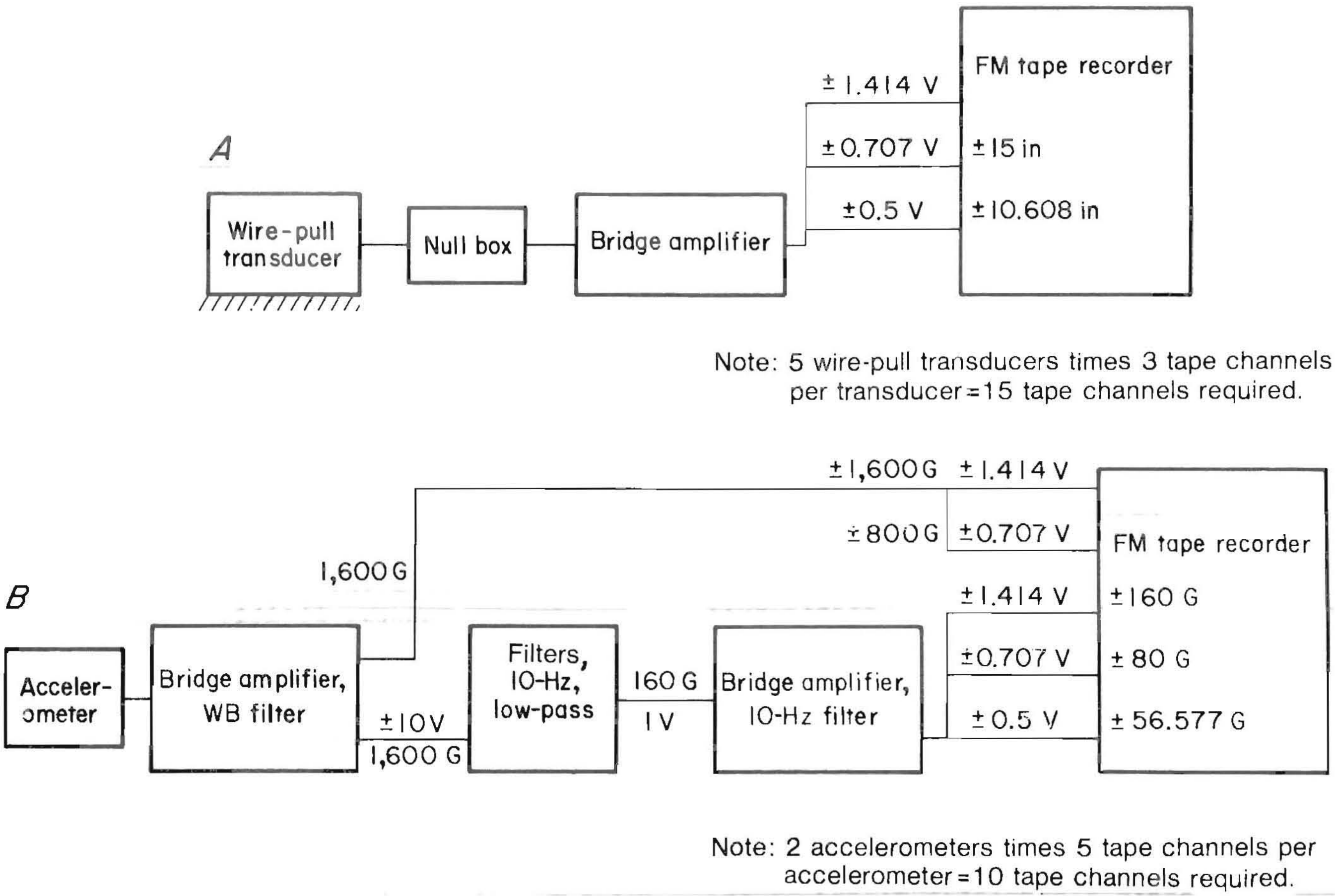

FIGURE 39.—Instrumentation and data acquisition system. *A*, Wire-pull transducer; *B*, accelerometer.

instrumentation systems. Five wire-pull transducers were used to measure the deflections of the arch canopy at the three locations specified in figure 38. The transducer at the crown (location C) measured "vertical" displacement, while the transducers at locations A and B measured both "vertical" and "horizontal" displacements. (The displacements are in quotations because the transducers only measured relative changes in wire pull length with respect to their anchorage position.) The data from each transducer were processed through an algorithm to obtain the desired output--the change in position of a point on the arch canopy in cartesian coordinates. The transducers were precalibrated prior to installation, and all transducers and cables were match-marked during calibration and installation to prevent channels from being crossed and to ensure that the calibrated transducer-cable combinations were never changed accidentally. The null box shown in figure 39 allowed for adjustment to a zero reference state. Movement of the wire inward caused negative voltages; outward movement caused positive voltages. A bridge amplifier was used to supply power to the transducers and to condition their signals. The gains were adjusted on the amplifier to allow ±30 in deflection to equal ±1.414 V from the amplifier. These signals were recorded on an FM tape recorder at three different input sensitivities. The voltage levels and corresponding engineering units are shown in figure 39*A*.

Three accelerometers were mounted to the crown of the arch canopies at location C, as shown in figure 38. In figure 39*B*, the first amplifier was used to power the transducer and for signal conditioning. The amplifier was set up to give full-scale output for ±1,600 G for a nonfiltered signal. The 1.414-V (±1,600 G) nonfiltered signal was sent directly to the tape recorder and recorded at two different input ranges. The ±10-V signal from this amplifier was fed to a filter. The filter was set for 10-Hz low pass. The output of the filter was sent to a second amplifier, which had gain settings set such that 160-G input (1 V) had a full-scale output of ±1.414 V. This signal was recorded at three different input levels by an FM tape recorder. Figure 39*B* shows the recorder input voltage levels and equivalent engineering units.

Photographic System

A permanent visual record of the dynamic response of the arch canopy to impact loading was desired and involved the use of the following photographic equipment and accessories. Figure 38 shows the transducers at locations A, B, and C. At these locations, incandescent panel-meter socket lights were also installed for a visual reference (fig. 40). Horizontal and vertical bars were installed in front of the arch canopy as a reference system. Luminous tape was attached to these reference bars to enhance their visibility.

Two types of cameras were used to document the impact tests. A large-format still camera was used to photograph the arch canopies before and after each impact. A 16-mm high-speed camera was used to photograph the dynamic response of the arch canopies to impact loading. The high-speed camera was operated at 48 frames per second. Appropriate lighting was used to reduce shadows to a minimum, achieve quality photographs, and ensure that the incandescent panel-meter socket

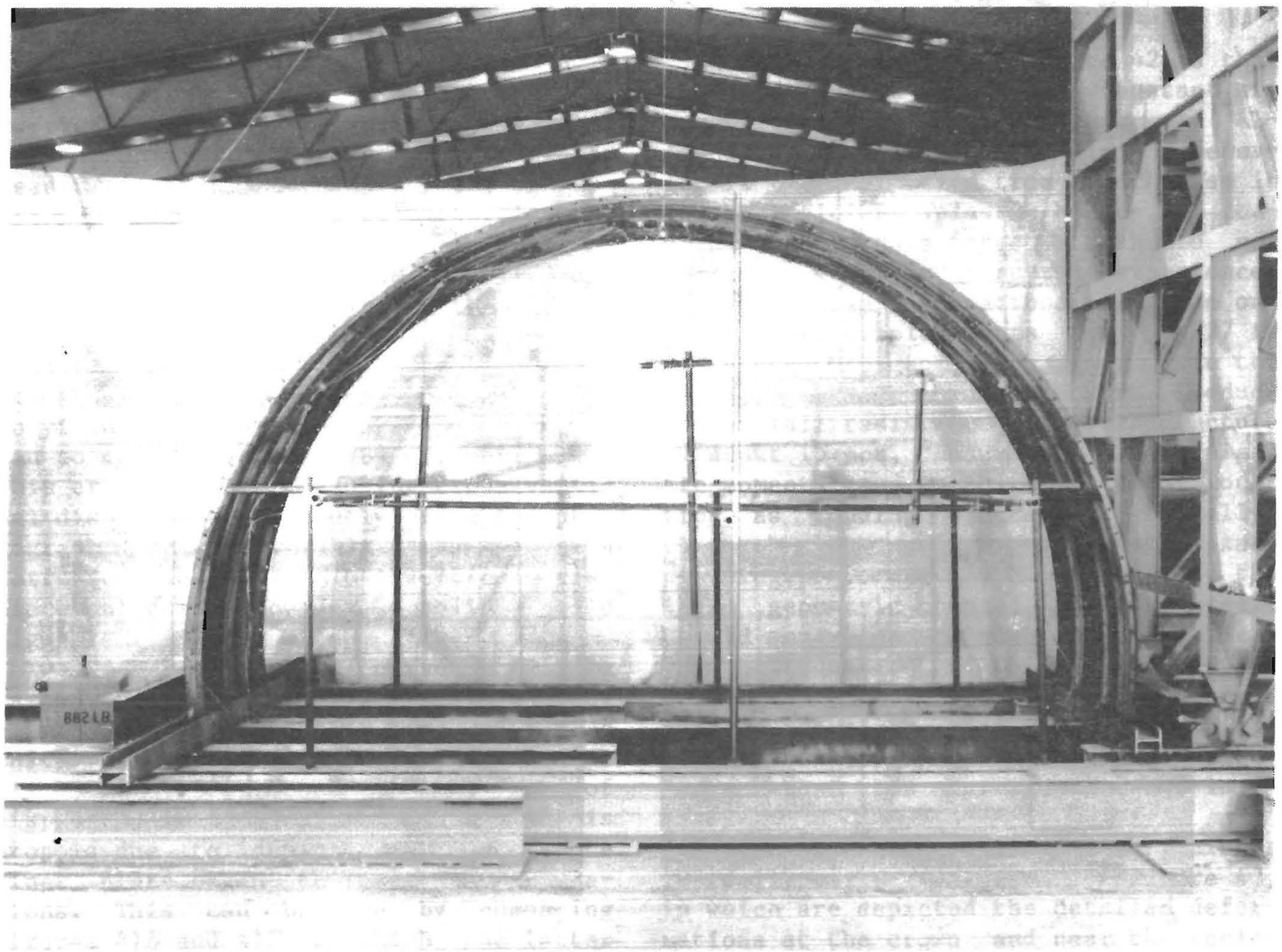

FIGURE 40.—Incandescent panel-meter light installations.

lights were detectable in the photographs. The cameras were positioned with their focal planes parallel to the center plane of the ITS and the axes of their lenses parallel to the longitudinal axis of the arch canopy.

TEST RESULTS

STATIC TEST

Perhaps the most eloquent statement of the performance of the liner plate arch under static load is given in figure 41. In these photographs the ram load varies from zero to a maximum value corresponding to its stroke limit and then is

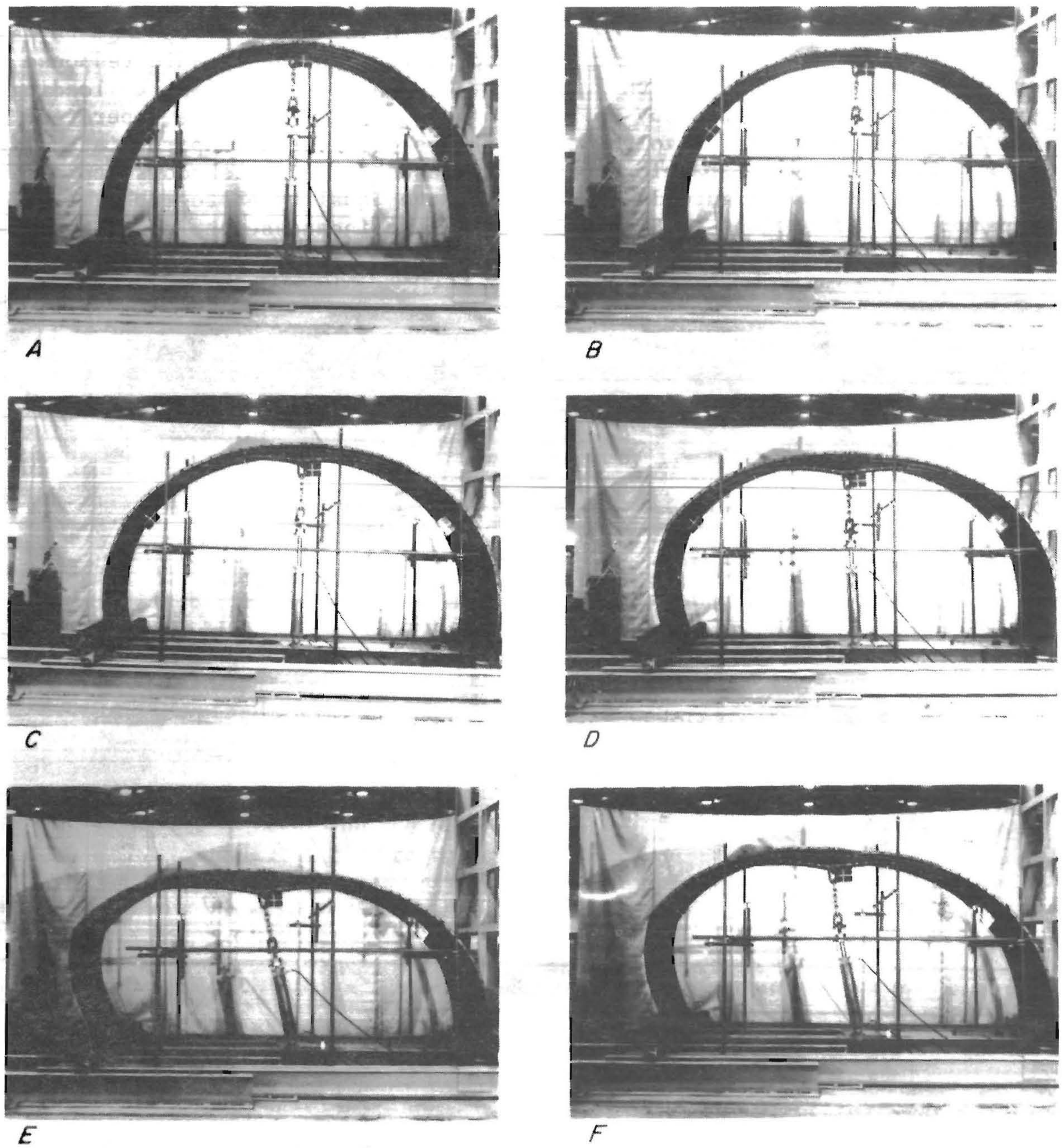

FIGURE 41.—Photographs of static test. ***A***, **No pull force;** ***B***, **18.8-kip pull force;** ***C***, **19.0-kip pull force;** ***D***, **23.3-kip pull force;** ***E***, **23.4-kip pull force;** ***F***, **pull force released.**

released to zero again. Based upon preliminary calculations, it was a foregone conclusion that the arch would experience lateral displacements before it reached its maximum strength, although to what extent was difficult to predict. This conclusion was amply confirmed.

Actually, the critical load at which buckling (the theoretical load at which lateral displacements become independent of the vertical load) would occur was estimated to be within 1.1 to 2 times the ultimate vertical load. It was expected, however, that owing to various imperfections the crown would begin to move laterally at some lesser load. By averaging the horizontal displacements at the intermediate locations A and B (fig. 38), it was found that these lateral displacements began to be significant at about 19,000 lbf after the crown had already deflected vertically about 16 in. Figure 41*D* is the first photograph for which the slope of the ram is discernible.

The sloping of the ram means that the vertical load on the arch was somewhat less than the recorded ram load. By measuring that slope from the photographs, it is possible to determine the true vertical load and also to determine the horizontal force that must be applied at the crown in order to maintain equilibrium.

In a similar fashion the vertical displacement transducer at the crown ceased to yield accurate data once the crown began to move laterally. By correcting for this error and by measuring crown vertical displacements directly from the photographs, it was possible to construct a corrected vertical load-vertical displacement curve shown as a solid line in figure 42.

Two features of this curve should be particularly noted. The first is that after reaching about 19 kips the load initially began to drop off, only to climb again after about 4 more inches of displacement. It is believed that this drop is due to local deformation as the liner plate began to lose its corrugations. This can be seen by comparing figures 41*B* and 41*C* in which the latter exhibits much larger displacements at the

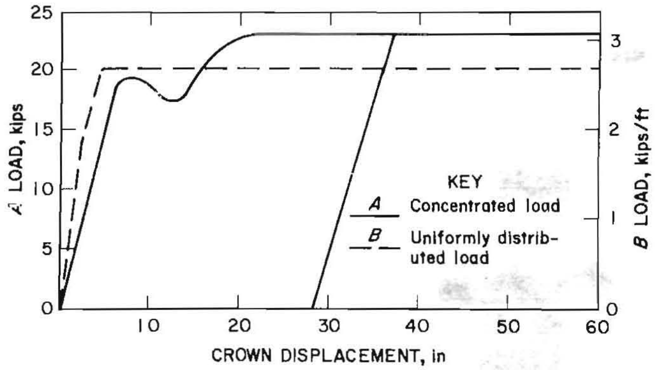

FIGURE 42.—Arch canopy force-crown displacement curves.

center than at the lip. The subsequent increase in load represents a transition from shell (local) to ring (general) behavior. If the shell had been line-loaded instead of loaded only on the center ring, it is doubtful this dip would have been observed.

The second feature of figure 42 is that the load remains essentially constant after the first 20 in of displacement. Although this kind of purely plastic behavior is to be expected from a mechanism in which sufficient plastic hinges of constant moment have formed, it should not remain so when the structure experiences the very large deformations that this one has. Crown displacements of 20 to 37 in correspond to geometric changes in the shape of the arch sufficient to reduce the overall resistance of the structure by about 15 pct, assuming constant plastic moments and no shift in their location as loading progresses. Actually, there should be a tendency for the plastic hinges to move upward owing to the large geometric change; this by itself should somewhat diminish the theoretical rate of reduction of the resistance, but it cannot account for it all. It seems likely, therefore, that the moment at the hinges increases owing to strain hardening and that this explains the flat load-displacement diagram.

Whether the concept of plastic hinges is really valid is raised by figure 43, in which are depicted the detailed deformations at the crown and near the region of maximum negative moment. Clearly,

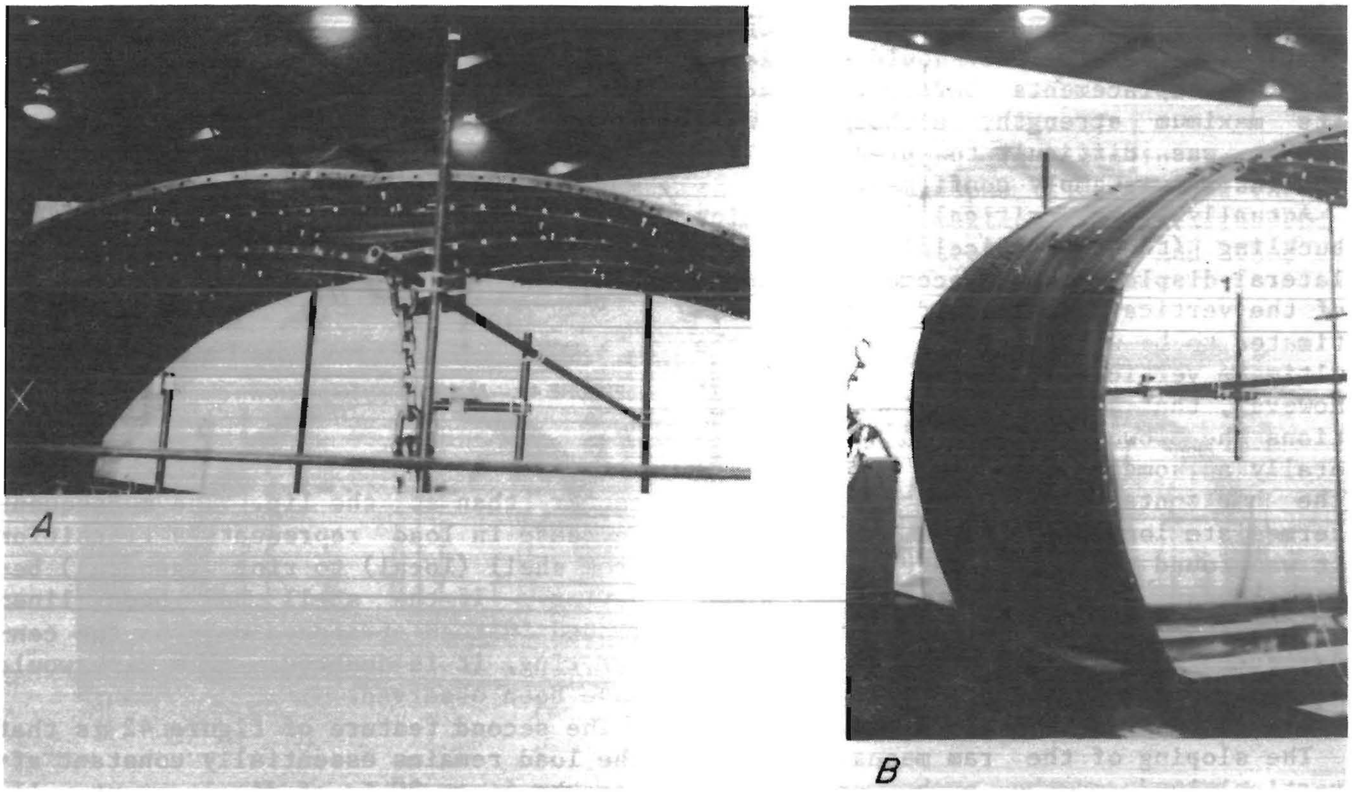

FIGURE 43.—Plastic hinge geometry. *A*, Site of maximum positive moment at the crown; *B*, site of maximum negative moment.

rotation is occurring at these locations, but its relation to the bending properties of corrugated plate may be nebulous.

For comparison, an idealized load deflection diagram has been superimposed as a dotted line on the experimental curve of figure 42. This curve assumes a line loading for which the entire arch deforms as though it were a two-dimensional structure. The bilinear "elastic" portion of the idealized curve is due to the assumed formation of a plastic hinge at the crown. A mechanism is formed when two more hinges form at about 55° on either side of the crown. Again, the stiffness (slope) of the idealized curve is greater than that of the experimental curve because the loading of the experimental arch was applied only over the center ring; thus, that ring was allowed to deflect relative to the remainder of the shell. The idealized curve levels off at a smaller maximum resistance because the nominal value of the yield point (33,000 psi) was used.

DYNAMIC TESTS

Nondestructive Impact Tests

As noted earlier, one purpose of the nondestructive impact tests was to determine the effective mass of the arch for use in energy-transfer calculations. Based upon the crown wire-pull displacement transducer outputs, only the data and results of these two tests are summarized in table 4.

It was only after an attempt was made to analyze the raw data that it was realized the arch had experienced permanent displacement under impacts that should have produced only an elastic response. This became evident from both tests when the arch, during a brief period for which the tup had rebounded upward free of the arch, oscillated for about 1-1/2 cycles about an equilibrium point that was not zero. This effect is demonstrated in figure 44, which shows the dynamic response of the crown due to the smaller

TABLE 4. - Nondestructive impact test data and results

	Test 1	Test 2
DATA		
Tup weight..........lbf..	882	3,150
Drop height..........in..	42	10
Displacement, in:		
Maximum..........	3.1	4.1
Maximum elastic..........	2.7	3.8
Permanent..........	0.4	0.3
Static[1]..........	0.2	0.7
Period, s:		
With tup[2]..........	0.189	0.281
Without tup (based on 1.5 cycles)..........	0.145	0.145
RESULTS		
Stiffness, effective, lbf/in:		
Based on static displacement..........	4,400	4,500
Based on both test periods with tup..........	5,374	5,374
Mass, effective, slugs:		
Based on stiffness and period with tup..........	20.8	10.2
Based on stiffness and period without tup..........	28.2	28.2
Based on both test periods with tup..........	31.2	31.2
Transmission ratio (r_t)[3]..........	0.48	0.77
Absorption ratio (E_a/E_g)[4]..........	0.49	0.74

[1]Based on tup removal after drop test (rebound).
[2]Based on 8 or more cycles.
[3]Based on effective mass of 30 slugs.
[4]With E_a based on effective stiffness of 5,374 lbf/in and the maximum elastic displacement.

882-lbf tup. This point is identified as permanent (crown) displacement in table 4 and can only be estimated to the nearest tenth of an inch, in view of its short duration. It will be shown later that this permanent displacement is probably not an instrument error.

The precision of 0.1 in is not out of line with that stated for the instrument (±0.05 in), but it has a profound effect on some of the calculations that follow from it. For example, the effective stiffness of the arch can be found by dividing the weight of the tup by the static displacement, i.e., the displacement about which the tup and the arch oscillate after all other transients have damped out. This figure is found by subtracting the permanent displacement from the equilibrium point for arch-with-tup oscillations and can only be expressed to the nearest 0.1 in. In both cases this means only one significant figure in the displacement and therefore only one significant figure in the stiffness, although two are shown. That the two stiffnesses are as close as they are is rather remarkable.

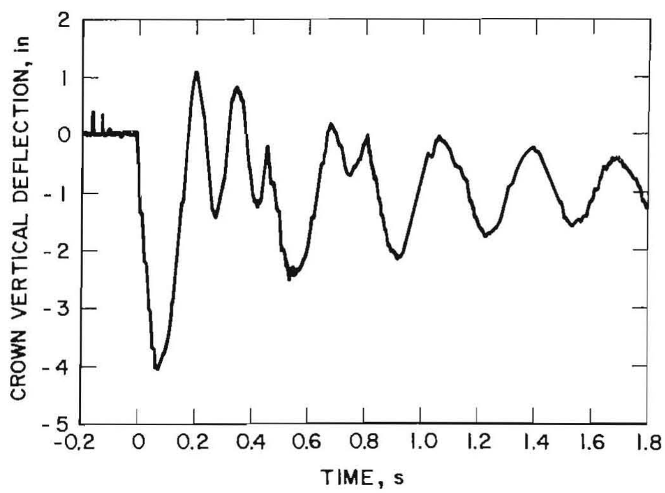

FIGURE 44.—Dynamic response of arch canopy at the crown.

The effective mass of the arch can be found from the following relation for a single-degree-of-freedom system:

$$\omega = \sqrt{\frac{K}{M}}, \qquad (14)$$

where ω = circular frequency ($2\pi/\tau$), rad/s,

τ = period of the system, s,

K = stiffness, lbf/in,

and M = mass (in this case $M_a + M_t$, the effective mass of the arch plus the mass of the tup), slugs.

Solving for M_a leads to

$$M_a = \frac{K\tau^2}{4\pi^2} - M_t, \qquad (15)$$

from which the tabular figures for effective mass, based on stiffness and period with tup, are calculated. Their wide disagreement reflects the uncertainty of the calculations. (Note: The figures for period with tup given in table 4 are justified at three significant figures.)

As an alternative to a reliance upon the uncertain stiffness of the structure, equation 15 can be written twice using the period and tup mass from each test and can be solved simultaneously for M_a and K. The solutions to these two equations are an effective mass of 31.2 slugs and a stiffness of 5,374 lbf/ft.

While these figures are probably the most reliable results shown for mass and stiffness in table 4, it should be realized that these values may not be all that accurate. The reason is that because the tup dimensions in the direction of the arch length were not equal (17 in for the 882-lbf tup and 25 in for the 3,150- and 3,350-lbf tups), the stiffnesses for each case will be somewhat different. Thus, it is reasonable to assume an effective mass for the arch of 30 slugs.

In addition, it will be noted that the stiffness given in table 4 differs significantly from the stiffness of 4,000 lbf/in shown in the initial slope of the static test results with a yet different loaded length (fig. 42) and from the idealized line-loaded stiffness of 8,100 lbf/in, also shown in figure 42. It was from the attempt to reconcile these differences in stiffness that it became apparent how sensitive the stiffness was to length of loading. It became obvious that it was not only more conservative to design for line loading, it was practically impossible to design any other way. Unfortunately, this fact became apparent only after the destructive impact tests were already completed.

Finally, in table 4 are given the transmission ratio, based upon an effective arch mass of 30 slugs, and the energy absorption ratio, based upon the energy under the static load-deflection curve (fig. 42) for the maximum elastic displacement. As previously noted, the stiffness, and hence the area under the static curve, should be greater for longer lengths of load. The absorption ratios given in table 4 (which are the least possible values available) may thus be less than actual by a significant amount, perhaps as much as 15 to 20 pct.

Destructive Impact Tests

Perhaps the most significant general observation to be made about the results of the four consecutive high-energy drop tests is that during the maximum crown deflection (the first quarter cycle when all of the damage is done) the arch displayed no significant tendency toward lateral displacement as it had in the static test. This visible behavior, shown in figure 45, was confirmed by the horizontal wire pull transducers at the two-third points, which in the first two drops (for which two such transducers were used) indicated a shift to the left of only 0.1 in. in the first drop and a shift to the right of only 0.25 in. in the second drop.

In table 5 are given all of the data from the four drops producing progressive failure of the arch. The displacements listed in rows 3, 4, and 5 were obtained from the photographs, both motion and still. The crown vertical displacement transducer broke during test 2 and failed

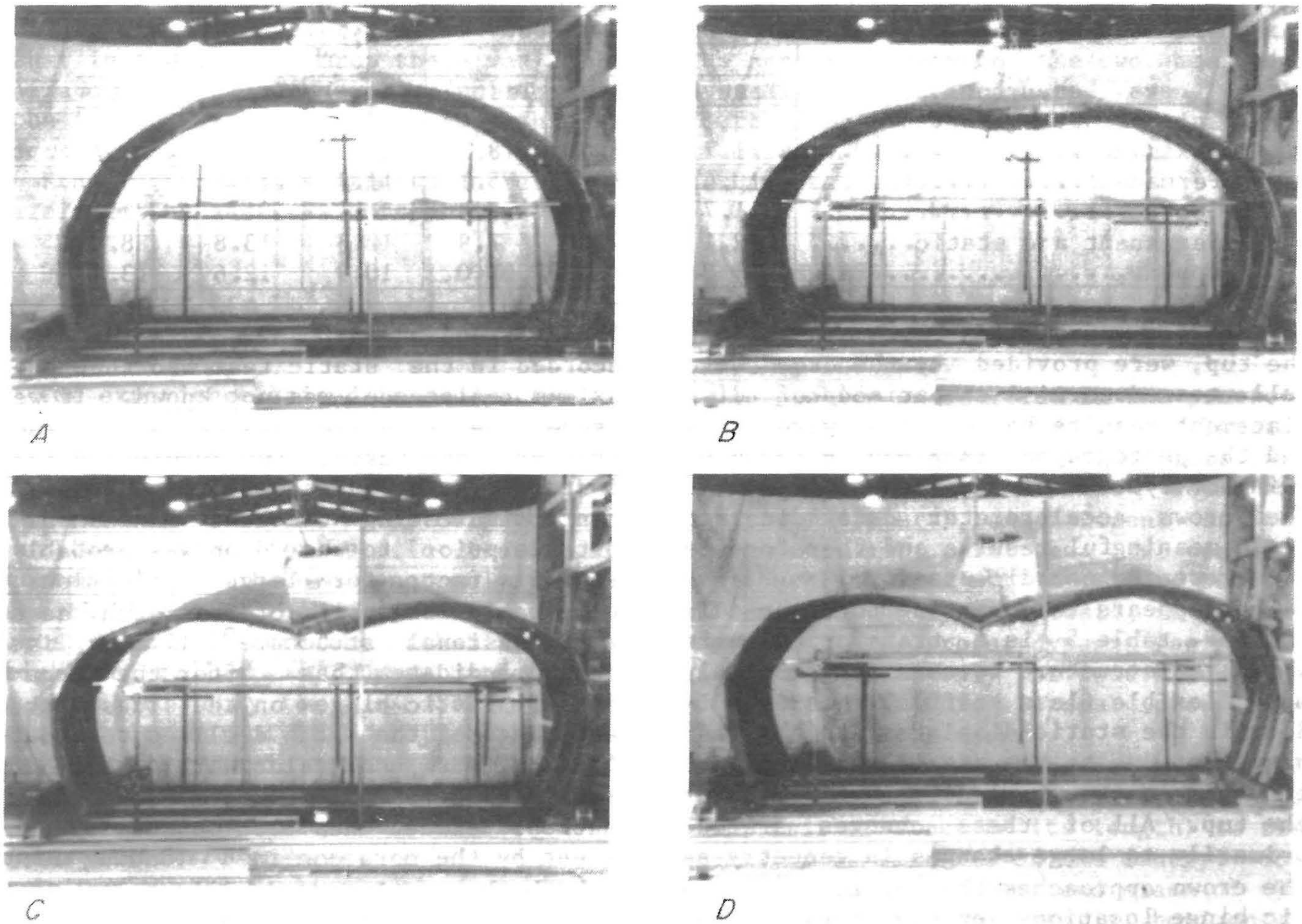

FIGURE 45.—Photographs of consecutive destructive impact tests. *A*, 11-ft drop test; *B*, 12-ft drop test; *C*, 11-ft drop test; *D*, 6-ft drop test.

TABLE 5. - Destructive impact test data and results

	Test 1	Test 2	Test 3	Test 4
DATA				
Tup weight....................................lbf..	3,350	3,350	3,350	3,350
Drop height....................................in..	132	144	132	72
Displacement, in:				
Maximum..	22.1	24	24.7	19.7
Maximum elastic................................	10.9	11	12.6	13.4
Permanent......................................	11.2	13	12.1	6.3
Static[1]......................................	1.1	1.4	1.7	2.6
Period, s:				
With tup.......................................	0.37	ND	0.46	0.57
Without tup....................................	0.19	ND	0.22	0.28
RESULTS				
Transmission ratio (r_t)...........................	[2]0.78	ND	ND	ND
Energy absorption ratio[3]...........................	0.75	0.76	0.84	ND

ND Not determined.
[1]Tup removed after drop test (rebound). [2]Based on effective mass of 30 slugs.
[3]Based on drop height and maximum displacement, stiffness, and maximum resistance of the structure.

TABLE 6. - Comparison of wire-pull and photographic measurements

	Wire-pull			Photographic		
	Test 1	Test 3	Test 4	Test 1	Test 3	Test 4
Displacement, in:						
Maximum..................	21.7	23.5	18.1	22.1	24.7	19.7
Permanent................	10.4	11.1	5.1	11.2	12.1	6.3
Static...................	1.7	2.9	2.8	1.1	1.7	2.6
Permanent and static.......	12.1	14.0	7.9	12.3	13.8	8.9
Rebound..................in..	11.3	12.4	13.0	10.9	12.6	13.4

to record. The periods, with and without the tup, were provided by the crown wire pull transducer. A comparison of displacement results between the wire pulls and the photographs is given in table 6. Owing to system noise the integration of the crown accelerometer data did not yield meaningful results and therefore is not presented nor discussed.

What appears in comparing the four columns of table 5 is a transformation in which the structure becomes progressively more flexible (less stiff). This is evident in the static displacement (the displacement due to the static weight of the tup) and in the periods with and without the tup. All of these changes are due primarily to large changes in geometry as the crown approaches the horizon of plastic hinge locations on the sides of the arch, and the structure becomes more of a rectangular frame than an arch.

Because the period without the tup was based on such a very short duration and thus was of low precision, no effort was made to calculate the effective mass or the transmission ratio for other than the first drop, for which the data from the nondestructive drops were used. It will be noted, however, that the ratio of the period with the tup to the period without the tup remained roughly 2 throughout the tests, indicating that the effective mass of the arch and the transmission ratio remains roughly the same in spite of the changes in geometry.

The energy absorption ratio, which is based on the drop height and the maximum displacement, stiffness, and maximum resistance of the structure (that is, the load at which load deflection diagram becomes horizontal), has been estimated for the first three drops. In the case of test 4 the displacement of the crown had already exceeded the maximum displacement recorded in the static test so that the maximum resistance was not known. If we assume the same resistance as in the three previous tests, the absorption ratio for test 4 turns out to be greater than 1, indicating that the maximum resistance prior to the drop was probably less. In fact, for large deflections, the static analysis of the arch as a two-dimensional structure[9] (i.e., line loaded) indicates that, after the formation of plastic hinges on the sides, the resistance of the arch declines to about 65 pct of its peak value within 60 in of crown deflection. It is quite possible that this effect has been masked to some extent by the more confined loading used in both the static and dynamic tests.

In the comparison between the wire pull and photographic data, table 6, it is seen that the results are quite consistent, within the precision of the two measuring systems, at least for the first and third drops.

Because both the permanent and static displacement measurements from the wire pull data rely on an estimation of the point about which the arch oscillates briefly while the tup is thrown back up into the air, their values are more uncertain. However, once the tup returns to the arch and remains with it, they both oscillate about a point equivalent to the sum of the permanent and static displacements. The agreement between the two measurement systems for the values of

[9]Assuming that the plastic hinge locations do not change and that the plastic moment does not change. It should be noted that the peak value of maximum displacement is more uncertain by both methods than a steady state value.

this sum is quite good for the first and third drops. However, it is suspected that in the fourth drop there was a slip of about 1 in. in the wire pull during the initial displacement because this sum errs by that much.

Finally, loss in stiffness is reflected again in the last row of table 6 wherein the rebound of the structure from maximum to permanent displacement is recorded. The agreement between the two measuring systems is quite good here, even for the fourth drop, because the slip error, if it exists, occurs in both terms that determine rebound.

RECOMMENDED TESTING PROCEDURES

STATIC TEST

The experience gained in conducting the above tests and in evolving appropriate design loading criteria has led to a better recognition of what should be required in future testing. The procedures outlined below for static testing are recommended to be used not only for research, but also for evaluating any products proposed for the rehabilition of high-roof-fall areas. Dynamic testing is recommended only for the continuing research purposes of this project. The objective of dynamic testing is to verify the design procedure here recommended and to determine whether it is sufficient to cover all circumstances.

As was noted earlier, the concept of line loading for both evaluation and design allows for a two-dimensional consideration of arch behavior, and for testing of canopies much shorter than would normally be encountered in practice. It also represents a more severe case than would be encountered in the field. In all of the following, line loading is assumed if not explicitly stated otherwise.

Liner Plate Arch

To achieve a line loading in the static test, a beam running the length of the crown and mounted on top of the arch canopy can be directly attached at its center to a point-loading system such as a hydraulic ram. The stiffness of the beam should be such that its center will deflect elastically not more than 1/200th of its length relative to its ends under the maximum loading the arch can reasonably be expected to take. The beam should be torsionally stiff enough to prevent its own lateral buckling and should also be attached to the canopy at least at its ends.

One problem encountered in the previous static test was the tendency for lateral buckling of the arch at loads less than the ultimate strength the arch would exhibit if lateral displacement at the crown were not permitted. Because lateral buckling did not appear to be significant in the dynamic tests, it is presumed that the proper dynamic resistance of the arch can only be determined from static tests in which lateral crown displacements do not occur.

To prevent such displacements, it will be neccessary to attach an adjustable strut (with respect to its length) to the crown lip at each end of the canopy. The strut should lie in the end plane of the canopy and should be nearly horizontal (within ±10°) over the range of anticipated crown vertical deflections from zero to $h-h_p$ or to the maximum stroke of the loading jack. The strut should be capable of resisting a load in either tension or compression of about 3 pct of the estimated total capacity (force not force per unit length) of the arch with a reasonable factor of safety. Because the strut will rotate as the arch deflects, it should be pin-connected at both ends.

As noted previously, the length of the test article can be as short as desired as long as it does not buckle out of the plane of its curvature. For a liner plate arch this might be only one ring, e.g., 18 in; however, three rings would be preferred in order to take advantage of some mutual reinforcement of flanges and still preserve symmetry of behavior with respect to its midlength plane.

The arch base supports should be similar to those employed in the field. In most instances, this will mean freedom of rotation but restraint against vertical and horizontal displacements. A precise duplication of field conditions is not required, only some reasonable approximation thereof. For example, test support displacements of an inch or less where none are allowed in the field, or vice versa, would not widely affect the results.

The loading system employed should be a hydraulic cylinder similar to that previously used. Deadweights should not be used as they may cause collapse as soon as the maximum load is reached. The cylinder should be equipped with a load cell to measure the applied force and should also be pinned at its base so that no lateral loads will be experienced when, due to vertical displacement, the struts allow the crown to move horizontally before they are adjusted. Although a tension ram below the crown, as used with the ITS, is here contemplated, a compression ram above the arch and mounted on an enclosing frame would also be acceptable, although this scheme is more susceptible to lateral buckling and would require more lateral restraint.

The primary displacements of interest are the crown vertical at midlength, and loading should proceed in increments of displacement, not force. (See the section entitled "Equal Increments of Vertical Deflection.") Some means of observing horizontal motion of the crown lip is required so that after each step of loading, the strut lengths can be adjusted to keep the crown in the same vertical plane throughout the test. Loading should continue until the stroke of the cylinder or the maximum allowable displacement is reached. In the first instance, an unloading curve should also be obtained, and perhaps the struts should be reset before loading begins again. Still photographs should be taken before loading, at maximum displacement, and in the unloaded condition for each cycle of loading. Some reference system should be included within the photos for checking displacements.

The final result of the static test should be a complete load-deflection curve for the arch canopy crown, a set of still photographs, and the actual yield strength of the material.

Steel Set Arch

Ideally, the testing of steel sets can be accomplished on a single set, with the total energy absorbed divided by the applicable spacing of the steel sets to obtain an energy per unit length. However, because of the tendency of the set to buckle out of its plane, one set will probably not be sufficient. Even two with the associated hardware in between may not be adequate, unless that hardware included crossed tie rods. This is because both sets buckling in parallel could move together with only the friction between the lagging timbers resisting their motion.

As an alternative, it may be possible to weld together intermittently the inside and outside flanges of two or three sets, thereby forming a box section and increasing by one to two orders of magnitude the out-of-plane bending resistance. In any event, all of the procedures outlined for liner plate are applicable, although only a single strut may be required if the welding option is followed. The load-displacement curve expressed in load per unit length can be found from dividing the load per steel set by the proposed spacing between sets.

SYMMETRICAL DYNAMIC TESTING: LINER PLATE AND STEEL SET ARCH

To ensure a line or near-line loading, the tup should be at least as long as the arch, or the tup should have attached to it a loading beam as long as the arch, with a stiffness comparable to that required for static testing. Attaching a beam to the arch instead of the tup is not recommended because it will increase the effective mass of the arch and alter the transmission ratio. No struts or crown lip displacement monitors are required, but base supports should be identical to those of the static test. Two nondestructive tests should be conducted with different tup weights to determine the effective mass of the arch. (The initial stiffness from the static test can be used with reasonable confidence to confirm this result.) Finally, the maximum drop distance possible, consistent with the energy capacity as given in the static test, should be used for the destructive test.

The instrumentation required for crown loading is the same as for static testing with the addition of high-speed movies. The film speed of 48 frames per second was barely sufficient to establish maximum displacement conditions in the previous tests.

FUTURE RESEARCH

TRANSMISSION RATIO

When a falling object strikes a structure, not all of its kinetic energy will be transformed into potential energy of deformation of the structure (13). The amount of kinetic energy lost during the impact is $1-r_t$, where r_t is the transmission ratio (see equation 5). The transmission ratio is an important parameter in the design of an arch canopy because it is used in determining how much energy an arch canopy must be capable of absorbing. Some of the nondestructive and destructive impact tests conducted have shown that the calculated values of r_t are conservative at times. This was determined from comparing the ratio of the kinetic energy of the tup at the instant of impact to the energy absorbed by the structure at maximum crown deflection with the calculated value of r_t given in equation 5. Because of the differences in transmission ratios (actual versus calculated), the decision was made to use the conservative value of r_t (equation 5) until an improved transmission ratio (if possible) can be developed through future tests and analyses.

TESTING OF ARCH CANOPY-BACKFILL SYSTEMS

Backfilling an arch canopy will resist outward displacements of the arch sides and discourage buckling. This resistance to outward displacement increases the stiffness and overall strength of the arch canopies. Future static tests will be conducted to provide a detailed understanding of the failure processes that a backfilled arch canopy undergoes. The static tests will also allow the increases in energy absorption capacities of the arch canopies to be determined. Dynamic tests will also be conducted to determine the dynamic response of arch canopy-backfill systems to impact loading and to establish the total amount of energies that the backfilled arch canopies can absorb. These tests may also be used to determine the parameters of backfill material such as density, compressive strength, and coefficient of friction, which are important in resisting outward movement of an arch canopy during dynamic loading.

PUNCTURE TESTING

All of the tests so far outlined have concentrated on the overall structural behavior of the arch. Another danger to be considered is that of a small rock falling a great distance and either penetrating the liner plate or hitting between steel sets and either penetrating the lagging or causing it to tear away the inside flange of at least one of the sets.

Whether such a rock would disintegrate on impact is impossible to predict. It is reasonable to suppose, however, that if the rock became dislodged from strata that were badly fractured but otherwise quite competent it might survive the impact.[10]

[10] Tests on small high-strength sandstone projectiles fired against a flexible beam at velocities of 66 to 83 ft/s survived with only minor damage (14). A significant percentage of their energies went into local deformation of the beam.

Assuming a maximum fall height of 80 ft, the rock would have a velocity of a little over 70 ft/s. Compared to projectile velocities for ordinance purposes, this is quite low. On the other hand, 80 ft greatly exceeds the height of the ITS.

A similar but not identical problem is that of a larger rock falling with a sharp corner at the point of impact. This condition corresponds to a lower velocity but perhaps higher energy.

ASYMMETRICAL DYNAMIC TESTING

One premise of the design criteria is that the worst condition that can be conjectured exists when a rock falls squarely on the crown and is brought to a complete stop by the arch canopy. Many arch-canopy-type structures are actually less stiff for lateral loadings than for vertical ones, and the effect of an off center rock delivering a glancing blow to the side of an arch must be determined.

For an off-center loading it is suggested that the tup, still applied as a line load, be dropped to impact the arch at a point where its slope is roughly 45°, if the arch geometry contains such a point. The bottom surface of the tup or beam should be roughly tangent to the arch at the point of impact in order to minimize energy losses due to local deformations. For this test horizontal displacements of the crown, of the point of contact, and of its counterpart on the opposite side should be recorded instead of crown vertical displacements. Movies will provide an indication of overall deflected shape and give some guidance as to what the hazards are under these conditions.

CONCLUSIONS AND RECOMMENDATIONS

A design procedure was developed for unbackfilled arch canopies constructed of liner plate and/or steel sets and lagging and subjected to impact loading at the crown. The design procedure is based on the concept that an arch canopy, in deflecting from the unloaded condition to maximum vertical displacement at the crown, absorbs strain energy, both elastic and plastic, and that this energy can be calculated from a static load-displacement diagram for the structure. An integral part in the development of any design procedure is the selection of the magnitude of the dynamic loads that the given structure is to be capable of absorbing. Based on a study of roof-fall rehabilitation accidents, a design energy level of 20,000 ft•lbf/ft was selected for demonstrating the design procedure.

Another important parameter in the design of an arch canopy is the protection height. The protection height limits the extent of maximum vertical displacement of the crown of an arch canopy and was selected to be 6 ft for discussion purposes. A protection height of 6 ft should protect a majority of mine personnel from injury due to crown displacement. The other important design parameters are the mass of the rock and the effective mass, stiffness, yield limit, and maximum resistance of an arch canopy, which can be obtained experimentally or analytically. The design procedure developed here for arch canopies gives mine personnel a method to select and design an arch canopy to meet the dimensional and functional requirements of the mine entry.

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APPENDIX A.--GLOSSARY

Arch. - a. A structure that produces horizontal converging reactions under vertical load. An arch tends to flatten out under load and must be fixed against horizontal movement at its supports (15).
b. Structurally, an arch is a piece or assemblage of pieces so arranged over an opening that the supported load is resolved into pressures on the side supports and practically normal to their faces (16).
c. A typically curved structural member spanning an opening and serving as a support (17).
d. A curvature having the form of an arch (17).

Arch canopy. - A structure constructed of liner plate and/or steel sets and lagging that is used in the rehabilitation of a high-roof-fall area to insulate a mine entry from a recurring roof fall. An arch canopy protects an entry from a roof fall but does not contribute to the stabilization of the mine opening.

Arch rib. - A steel set used in conjunction with a liner plate arch; acts as the main load-bearing member of the entire support system.

Backfill. - Mine waste or other material placed around the arch canopy to partially dissipate the energy of a roof fall and to increase the stiffness of an arch canopy by resisting its side buckling during loading.

Course. - A single circumferential section of a steel set arch, composed of a steel set, lagging, tie rods, and/or spacers.

Dynamic load. - A load whose magnitude, direction, or point of application varies with time (18).

Forepoling. - Sharpened planks or steel sections driven into the soft ground or rubble of headings as a protection against sloughing material.

High-roof fall. - A roof fall that creates a mine entry height that exceeds the operational limit of the mine's bolting machine and makes scaling and resupport operations extremely difficult or impossible.

Impact load. - A force producing an essentially instantaneous velocity and no initial displacement in a structure at the instant of impact.

Joint. - Connection that joins and holds two or more structural members together.

Lagging. - a. Members of a tunnel support that span the spaces between the main supporting ribs (19).
b. Wood or other structural materials spanning the area between ribs (11).

Liner plate. - a. Formed steel unit used to line or reinforce a tunnel or other openings. Steel liner plates are produced in two general designs: (1) four-flange type with abutting end joints, and (2) two-flange type with lapped offset end joints (10, 20).
b. Plates that can be fastened together to support the arch, sides, and in some cases the invert of a tunnel (11).

Liner plate arch. - An arch constructed of liner plates.

Rehabilitation. - The state or process of a mine entry restored to a condition of useful and constructive activity (17).

Resupport. - The state or process of a mine opening being supported after a roof fall has occurred.

Rib. - See steel set.

Ring. - A single circumferential section of a liner plate or steel set arch, comprised of individual segments bolted together.

Set. - See steel set.

Spacers. - A minor component of the support system that prevents lateral bending of the ribs about their minor axis, thereby improving their capacity to carry loads by column action, and assists mine personnel to properly space the ribs and to install them at right angles to the centerline of the tunnel, both vertically and horizontally (11).

Steel set. - A term used to identify a single structural support, composed of a single or an assemblage of straight and/or curved steel flexural members of constant or variable cross section(s).

Steel set arch. - An arch constructed of steel sets, lagging, tie rods, and/or spacers.

Steel support. - See steel set.

Tie rods. - Tension members between sets to maintain spacing. These pull the sets against the struts or spacers (11).

Tup. - An object that is dropped from above a test article to create an impact load.

APPENDIX B.--SYMBOLS AND ABBREVIATIONS USED IN THIS REPORT

d_h - drop height of tup.

E_a - Energy absorbed by arch canopy.

E_g - gross energy available to deform arch canopy.

E_p - potential energy.

G - acceleration due to gravity.

γ - density of rock.

H - void height.

h - height of arch canopy.

h_p - protection height.

K - stiffness.

M - mass.

M_a - effective mass of arch canopy.

M_r - mass of rock.

M_t - mass of tup.

ω - circular frequency.

π - pi.

Pi - used in reference to a length of liner plate, where pi is equivalent to 3.14 in.

R_m - maximum resistance.

r_t - transmission ratio.

τ - period of vibration.

V_o - instantaneous velocity.

vol - volume.

W - weight of rock.

W_a - effective weight of arch acnopy.

W_t - weight of tup.

y_a - static deflection of arch canopy due to effective weight of arch canopy.

y_e - displacement of arch canopy at yield point.

y_s - static deflection of arch canopy.

y_{max} - maximum displacement of arch canopy.

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