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EXTRUDED TUNNEL LINING SYSTEM PHASE I - CONCEPTUAL DESIGN AND FEASIBILITY TESTING

Foster-Miller Associates, Inc.
350 Second Avenue
Waltham MA 02154

SEPTEMBER 1979

FINAL REPORT

Prepared for

U.S. DEPARTMENT OF TRANSPORTATION
URBAN MASS TRANSPORTATION ADMINISTRATION
Office of Technology Development and Deployment
Washington DC 20590

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16. Abstract

The Extruded Tunnel Lining System (ETLS) has been conceived as a means of continuously placing the final concrete tunnel lining directly behind a tunnel boring machine. The system will shorten the time required to excavate and line a tunnel section, eliminate the need for primary supports, and thus significantly reduce the cost of tunnel construction. With the goal of demonstrating the feasibility of this concept, the Urban Mass Transportation Administration has contracted with Foster-Miller Associates, Inc. to design, fabricate, and test such a tunnel lining system designed specifically for rock tunnel application. The scope of the program and its three phases are: Phase I: R&D - Resolution of basic technological problems; Phase II: System design, fabrication, and test; and Phase III: System modification, test, and demonstration. This report describes the work completed during the first phase of the program. The report presents a state-of-the-art review of related construction practices, concrete technology, and tunnel liner design methods. The ETLS conceptual design is described. In addition, the results of a development program carried out to formulate a rapid setting concrete for use with the ETLS and a test program to demonstrate the ability to slipform and place that concrete under pressure in a closed form are presented. The ability to slipform a well consolidated slab of the rapid setting Very High Early Cement (VHEC) concrete was demonstrated. The slipformed concrete gained strength rapidly and was self-supporting approximately one hour after the initial addition of water to the concrete. This document contains a Listing of References and the appendix is a Report of New Technology.

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16. Abstract

This report describes the work completed during the first phase of a three phase program to design, develop, fabricate, test and demonstrate a system for placing a continuously extruded tunnel lining.

The report presents a state-of-the-art review of related construction practices, concrete technology and tunnel liner design methods. The extruded tunnel lining system (ETLS) conceptual design is described. In addition, the results of a development program carried out to formulate a rapid setting concrete for use with the ETLS and a test program to demonstrate the ability to slipform and place that concrete under pressure in a closed form are reported.

The ability to slipform a well consolidated slab of the rapid setting Very High Early Cement (VHEC) concrete was demonstrated. The slip-formed concrete gained strength rapidly and was self-supporting approximately 1 hr. after the initial addition of water to the concrete.

17. Key Words
tunnel lining, concrete slipforming, extruded concrete, rapid setting concrete, machine bored tunnel

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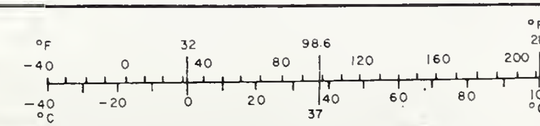
Approximate Conversions to Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

*1 in = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.

Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



METRIC CONVERSION FACTORS PAGE

PREFACE

This report was prepared by Foster-Miller Associates, Inc. (FMA), Waltham, Massachusetts under contract DOT-TSC-1516 to the U.S. Department of Transportation's Urban Mass Transportation Administration Office of Technology Development and Deployment. The contract was managed by the Transportation Systems Center (TSC), Cambridge, Massachusetts. Mr. G. Saulnier of TSC was the contract technical monitor.

The objective of the contract is to design, develop, fabricate, test and demonstrate a system for placing a continuously extruded tunnel liner. The program will be carried out in three phases. This report describes the results of the state-of-the-art review, concrete development and concept feasibility testing conducted during the first of these phases.

CONTENTS

Section		Page
1.	INTRODUCTION	1-1
	1.1 The Extruded Tunnel Lining System	1-2
	1.2 Conventional Tunnel Lining Systems	1-5
	1.2.1 CIP Concrete Tunnel Lining	1-5
	1.2.2 Precast Concrete Liners	1-7
	1.2.3 Shotcrete	1-8
	1.3 State-of-the-Art Review	1-10
	1.4 Test Program	1-12
2.	RELATED CONSTRUCTION PRACTICES	2-1
	2.1 Vertical Slipforming	2-1
	2.1.1 General Description	2-1
	2.1.2 Process Control	2-4
	2.1.3 Application to ETLS	2-6
	2.2 Horizontal Slipforming	2-7
	2.2.1 General Description	2-7
	2.2.2 Process Control	2-7
	2.2.3 Application to ETLS	2-9
	2.3 Cast-in-Place (CIP) Pipe	2-9
	2.3.1 General Description	2-9
	2.3.2 Applications to the Extruded Liner	2-11
	2.4 Soviet "Pressed Concrete" Continuous Tunnel Liner	2-13
	2.4.1 General Description	2-13
	2.4.2 System Specifications	2-16
	2.4.3 Applications to ETLS	2-17

CONTENTS (Cont.)

Section		Page
2.5	Tunneling Environment	2-18
2.5.1	Tunnel Boring Machines.................	2-18
2.5.2	Trailing Gear.........................	2-19
2.5.3	Ambient Conditions....................	2-23
3.	CONSIDERATIONS FOR LINER CONCRETE	3-1
3.1	Mix Design Requirements	3-2
3.2	Cement Candidates	3-3
3.2.1	Influence of Cement on Concrete	3-3
3.2.2	Reg-Set Cement	3-4
3.2.3	Portland Type III.....................	3-5
3.2.4	Aluminous Cements	3-7
3.2.5	Aluminous - Portland Type I Combinations	3-8
3.2.6	Specialized Highway Patching Materials	3-9
3.2.7	Duracal...............................	3-12
3.2.8	Very High Early Cement (VHEC).........	3-12
3.2.9	German and Japanese Reg-Sets	3-12
3.3	Admixtures, Aggregates and Water	3-13
3.3.1	Admixtures	3-13
3.3.2	Aggregates	3-16
3.3.3	Water	3-18
3.4	Mix Design for Pumpability	3-19
3.4.1	Basic Principles	3-19
3.4.2	Mix Design Method	3-20
3.4.3	Pumpable Mix Characteristics..........	3-22
3.5	Fiber Reinforcement	3-22
3.5.1	Background	3-22
3.5.2	Mixing and Placement	3-24
3.5.3	Strength of Fiber Reinforced Concrete	3-25
3.5.4	Post-Cracking Strength of Fiber Reinforced Concrete	3-26

CONTENTS (Cont.)

Section		Page
	3.6 Summary	3-31
4.	TUNNEL SUPPORT AND LINER DESIGN	4-1
	4.1 Introduction	4-1
	4.1.1 Liner Design - State-of-the-Art.......	4-1
	4.1.2 Considerations for the Extruded Tunnel Liner	4-3
	4.2 Design Rock Loads	4-4
	4.2.1 Maximum Uniform Rock Load	4-4
	4.2.2 Concentrated Rock Loads	4-6
	4.2.3 Initial Rock Loads	4-11
	4.2.4 Loads Due to Elastic Deformation	4-12
	4.3 Characterization of Liner Behavior	4-12
	4.3.1 Moment-Thrust Envelope	4-12
	4.3.2 Evaluation of Rock-Liner Interaction	4-14
	4.3.3 Recommended Design Procedure..........	4-15
	4.4 ETLS Liner Thickness and Strength Evaluation	4-19
	4.5 Methods to Reduce Concentrated Rock Loads....	4-27
	4.6 Conclusions and Recommendations	4-29
5.	ETLS SPECIFICATIONS	5-1
	5.1 Tunneling Conditions	5-1
	5.1.1 Rock Type and Quality	5-1
	5.1.2 Depth of Cover	5-4
	5.1.3 Water Inflow	5-4
	5.1.4 Depth Below Water Table	5-4
	5.1.5 Nominal Tunnel Diameter	5-4
	5.1.6 Minimum Radius of Curvature...........	5-4
	5.1.7 Ambient Temperatures	5-5

CONTENTS (Cont.)

Section Page

5.2 Tunnel Boring Machine 5-5

5.2.1 TBM Geometry 5-5
5.2.2 Advance Rate 5-5
5.2.3 Bore Diameter Variation................ 5-5
5.2.4 Bore Surface Roughness 5-6
5.2.5 Gripper Slip 5-6
5.2.6 TBM Alignment Deviation 5-6
5.2.7 Shove Length 5-7
5.2.8 Routine Shutdown Periods 5-7

5.3 Extruded Tunnel Lining System 5-7

5.3.1 Liner Thickness 5-7
5.3.2 Maximum Lining Rate 5-8
5.3.3 Minimum Lining Rate 5-8
5.3.4 Tunnel Alignment Correction 5-8
5.3.5 ETLS/TBM Relative Motion 5-9
5.3.6 Slipform Design Pressure 5-9

5.4 Liner Concrete 5-9

5.4.1 Workability 5-9
5.4.2 Time to Self Support 5-9
5.4.3 Final Concrete Strength 5-9

6. TECHNOLOGY REQUIREMENTS AND TEST PLAN 6-1

6.1 Identification of Requirements 6-1
6.2 Test Plan 6-2

7. CONCRETE FORMULATION TESTS 7-1

7.1 Time of Setting Determination 7-2

7.1.1 Introduction 7-2
7.1.2 Method of Testing 7-3
7.1.3 Results of Testing 7-4

Section		Page
7.2	Materials	7-14
7.2.1	Aggregates	7-14
7.2.2	Cements and Fly Ash	7-16
7.3	Initial Strength-Time Development and Workability Measurements	7-17
7.3.1	Introduction	7-17
7.3.2	Testing Procedure	7-18
7.3.3	Results of Testing	7-19
7.4	Final Mix Design - VHEC Concrete	7-24
7.5	Strength-Time Testing	7-27
7.5.1	Casting, Curing, and Testing	7-27
7.5.2	Results of Testing	7-27
7.6	Durability Tests	7-30
7.6.1	Resistance of VHEC Concrete to Prolonged Water Exposure	7-30
7.6.2	Sulfate Resistance of VHEC Concrete	7-31
8.	CONCRETE DISTRIBUTION AND SLIPFORMING TESTS	8-1
8.1	Introduction	8-1
8.2	Concrete Distribution Tests	8-4
8.2.1	Distribution Test Plan	8-4
8.2.2	Distribution Test Rig	8-4
8.2.3	Distribution Test Results	8-8
8.3	Concrete Slipforming Tests	8-10
8.3.1	Slipforming Test Plan	8-10
8.3.2	Slipforming Test Rig	8-13
8.3.3	Slipforming Test Results	8-16
8.3.4	Forces on Slipform Advancing Mechanism	8-27

CONTENTS (Cont.)

Section		Page
9.	CONCEPTUAL SYSTEM DESIGN	9-1
	9.1 General System Description	9-1
	9.1.1 System Configuration	9-1
	9.1.2 System Operation	9-3
	9.2 Component Descriptions	9-5
	9.2.1 Independent Slipform	9-5
	9.2.2 Bulkhead	9-8
	9.2.3 Concrete Handling System	9-9
	9.3 ETLS Control System	9-10
10.	REFERENCES	10-1
APPENDIX - REPORT OF NEW TECHNOLOGY		A-1

LIST OF FIGURES

Figure		Page
1	ETLS - OVERALL CONFIGURATION	1-3
2	CROSS SECTION OF BULKHEAD AND SLIPFORM	1-4
3	CONVENTIONAL CIP TUNNEL LINING	1-6
4	PRECAST CONCRETE SEGMENTS	1-7
5	SHOTCRETE LINING BEHIND A TBM	1-9
6	DETAIL OF VERTICAL SLIPFORMING EQUIPMENT	2-2
7	VERTICAL SLIPFORMING EQUIPMENT AND OPERATION (3)	2-3
8	VERTICAL SLIPFORMING OF A SHAFT	2-5
9	HORIZONTAL SLIPFORMING OF A MEDIAN BARRIER (4)	2-8
10	CONTINUOUS PLACEMENT OF CAST-IN-PLACE PIPE	2-10
11	PROPOSED FORM SYSTEM FOR MX PROJECT	2-12
12	SOVIET LINING SCHEME	2-14
13	PRESSED CONCRETE APPROACH	2-15
14	OVERALL LAYOUT OF THE SOVIET LINING SYSTEM	2-17
15	ROBBINS MODEL 201-158 TUNNEL MACHINE	2-20
16	ROBBINS MODEL 233-172 TUNNEL MACHINE	2-21
17	TBM AND TRAILING GEAR	2-22
18	SETTING TIMES OF MIXTURES OF PORTLAND CEMENT AND HIGH-ALUMINA CEMENT (17)	3-10
19	STRENGTH-TIME RELATIONSHIP OF PORTLAND/ALUMINOUS MIXTURES (17)	3-11

LIST OF FIGURES (Cont.)

Figure		Page
20	EFFECT OF CEMENT CONTENT ON SLUMP LOSS (22) IN SUPERLASTICIZED CONCRETE	3-17
21	COMPRESSIVE STRENGTH FOR CONTROL SPECIMENS (32)	3-27
22	MODULUS OF RUPTURE FOR CONTROL SPECIMENS (3 BY 3 BY 15 IN.) (32)	3-28
23	MODULUS OF RUPTURE FOR LARGE BEAMS (4 BY 6 BY 64 IN.) (32)	3-29
24	LOAD-DEFLECTION CURVE IN FLEXURE FOR PLAIN AND FIBROUS CONCRETE	3-30
25	EFFECT OF VOLUME OF FIBERS ON FLEXURAL TOUGHNESS AND STRENGTH (33)	3-32
26	UNIFORM ROCK LOAD	4-5
27	STAND-UP TIME AS A FUNCTION OF ROCK CLASS AND UNSUPPORTED WIDTH OF TUNNEL ROOF (40)	4-8
28	RELATIONSHIP OF ROCK-LOAD FACTORS AND RQD (40)	4-9
29	TYPICAL CONCENTRATED ROCK LOADS	4-10
30	TYPICAL MOMENT-THRUST DIAGRAM	4-13
31	LINER CONFIGURATION USED IN ANALYSIS	4-16
32	COMPARISON OF LINEAR AND NONLINEAR LINER BEHAVIOR	4-17
33	LINER COEFFICIENTS VERSUS FLEXIBILITY RATIO FOR A UNIFORM LOADING CONDITION (38)	4-20
34	LINER COEFFICIENTS VERSUS FLEXIBILITY RATIO FOR A NONUNIFORM LOADING CONDITION (38)	4-21
35	MOMENT-THRUST DIAGRAMS FOR VARIOUS SECTION THICKNESSES	4-23

LIST OF FIGURES (Cont.)

Figure		Page
36	EVALUATION OF LINER CAPACITY	4-24
37	DESIGN LOADS 6 IN. LINER IN POOR ROCK	4-26
38	DESIGN LOADS 8 IN. LINER IN POOR ROCK	4-28
39	OVERALL TEST PLAN	6-4
40	VARIATION IN SET TIME OF ALUMINOUS CEMENT MORTARS FOR DIFFERENT LITHIUM CARBONATE CONTENTS	7-7
41	VARIATION IN SET TIME OF VHEC MORTARS FOR DIFFERENT WATER-CEMENT RATIOS AND WATER TEMPERATURES	7-13
42	RECOMMENDED COMBINED NORMAL WEIGHT AGGREGATE GRADATION FOR PUMPED CONCRETE (ACI-304-74)	7-16
43	EARLY STRENGTH-GAIN WITH TIME FOR TYPICAL FORMULATIONS	7-23
44	VHEC CONCRETE COMPRESSIVE STRENGTH	7-29
45	EXPANSION OF VHEC CONCRETE	7-32
46	WEIGHT CHANGE FOR VHEC CONCRETE	7-33
47	CONCRETE PUMP	8-2
48	PROPORTIONING MIXER	8-3
49	GENERAL EQUIPMENT ARRANGEMENT	8-3
50	DISTRIBUTION TEST PLAN	8-5
51	DISTRIBUTION TEST RIG - CUTAWAY	8-6
52	DISTRIBUTION TEST RIG (back view)	8-7
53	DISTRIBUTION TEST RIG (side view)	8-7

LIST OF FIGURES (Cont.)

Figure		Page
54	VHEC DISTRIBUTION TEST SLAB	8-11
55	SLIPFORMING FINISHED LINER	8-12
56	SLIPFORMING TEST RIG - CUTAWAY	8-14
57	SLIPFORMING TEST RIG	8-15
58	CROSS SECTION OF SIMULATED SEGMENT	8-15
59	PENTROMETER TEST OF SLIPFORMED VHEC CONCRETE	8-18
60	INTERFACE BETWEEN TWO CONCRETE PLACEMENTS	8-20
61	BOND BETWEEN TWO CONCRETE PLACEMENTS	8-20
62	CROSS SECTION OF SLIPFORMED SLAB	8-22
63	CROSS SECTION OF SLIPFORMED SLAB	8-22
64	FRACTURE PLANES PLACED IN SLIPFORMED SLAB	8-23
65	SLIPFORMED SLAB SLUFFED FROM TEST BED SURFACE	8-25
66	UNDERSIDE OF SLIPFORMED SLAB	8-25
67	SELF-SUPPORTING SLIPFORMED SLAB	8-26
68	ETLS COMPONENTS	9-2
69	SLIPFORMING SEQUENCE	9-4
70	SLIPFORM	9-6
71	END VIEW OF LOCATING PAD	9-7
72	BULKHEAD CONFIGURATION	9-8

LIST OF TABLES

Table		Page
1	EVALUATION OF ALTERNATIVE CEMENTS	3-6
2	ROCK QUALITY DESIGNATIONS	4-7
3	ETLS SPECIFICATIONS	5-2
4	RESULTS OF THE TIME OF SETTING TESTS FOR ALUMINOUS CEMENT	7-5
5	RESULTS OF TIME OF SETTING TESTS FOR PORTLAND TYPE III CEMENT MODIFIED WITH CALCIUM CHLORIDE ...	7-8
6	RESULTS OF TIME OF SETTING TESTS FOR PORTLAND TYPE I - ALUMINOUS CEMENT COMBINATIONS	7-10
7	RESULTS OF THE TIME OF SETTING TESTS FOR VERY HIGH EARLY CEMENT	7-12
8	SIEVE ANALYSIS OF SAND AND GRAVEL.................	7-15
9	PROPERTIES OF SAND AND GRAVEL	7-15
10	INITIAL ALUMINOUS CEMENT CONCRETE FORMULATION TRIALS	7-20
11	INITIAL VHEC CONCRETE FORMULATION TRIALS	7-22
12	FINAL VHEC CONCRETE FORMULATION TRIALS	7-25
13	FINAL MIX DESIGN[a]	7-26
14	COMPRESSION AND FLEXURAL STRENGTH OF THE VHEC CONCRETE	7-28
15	COMPRESSIVE STRENGTH OF THE VHEC CONCRETE DURABILITY SPECIMENS	7-31

1. INTRODUCTION

The Extruded Tunnel Lining System (ETLS) has been conceived as a means of continuously placing the final concrete tunnel lining directly behind a tunnel boring machine (TBM). The system will shorten the time required to excavate and line a tunnel section, eliminate the need for primary supports and thus significantly reduce the cost of tunnel construction. With the goal of demonstrating the feasibility of this concept, the Urban Mass Transportation Administration (UMTA) of the Department of Transportation has contracted with Foster-Miller Associates, Inc. (FMA) to design, fabricate and test such a tunnel lining system designed specifically for rock tunnel application. Under FMA direction, a team, composed of Underground Technology Development Corp., Ewing-Records and Associates, the Robbins Company and consultants from the University of Illinois has been formed to pursue design and fabrication of the ETLS.

The scope of the program and its three phases are summarized below.

	Phase	Completion Date
I	R&D - Resolution of basic technological problems	6/79
II	System design, fabrication and test	9/80
III	System modification, test and demonstration	9/81

Phase I effort was divided into four major tasks:

a. Task A - Technology Development Plan - Develop system specifications and testing requirements based on state-of-the-art review of tunneling and related construction practices.

b. Task B - Detailed Test Program Plan - Formulate test plan and design required test facilities.

c. Task C - Test Program - Conduct proposed tests.

d. Task D - Evaluation and Phase Report - Revise system specification based on results of test program and prepare a final report.

This report summarizes the results of all work conducted during Phase I.

1.1 THE EXTRUDED TUNNEL LINING SYSTEM

The proposed system concept is depicted in Figure 1 which shows the integrated TBM-ETLS deployed at the working face of a tunnel. The major components of the system are:

a. The "independent" slipform which shapes and supports the fresh concrete. The name derives from the relative motion permitted between the bulkhead and the slipform.

b. The annular bulkhead which provides the containment for the leading edge of the fresh concrete.

c. The concrete handling system which includes all equipment for transporting, batching, mixing, pumping and placing the special concrete mix within the slipform.

A typical cross section of the bulkhead and slipform is shown in Figure 2. The hydraulic cylinders which permit relative motion between the bulkhead and slipform and the hydraulic jacks and

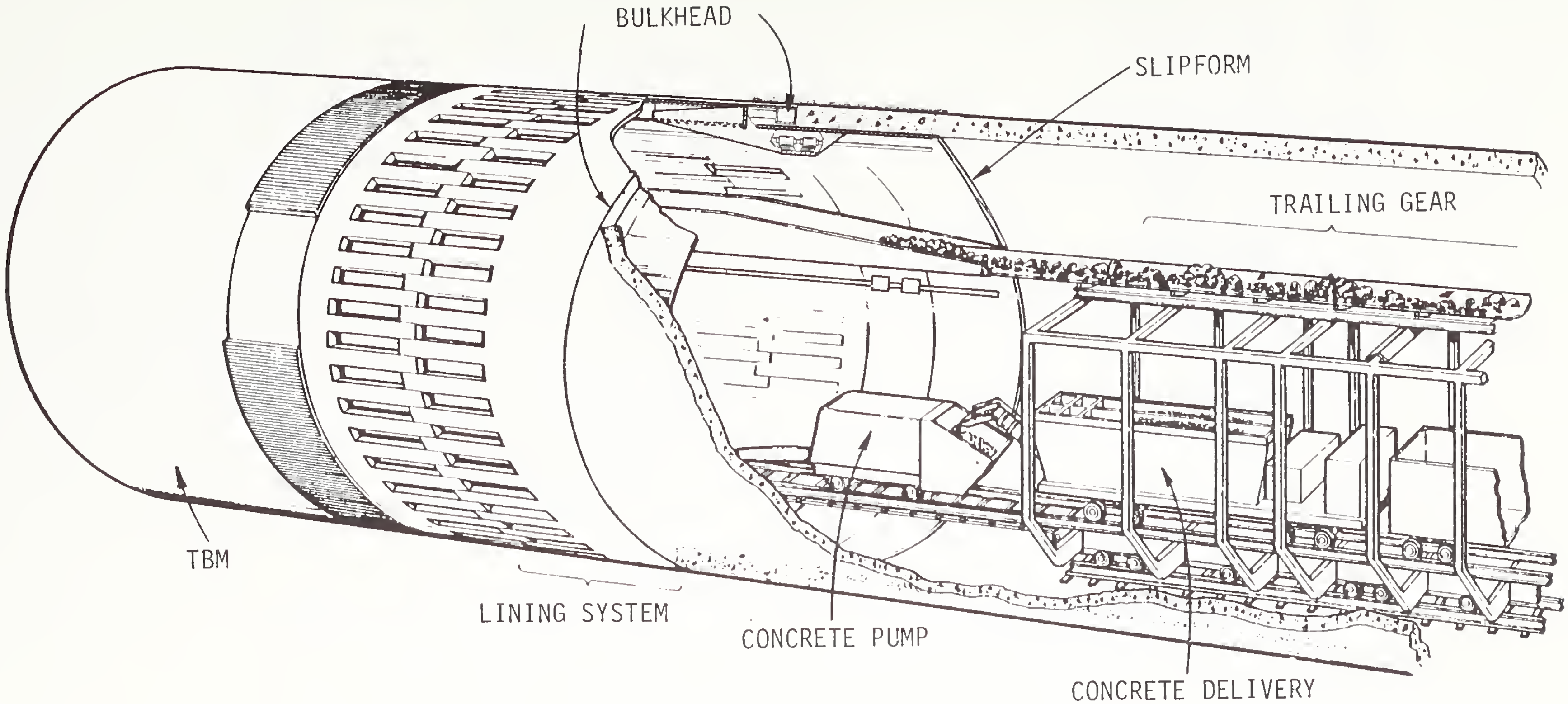


FIGURE 1. ETLS - OVERALL CONFIGURATION

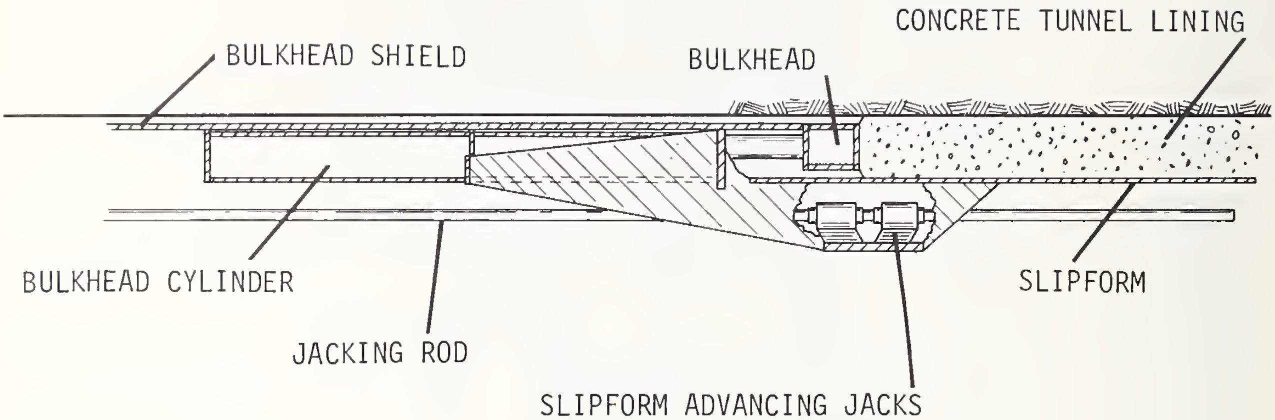


FIGURE 2. CROSS SECTION OF BULKHEAD AND SLIPFORM

jacking rods which connect the ETLS to the TBM are also shown. The remaining key component of the system is the concrete that will be used. This concrete must be formulated from a rapid setting cement and should have an initial set time in the 20 to 40 min range. This rapid set is necessary to achieve ETLS advance rates comparable to those of the TBM's in rock.

The ETLS design specifications are discussed in Section 5 of this report.

A detailed description of the ETLS design and operation is presented in Section 9.

1.2 CONVENTIONAL TUNNEL LINING SYSTEMS

Conventional tunnel lining/ground support techniques employing cast-in-place (CIP) concrete, precast concrete segments, and shotcrete were reviewed with the purpose of extracting any technology applicable to the ETLS. A qualitative comparison between these systems and the ETLS was also made in order to identify areas in which the ETLS offered the highest potential savings.

1.2.1 CIP Concrete Tunnel Lining

The CIP system illustrated in Figure 3 is installed long after the tunnel has been excavated and is considered a secondary ground support system. The primary support is usually provided by steel sets, however, in some cases, rock bolts and steel straps are used. Clearance requirements between the concrete slick line and the steel sets require minimum liner thicknesses in excess of 12 in. Deterioration of the lagging used with steel sets requires contact grouting to be used to prevent undesirable ground movement.

Compared to the CIP liner system, the ETLS offers the following potential advantages.

a. No primary support required - savings of labor and material

b. Final lining cycle eliminated - shortens project time and saves interest costs

c. Eliminate long-term rock exposure - minimizes rock load and ground movement

d. Thinner liner can be placed - reduces excavation

e. No need for contact grouting.

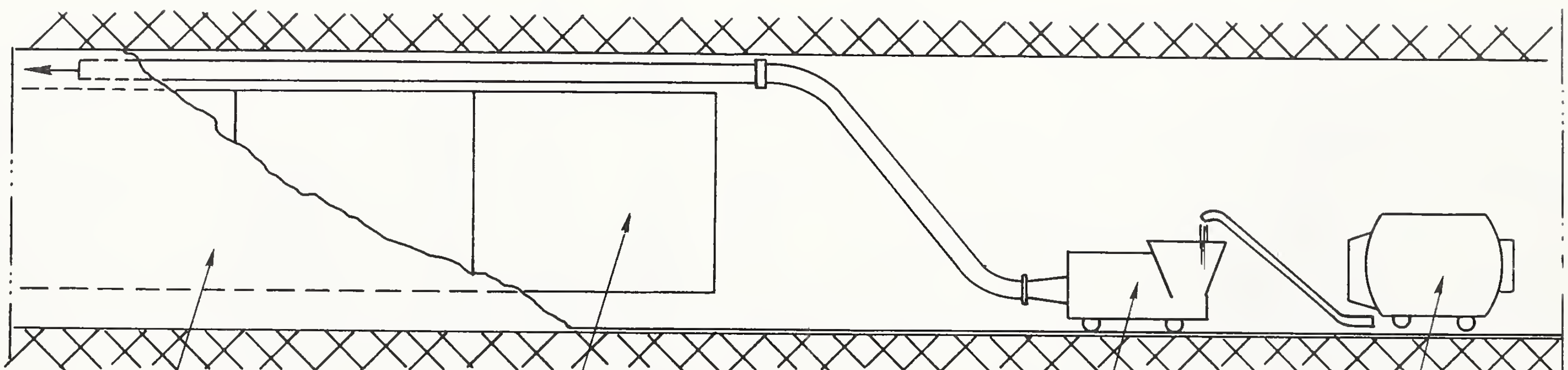


FIGURE 3. CONVENTIONAL CIP TUNNEL LINING

1.2.2 Precast Concrete Liners

Precast concrete liners were originally developed for soft ground tunnels, but have recently found applications in rock tunnels. The liner is made up of concrete segments which have been fabricated to close tolerances and which are erected within the tail shield of a TBM. A precast lining system is shown in Figure 4. Many current designs do not enable boring to continue while the liner is being erected; thus, the TBM advance rate is significantly restricted. The concrete segments must be reinforced to react the TBM loads as it shoves off the liner (in soft ground) and to prevent damage during handling. The completed liner must be backpacked and grouted to ensure uniform loading.

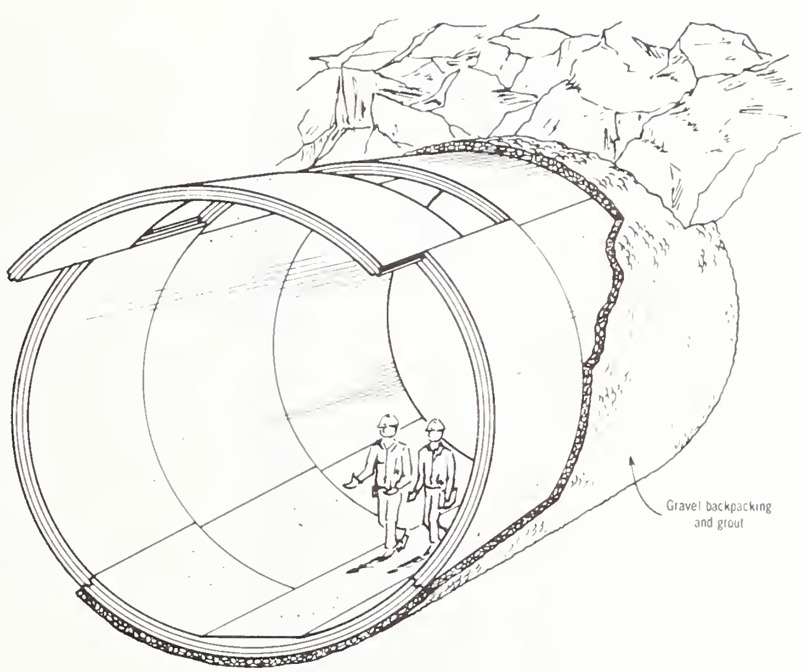


FIGURE 4. PRECAST CONCRETE SEGMENTS

Compared to the precast concrete liner, the ETLS offers the following potential advantages.

a. Higher potential advance rates - TBM can advance continuously

b. Reduced material cost - no factory cost, elimination of reinforcement for handling, no hardware for bolted segments, no pea gravel or grout

c. Reduce labor - no segment erection, no backpacking and grouting, no installation of special elastomeric joints

d. Simpler Materials Handling - segments are bulky over long hauls, and are subject to damage in transit.

1.2.3 Shotcrete

Shotcrete has been used extensively in Europe for support and lining of rock tunnels. While most of the application has been with drill and blast operations, Figure 5 illustrates a TBM equipped with a shotcrete ground support system.

While the shotcrete process is capable of placing a thin liner close to the tunnel face, it has the disadvantages of being a dirty, noisy operation not well suited for the machine driven tunnel environment. It is also difficult to place a liner of uniform thickness with shotcrete. Liners more than a few inches thick require multiple passes to avoid sluffing.

The ETLS offers the following advantages over shotcrete.

a. Reduced material cost (no rebound loss)

b. Guaranteed uniform thickness

c. Clean face operation

d. Full thickness can be placed in a single pass.

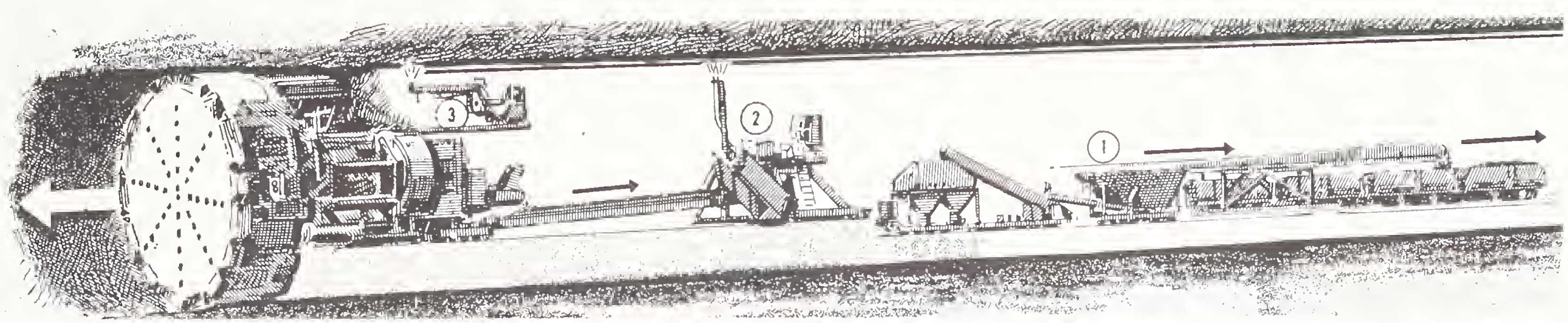


FIGURE 5. SHOTCRETE LINING BEHIND A TBM

1.3 STATE-OF-THE-ART REVIEW

Major features and potential problem areas of the ETLS design were identified and a state-of-the-art review was conducted to identify technology from related fields that could be utilized in the design and those areas in which testing was needed to supplement current technology.

The state-of-the-art review concentrated on the three major information areas listed below. These topics are discussed in detail in Sections 2, 3 and 4 respectively.

a. Related Construction Practices - to specify the tunneling environment, required interfaces with tunneling equipment and applicability of current concrete placement and tunnel lining techniques to the ETLS design.

b. Liner Concrete - to determine the strength versus time and workability characteristics of available concrete formulations to permit selection of the one best suited for use with the ETCS.

c. Tunnel Ground Support and Liner Design - to establish the ETLS liner thickness, strength and reinforcement specifications.

Items investigated and the major conclusions reached during the state-of-the-art review are listed below.

<u>Slipforming a Full Round Liner</u>

a. Conceptually similar to vertical slipforming

b. Concrete distribution is a major potential problem*

c. Control of concrete set time, and required concrete strength for self-support, are critical.*

*The items marked by an asterisk (*) were identified as requiring further evaluation in the test program, Task C.

Interface with Tunneling Environment

a. ETLS can be designed for general TBM and trailing gear features

b. Ambient conditions are generally favorable for concrete placement.

Liner Design Requirements

a. Six inch thick liner is sufficient

b. Fiber reinforcement does not contribute to liner performance.

Concrete Formulation

a. Superplasticizers and vibration can assist concrete distribution*

b. Regulated Set Portland Cement, initially proposed, is unsuitable

c. Alternatives exist, but must be evaluated.*

Concrete Handling

a. Concrete handling for the ETLS is more demanding than in conventional construction applications*

b. Off-the-shelf equipment is available

c. Minor modifications and adaptations will be required.

Lining Equipment Design

a. Forces on mechanism used to advance ETLS require evaluation*

b. Seals - some technology available*

*The items marked by an asterisk (*) were identified as requiring further evaluation in the test program, Task C.

c. Vibration of concrete - further evaluation required*

d. Form length and taper - depends on advance rate and concrete self-support.*

1.4 TEST PROGRAM

The test program plan is discussed in Section 6, in essence it was devised to address the following topics.

a. Formulation of a concrete for early workability and rapid set

b. Development and evaluation of concrete placement and distribution techniques

c. Evaluation of concrete self-support and required slip-form dimensions at proposed system advance rates

d. Evaluation of concrete pressure and drag loads on the slipform and bulkhead.

In an effort to find a concrete with the desired early workability and rapid set, several commercially available rapid set cements were tested, with accelerators or retarders as appropriate, for set time, workability and early strength. These tests, which lead to the formulation of a concrete from Very High Early Cement (VHEC) that met all ETLS requirements, is discussed in Section 7.

Questions regarding concrete distribution within a closed form, the feasibility of slipforming a rapid setting concrete, and the self-support capability of that concrete were also addressed during the test phase. Because of problems modeling systems using concrete, due to size effects of paste and aggregate, it was not desirable to use a scale model as the test device. To avoid this problem of size effect, separate tests were conducted to determine

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*The items marked by an asterisk (*) were identified as requiring further evaluation in the test program, Task C.

the factors effecting distribution and slipforming. The use of separate test facilities, representing partial segments of the liner, permitted full scale liner thickness and concrete aggregate to be tested without resorting to a full scale system mockup. These tests, which demonstrated the feasibility of distributing a rapid setting concrete in a closed form and of slipforming the same concrete, are discussed in Section 8.

The results for the test program and the information collected during the state-of-the-art review were incorporated into a revised ETLS design concept. This concept, which is discussed in detail in Section 9, will be designed, fabricated and tested.

2. RELATED CONSTRUCTION PRACTICES

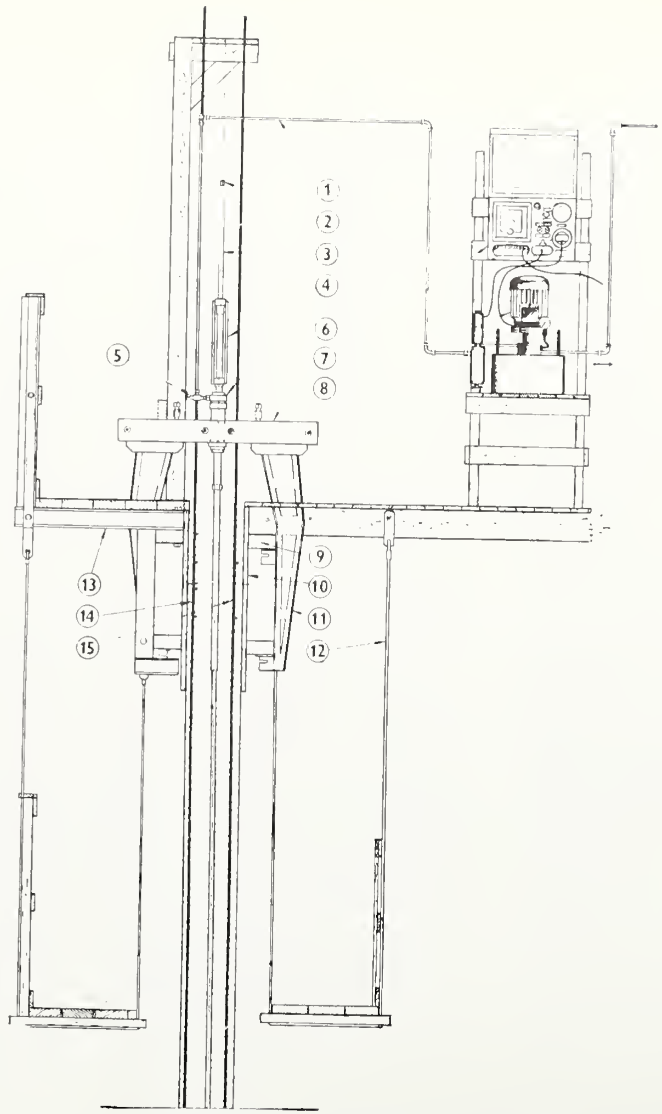
The ETLS concept is unique in many aspects. A survey of current construction practices revealed techniques, which while different on the whole, are similar to the proposed ETLS in some respects. In order to draw on the experience available, we have reviewed what we believe to be the construction practices most relevant to the ETLS. These are discussed in the following subsections.

2.1 VERTICAL SLIPFORMING

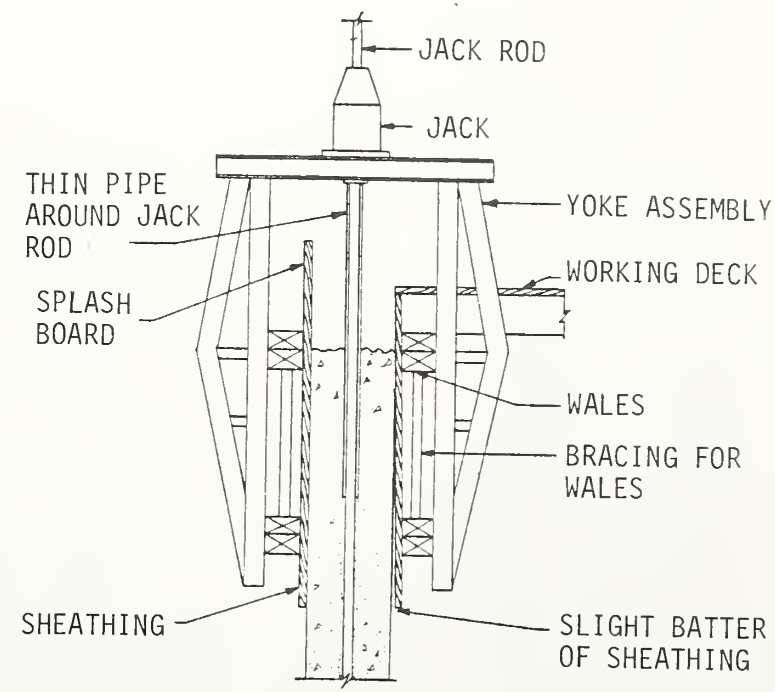
2.1.1 General Description

Vertical slipforming is a standard construction practice for tall structures with relatively uniform or moderately varying horizontal cross sections. The technique has been applied to both free standing (silos, towers, etc.) as well as partially constrained (shafts) structures. In principle, this process is very similar to that proposed for the ETLS. The primary differences arise from the change in the direction of slipforming with respect to gravity. The following paragraphs describe the details of the vertical slipform process, and discuss their relationship to the proposed ETLS.

Figures 6 and 7 illustrate the general principle of vertical slipforming, as well as the equipment employed in its application. The entire form, work deck, and finishing scaffold is supported by rods, usually embedded in the concrete. The form structure is advanced on these rods by jacks which climb the rods in small (about 1 in.) increments. Concrete is introduced into the top of the form with a rotating chute fed by a bucket, or with a flexible



a) GENERAL VERTICAL SLIPFORM CONFIGURATION (1)



b) DETAIL OF FORM AREA (2)

FIGURE 6. DETAIL OF VERTICAL SLIPFORMING EQUIPMENT

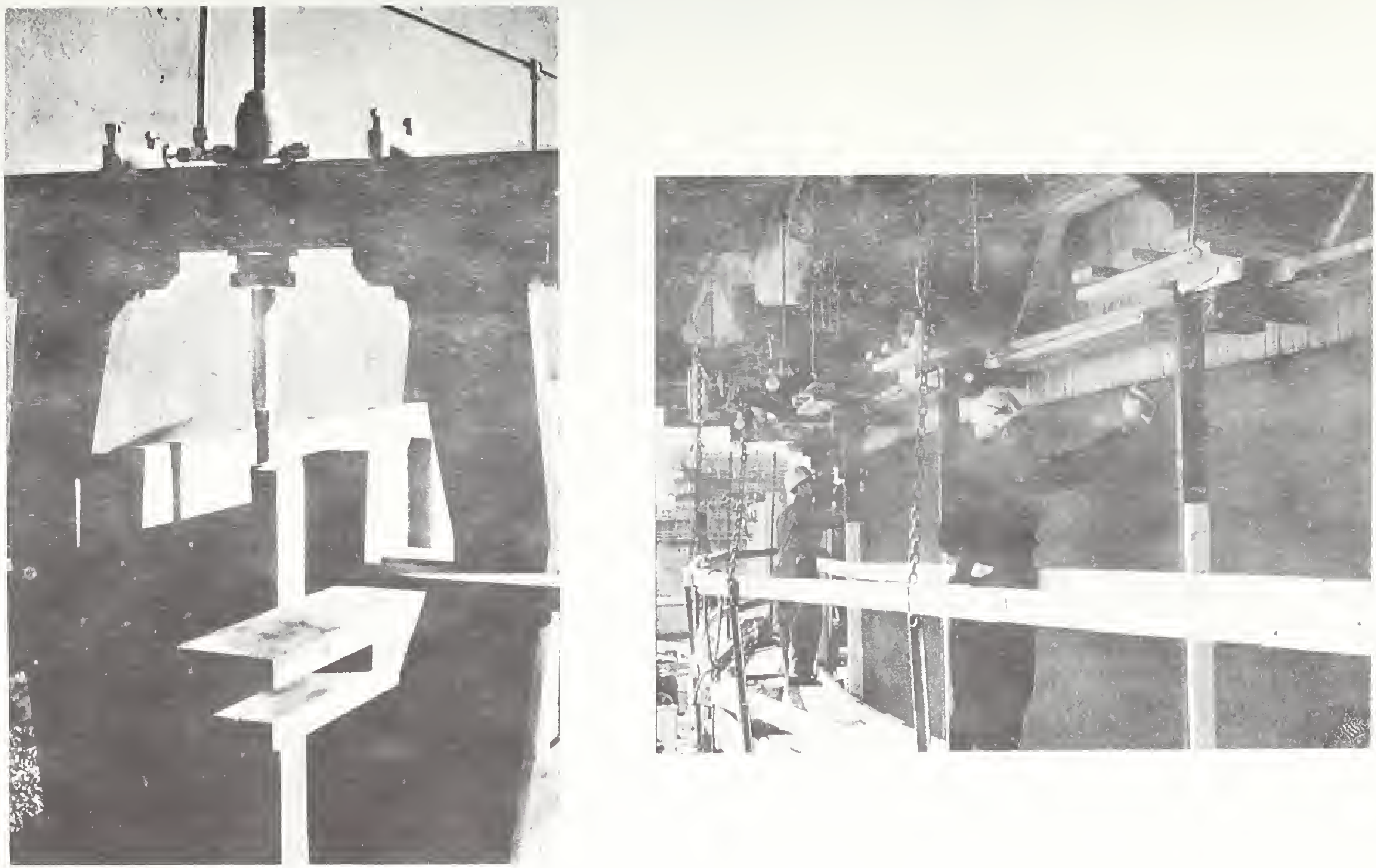


FIGURE 7. VERTICAL SLIPFORMING EQUIPMENT AND OPERATION (3)

hose fed by a concrete pump. The freshly placed concrete is consolidated with immersion (internal) vibrators. The form is continuously raised and the concrete, which is deeper in the form, begins to set. Set normally occurs within 12 to 18 in. from the bottom of the form. Once set concrete is capable of supporting itself, the form is no longer required. Forms are normally designed with a batter (opening taper) on the order of 1/4 in. in 4 ft of length, so that they don't drag on the set concrete.

A vertical shaft slipform is shown schematically in Figure 8. The primary difference between this and the process described above is that only the inner form is required. The jacking of the form on rods suspended from the collar is another variation.

2.1.2 Process Control

Vertical slipforming is more art than science; control of the process relies heavily on visual observation and feedback. The set point of the concrete in the form is determined by the depth of penetration of a rod driven by hand down through the fresh concrete. The degree of set of the free standing concrete is determined by thumb pressure, and its visual appearance can be related to the overall concrete quality. This type of access and feedback, whether manual or automated, should be incorporated into the ETLS system.

Alignment and level are generally maintained using level control devices, plumb bobs, and occasionally more sophisticated surveying equipment. The circumferential array of jacks is controlled from a central console, and can be finely controlled to correct the form position. ACI recommends tolerances for vertical slipform work of ±3/8 in. in the thickness and 1 in. deviation from plumb per 50 ft of height.(2)

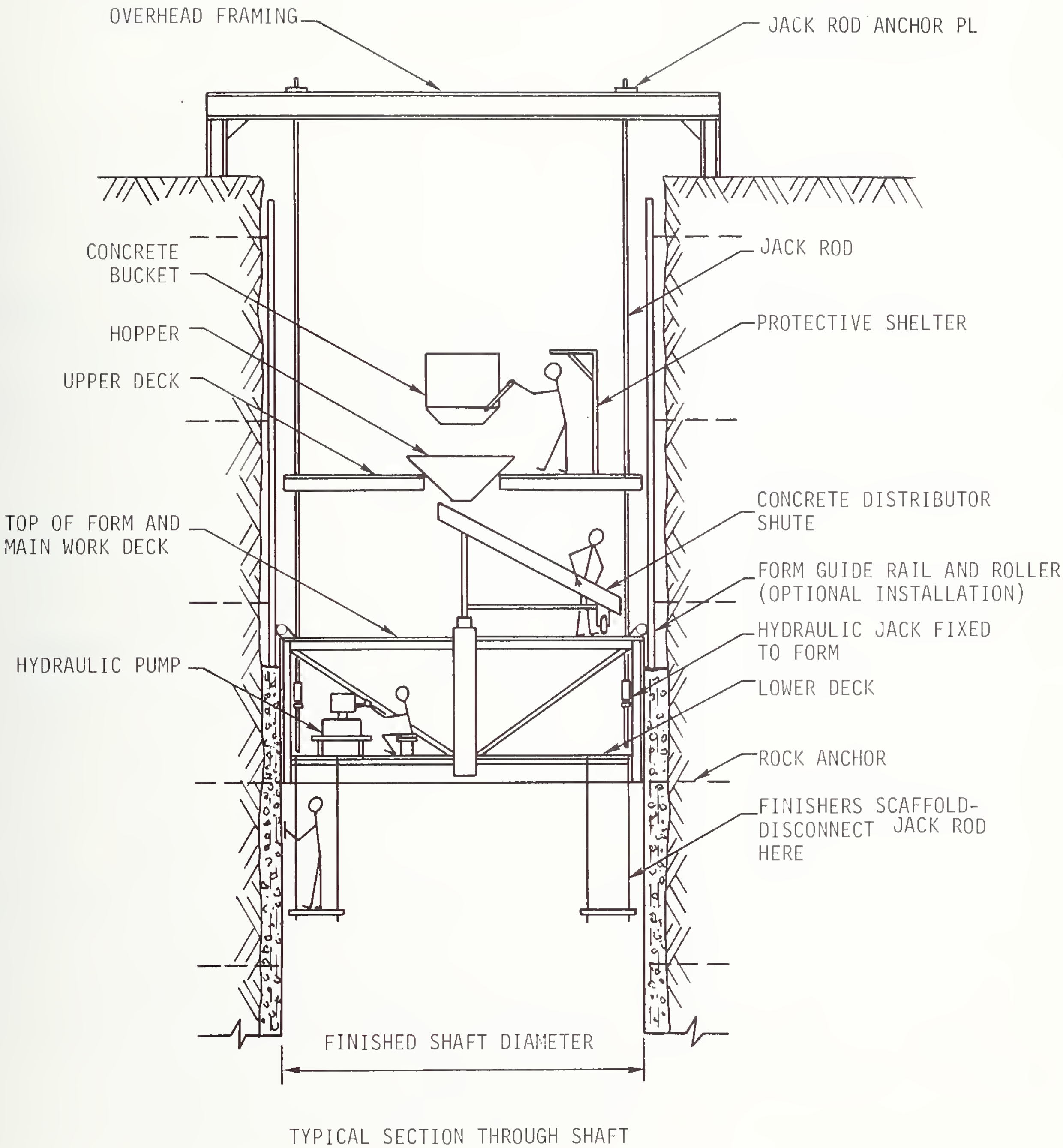


FIGURE 8. VERTICAL SLIPFORMING OF A SHAFT

Vertical slipforms tend to twist about the vertical axis unless some circumferential constraint is provided. Structural elements, such as channel selections, are sometimes embedded in the concrete to provide a reaction point for a jack or comealong used to constrain the form.

2.1.3 Application to ETLS

The following aspects are of particular interest for the ETLS application.

2.1.3.1 Form Length and Advance Rates - Vertical slipforms usually range from 4 to 8 ft in length, with advance rates of 6 to 24 in./hr. Higher rates (up to 48 in./hr) have been reported, but are not standard. The advance rate of conventional vertical slipforms is normally limited by the speed with which the men on the work deck can place concrete. Consequently, there are no great demands on accelerated set concrete. In principle, with the proper combination of form length and concrete set time, the slipform rate is not limited. This fact will be utilized in matching the ETLS form design and concrete formulation with the desired advance rates.

2.1.3.2 Concrete Control - *Careful control of the concrete is the most critical element in a successful slipforming operation.* An on-site batch plant can be used to overcome the problems associated with variations in age of concrete delivered by transit mix trucks. In any case, a full-time concrete technician on the job should continuously monitor slump and set time, and carry out mix proportion adjustments to account for changing concrete behavior due to ambient temperature and humidity changes.

The tunnel environment should provide relatively constant temperature and humidity, and should therefore eliminate some of the control problems. Nevertheless, it will be important to exercise close control over the batching and mixing of the concrete.

2.2 HORIZONTAL SLIPFORMING

2.2.1 General Description

Horizontal slipforming is a common practice for cast-in-place (CIP) pavements, canals, curves, and median barriers. Although it goes by the same name, the process is quite different from vertical slipforming. The only similarity is that both employ a continuously moving form.

Horizontally slipformed concrete leaves the slipform machine compacted to such a degree that it can support itself. Figure 9 illustrates the type of self-supporting structure that can be cast in this fashion. The horizontally slipformed concrete has not reached a chemical set as in the vertical slipform. Thus the advance rate is limited only by how fast the concrete can be delivered to and placed by the machine. Curbers, for example, can go 2800 ft in a 6-hr shift and up to 35 ft/min.

2.2.2 Process Control

The two basic approaches to horizontal slipforming are:

a. Extrusion Process - In the extrusion process, the form propels itself forward on the placed concrete. Very low slump concrete (about 1/4 in.) is used. It is packed into a tapered mold with an auger. The reaction of the auger against the placed concrete drives the machine forward.

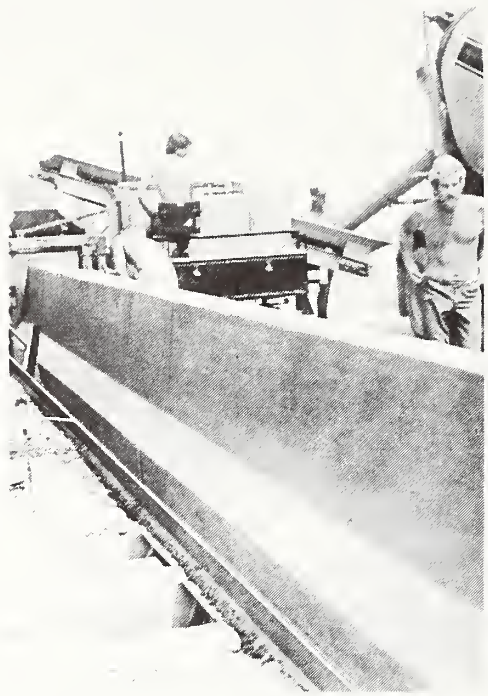


FIGURE 9. HORIZONTAL SLIPFORMING OF A MEDIAN BARRIER (4)

b. "Slipform" Process - This process uses higher slump concrete (3 to 4 in.), and consolidates it into the mold using continuously operated immersion vibrators mounted to the machine structure. The machine propels itself on tracks.

Regardless of the approach, concrete placed through horizontal slipforming is essentially zero slump concrete, supporting itself through its stiffness. Because it is not set, it can only support limited weight, and it is highly sensitive to external disturbance. In one case, for example, the rebar of a median barrier being horizontally slipformed, was touched by a vibrator on the slipform machine. As a result, 100 ft of slipformed concrete collapsed due to vibration transmitted through the rebar.

2.2.3 Application to ETLS

The process is of interest to the ETLS development. We can conceive of an extruded liner supporting itself upon leaving the slipform due to a combination of stiff concrete and chemical set. Use of low water/cement ratios, superplasticizers, and vibrators will be investigated for this purpose.

2.3 CAST-IN-PLACE (CIP) PIPE

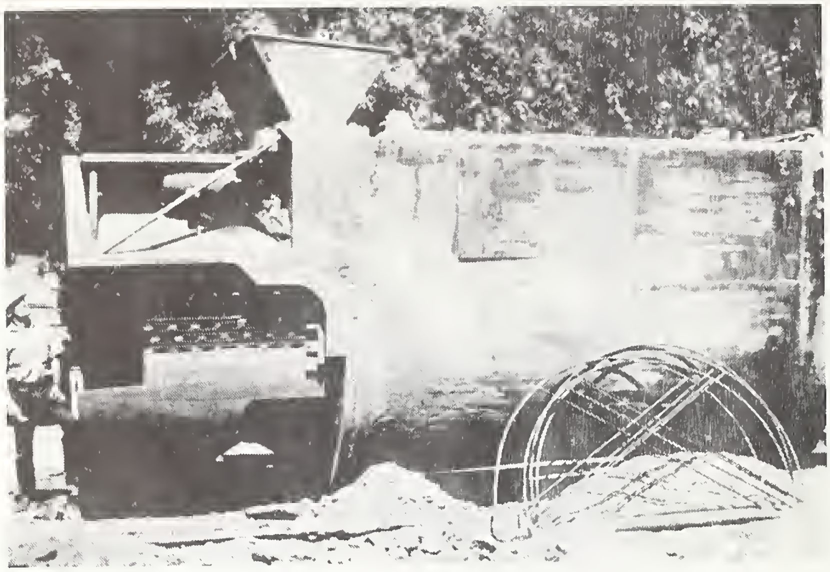
2.3.1 General Description

A number of techniques have been developed in recent years for the continuous rapid placement of large diameter pipe. The most frequent applications are for irrigation in the southwest regions of the United States. These methods are often referred to as slipforms. In fact, only a portion of the pipe section is slipformed; the remainder is supported by fixed forms which are later advanced.

The methods normally involved casting a pipe in a "U" shaped excavated trench. A typical piece of equipment for this application is shown in Figure 10. The front end of this machine ("boat," right side of photo) holds the trench open and guides the machine in the trench. Concrete is delivered through the hopper in the top, and is forced around the periphery in a variety of ways. The concrete is cast at the rear of the machine as follows:

a. Inside Diameter

1. Bottom 120 deg - slipformed with mandrel attachment to the machine.



a) MACHINE USED TO FORM CAST-IN-PLACE PIPE BY SINGLE STAGE PROCESS USING METAL FORMS. ALUMINUM FORMS 6 FT LONG ARE FED INTO MACHINE OVER ROLLERS ATTACHED TO MANDREL. METAL STRUTS TO SUPPORT UPPER FORMS ARE SHOWN LEANING AGAINST THE MACHINE (2)

b) PLACEMENT OF CAST-IN-PLACE CONCRETE PIPE. CONCRETE, TRANSPORTED BY TRUCK MIXER, IS BEING DUMPED FROM THE LEFT SIDE INTO THE PLACING MACHINE, PX-D-34074 (5)

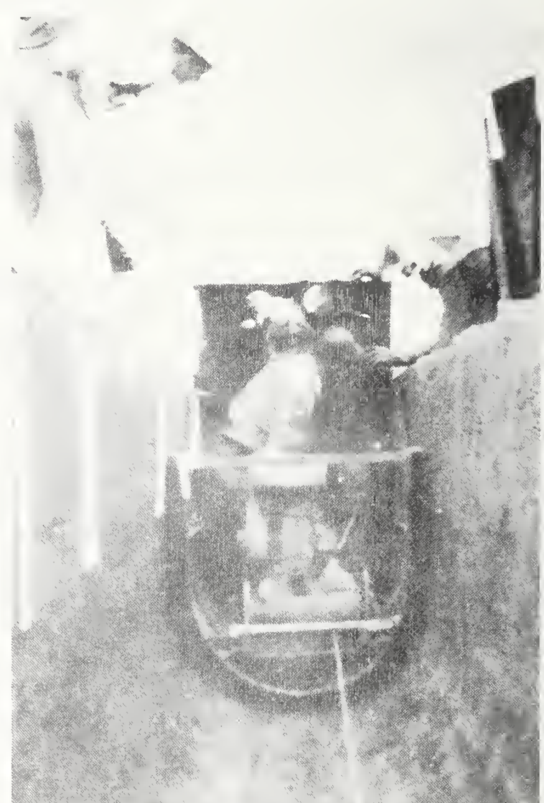


FIGURE 10. CONTINUOUS PLACEMENT OF CAST-IN-PLACE PIPE

2. Top 240 deg - stationary forms attached over the mandrel inside of the boat. Later retracted and reused.

b. Outside Diameter

1. Bottom 240 deg - trench acts as form.

2. Top 120 deg - slipformed by trailing arch-shaped form.

Tolerances on these types of structures are normally rough, but rates of 600 ft/day are routinely achieved (6). In some applications, a long inflated balloon is used as a replacement for the fixed form section.

The largest scale project of this type, the Air Force MX Project, involves a 15 ft diam fiber-reinforced concrete tunnel. Figure 11 shows the general equipment arrangement. The technique is identical to that discussed above, with the exception of its size and the use of fiber reinforced concrete.

2.3.2 Applications to the Extruded Liner

The aspects of the systems described above which are of interest to our development of the ETLS are:

a. Distribution of concrete

b. Bulkheads and seals

c. Use of fiber reinforced concrete.

These CIP pipe systems require introduction of concrete from a central hopper at the top, and uniform distribution of that concrete throughout the region between the slipform, a bulkhead, and the excavated surface. Concrete distribution is one of the major technical unknowns facing the ETLS. The CIP pipe techniques generally employ some type of mechanical consolidator buried in the concrete. Varieties include that shown in Figure 11, vertical stuffing augers, and a patented system using a wiggling rebar.

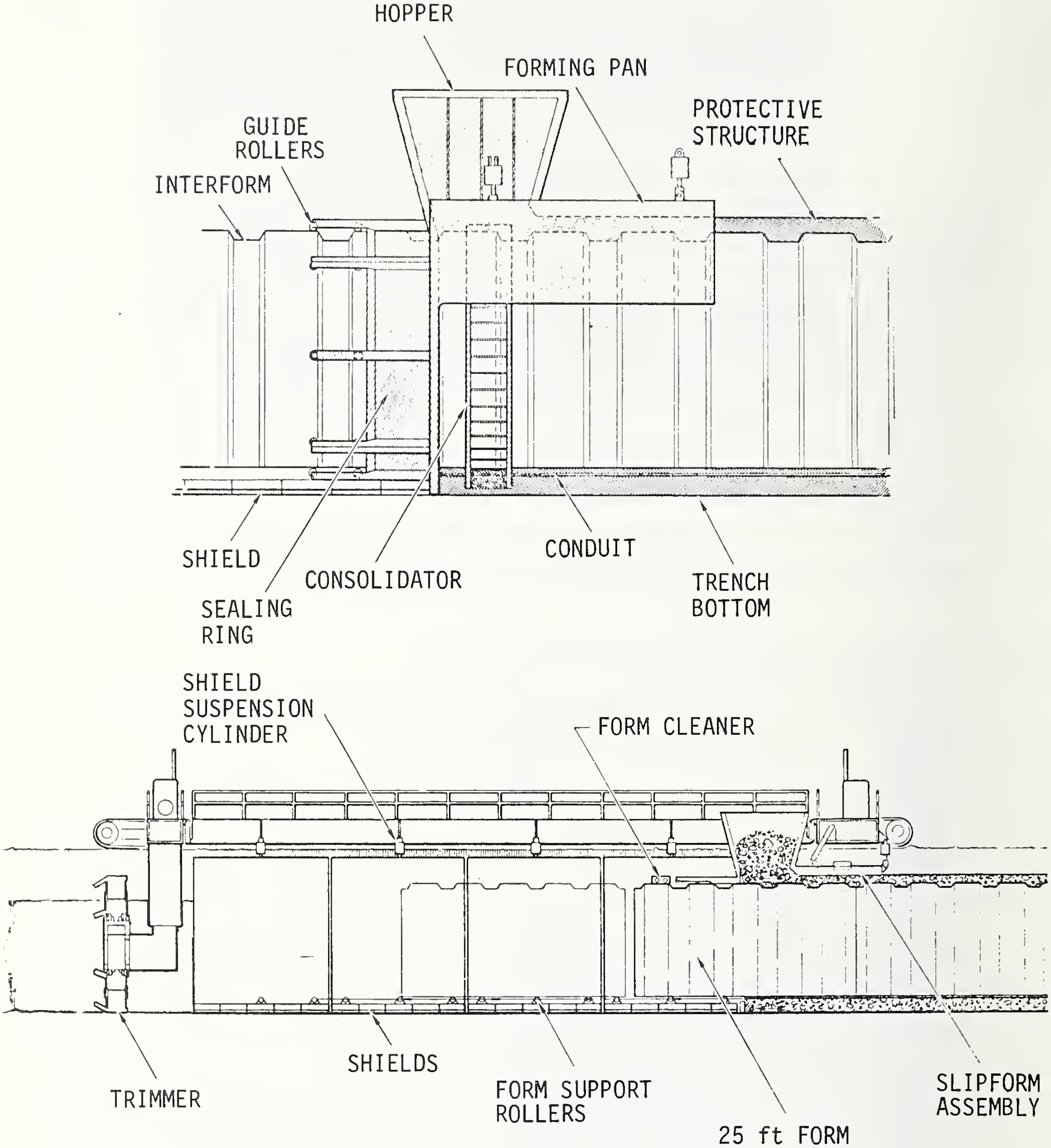


FIGURE 11. PROPOSED FORM SYSTEM FOR MX PROJECT

The problem with these techniques is that by being embedded in the concrete, they do not lend themselves for use with a system that may have frequent shutdowns or employ rapid setting concrete. Consequently, some alternative approach will be required for the ETLS.

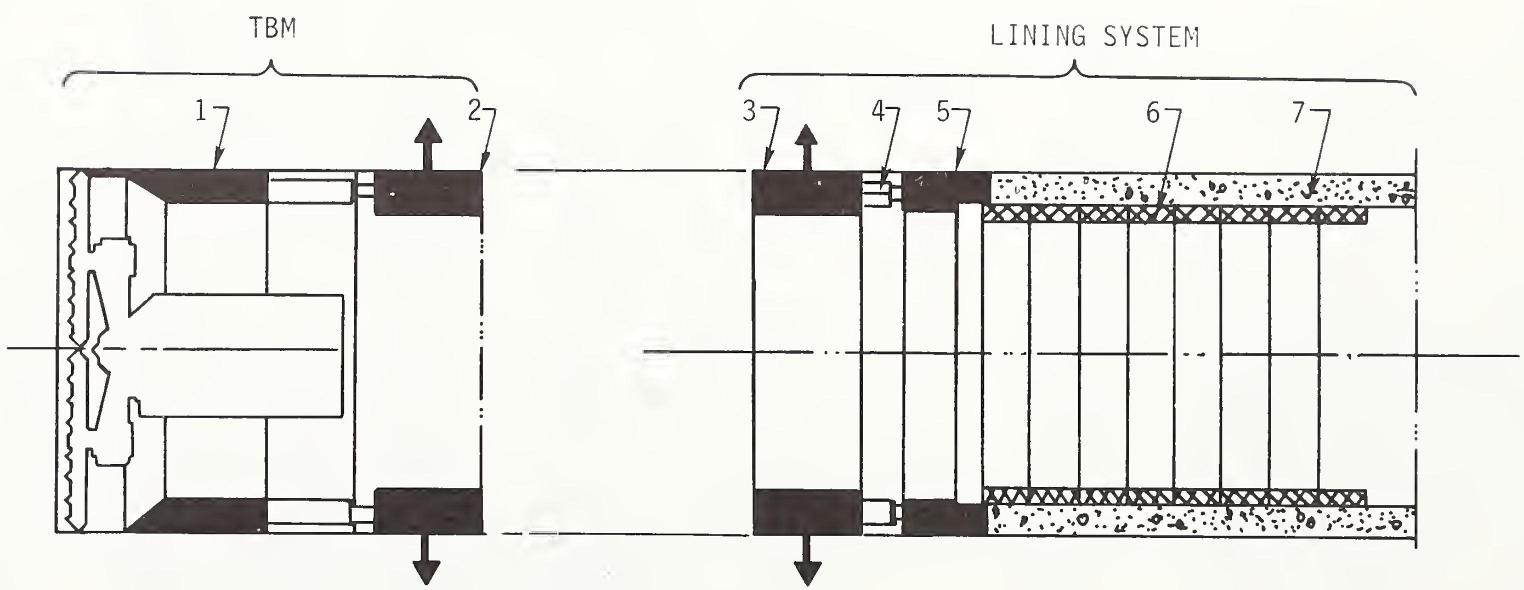
CIP pipe systems all employ a traveling bulkhead to support the vertical surface of the poured concrete. Leakage of concrete, under its own pressure head, past the bulkhead does not appear to be a problem. It may be a problem in the ETLS since significantly higher concrete pressures are anticipated.

2.4 SOVIET "PRESSED CONCRETE" CONTINUOUS TUNNEL LINER

2.4.1 General Description

The Soviet Union has developed a continuous CIP concrete tunnel lining system which apparently has been applied to soft ground, rock, and mixed tunneling conditions. Due to the limited information available, we are not able to assess the system from any practical point of view. The following comments are based on a proposal submitted by the Union of Soviet Socialist Republics to the United States Department of Transportation.

The Soviet system places concrete continuously behind a mechanized shield or TBM as shown in Figure 12. The form is essentially a jump form made up of a series of sections ("shutters") which leapfrog from the rear where the concrete has set to the front where new concrete will be poured. The front of the form is bounded by a bulkhead, or "pressing ring." Concrete is placed into the void between the previous pour, the newly erected shutter, and the pressing ring, using an air placer. It is then subjected to high pressure (~200 lb/in.^{2}) by the pressing ring and compacted as shown in Figure 13. The pressed concrete is well consolidated



1 - MECHANIZED SHIELD
2 - CARRIER RING OF A SHIELD
3 - CARRIER RING OF PRESSING DEVICES
4 - PRESSING JACKS
5 - PRESSING RING
6 - SHUTTER
7 - COMPLETED TUNNEL LINING

FIGURE 12. SOVIET LINING SCHEME

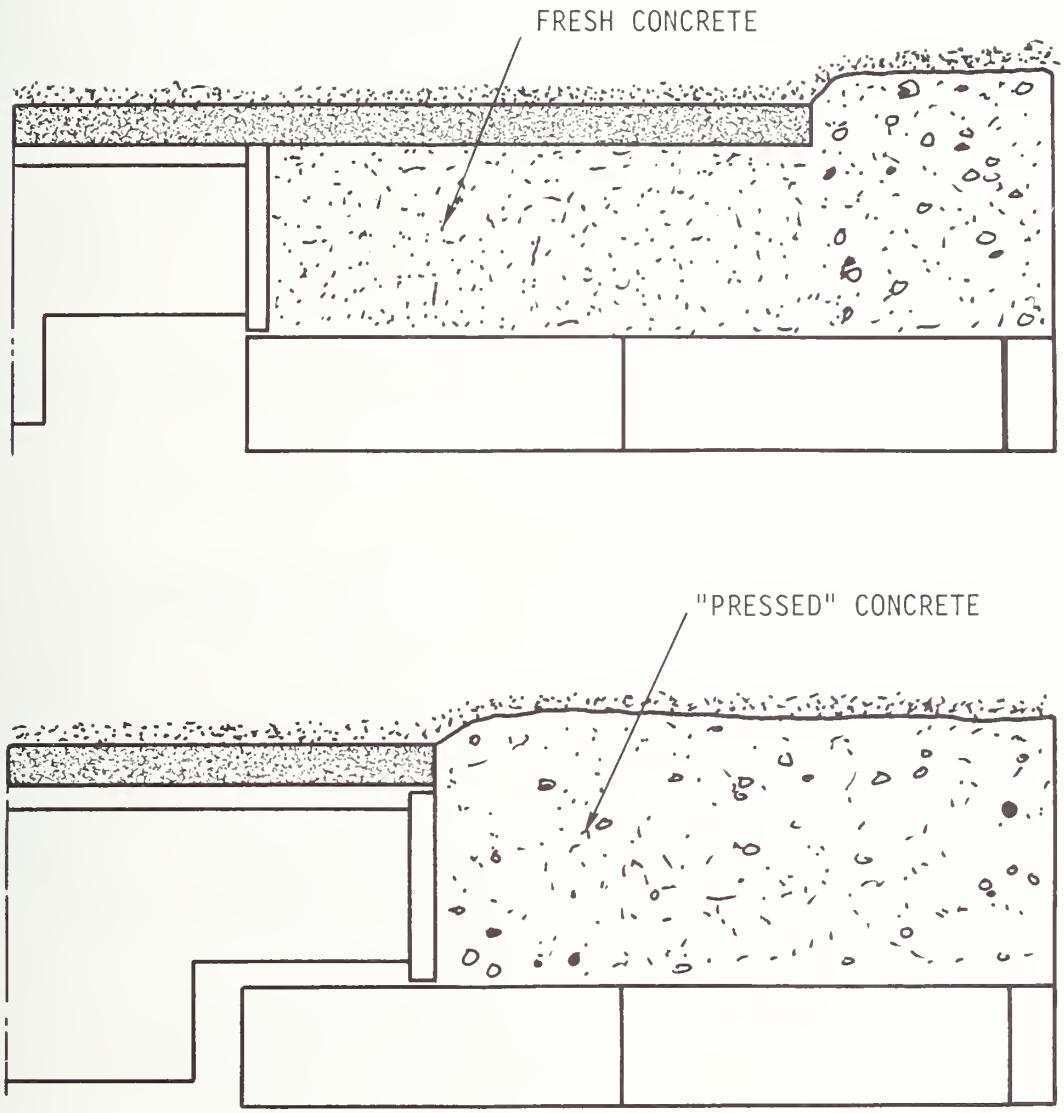


FIGURE 13. PRESSED CONCRETE APPROACH

and maintains its vertical boundary while the pressing ring is released and a new void is created for the next pour. For soft ground, the concrete reaction drives the shield forward, and the "pressed concrete" is forced into the tail void. For rock, the pressing ring reacts against an independent set of grippers ("carrier ring") which is independent of the TBM (see Figure 12).

Figure 14 shows the overall system layout for rock application. Concrete is supplied by a series of placer cars brought to the heading. Muck is conveyed through the forms and over the placer cars by conveyor to muck cars. A transportation bridge mounted along the tunnel axis provides a path for the shutter removal and placement mechanism.

As of August 1976, the Soviets claim to have lined 4.7 mi of soft ground and rock tunnel using this system.

2.4.2 System Specifications

The stated specifications for the Soviet rock system are listed below:

a. Excavated diameter (19 ft)

b. Lined diameter (17 ft)

c. Pressure on concrete (up to 215 lb/in.^{2})

d. Thrust on carrier ring (computed) (1.6 million lb)

e. Average advance rate (up to 33 ft/day)

f. Total manpower at the face (including excavation and mucking) (4 to 6 men)

g. Minimum curve radius (1150 ft)

h. Lining system power requirement (70 kVA)

i. Lining system weight (330 tons)

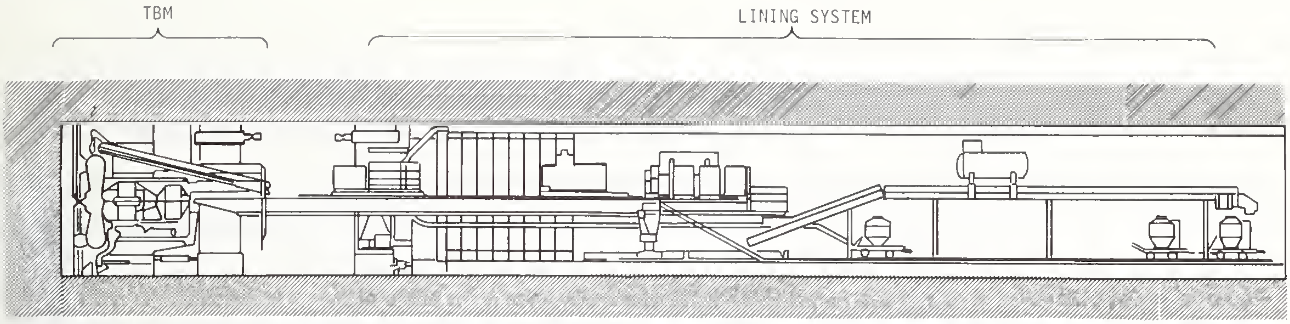


FIGURE 14. OVERALL LAYOUT OF THE SOVIET LINING SYSTEM

j. Final concrete strength (4000 to 6500 $lb/in.^2$)

k. Shutter width (2 ft)

l. Number of shutters (14).

2.4.3 Applications to ETLS

Taken at face value, the Soviet system is massive and unimpressive in its advance rate capability. However, the system has the following features which are particularly of interest to us.

a. Filling the Form - Figure 14 indicates the form is filled through a single injection port in the pressing ring at the top. The details of filling and the avoidance of segregation and voids are not discussed in the Soviet proposal.

b. Use of Pressure to Stabilize Concrete - The "pressed concrete" supports itself due to the removal of excess water by the pressure. It becomes, essentially, zero slump concrete. The pressures used by the Soviets are high, and hence the associated hardware is massive. This principle, however, may be used at lower pressures to advantage in the ETLS design.

c. Lining System Independent of the TBM - The lining system propels itself independently of the TBM with its own gripper section. The concept of independence is attractive, but the cost of the hardware is high. It may be more desirable to react off of the TBM grippers using a system that allows for a reasonable amount of relative motion.

d. Bulkhead Seals - We are interested in the method used to seal between the pressing ring and the rock, particularly under these high pressures. Unfortunately, no detail has been presented in this regard.

The above features have been given consideration in the development of our design concept.

2.5 TUNNELING ENVIRONMENT

Prior to proceeding with a detailed conceptual design effort of the ETLS the tunneling environment in which the system would operate was studied. This study identified the conditions with which the ETLS would have to cope and the peculiarities of the equipment with which it would interface. System interfaces with various TBM and trailing gear designs along with the ambient conditions in which it will operate are discussed in the following subsections.

2.5.1 Tunnel Boring Machines

While there has been considerable discussion of a "universal" TBM design, there is no current trend in that direction. The TBM is usually custom designed for a particular project and a number of different design approaches are currently popular among the various equipment designers.

The basic TBM design is determined by the type of ground support that will be employed. The two most popular designs are illustrated in Figure 15, an open machine designed for use with steel sets, and Figure 16, a fully shielded machine designed for use with precast segments.

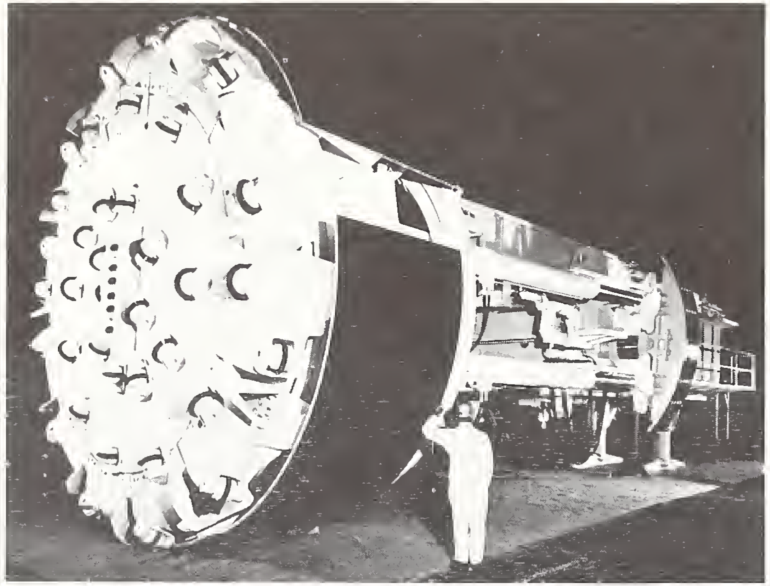
In spite of the fact that the TBM is usually custom designed for a particular project the design for that project could vary significantly depending on the manufacturer. Thus, in order to maximize the potential number of projects on which the ETLS could be employed it will be designed around a general TBM configuration, rather than tailored to a specific design.

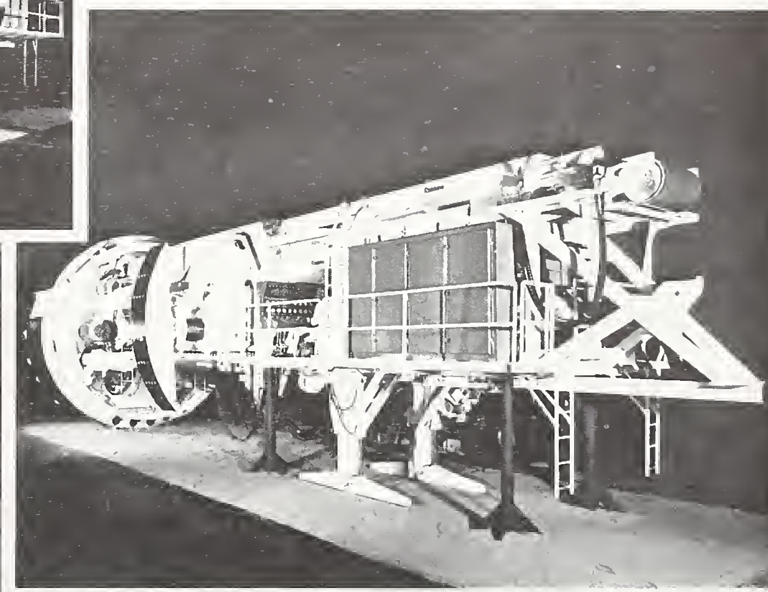
2.5.2 Trailing Gear

As with the TBM, the trailing gear is usually custom designed for a particular project. There are several common designs and the one selected will depend on the tunnel diameter and anticipated advance rates. A typical TBM/trailing gear configuration is shown in Figure 17.

The ETLS will be located between the TBM and trailing gear and regardless of the particular design of this equipment provisions must be made to pass muck, ventilation and auxiliary services through the region of the tunnel occuppied by the ETLS. The ETLS will be designed with this requirement in mind; here again the design will be general enough to interface with many TBM/trailing gear combinations.

The ETLS will place one requirement on the trailing gear that is to provide a continuous flow of concrete to the ETLS concrete pump. There are several trailing system designs that can accomplish this; the one which best suits the remaining conditions imposed by the TBM and tunnel diameter will be selected.





ROTARY ROCK BORER
DIAMETER 20 ft (6,14 m)

FIGURE 15. ROBBINS MODEL 201-158 TUNNEL MACHINE

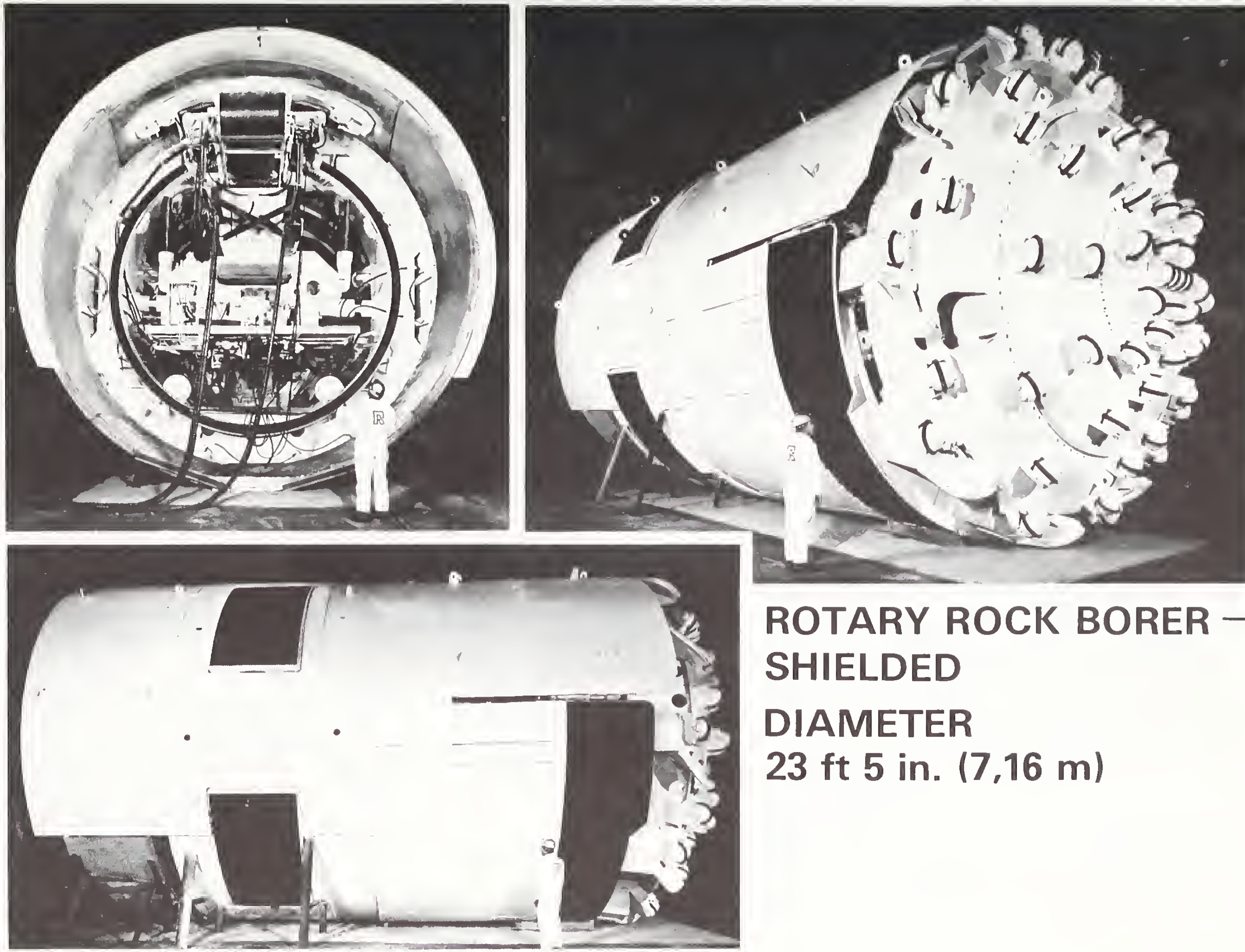


FIGURE 16. ROBBINS MODEL 233-172 TUNNEL MACHINE

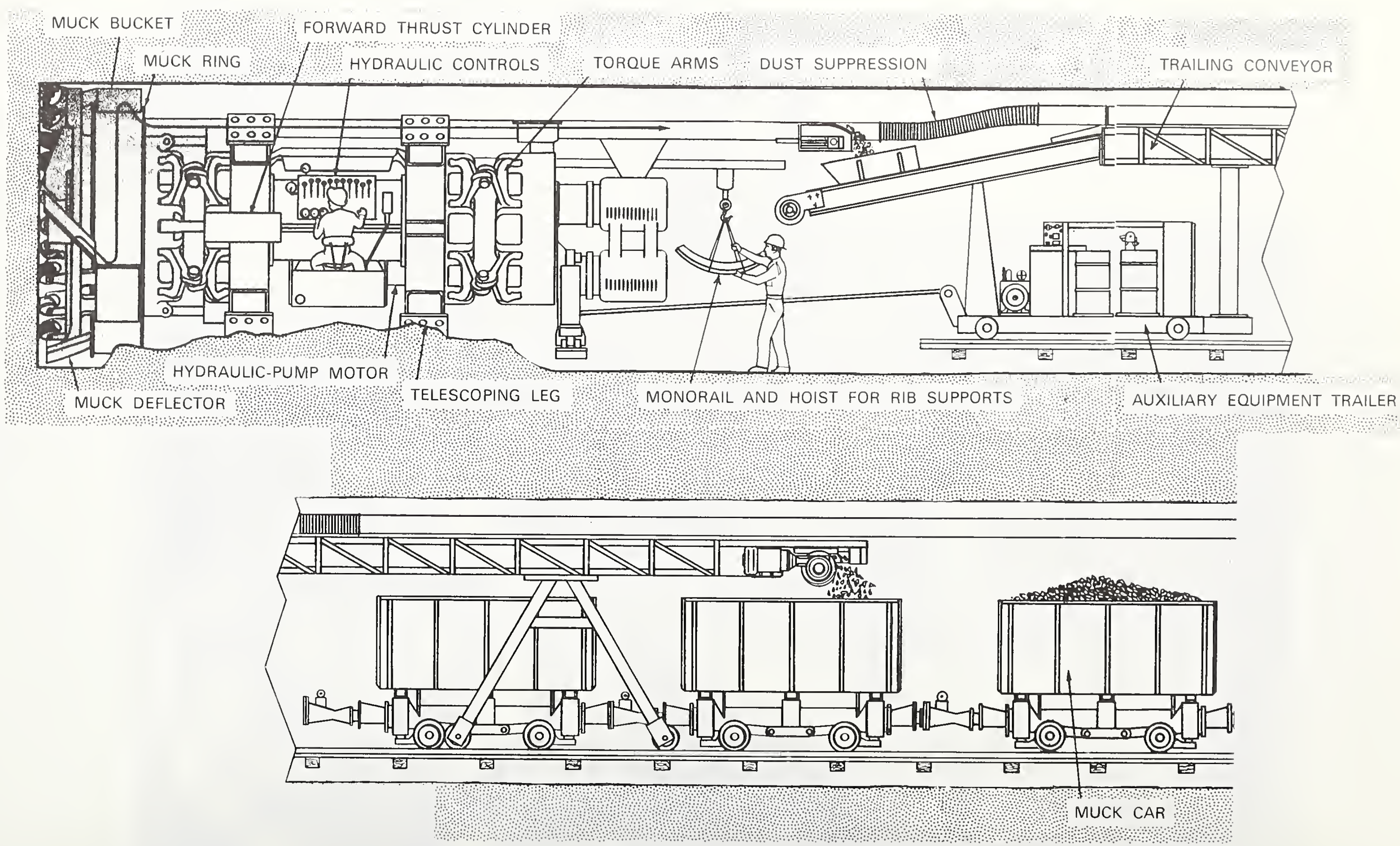


FIGURE 17. TBM AND TRAILING GEAR

2.5.3 Ambient Conditions

The ambient conditions in which the ETLS will operate are not significantly different than those experienced during cast-in-place liner construction.

The temperature will be relatively constant, although somewhat higher than that experienced during cast-in-place operations. The higher temperature is due to heat generated by the TBM auxiliaries and power supply. Air velocity will also be higher than during cast-in-place operations due to the increased ventilation required close to the tunnel face for dust control.

This combination of higher temperature and air velocity might cause some acceleration of the concrete curing rate, however, the effect will be minimized by the high humidity normally present in the tunnel. It can easily be eliminated with the application of a curing agent if it becomes a problem.

Standard means of water control can be utilized to prevent damage to the extruded liner. However, these controls should be applied as soon as the wet area becomes accessible behind the TBM in order not to interfere with the ETLS advance.

3. CONSIDERATIONS FOR LINER CONCRETE

Parker (7) proposed a system for extruding an idealized, fiber reinforced concrete in a continuous slipforming operation. This proposal included recommendations for a pumpable concrete with favorable workability coupled with a fast-setting, rapid-hardening behavior. Fiber reinforcement was proposed as a means of eliminating conventional reinforcing bars. Attractive properties of increased flexural and post-cracking strength made the inclusion on fibers appear to be a reasonable alernative to reinforcing steel. The results of recent large scale tests simulating liner loading conditions indicate that liners made of fiber reinforced and plain concrete behave similarly at failure (8). The need for conventional reinforcement was also questioned as a result of these tests, which showed that although conventionally reinforced liners showed improved qualities over plain or fiber reinforced sections, the differences would be minimal for in situ loading conditions.

The mix design requirements for a concrete suitable for ETLS application are discussed in the following subsection. The remaining subsections of this chapter discuss the influence of the individual mix constituents on the final mix formulation. In areas where the behavior of a particular material will strongly influence the end result, more extensive discussion in included. Because the presence of fibers complicates both the fresh and hardened properties of the concrete, a discussion of the strength, post-cracking behavior and handling problems is included. A concrete mix design procedure is described summarizing the work of previous researchers.

3.1 MIX DESIGN REQUIREMENTS

In addition to being pumpable and workable enough to permit proper placement and consolidation within the slipform, a concrete mix must meet the following requirements in order to be a suitable material of an extruded tunnel liner.

a. Set control - The cement should cause the concrete to set in a range of 20 to 40 min, with adequate workability as close to set time as possible.

b. Strength gain - In order to support construction loadings, the concrete should exhibit a rapid strength gain from set so that a compressive strength of 500 $lb/in.^2$ is met in about 2 hr.

c. Ultimate Strength - For good quality tunnel concrete, the final concrete compressive strength should be about 5000 $lb/in.^2$ at 28 days.

d. Durability - The constituents of the mix should be compatible and produce a durable final product not sensitive to deterioration under water or under sulfate attack.

e. Shrinkage - The concrete should not undergo excessive volume changes in the plastic or hardened states.

The cement constituent has a major effect on the behavior of concrete, especially at early ages. Previous research (7,9,10) studied the effectiveness of a variety of regulated-set (reg-set) cements. The earliest work (7) demonstrated the potential usefulness of these products. Reg-set is no longer manufactured in the United States and the only remaining stockpiles are unsuitable for ETLS application because of their extremely rapid ($\sim$10 min) set times. This development led to an investigation to locate the cement, currently in production, which best satisfies the established behavior criteria. Information on a variety of possibilities has been gathered and is presented in the following subsection. In almost every case, early strength characteristics are not well documented. Consequently, the need for a laboratory study to determine the best available cement material was identified.

3.2 CEMENT CANDIDATES

3.2.1 Influence of Cement on Concrete

Cement is one of the most influential constituents in a concrete mix. Properties most directly effected by different cements are:

a. Set time

b. Rate of strength gain

c. Ultimate strength

d. Durability

e. Shrinkage.

Other properties effected to a lesser degree include workability and pumpability.

Many cements are not able to meet the ideal mix design requirements, discussed in the preceding section, without the aid of admixture products to supplement inadequate behavior. A number of cement options, which were investigated as part of Task A, along with an assessment of the performance characteristics of each are summarized in Table 1. Options with an asterisk (*) indicate those selected for laboratory evaluation.

TABLE 1. EVALUATION OF ALTERNATIVE CEMENTS

CEMENT[1]	20 TO 40 MIN SET TIME	500 $LB/IN.^2$ IN 2 HR	5000 $LB/IN.^2$ ULTIMATE STRENGTH	DURABLE	RELATIVE COST TYPE I = 1
Current Reg-Set	No	?[2]	Yes	Yes	2
Aluminous, Accelerated*	Yes	?	?	Yes	3-4
Duracal, Retarded	No	?	Yes	No	3
Very High Early Cement**	Yes	?	?	?	3-4
Portland/ Aluminous Mixtures*	Yes	Yes	?	?	1-3
Highway Patching Materials	Yes	?	?	?	3-20
Type III, Accelerated*	?	?	Yes	?	1
Type I	No	No	Yes	Yes	1

[1]Foreign reg-set has not been included due to lack of information. Type I cement is listed for comparison only.

[2]? denotes uncertainty, laboratory testing would clarify.

*To be tested in the laboratory. **Retarded mix to be tested in the laboratory.

3.2.2 Reg-Set Cement

Reg-set portland cement is a quick-setting, rapid-hardening cement that was developed by the Portland Cement Association. The cement is formulated with a quantity of calcium fluoroaluminate which is responsible for its unique behavior.

Because of its fast-setting, rapid-hardening characteristics, reg-set cements were studied for potential use with the extruded tunnel liner concept. A number of cement companies have manufactured reg-sets in the past with variable set times and workability ranges. Initial studies conducted at the University of Illinois (7,9) studied the behavior of two different reg-sets. One cement was manufactured by the General Portland Cement Company and the other by the Huron Cement Company. These cements were found to have handling times from 8 to 35 min with no retarding admixtures and $60^{\circ}F$ water temperatures. General Portland reg-set could attain handling times of about 25 min with small amounts of retarding citric acid, and $70^{\circ}F$ mix water temperatures. Retardation of the reg-set from the Huron Cement Company was not quite as effective requiring larger amounts of citric acid and cooler water temperatures ($50^{\circ}F$) for similar workability. Concretes made with both types of cements gained compressive strengths of 1000 $lb/in.^2$ in about 1-1/2 hr.

More recent research at the University of Illinois (10,11) has indicated that extremely rapid set times and short workability ranges characterize the most recent burn of reg-set from Huron. Water temperatures of $32^{\circ}F$ and citric acid contents of 0.3 to 0.5 percent by weight of cement gave erratic strength results and average set times about half that of the previous investigation. The latest study (12) indicated, conclusively, that currently available reg-set materials set too rapidly for handling and batching in large quantities.

Reg-set is no longer manufactured in the United States and the only stockpile of available material, which was manufactured 5 years ago, exists at the Huron Cement Company. Since this is the same material that was recently tested at the University of Illinois, further investigation for its use in conjunction with the ETLS was unwarranted.

Many cement companies have the facilities to manufacture reg-set to a prescribed set of specifications. A normal cement production run takes place over a period of days. With production quantities going as high as 700 tons/day, this creates large amounts of material. A tunnel lining 6 in. thick at 20 ft in diameter would require about 2000 tons/mi of cement for a seven bag mix. Clearly, the use of reg-set cement for tunnel lining would provide a market that would encourage its manufacture.

The lack of availability of new reg-set cements and the conclusions drawn by recent investigators required a search for an alternative cement that would meet the desired criteria. Cements available on the market set either too slowly (normal setting) or too rapidly (rapid setting) to meet the ETLS requirements and consequently, require acceleration, or retardation. Acceleration or retardation can be accomplished by using different admixtures, specific curing methods, and various mix water temperatures. In most cases quantitative data for workability ranges and strength-time data are not available. The final analysis for determining the adequacy of a particular cement required laboratory investigation.

3.2.3 Portland Type III

High early strength portland cement can be accelerated with the use of calcium chloride or similar salts. The increase in strength as a result of the addition of calcium chloride depends on the temperature of the concrete. An addition of 2 percent salt to a mix at 55°F will increase the 1 day strength about 140 percent, but the same addition to a mix at 120°F will increase the 1 day strength about 50 percent. The effect of adding 1 percent calcium chloride has the same effect of raising the mix temperature by 11 deg (13). Thus if the mix is already fairly warm, the salt cannot be very effective. The best results occur when 2 percent calcium chloride is added to rich mixes at low water-cement ratio. Additions in excess of 2 percent may cause flash setting.

A combination of high cement content, very low water-cement ratio and a superplasticizing admixture produces a concrete that is very workable for a limited time period. This period is followed by a sudden loss of slump or stiffening of the concrete. The resulting stiffness is dependent on the particular cement used, but in general, cements with high C_3A content stiffen more rapidly (14). Values of unconfined compressive strength for stiffened no-slump mixes have been found (15) to be about 6 to 10 lb/in.^{2}. This may be adequate for self-support. With the addition of heat by steam curing or form heating, strength development may be sufficient.

A combination of Type III cement, superplasticizer and calcium chloride may produce a useful concrete. A workable mix, formulated at low water-cement ratio with the aid of a superplasticizer, may be adequately accelerated with calcium chloride. Laboratory testing would be necessary to evaluate any such combinations.

3.2.4 Aluminous Cements

Aluminous or high alumina cement is a combination of alumina and lime, usually in equal quantities. This material sets no more rapidly than conventional portland cements, but once set hardens so rapidly that it can gain up to 90 percent of its ultimate strength in 24 hr (16). The cost of aluminous cement is about three times that of portland Type I. Lithium carbonate may be used as a set accelerator and will accelerate early strength gain. An addition of a superplasticizing admixture may be necessary to maintain workability. Aluminous cements tend to convert to a weaker structure with some loss of ultimate strength when subjected to increased temperatures. Under the most favorable conditions, as might be expected in tunnels, conversion will still take place but at a very slow rate. Nonetheless, strength losses of about one-third the 3 to 12 month strength may be encountered (16). Manufactureres recommend that the concrete be designed for the converted strength. They recommend a seven to eight bag mix at a water-cement ratio no greater than 0.40 to produce a final converted strength of about 5000 $lb/in.^2$. On the positive side, aluminous cements have excellent resistance to sulfate attack. In addition, in the presence of sulfate waters, the 20 year loss in strength due to conversion may not be significant and limited to about 20 percent of its maximum strength (16). Care must be taken to keep the concrete cool in the initial 24-hr period after placement so that the heat of its own hydration will not cause immediate conversion and loss of strength.

3.2.5 Aluminous - Portland Type I Combinations

A combination of aluminous and portland cements produces a mixture that is more rapid-setting than either of the two cements independently. For various portland and aluminous combinations in a range from about 20 percent aluminous and 80 percent portland to 80 percent aluminous and 20 percent portland the set times are accelerated and the strength gain is more rapid. It should be noted that in mixtures high in portland cement the setting time is quite rapid but final strengths are low. For aluminous rich combinations the reverse is true. Figure 18 shows setting times of mixtures of portland and aluminous cement and Figure 19 shows the development of strength (17). For the specific combination of 60 percent portland and 40 percent aluminous, there is a rapid strength gain in the first few hours followed by small increases at advanced ages. At 28 days the strength is about 40 percent of the pure portland mix and 20 percent of the pure aluminous mix. In mixes with a larger portion of aluminous material final strength is increased somewhat, but set time is delayed to at least 1 hr. Therefore, if high early strength is necessary, it can be developed only at the direct expense of final strength. A combination of cements and accelerating admixture may overcome these problems although this has not been documented. A trial and error batching method with specific brands of cement would be necessary to assess the usefulness of aluminous-portland combinations.

3.2.6 Specialized Highway Patching Materials

These materials are primarily used for emergency patching on concrete roadways. Many have set times that are less than 15 minutes. This fast setting characteristic is usually of little consequence since they are almost always handled in small volumes. Independent data on strength development and durability behavior is not readily available although some studies (18,19) comparing their relative effectiveness to conventional products have been completed. Because specialized patching cements are manufactured in limited quantities, many are prohibitively expensive for use in large construction operations.

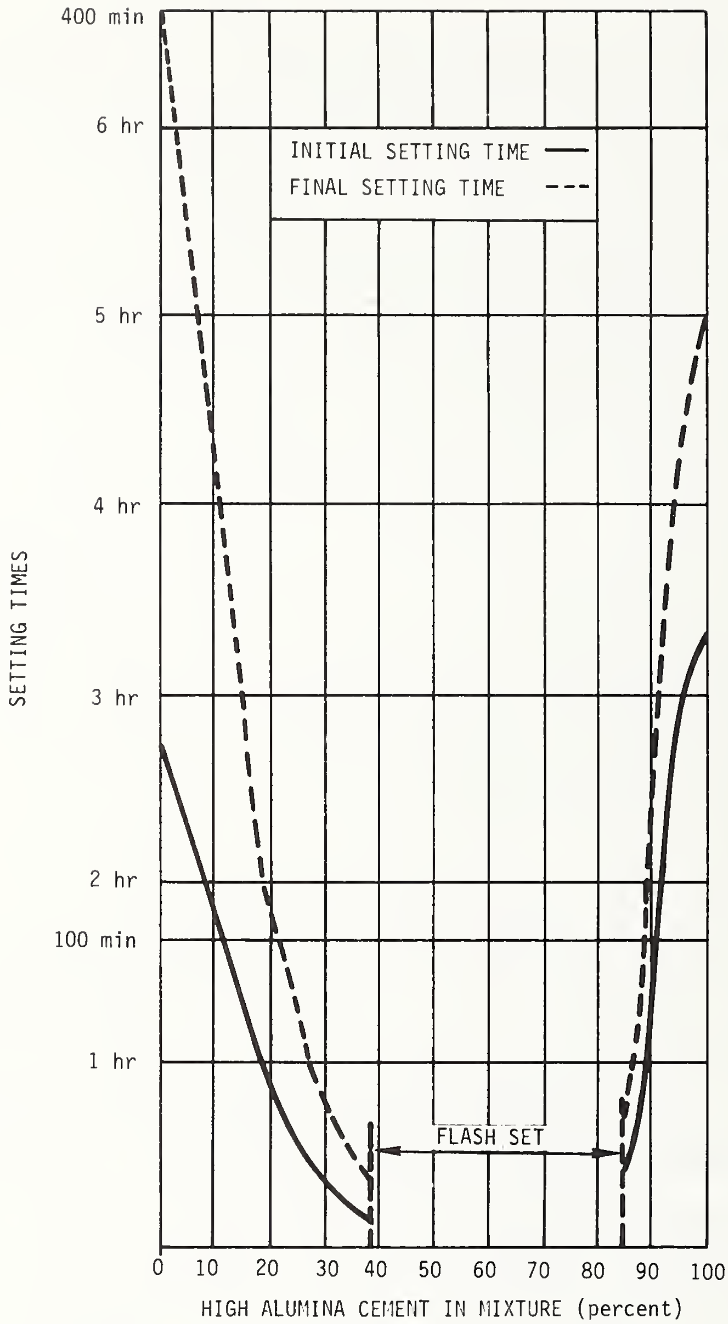


FIGURE 18. SETTING TIMES OF MIXTURES OF PORTLAND CEMENT AND HIGH-ALUMINA CEMENT (17)

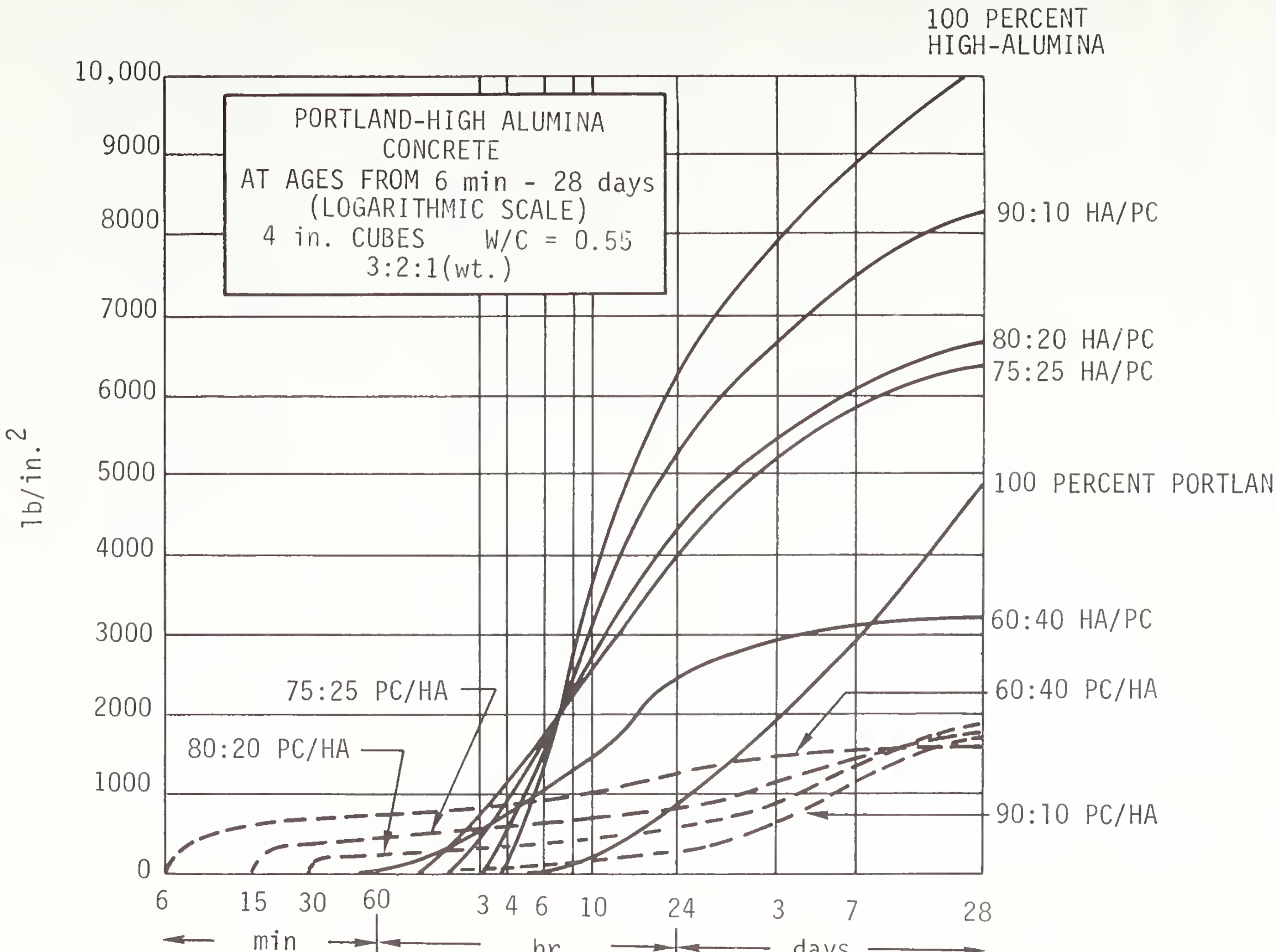


FIGURE 19. STRENGTH-TIME RELATIONSHIP OF PORTLAND/ALUMINOUS MIXTURES (17)

3.2.7 Duracal

This material is a product from United States Gypsum. Duracal is a combination of portland Type V cement and gypsum. Work at the University of Illinois (10,11) has indicated that problems, similar to those encountered with reg-set, are apparent relative to set time and workability. Citric acid was used as a retarder but adequate mix designs for large scale use were never attained. Unsatisfactory durability behavior under water (18,19) has made Duracal a doubtful cement for use in underground structures.

3.2.8 Very High Early Cement (VHEC)

A product of United States Gypsum, this cement is a quick-setting, rapid-hardening material that can be retarded to some degree with citric acid and cooler water temperatures. Set times without additives and with $70^{\circ}F$ mix water is about 15 min. Compressive strength values of 3000 $lb/in.^2$ can be obtained in about 3 to 5 hr. The manufacturer has indicated that VHEC can be adequately retarded using citric acid, for placement by pumping, although retardation will hinder strength development. A series of laboratory tests, discussed in Section 7, were conducted to provide sufficient information on the workability and early strength gain characteristics of VHEC and to assess its suitability for use with the ETLS.

3.2.9 German and Japanese Reg-Sets

Both the Heidelberg Cement Company of Germany and the Onada Cement Company of Japan are presently manufacturing reg-set in their native countries. The lack of literature available in this country has made it difficult to secure information regarding the set characteristics and strength development for these products.

No data has been obtained although efforts are being made to acquire such information. It is possible that these cements may exhibit the desired qualities. The cost of importation may preclude their use.

3.3 ADMIXTURES, AGGREGATES AND WATER

3.3.1 Admixtures

A variety of admixtures can be used to alter the properties of the concrete both in the plastic and hardened states. The materials presented in this subsection are limited to those admixtures referred to in the discussion of cements, that is, those that either control the setting characteristics of a particular cement or increase the workability of the plastic concrete.

Citric acid can be used as an effective retarder for reg-set cements, Very High Early Cement, and Duracal. It has been used in an amount equal to about 0.3 to 0.5 percent by weight by cement for reg-sets with varying results on controlling the set time (9,10,12). The actual value of retardation is dependent on cement type, cement fineness and water temperature. Citric acid additions in excess of 0.5 percent will further delay set, however, at a direct reduction in gain of early strength.

Calcium chloride can be used to accelerate portland cements. This additive is most effective in cold casting temperatures, and in terms of effectiveness, amounts to the equivalent of an 11°F rise in mix temperature per 1 percent addition by weight of cement (13). The limiting maximum value is usually 2 percent. Larger dosages can cause flash setting. Usually a 1 percent addition will accelerate set time by about 1-1/2 hr (13). Strength gain of concretes with calcium chloride is dependent on mix temperature but increases as much as 150 percent of one-day strength are possible. Calcium chloride may exhibit some undesirable qualities. It may lead to advanced corrosion of steel reinforcement, lower resistance to sulfate attack, and increase drying shrinkage. The extent of these effects is dependent on the particular mix design.

Lithium carbonate has been recommended by an aluminous cement manufacturer as an effective means of accelerating set for aluminous cements. The quantities of addition are small, generally in a range of 2 to 10 g per sack of cement. This creates a problem of uniform dispersion. The best dispersion can be obtained by adding the chemical to the mix water.

Fly ash can be used either to make up for deficiencies of fines in the sand or as a cement replacement. Being a pozzolan, fly ash does not exhibit cementitious behavior by itself. In the presence of a cement, however, fly ash will combine with the products of hydration and will gain cementitious properties. When added to a mix, fly ash improves workability and pumpability and results in stronger concrete with smaller shrinkages (11). Concretes with fly ash additions tend to gain strength more slowly than conventional mixtures.

Superplasticizers - A number of workability admixtures have been used over past years to either maintain constant workability at reduced water content or increase workability at constant water content. The most recent development in workability agents is the superplasticizer or super water-reducer. Generally, there are two major types of superplasticizing admixtures. They are the sulfonated melamine-formaldehyde condensates and the sulfonated naphthalene-formaldehyde condensates. A variety of admixture manufacturers market one of the two products under various trade names. Both types of superplasticizers have the effect of dispersing the cement particles. The dispersion effect is produced when the admixture is adsorbed on the surface of the cement particles causing them to become mutually repulsive (20). In this manner, superplasticizers, when added to normal concrete, impart extreme workability, and allow large water reductions beyond the range of normal water-reducers (20). When superplasticizers are used to produce high workability, the concrete will take on a flowing behavior, remain cohesive, and not be prone to segregate. This property will be of particular value in aiding the distribution of concrete in the ETLS form.

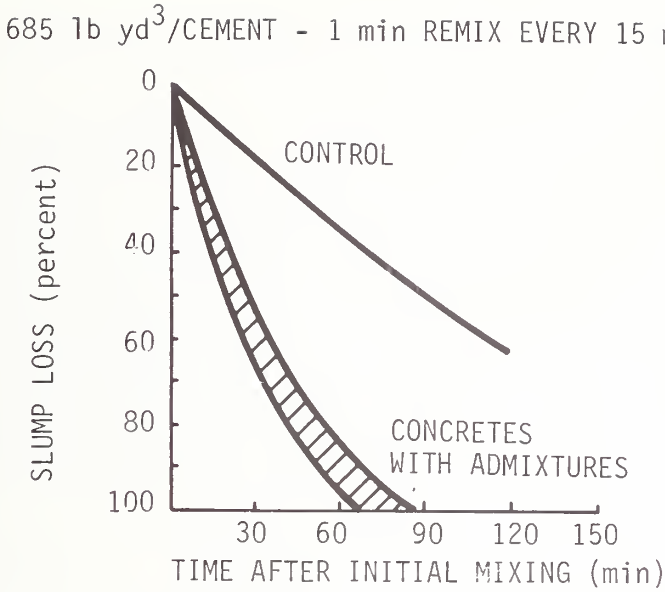
Superplasticizers can be used to formulate very low water-cement ratio concretes with adequate workability for placement. Water reductions of up to 20 to 30 percent can be accomplished with increasing admixture additions, while maintaining normal workability (20,21). Reducing water-cement ratio is preferable to increasing cement content for strength gain. High cement content will result in higher heat of hydration and cause excessive shrinkage. Normally, however, concretes with low water-cement ratio have undesirable workability (for example, zero slump) for many applications. The use of superplasticizers to increase workability eliminates this problem, and makes this approach feasible for the ETLS.

Whether superplasticizers are used as a water-reducer, a workability agent, or a combination of the two, superplasticized concrete may exhibit a rapid loss of workability. For our application, after the concrete has been distributed in the form, any rapid stiffening would be advantageous. The amount of loss of workability, which is usually measured by slump, is dependent on a number of variables. A recent investigation (14) has shown a definite increase in the rate of slump loss with high cement content. Figure 20(a) shows the rate of slump loss for a seven bag concrete mix. In contrast, the results of another study (22) indicate that mixes with higher cement contents retain their workability longer than those of low cement content. This can be seen in Figure 20(b). In addition, increases in ambient casting temperature has been cited by many (20,22,23) as a definite influence in reducing the duration of the fluidifying effect of the superplasticizer. Cements high in C_3A content also tend to lose slump rapidly (14).

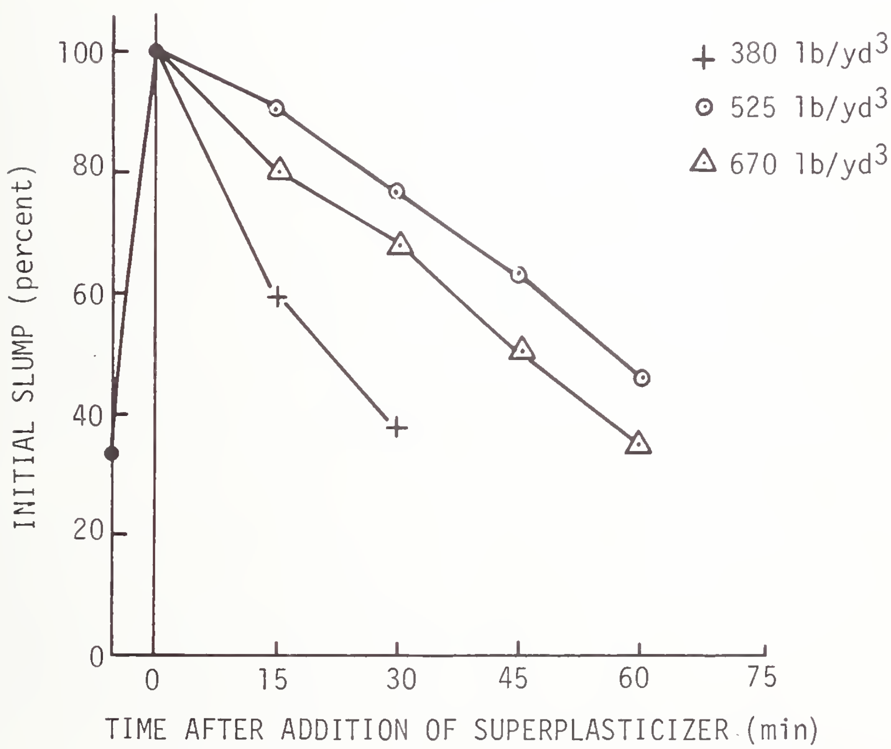
For applications of placement by pumping, superplasticizers appear to be beneficial. Reductions in pumping pressures of 25 to 35 percent over conventional mixes were observed in tests in Japan (23). Similar experiences have been observed in Germany (24) and England (25).

3.3.2 Aggregates

The properties and gradation of the fine and coarse aggregate becomes very important when formulating pumpable concrete. Fine aggregate should conform to ASTM C33 (30) and have a smooth gradation curve without deficiency or suplus of any particular size (26). Material retained on the No. 100 and No. 200 sieves are very important. If deficiencies of these sizes occur, fly ash or other fine materials may be appropriate for addition. Coarse aggregates should preferably be of rounded shape. The gradation of the coarse aggregate is not as critical as that of the fine, but a well-distributed even gradation will result in a dense aggregate mixture. The maximum size of coarse aggregate



a) SLUMP LOSS OF RICH CONCRETE (14)



b) VARIATION OF SLUMP LOSS WITH CEMENT CONTENT (22)

FIGURE 20. EFFECT OF CEMENT CONTENT ON SLUMP LOSS (22) IN SUPERPLASTICIZED CONCRETE

is usually dictated by the size of the pump delivery line. The maximum size should normally not be greater than 40 percent of the line ID for well-rounded aggregates. The quantity of coarse aggregate must be reduced as the maximum size decreases (26). This is a consequence of the shift in proportion of coarse and fine aggregate that produces minimum void. A denser combination of aggregates improves the pumpability. A combination of a particular coarse and fine aggregate will produce the smallest void volume, hence the densest mix. This is a very important parameter in mix design formulation for pumpability and is discussed further in subsection 3.4.

3.3.3 Water

General guidelines concerning water quality for conventional concretes should be followed regardless of the constituents of the mix. Potable water is always usable and some nonpotable supplies may be acceptable if adequate testing so indicates.

The temperature of the concrete mix can effectively be controlled by the water temperature. Increasing the mix temperature reduces the time of set and increases the rate of hardening, although mix temperatures in excess of $90^{\circ}F$ may be difficult to handle and have reduced ultimate strength (27). Cooler mix temperatures improve workability and delay the set.

Depending on the type of cement and admixtures used, the temperature of the mix, controlled by the mix water, can be manipulated to proper advantage. Water is the easiest portion of the mix to heat or cool and is the most effective because of its high specific heat value.

3.4 MIX DESIGN FOR PUMPABILITY

3.4.1 Basic Principles

Concrete that is formulated for pumpability must be well-proportioned, dense and plastic (26). The concrete should be well-mixed before entering the pump and should not be too stiff or too wet. Concrete can be pumped at a wide range of slump, although a slump of about 3 to 5 in. is most common. Because of these requirements, pumpable concrete is inherently of good quality in terms of workability and strength.

The ability of a concrete mix within a pipeline to transfer applied force through the water phase to the remaining particles is an important parameter controlling pumpability. Properly formulated concrete will flow through the pipeline as a slug. Failure can occur if the mix is unable to contain the water or if pipeline friction is too great (28). These two types of failures are referred to as segregation and friction failures, respectively. The ideal solution therefore is to produce maximum density of solids in the mix with minimum voids, and minimal frictional resistance against the pipe wall with a low surface area of the aggregate (29).

Designing a pumpable concrete can be approached systematically if the influence of certain factors such as aggregate shape, maximum size, gradation, cement content and type, and water content are known. If the concrete will include fibrous reinforcement or admixtures, this will change the resulting proportions of the mix design to some degree, however, the actual procedure for developing a pumpable mix will be identical.

3.4.2 Mix Design Method

A method to formulate pumpable mixes has been developed by others (28). The following procedure is a summary of that method.

3.4.2.1 Select Aggregates - The fine aggregate should conform to ASTM C33 (30) and have a smooth gradation curve. Coarse aggregate should preferably be of rounded shape. Guidelines set down by ACI Committee 304 (26) should be followed.

3.4.2.2 Experimentally Determine Voids - Perhaps the most important parameter controlling pumpability is the minimization of the void volume in the aggregate particle array. For a specific combination of aggregates, a minimum void volume can be determined experimentally. The ratio of fine to coarse aggregate that yields a minimum void volume will vary depending on the maximum aggregate size and the fiber volume, if fibers are specified.

3.4.2.3 Adjust Aggregate Proportions - In the final concrete mix, the particles will be separated by the paste. Thus, the densest concrete will be obtained when the coarse aggregate content is increased beyond that which produces minimum voids. This amount should equal about 2 to 5 percent of the total aggregate portion (31).

3.4.2.4 Determine Volume of Paste Required - Once the combination of constituents that provides minimum voids has been experimentally obtained, the volume of paste, consisting of cement, water, and pozzolan or other fine material if used, must exceed the void volume by a slight amount. The excess is required due to separation of the particles by the paste and for coating of all of the particles. The amount of the excess is dependent upon the properties aggregates and the fiber used. Generally, the excess can be as low as 3 to 5 percent, but as high as 9 percent of the total volume of the mix, for some fiber aggregate combinations. Air content is not considered as part of the paste for these values, due to reduction to insignificant volume under pumping pressures.

3.4.2.5 Select Water Content - For a specified volume of paste, the amount of water necessary will be dictated by workability requirements. The amount of water is dependent on the type of cement used, water temperature, total aggregate gradation, and fiber content. Generally, finer cements and lower fineness modules of the combined aggregates will increase water demand. Specific values are dependent on particular aggregates. The inclusion of fibers will increase water requirements substantially.

3.4.2.6 Determine Cement Content - To determine the amount of cement required, the value of a water-cement ratio should be selected to provide for adequate strength and durability. The value of cement dictated by the water-cement ratio may not, in some cases, provide adequate total paste volume. It would then be necessary to add more cement, reducing the water-cement ratio or to hold the water-cement ratio constant and add a pozzolan or finely powdered material to provide the necessary increase in paste volume.

3.4.2.7 Trial Batch - Trial batching of any mix combination will be necessary to verify proper consistency and workability. If adjustments are required it should only be necessary to make minor changes in the mix design. Mixes without fibers will exhibit a much wider range of pumpable mixes, with a given set of constituents. Nonetheless, proper evaluation of mix design parameters is necessary to formulate a pumpable mix.

3.4.2.8 Addition of Superplasticizers - For applications in which improved workability or substantial water-reduction is desired, such as for the ETLS, superplasticizers may be a useful addition to a mix. This step should generally be included after an initial trial batch. If reductions in water content are desired, the remaining volume should be filled by relative proportions of all the other constituents.

3.4.3 Pumpable Mix Characteristics

In general, pumpable mixes will contain more fine material than conventional mixes. This in turn will require more paste volume to coat all of the particles. Pumpable mixes by nature of their careful formulation, are not prone to segregation and are fairly easily remolded.

If a mix design requires fibrous reinforcement, large changes in proportions usually result. This is mainly due to increased void volume. In addition, the proportion of fine aggregate at minimum voids is increased for fibrous mixes. High void volume and larger amounts of fine aggregate require much more paste than conventional, plain mixes. Fibrous mixes are harsh and are not as easily remolded as plain mixes. Much more care is required for formulating fibrous mixes and the range of pumpable fibrous mixes is much more limited.

3.5 FIBER REINFORCEMENT

3.5.1 Background

The inclusion of fibers in tunnel lining concrete was first proposed by Parker (7). The purpose of fibers, as originally intended, was to eliminate the need for conventional reinforcement which would have been virtually impossible to place in a continuous slipform system. Research (32) conducted since the first developments, has confirmed definite post-cracking behavior of ductility and toughness for fibrous mixes but has also determined that pure strength in compression and flexure is not significantly increased.

In a redundant liner structure, behavior at failure for plain and fibrous concrete is similar. Both materials develop a singular large tension crack which upon further propagation, continually diminishes the size of the compression zone causing eventual crushing failure. By nature of the redundancy of the structure, redistribution of movement occurs prior to failure allowing adjacent sections to share moment. Redistribution will occur with or without fibrous reinforcement.

The addition of fibers into conventional concrete significantly alters both the plastic and hardened properties. Direct effects in plastic properties include:

a. Complication of mix design

b. Reduced workability

c. Difficulty in remolding and in consolidation

d. A necessity for more precise batching to ensure pumpability.

Fibrous mixes require more paste and a change in total aggregate gradation. Compressive and flexual strength of fibrous mixes are not significantly greater than plain concretes. Increases in strength are much more sensibly gained by additions in cement content. Fibrous concretes exhibit definite post-cracking strength, the magnitude of which is dependent on fiber content, geometry, and composition. Toughness may be increased at least an order of magnitude over that of plain mixes.

These properties are discussed in further detail in the following subsections.

3.5.2 Mixing and Placement

Mixing and placement of fiber reinforced concrete can be markedly different than plain concretes. Fibrous mixes are harsh and require a larger amount of paste to coat all of the particles. The addition of fibers causes a sharp reduction in workability. For example, if a typical plain concrete mix is formulated to a slump of about 8 in., the addition of about 1 percent fibers (by volume) will reduce the slump to about 4 in.

Handling fibers is difficult and can cause injury if safety precautions are not taken. Hand and eye protection should be worn at all times when handling fibers.

Formulation of a mix design is much more complex if fibers are included. Generally, fibrous mixes will contain higher amounts of fine aggregate and increased cement content.

If fibrous concrete is to be pumped, much more care is required to develop a pumpable mix. As was discussed, proper attention to variables such as fiber content, aggregate size, shape, and gradation, void volume and cement content is necessary to formulate a pumpable mix. Mixes that are pumpable without fibers may not be pumpable at all if only a small volume of fibers are added to the mix.

Batching mixes with fibers can be much more difficult than conventional mixes. Fibers can be added to the mix after the addition of all of the dry constituents, during the water addition, or after all the water has been added. In all cases, fibers should be added with the use of a vibrating screen so as to disperse the individual fibers as much as possible. If adequate dispersion is not attained, the fibers will tend to cling together and ball-up. If this occurs, the full benefit of the fibers will not be realized and the mix may not be pumpable.

Inadequate methods of addition of fibers are not the only reason that balling occurs. The tendency to ball-up is very much dependent on the aggregate shape and gradation. Mixes high in coarse aggregate content will be subject to extensive balling. Crushed aggregates increase the likelihood of balling. In addition, if the maximum aggregate size approaches the size of the fibers, problems may occur. Laboratory tests with 1 in. maximum size aggregate and 1 in. fibers showed increased incidence of balling although the aggregate was rounded in shape and was well-graded (28). Generally, balling can be minimized with careful aggregate gradation and proper batching sequences.

Compaction of the plastic fiber reinforced concrete can be difficult because of the resistance of the mix to remolding. Improper compaction can lead to an undesirable hardened matrix. Much more vibration energy and manpower is required to handle fibrous concrete than for conventional concrete.

3.5.3 Strength of Fiber Reinforced Concrete

The compressive and flexural strength of fiber reinforced concrete has been found to be slightly greater than that of plain concretes. In a recent study (32) a comparison between mixes with various fiber types and plain mixes was made. The following discussion summarizes these results.

All of the fibrous mixes had the same mix proportions. The plain mixes were formulated to be as similar to the fiber mixes as possible. One mix (P1) was based on the same water/cement ratio as the fibrous mixes, (with a lower cement content) and the other (P2) was proportioned exactly like the fibrous mixes, but without the fiber.

Figure 21 compares the compressive strengths of the various fiber type mixes, labeled F1 to F6, for 0.9 percent fibers and 1.5 percent fibers. Figure 22 compares the flexural strengths for 3 by 3 by 15 in. specimens and Figure 23 compares flexural strengths for larger 4 by 6 by 64 in. specimens for these mixes. Although the author cautions against the use of the data to rank specific fiber types on the basis of strength, some interesting conclusions may be drawn.

For the compressive strengths (Figure 21), the values are all quite similar and in the range of 7000 to 8000 lb/in.^{2}. The increased values at higher fiber content can be attributed to higher cement content. Flexural results (Figure 22) indicate that in general, fibrous mixes are slightly stronger than plain mixes and that the bent end type, F6, is substantially stronger. The results of the large specimen flexural tests (Figure 23) indicate that again, fibrous mixes attained higher strength than plain mixes, however all of the values are substantially lower. This makes the relative difference between mixes slightly smaller. The author cites several reasons for the strength reduction due to size effects.

Thus, if strength were to be the main criteria for design, the addition of fiber would not be the best solution. Improvements in strength could be much more economically made up with addition of cement.

3.5.4 Post-Cracking Strength of Fiber Reinforced Concrete

Plain concrete is a brittle material and is not able to offer sizeable resistance to flexural loading. A common tool to compare the flexural behavior of plain and fiber reinforced concretes is the flexural load-deflection curve. Figure 24 represents typical curves for a plain concrete and a fiber reinforced concrete. Note

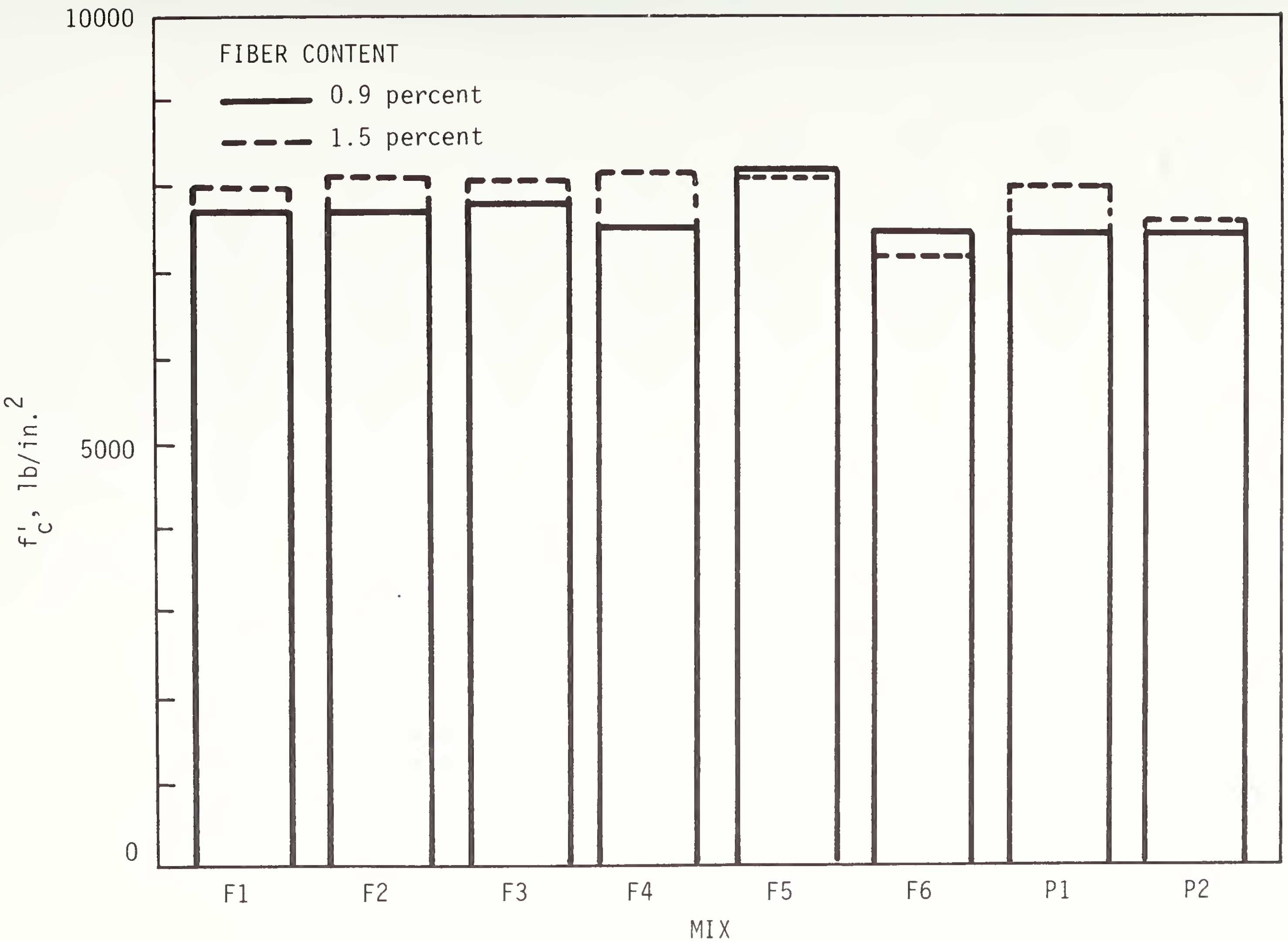


FIGURE 21. COMPRESSIVE STRENGTH FOR CONTROL SPECIMENS (32)

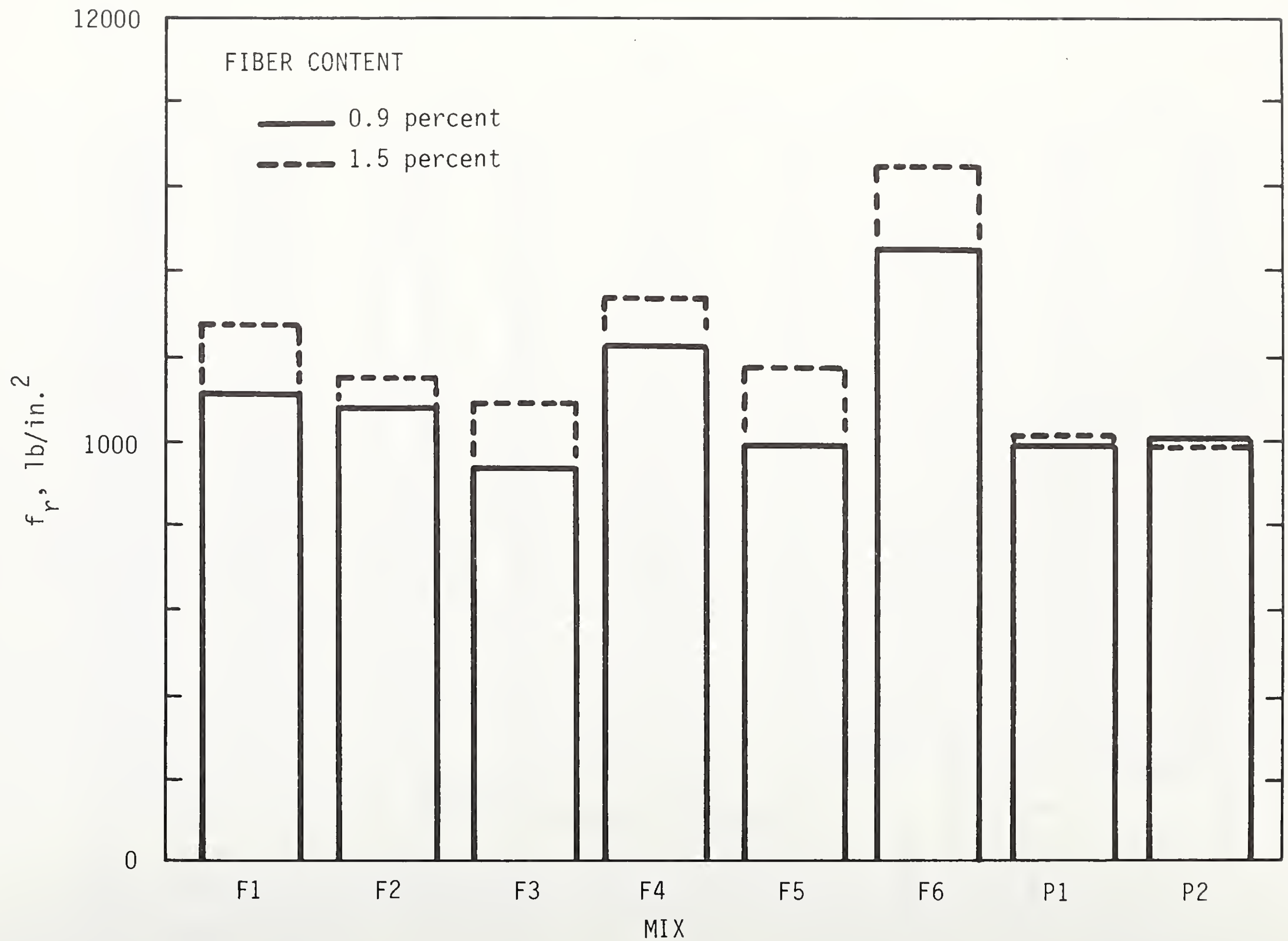


FIGURE 22. MODULUS OF RUPTURE FOR CONTROL SPECIMENS (3 BY 3 BY 15 IN.) (32)

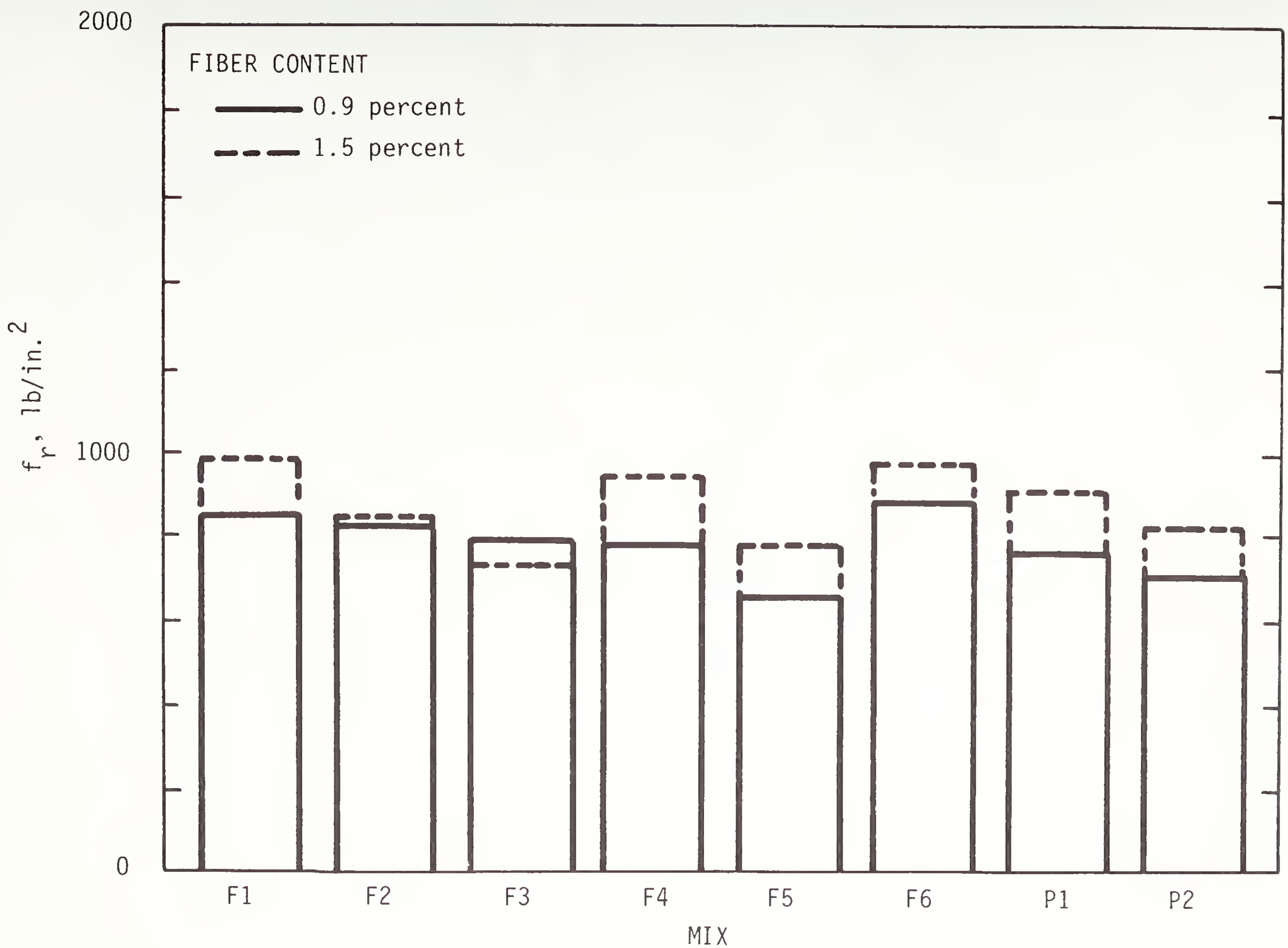


FIGURE 23. MODULUS OF RUPTURE FOR LARGE BEAMS (4 BY 6 BY 64 IN.) (32)

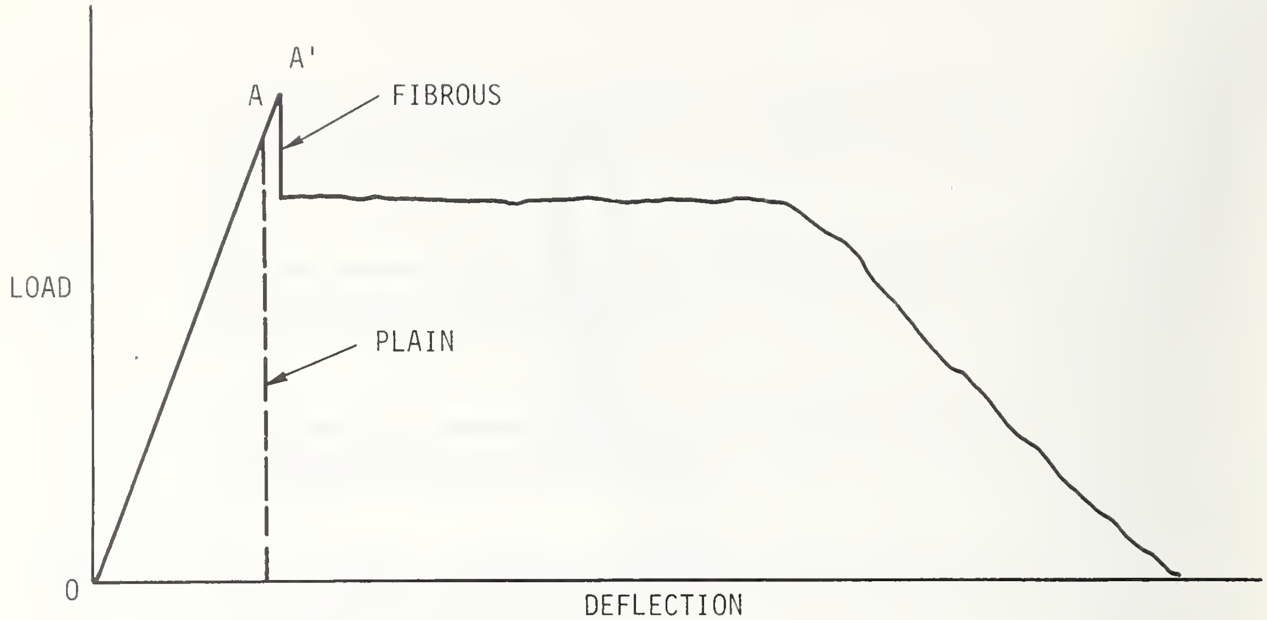


FIGURE 24. LOAD-DEFLECTION CURVE IN FLEXURE FOR PLAIN AND FIBROUS CONCRETE

the linear elastic region up to a peak, known as the cracking load (A). The cracking load for fibrous concrete (A') is slightly higher than that for the plain mix due to increased flexural strength. This peak is followed by a sudden load reduction. For plain concrete this reduction is a complete loss of strength and the specimen is considered to have failed. Fibrous concrete will also exhibit a load reduction but maintains a residual capacity at increased deflection. Depending on the exact fiber geometry and content, the flat region may continue for three to four times the cracking deflection. For fibers with bent ends, the load drop is similar, but in the region beyond cracking an increase in load is evident. This post-cracking load is higher than the initial peak. For this particular fiber type, a constant load range or plateau was observed for deflections five to seven times the deflection at cracking (32). As the fibers gradually pull out of the matrix, the load carrying capacity is slowly reduced and the curve returns to zero. For increased fiber anchorage qualities, the descending branch will decay much more slowly. Clearly this can be assessed as a ductile mode of failure.

Toughness is defined as the energy absorbing quality of the concrete and is graphically represented by the area under the flexural load-deflection curve. For plain concrete, the toughness would be equal to the triangular area OAB in Figure 24. For fibrous mixes, the additional toughness corresponds to the remaining area. This additional energy absorptive capacity is developed in debonding and stretching of the fibers (33). The effect of fiber content on flexure and toughness is shown in Figure 25. Toughness increases rapidly with higher fiber content but flexural strength is increased only slightly. The presence of fibers is much more important in terms of post-cracking behavior than in strength alone.

3.6 SUMMARY

Cements suitable for use in the ETLS should produce a concrete that:

a. Has a set time of 20 to 40 min with adequate workability prior to initial set.

b. Attains 500 $lb/in.^2$ compressive strength in about 2 hr.

c. Will have a 28 day ultimate strength of 5000 $lb/in.^2$.

d. Will exhibit desirable properties of durability and minimal shrinkage.

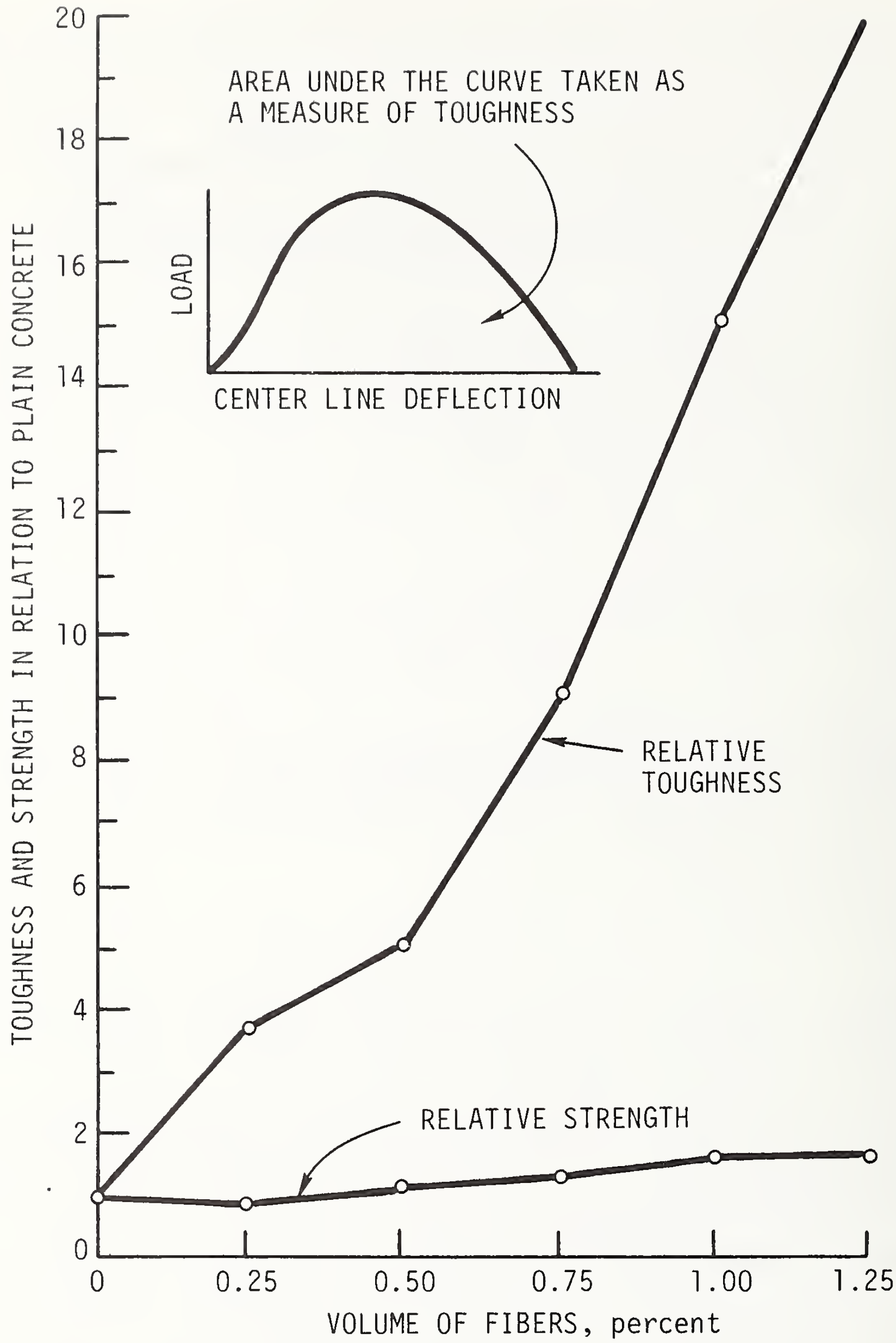


FIGURE 25. EFFECT OF VOLUME OF FIBERS ON FLEXURAL TOUGHNESS AND STRENGTH (33)

The cement will also have a minor effect on the workability and pumpability characteristics of the resulting concrete. Consequently, a cement should be chosen that will not substantially alter these plastic properties.

Materials that may exhibit these qualities are:

a. Aluminous cements with accelerating admixture of lithium carbonate and a superplasticizer

b. Aluminous-portland combinations

c. Type III cement with accelerators, superplasticizers and possible heat curing

d. VHEC sufficiently retarded with citric acid and cold water

e. Foreign reg-sets.

The cement requirements for the ETLS are unique, and consequently, as seen in Table 1, the data available on which to base a selection was sketchy. Therefore, a testing program to evaluate these alternatives and select a final material was required. The test program and results are discussed in detail in Section 7.

4. TUNNEL SUPPORT AND LINER DESIGN

4.1 INTRODUCTION

The ETLS will cast a continuous concrete tunnel liner closely behind a TBM. This tunnel liner will take the place of the two stages of ground support (primary and secondary) typically provided in rock tunnels. It should therefore meet the strength requirements normally imposed on both the primary and secondary tunnel supports. This section presents a development of the thickness, strength, and reinforcement requirements for the extruded liner based on its unique characteristics.

4.1.1 Liner Design - State-of-the-Art

A wide variety of rock tunnel support designs exist, ranging from the thin (2 to 6 in.) shotcrete linings used in Europe and Sweden (34,35) to conventionally reinforced CIP concrete liners with average thickness in excess of 18 in. in the United States (36). Precast concrete segmented liners fall between these two extremes with thicknesses ranging from 7 to 14 in., both plain and conventionally reinforced, for tunnels approximately 20 ft in diameter. The fact that each of these support methods have proved satisfactory leads one to question their apparent inconsistency. The following reasons, most of which are not related to rock support requirements, are often given to explain these differences.

a. Concrete placement - Space constraints often dictate liner thickness.

b. Segment handling - Reinforcement is often used to permit segment handling and to react TBM thrust.

c. Contracting practice - European turnkey operation presents less risk and encourages trial of new techniques.

d. General conservatism on transit work - Safety of general public and prevention of settlement in urban areas.

Perhaps the major reason for this variation is the manner in which the ground-liner interaction is taken into account. In a recent article, (37) Brierley discussed the effect of not considering this interaction on a 70 ft diam semicircular arch such as would be found in a transit station. When subjected to a uniform load equivalent to 30 ft of rock, the following liner thrusts, moments and deflections resulted.

	Max Thrust (kips)	Max Moment (in.-kips)	Max Deflection (in.)
a. Free Standing Lining	785	17,850	7.21
b. Liner with Ground Interaction	740	190	0.16

These results indicate that variations in the method used to account for the ground-liner interaction can cause significant differences in the moment and deflection for which the liner is designed. This could lead to concern over unacceptable settlement and to specification of unnecessary reinforcement. The tunnel liner has generally been designed as a relatively rigid structure with an assumed vertical and horizontal load. The horizontal load was selected as some fraction of the vertical load in an attempt to account for some interaction between the ground and the liner. This approach does not fully account for the ground-liner interaction and has lead to overly conservative designs. Except in cases where it has lead to specification of reinforcement in the liner, this conservatism has not caused excessive cost penalties with conventional CIP concrete liners since current concrete placement techniques usually require liners to be even thicker than the conservative design approach. For example, a 20 ft diam tunnel would require approximately a 12 in. minimum liner thickness based on placement considerations.

4.1.2 Considerations for the Extruded Tunnel Liner

The ETLS will not be limited by the concrete placement techniques currently used for CIP tunnel liners. This is due primarily to the shortness of the form, and the absence of large primary support elements. Therefore, it will be capable of casting liners which are significantly thinner than those currently specified for tunnel support. In order to take advantage of this capability, a more thorough analysis, considering not only the tunnel geometry and loading, but also the material properties of the ground mass and its interaction with the liner, was used (34). The analysis techniques which we use have been developed through research over the past several years, and are currently finding acceptance in the tunneling industry. As an example, this type of analysis has been used to evaluate linings for the Washington Metro stations (38).

In the following subsections, the rock loads to which the ETLS liner will be subjected are estimated and the liner behavior under these loads is discussed. The moments and thrusts resulting from different loading conditions are compared to the capacity of several plain and fiber reinforced concrete liner sections. Based on this comparison, the recommended thickness, strength and material for the ETLS liner were selected.

4.2 DESIGN ROCK LOADS

The loads to which tunnel supports in rock tunnels are subjected have been termed rock loads (35). The term "rock load" indicates the height of the mass of rock which tends to loosen and drop out of the roof. If no support were applied, this mass of rock would drop into the tunnel in increments and the tunnel would gradually assume the character of an irregular vault. The rock load depends on accidental details such as the spacing and orientation of the joints, and will thus change from point to point. The extent to which the rock is disturbed by the excavation process or the degree to which loosening is allowed to proceed before installation of supports will also cause the rock load to vary.

In order to determine the liner dimensions and strength development characteristics of the concrete specified for the ETLS, the maximum uniform, concentrated and initial rock loads have been estimated as discussed below. These loads were used to evaluate the required thickness and strength of the liner.

4.2.1 Maximum Uniform Rock Load

Figure 26 represents the maximum loading to which the lining could be subjected. A rock load of height H is assumed to act as a uniform vertical pressure over the full width of the tunnel. The method proposed by Terzaghi (35) is generally used to determine H, the rock load height, once the rock characteristics are known. The fact that the ETLS will be used only with a TBM limits the range of rock characteristics that must be considered.

An evaluation of TBM applications indicates that the rock mass must have a significant (greater than 30 min) stand-up time for a TBM to be effective. This stand-up time is necessary to

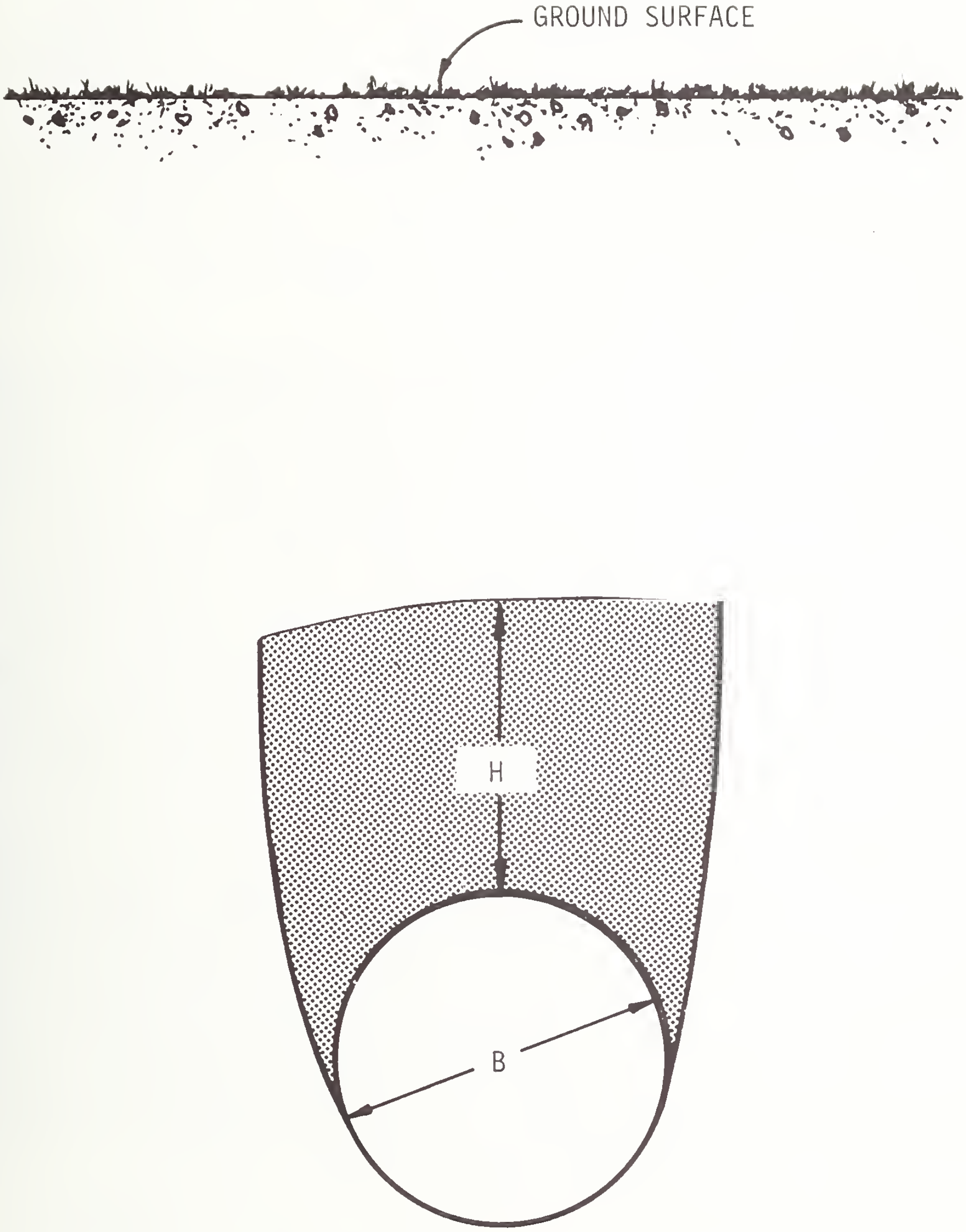


FIGURE 26. UNIFORM ROCK LOAD

prevent the cutters from becoming jammed and to allow adequate time for support installation. This limit on stand-up time places a lower limit on the rock quality that must be considered when establishing the design rock load. The information necessary to correlate stand-up time and rock quality designation (RQD) is presented in Table 2 and Figure 27. The 20 ft diam tunnel of interest with a stand-up time of at least 30 min falls in Lauffer's Rock Class D (Table 2), which places a lower limit of approximately 50 on the RQD to be considered.

In addition to limiting the range of rock quality that must be considered, the use of a TBM allows the design rock loads to be reduced below those estimated by Terzaghi (39). This reduction is due to the fact that the TBM disturbs the rock mass to a much lesser degree than conventional drill and blast techniques on which Terzaghi's estimates are based. The rock load factors recommended by Deere (40) for the TBM tunneling method are compared with Terzaghi's in Figure 28. With an RQD of 50, the maximum rock load that need be designed for is H = 1.0B, where B is the diameter of the tunnel in feet.

The rock loads estimated by Terzaghi and Deere are intended for use in designing steel sets and, as such, add a degree of conservatism when used in conjunction with the ETLS. This conservatism results from the fact that the ETLS will prevent the loosening which occurs after the installation of steel sets as a result of the crushing or deterioration of the wood lagging. Less loosening implies a lighter rock load.

4.2.2 Concentrated Rock Loads

The liner must also be evaluated under concentrated rock loads such as would result from the loosened rock wedges shown in Figure 29. The rock loads resulting from these wedges are

TABLE 2. ROCK QUALITY DESIGNATIONS

FRACTURE SPACING				TERZAGHI				LAUFFER
CM	IN.	RQD (%)	ROCK DESCRIPTION	ROCK LOAD, H INITIAL	ROCK LOAD, H FINAL	REMARKS		ROCK DESCRIPTION
100			1 Hard and intact	0	0	Generally No Side Pressure	Lining only if spalling or popping	A Stable
50	2	98	2 Hard strat-ified or schis-tose	0	0.25 B	Generally No Side Pressure; Erratic Load Changes from Point to Point	Spalling Common	
	1	95, 90	3 Massive moderately jointed	0	0.5 B	Generally No Side Pressure; Erratic Load Changes from Point to Point	Side pressure if strata inclined some spalling	B Unstable after long time
20	6	75	4 Moderately blocky and seamy	0	0.25 B to 0.35 B	Generally No Side Pressure; Erratic Load Changes from Point to Point		C Unstable after short time
10		50	5 Very blocky seamy and shattered	0 to 0.6 B	0.35 B to 1.1 B	Little or no side pressure		D Broken
5	2	25, 10	6 Completely crushed		1.1 B	Considerable side pressure. If seepage continuous support.		E Very Broken
		2	7 Gravel and Sand	0.54 B to 1.2 B	0.62 B to 1.38 B	Dense		
2	1			0.94 B to 1.2 B	1.08 B to 1.38 B	Loose		

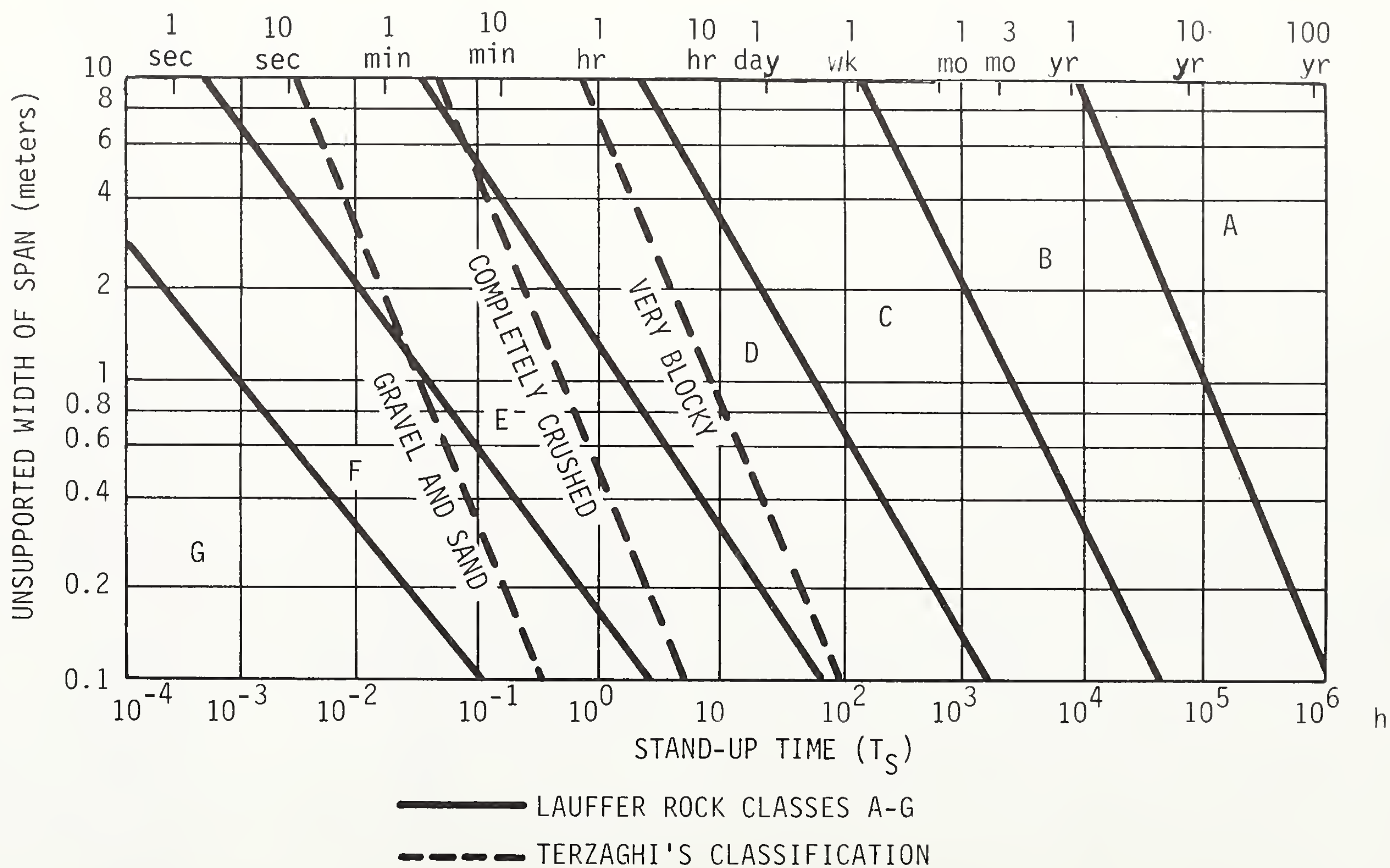


FIGURE 27. STAND-UP TIME AS A FUNCTION OF ROCK CLASS AND UNSUPPORTED WIDTH OF TUNNEL ROOF (40)

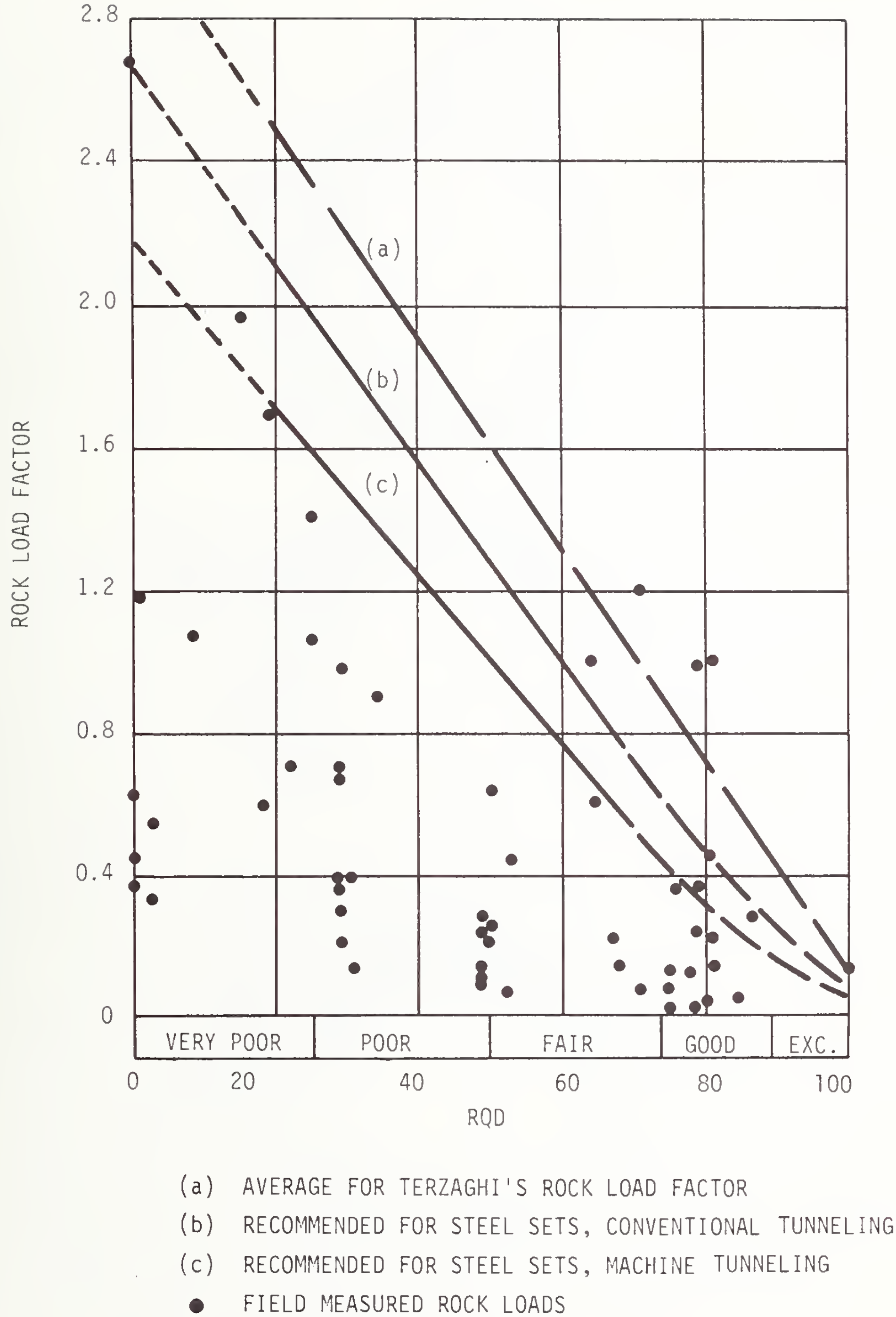


FIGURE 28. RELATIONSHIP OF ROCK-LOAD FACTORS AND RQD (40)

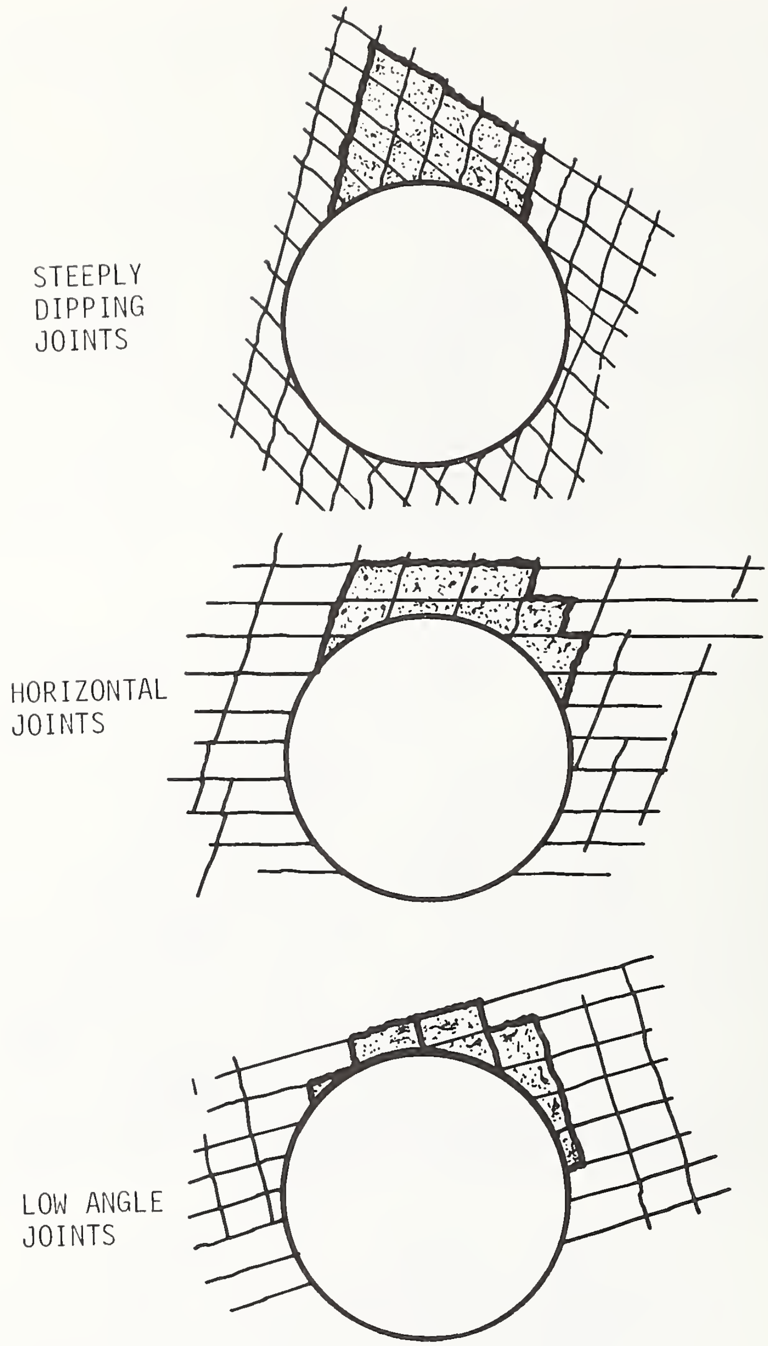


FIGURE 29. TYPICAL CONCENTRATED ROCK LOADS

the loads that can realistically be expected to act on the liner. If loosening of the rock were allowed to continue, these wedges would grow and the maximum uniform rock loads described in the preceding section would result.

The size, shape and orientation of these wedges depends on characteristics of the local geology such as fracture spacing and joint orientation with respect to the tunnel. Several different wedges based on the most unfavorable geologic conditions should be analyzed to ensure acceptability of the tunnel liner. The rock loads due to these wedges should be in the range of 0.15B to 0.6B (41).

4.2.3 Initial Rock Loads

In addition to developing the final strength required to support the long-term loads described above, the liner must gain strength rapidly enough to support any initial load imposed on it as it leaves the slipform. Terzaghi's estimate of these initial loads is presented in Table 2. It can be seen that these loads are zero in all but the worst rock conditions in which the ETLS will be employed. Terzaghi's estimates are based on tunnels constructed by the drill and blast method and are thus conservative for a bored tunnel, since the TBM does not significantly disturb the rock mass. Thus, for most applications, the liner need only be self-supporting as it leaves the slipform.

In the poorer quality rock, rock bolts could be used to support the roof until the liner is cast and thus will minimize the initial load on the liner. Experience with shotcrete (42) indicates that even the relatively low shear strength of the liner as it exits the slipform will be sufficient to prevent loosening of any small rock wedges that are not supported by rock bolts.

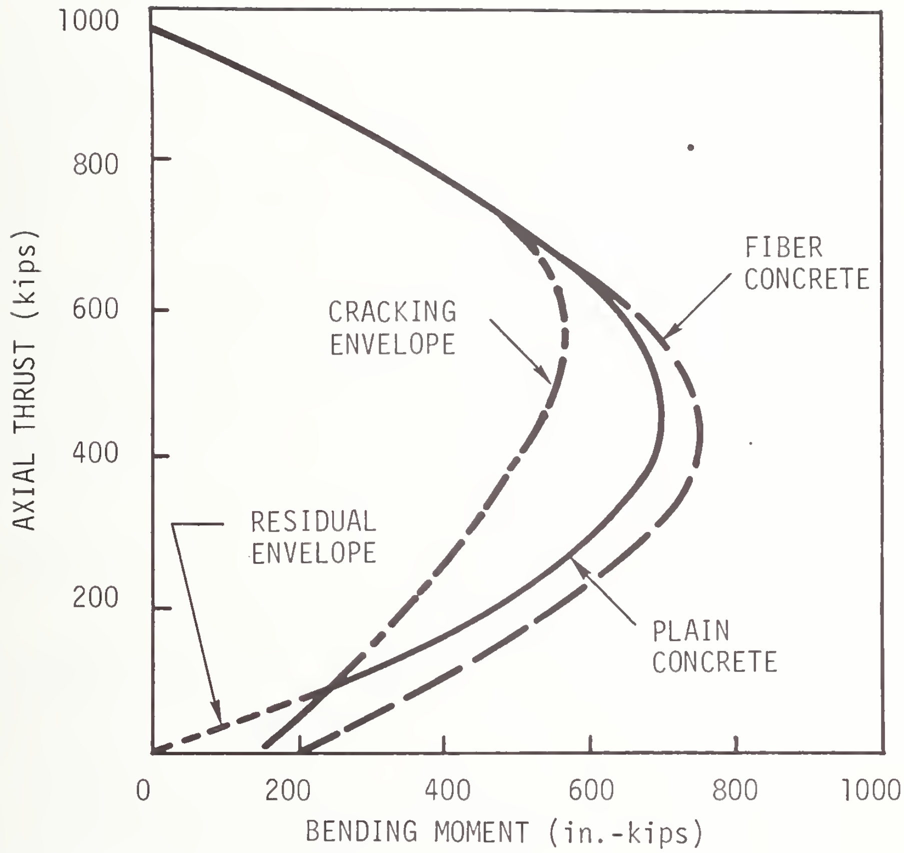
4.2.4 Loads Due to Elastic Deformation

In addition to the rock loads discussed above, certain tunnel support systems can be subjected to loads resulting from elastic deformation of the tunnel wall caused by stresses resulting from excavation or residual stresses in the rock mass. These loads are not significant for a shallow transportation tunnel. It has been estimated (41) that in a 70 ft wide station in a rock mass having a deformation modulus, E_r, of 500,000 lb/in.^{2}, such loads would cause only a 0.05 in. deflection of the rock. Virtually all of this deformation takes place within a distance one to one and a half diameters behind the tunnel face (36). Since the ETLS will place the liner in the region one to two diameters behind the face, loads resulting from elastic deformation of the rock mass need not be considered in the ETLS liner design.

4.3 CHARACTERIZATION OF LINER BEHAVIOR

4.3.1 Moment-Thrust Envelope

The moment-thrust or interaction diagram is a convenient device for comparing the imposed loads with the load-carrying capacity of a section. A typical moment-thrust failure envelope for a plain concrete section is shown in Figure 30. This envelope is defined as the locus of points representing the moment and thrust at failure of a statically determinate structure. If the moment-thrust load path intersects the envelope below the point of maximum moment, referred to as the balance point, the section will suffer tensile cracking and large rotations, and will then collapse due to bending failure. If the moment-thrust load path intersects the envelope above the balance point, the section will have reached its ultimate compressive strength, and will fail in thrust, at a relatively low rotation.



SECTION 36 in. BY 6 in.
$f_c' = 4500$ lb/in.^{2} FOR PLAIN AND 1.5 PERCENT FIBER

FIGURE 30. TYPICAL MOMENT-THRUST DIAGRAM

The behavior of a statically indeterminate structure, such as the highly redundant tunnel liner, is more complex. This structure has the ability to redistribute moment from highly stressed to less stressed sections by developing plastic hinges. When the moment capacity of a section is reached below the balance point, a further increase in thrust will not cause bending failure and collapse, but will result in rotations at that critical section that will allow additional passive reactions to develop between the rock and the lining. Moment will be distributed and the lining will continue to support higher thrusts until the critical section deforms to the point that only a small compressive area remains in the section. At this point, the thrust reaches the compressive capacity of the remaining compression area and a thrust failure ensues. Tests performed on 10 ft diam plain, fiber and conventionally reinforced concrete linings at the University of Illinois have verified this nonlinear behavior (34,43).

4.3.2 Evaluation of Rock-Liner Interaction

When calculating the thrust, moment and deflections to which the liner will be subjected, it is necessary to account for the liner flexibility and its interaction with the rock mass. A linear structural-frame analysis, such as STRUDL, can be used to estimate thrust, moment and deflection due to applied rock loads. In such an analysis, the lining is approximated as a series of continuous beam elements and the rock mass as a series of springs (bar elements). Active loads are applied at nodal points. As the lining deflects, some spring elements develop a passive reaction. Other radial spring elements may go into tension, in which case, the radial spring element is released from the lining to simulate the inability of the lining-rock contact to transmit significant tensile stresses. Figure 31 illustrates the idealized lining configuration used in the STRUDL analysis. A more complete description of the application of the STRUDL method to a tunnel lining can be found in Brierley (38). This analysis shows that in the elastic zone, where strains are less than 0.0015 in./in. and the lining is not cracked, the increases in lining thrust and moment are proportionate and the eccentricity, the ratio of moment to thrust, is constant. The linear analysis predicts linear moment-thrust paths for all sections and does not properly represent the liner behavior as the moment-thrust envelope is approached. In that region, the liner will be cracked and compressive strains may be high enough to cause nonlinear behavior, thus the eccentricity will change. In general, the lining will become more flexible as the envelope is approached and the additional rotations that develop will cause an increase in the passive reactions of the rock mass and will tend to decrease the eccentricity. This behavior and the structural redundancy of the liner, discussed above, allows additional loads to be carried even after the linear moment-thrust path intersects the moment-thrust failure envelope. The linear and nonlinear liner behavior are compared in Figure 32.

4.3.3 Recommended Design Procedure

A nonlinear analysis which adequately represents the liner behavior is not available. However, the following design procedure has been suggested by Cording (41) to ensure the liner serviceability based on available linear analysis.

a. Estimate the uniform and concentrated rock loads based on the most unfavorable geological conditions expected in the tunnel.

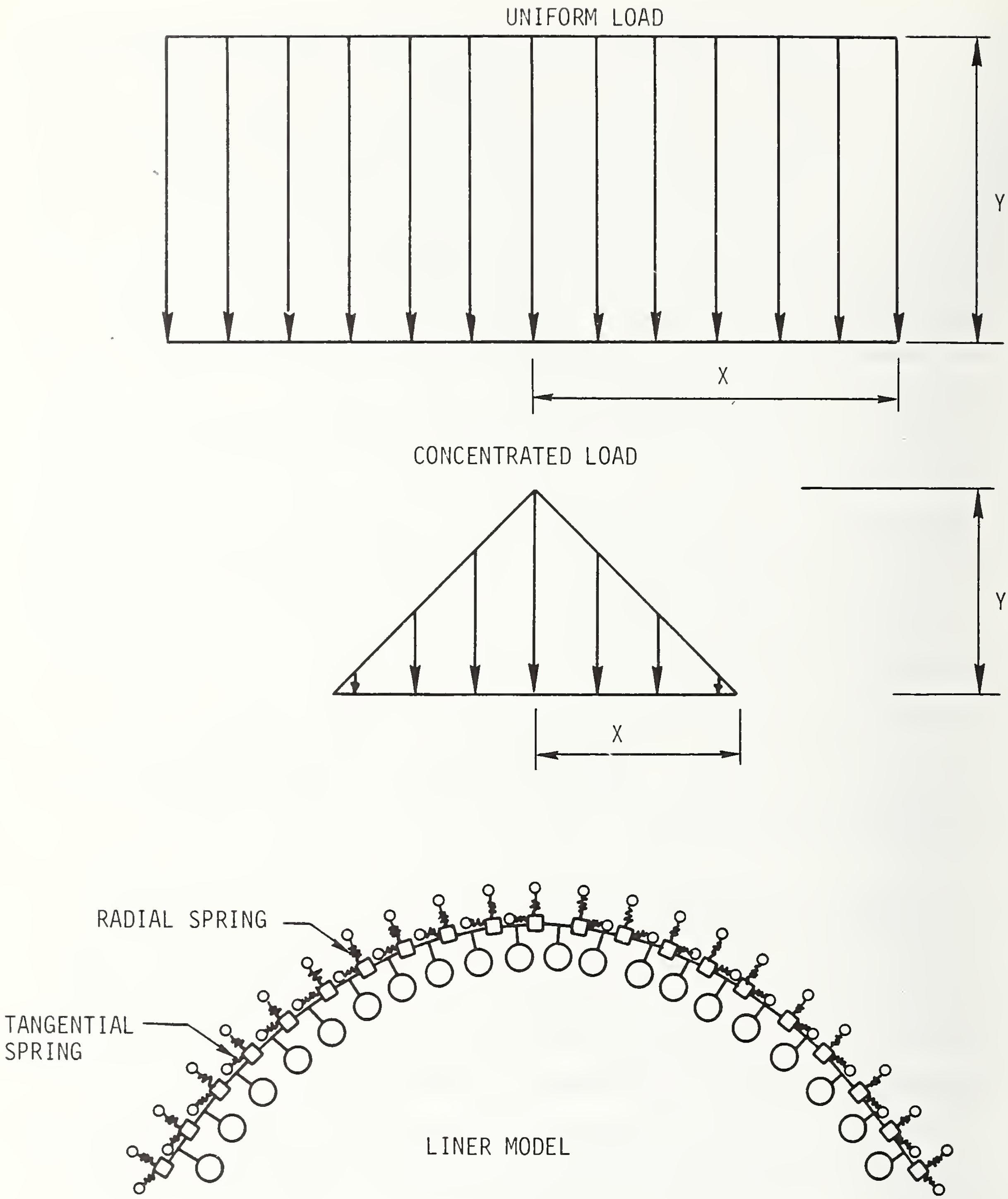


FIGURE 31. LINER CONFIGURATION USED IN ANALYSIS

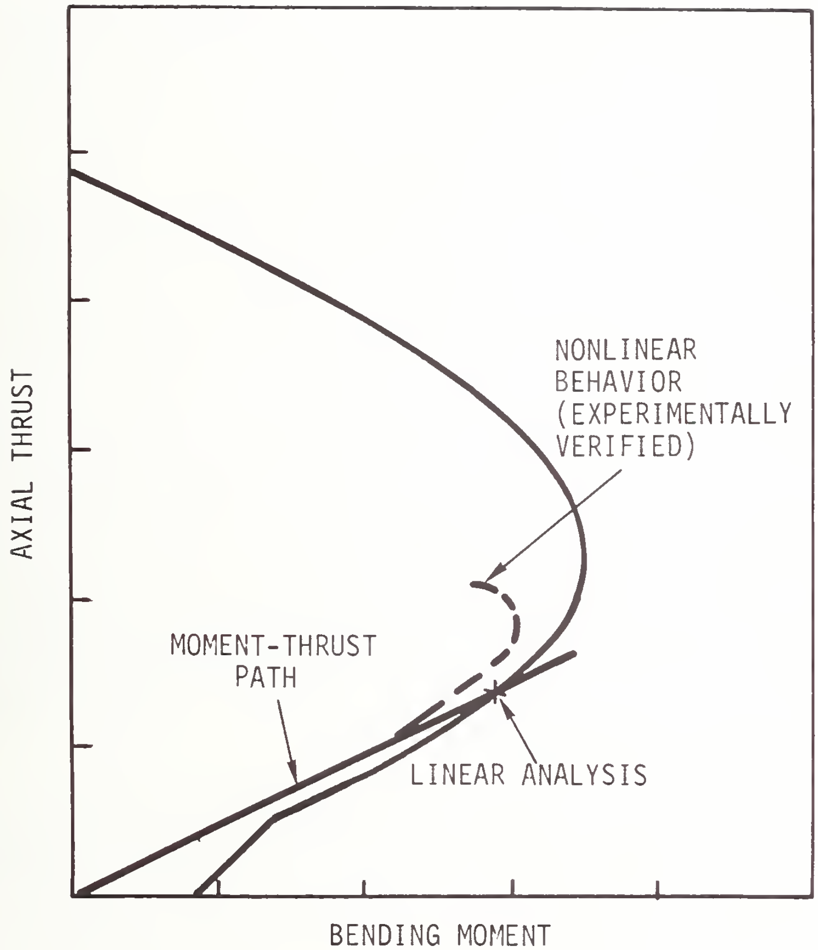


FIGURE 32. COMPARISON OF LINEAR AND NONLINEAR LINER BEHAVIOR

b. Construct the moment-thrust interaction diagram using the selected dimensions of the lining cross section.

c. Estimate moment, thrust, and eccentricity, from a linear structural analysis such as STRUDL, for the uniform and concentrated loading conditions.

d. Multiply the thrust and moment determined for the maximum uniform loading condition by a factor of 1.2 to account for inaccuracies in estimating the eccentricity. Plot 1.2 times the thrust and moment on the interaction diagram.

e. Select critical sections where moment and thrust are high for concentrated rock loads. Multiply the thrust and moment for each of these sections by a factor of 2. This factor is intended to account for inaccuracies in estimating maximum rock load and eccentricity and to ensure that the actual thrust is well within acceptable limits for cracking and deflection of the liner. Plot twice the thrust and moment on the interaction diagram.

f. If the lines of constant eccentricity, passing through the plotted points, intersect the interaction envelope below the balance point, the balance point thrust will be the limiting thrust.

g. If the lines of constant eccentricity, passing through the plotted points, intersect the interaction envelope above the balance point, the thrust at the point of intersection will be the limiting thrust.

This procedure has been used to evaluate the ETLS liner behavior under the rock loads discussed in subsection 4.2 and to establish the recommended liner strength and dimension specifications. The results of this analysis are presented in the following subsection.

4.4 ETLS LINER THICKNESS AND STRENGTH EVALUATION

In order to apply the liner design criteria discussed in the preceding section to the ETLS liner, data from the results of a linear structural analysis of the Washington Metro station linings performed by Cording (41) have been used. The results presented in nondimensional form in Figures 33 and 34 can be used to determine the moment, thrust and deflection resulting from any uniform or concentrated rock load. Any liner thickness can be evaluated, since the flexibility ratio of the rock to the liner is the dominant variable.

The flexibility ratio must be based on the in situ modulus of the rock mass, E_R. The intact rock modulus of the gneisses and schists commonly encountered in Washington is approximately 5×10^6 lb/in.^{2}. To obtain the in situ modulus, the intact rock modulus must be reduced by a factor that accounts for the joints in the rock mass. For RQD values ranging from 25 (poor) to greater than 90 (excellent) the in situ modulus was estimated by Brierley (38) to range from 100,000 to 1,000,000 lb/in.^{2}. Since the ETLS will be limited to machine bored tunnels (RQD $\geq$50), 500,000 lb/in.^{2} has been used for the in situ modulus in this analysis.

The following rock loads were calculated for a 20 ft diam tunnel using the method discussed in Subsection 4.2. The density of the rock mass (γ) was assumed to be 175 lb/ft^3.

a. Maximum uniform rock load $1.0\gamma B$ = 3500 lb/ft^2

b. Concentrated rock load $0.6\gamma B$ = 2100 lb/ft^2

The liner section used for analysis was 3 ft long and 6 in. thick. Its elastic modulus, E_ℓ, was assumed to be 4 by 10^6 lb/in.^{2}, which coupled with the liner dimensions and in situ rock modulus yields flexibility ratio of approximately 2000.

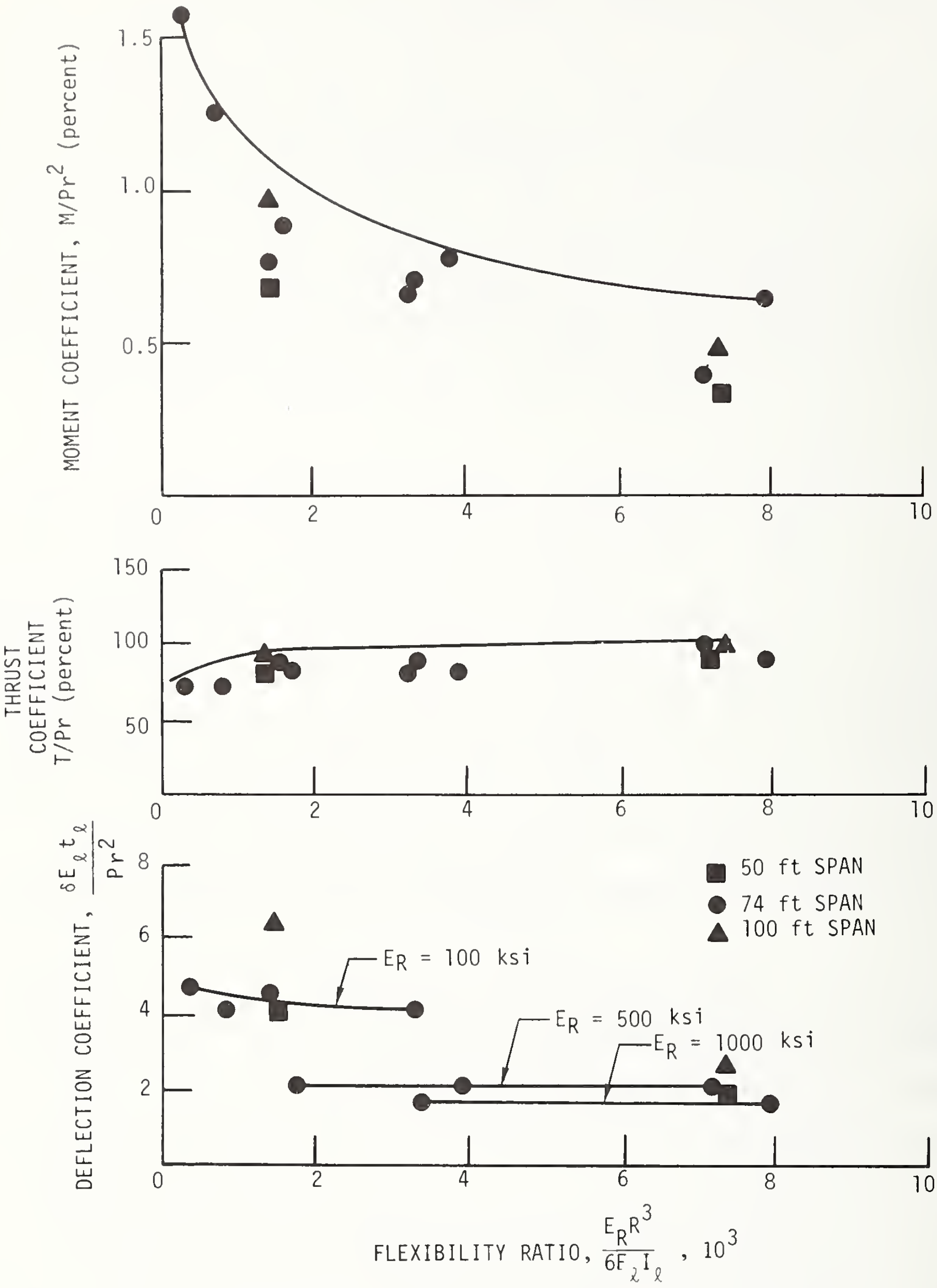


FIGURE 33. LINER COEFFICIENTS VERSUS FLEXIBILITY RATIO FOR A UNIFORM LOADING CONDITION (38)

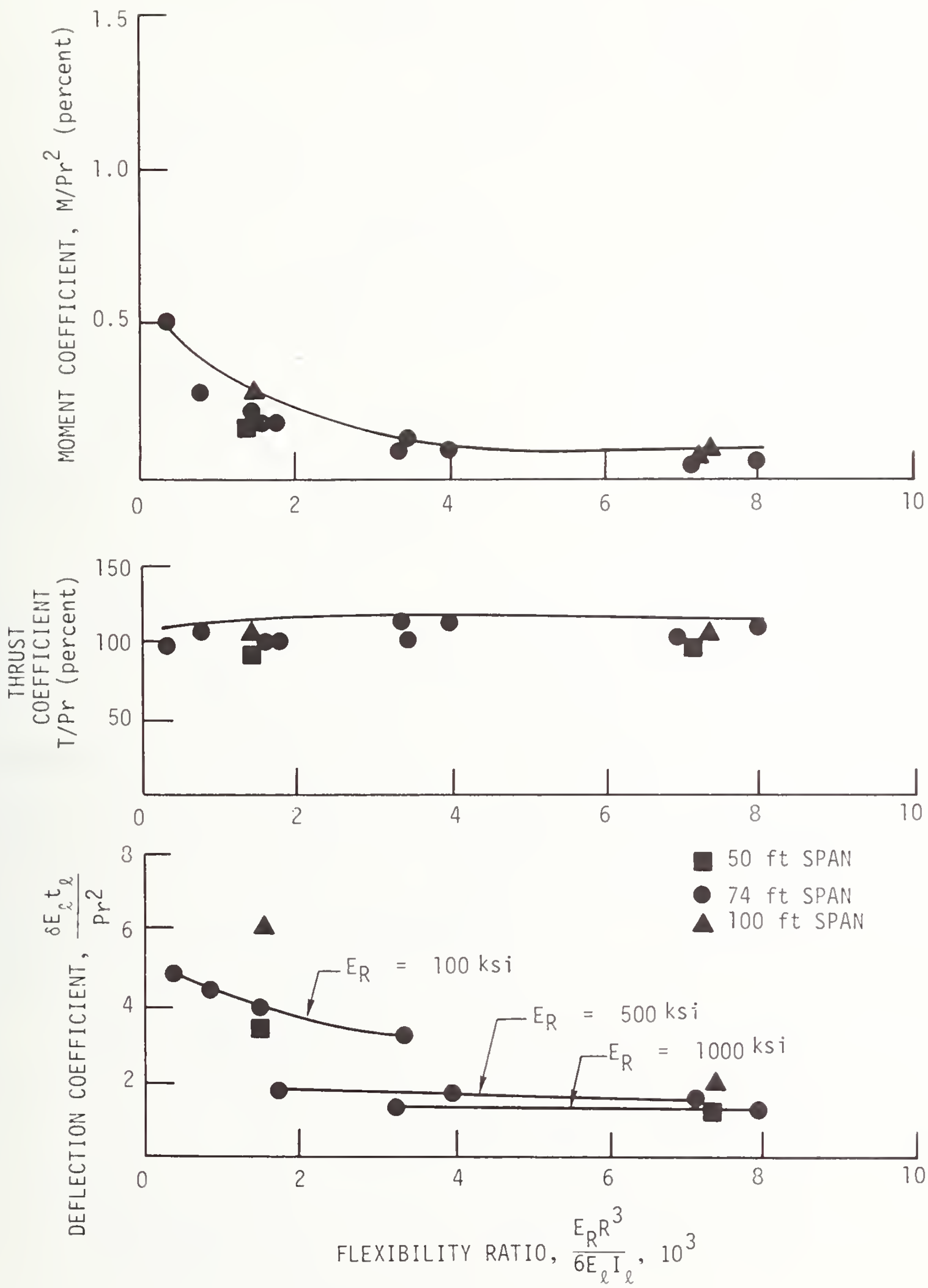


FIGURE 34. LINER COEFFICIENTS VERSUS FLEXIBILITY RATIO FOR A NONUNIFORM LOADING CONDITION (38)

The resulting thrusts, moments and displacements, as determined from Figures 33 and 34 are summarized below. In these figures, the rock load is represented by P.

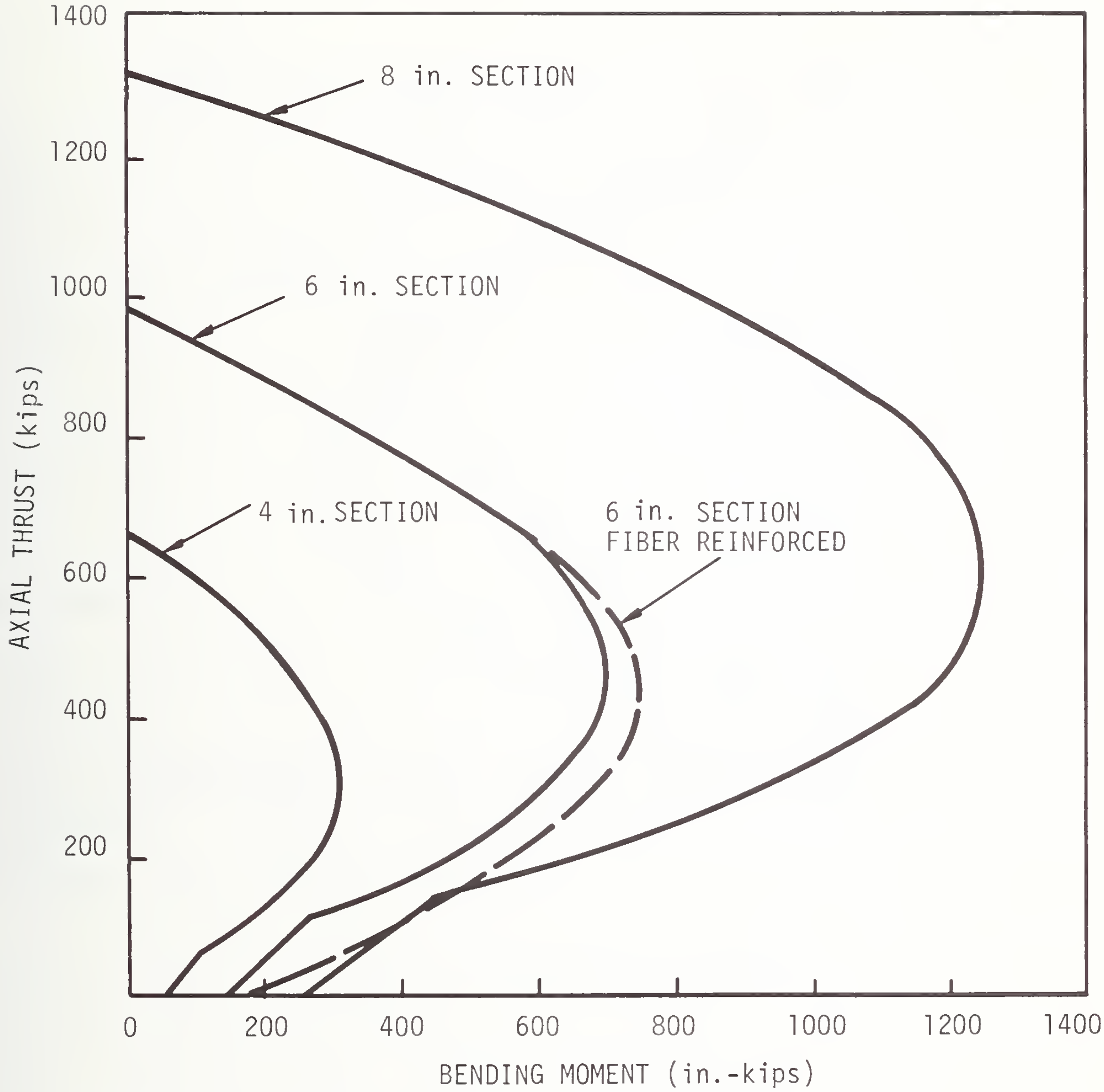
a. Maximum Uniform Rock Load:

Thrust	$1.1\ Pr$	=	113 kips
Moment	$0.002\ Pr^2$	=	24 in.-kips
Deflection	$1.9\ Pr^2/E_\ell t_\ell$	=	0.026 in.

b. Concentrated Rock Load:

Thrust	$1.0\ Pr$	=	62 kips
Moment	$0.0095\ Pr^2$	=	68 in.-kips
Deflection	$2.1\ Pr^2/E_\ell t_\ell$	=	0.017 in.

These loads were compared to the moment-thrust interaction diagrams for various liner thicknesses shown in Figure 35, the 6 in. thick section was selected as most suitable. Using the design procedure discussed in Subsection 4.3, the thrusts and moments for these loading conditions are plotted on typical moment-thrust interaction diagrams (Figure 36) for a 3 ft long, 6 in. thick liner section of both plain and fiber reinforced concrete. These interaction diagrams were calculated from experimentally determined concrete stress-strain curves as part of an experimental tunnel liner program at the University of Illinois (34,43). The compressive strength (f'_c) for both the plain and fiber reinforced sections was 4500 lb/in.^{2}. It can be seen from Figure 36 that even with the recommended design factor of 1.2 for uniform loading and 2 for concentrated loading, the failure envelope is not approached for either the plain or fiber reinforced sections. Serviceability of the plain concrete liner is assured by the



SECTION 36 in. LONG
$f_c' = 4500$ lb/in.^{2}

FIGURE 35. MOMENT-THRUST DIAGRAMS FOR VARIOUS SECTION THICKNESSES

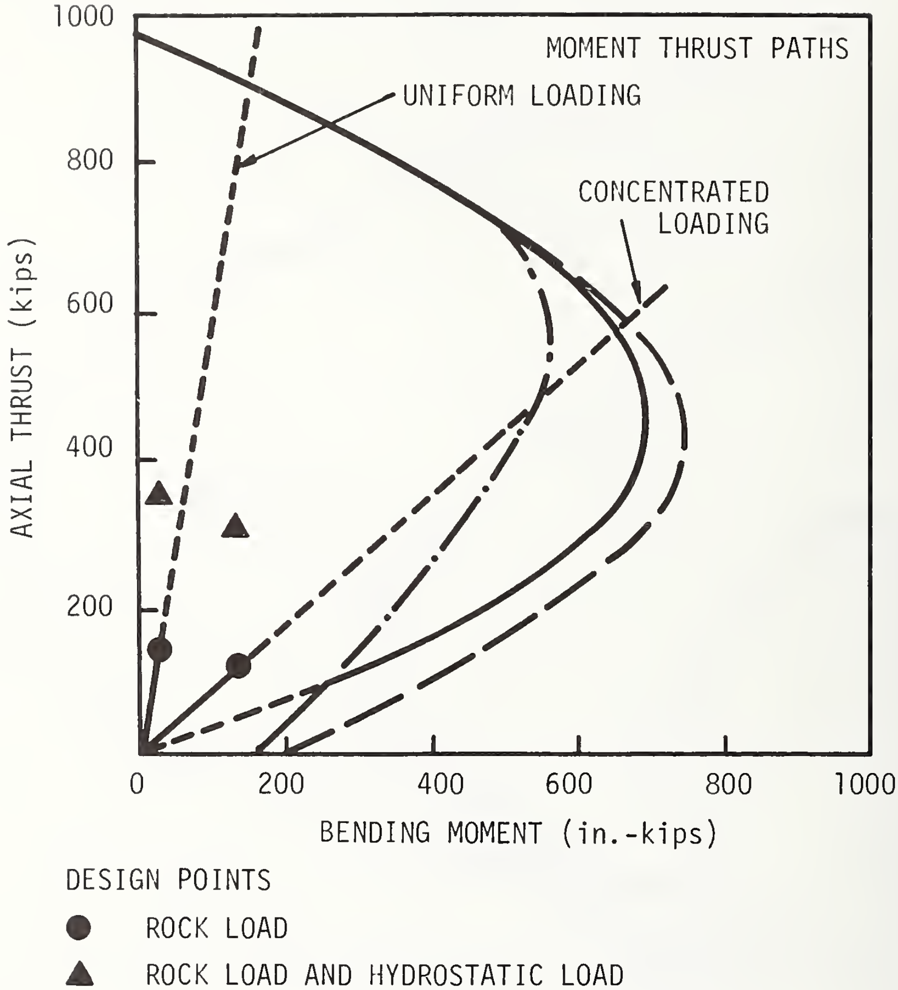
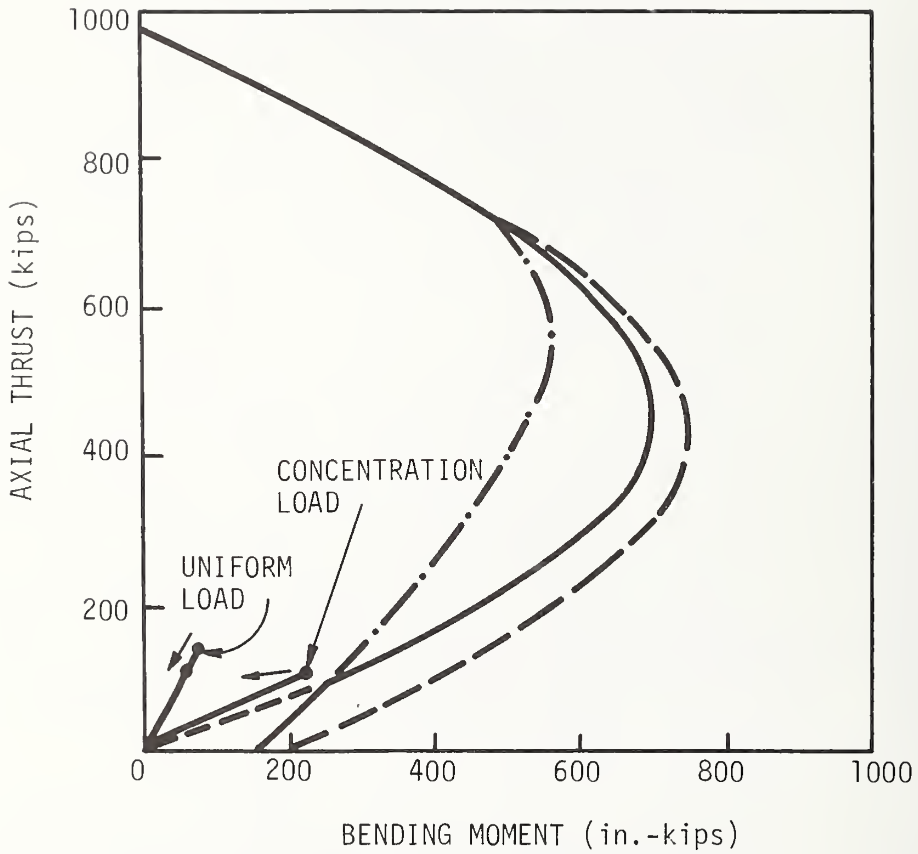


FIGURE 36. EVALUATION OF LINER CAPACITY

significant margin between the design moment and thrust point and the plain concrete cracking envelope. The plain concrete liner is adequate even if it is conservatively assumed that it has zero tensile strength.

The adequacy of these two liner sections has also been investigated under two additional loading situations. In the first case, it is assumed that the tunnel is below the water table and a hydrostatic load of 100 ft of water was applied in addition to the rock loads previously considered. In the second case, it is assumed that the in situ rock modulus was only 100,000 $lb/in.^2$ which, in turn, reduced the flexibility ratio to approximately 400. The moments and thrust resulting from these two loading conditions are plotted on the interaction diagrams in Figures 36 and 37. It can be seen from these figures that the plain and fiber reinforced liner sections are both adequate in these two additional loading cases. The hydrostatic load is essentially uniform, it increases the thrust with very little increase in moment, thus decreasing eccentricity. On the other hand, reduction of the in situ rock modulus increases the eccentricity by reducing the passive reaction of the rock for a given liner displacement. Thus, for a given thrust, more displacement occurs and hence larger moments result.

In the case of the low rock modulus, one might select the fiber reinforced concrete to provide a greater margin between the moment and thrust resulting from the design loads and the moment-thrust envelope. Before that selection is made, it should be remembered, as was discussed in subsection 4.3, that intersection of the load path with the moment-thrust envelope of the section does not imply failure of a highly redundant structure such as a tunnel liner. If the desire is to prevent tensile cracking of the liner, the techniques discussed in the following subsection



ARROWS INDICATE DIRECTION DESIGN POINTS MOVE WHEN ROCK BOLTS ARE USED

FIGURE 37. DESIGN LOADS 6 IN. LINER IN POOR ROCK

offer a more cost-effective means of providing margin against cracking than use of fiber reinforcement. Alternately, the liner thickness could be increased to 8 in. to gain the desired design margin. The moments and thrusts for the uniform and concentrated loading conditions are shown in Figure 38; note that these loads are slightly different than those in Figure 37 because of the decreased flexibility of the 8 in. liner.

The high eccentricity of the moment-thrust path in poor quality rock suggests that a 6 in. thick plain concrete liner may not be adequate in the portal regions of a tunnel. Severe weathering in these regions leads to very poor rock quality and the shallow cover can lead to undesirable load distributions. Both of these conditions cause high eccentricities which may lead to excessive cracking of the liner in these regions. While the liner would not fail, this cracking is not desirable and consideration should be given to using a conventionally reinforced CIP liner in these regions of the tunnel.

4.5 METHODS TO REDUCE CONCENTRATED ROCK LOADS

There are common tunnel support practices such as rock bolting and deep grouting that can be used with the ETLS to ensure even greater confidence in the liner in poor rock conditions. Rock bolts could be installed close to the face prior to liner installation. Deep grouting could be performed after the liner had been installed. Both of these procedures increase the load carrying capacity of the rock mass and will reduce the rock load and ensure that what load remains will be distributed more uniformly. The effect of these procedures on the thrust and moment to which the liner is subjected is illustrated in Figure 37.

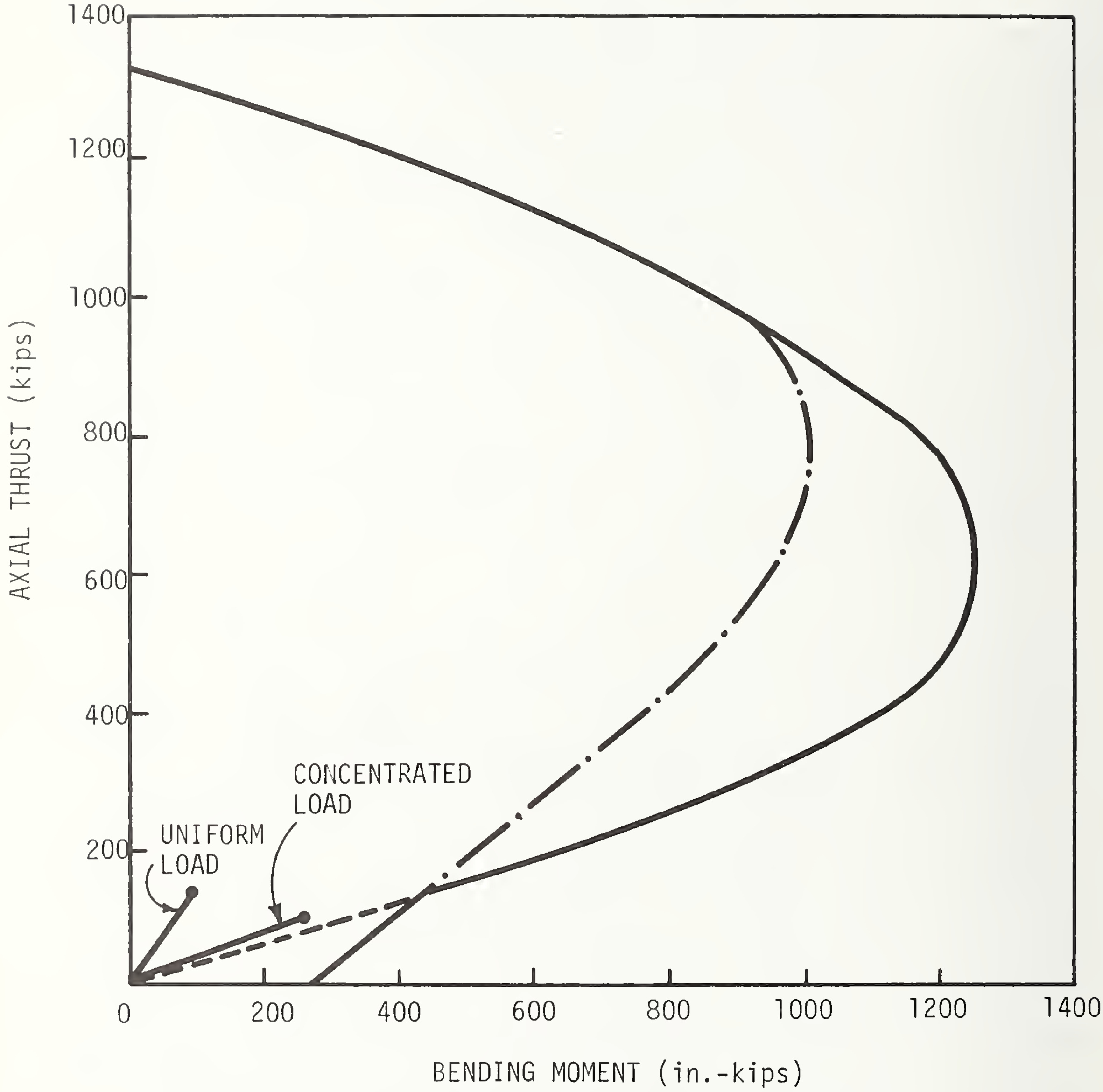


FIGURE 38. DESIGN LOADS 8 IN. LINER IN POOR ROCK

The decreased eccentricity of the resulting moment and thrust moves the load path away from the moment-thrust envelope and thus increases the design margin and provides more confidence in the liner design.

4.6 CONCLUSIONS AND RECOMMENDATIONS

The review of the current state-of-the-art of tunnel support design has shown that a significant reduction in the demands placed on the support can be achieved if the support is flexible relative to the medium it is supporting. Recently developed analytical methods which consider not only the tunnel geometry and loading, but also the material properties of the medium being supported and its interaction with the liner, have shown that the expected loads for a 20 ft diam tunnel can be carried by a 6 in. thick liner of plain concrete. This fact coupled with experimental results (34,43) which show that the addition of fibers neither changes the failure mode of the liner nor significantly increases its strength (see Section 3), lead to FMA's recommendation that steel fiber reinforcement not be used in the ETLS liner design. The elimination of steel fibers in the ETLS liner will reduce the tunnel liner material cost by a factor of two and significantly reduce material handling costs and potential problems.

The use of a TBM not only minimizes the long-term rock load to which the liner will be subjected, but practically eliminates the rock load on the liner as it leaves the slipform. Thus, the liner need only develop enough strength to be self-supporting before it clears the slipform.

5. ETLS SPECIFICATIONS

The specifications presented in this section are based on the background information discussed in Sections 2, 3, and 4 combined with FMA's assessment of the performance requirements for an economically attractive system. These specifications are oriented toward urban rapid transit tunnel applications, but do not exclude other possible applications. The specifications, summarized in Table 3, are divided into the following categories:

a. Tunneling Conditions - defines the ground conditions and environment in which the ETLS must function.

b. Tunnel Boring Machine - defines TBM performance and design parameters which impact ETLS design requirements.

c. Tunnel Lining System - defines the design and operational requirements for the ETLS components.

d. Liner Concrete - defines the concrete mix and performance requirements.

The subsections following the table give the rationale used to set each specification.

5.1 TUNNELING CONDITIONS

The first generation ETLS will be used in machine bored rock tunnels. The following parameters define the environment for that application.

5.1.1 Rock Type and Quality

The use of a TBM limits the range of rock conditions in which the ETLS must be capable of operating. As discussed in Section 3, the RQD of the rock formation should be greater than ~50 for a TBM to be effective.

TABLE 3. ETLS SPECIFICATIONS

Tunneling Conditions	
Rock type and quality	Any that can be machine bored
Depth of cover	One diameter to any depth where squeezing ground is not encountered
Water inflow	Any that can be handled in a TBM operation
Maximum external head for a water tight tunnel	100 ft on final liner
Nominal bore diameter	15 to 21 ft
Bore diameter variation	-1 1/2 in. to 100 ft
Bore surface roughness	1/2 in. to 1 in. maximum pitch
Rock temperature	50° to 70°F
Ambient temperature	55° to 90°F
Minimum tunnel radius of curvature	750 ft
Tunnel Boring Machine	
Type	Any
Maximum advance rate	12 ft/hr
Gripper slip	
Longitudinal	1 to 2 in.
Circumferential	1/4 to 1/2 in.
TBM deviation from alignment bullseye	3 in.
TBM backup for cutter replacement	48 in. maximum
Shove length	2 to 6 ft
Routine shutdown period	15 min to 2 hr

TABLE 3. ETLS SPECIFICATIONS (CONTINUED)

Lining System	
Lining thickness	6 to 12 in.
Variation in lining thickness across tunnel section	±3 in.
Maximum lining rate	12 ft/hr
Minimum lining rate	2 ft/hr
Lining system to TBM travel	8 ft
Lining Material	
Concrete workability	20 to 25 min
Tolerance on concrete set time	40 to 60 min
Final concrete strength (f_c')	4000 to 6000 lb/in.^{2}

5.1.2 Depth of Cover

Shallow tunnels, less than 1 diam below the surface, could impose undesirable non-uniform loading conditions on the liner. The results of the analysis presented in Section 3 indicate that, except in squeezing ground, there is no maximum limit on the depth of cover in which the ETLS could be used.

5.1.3 Water Inflow

Distributed seepage into the tunnel should not pose a problem for the ETLS. Local spouts (>~10 gal/min) will probably require special treatment before the liner progresses. Water control techniques similar to those used with conventional cast-in-place concrete liners could be used with the ETLS.

5.1.4 Depth Below Water Table

Water pressures greater than concrete pressure during placement may degrade the strength and quality of the liner. However, it usually takes some time for the hydrostatic pressure to develop, allowing time for the liner to set and gain strength. The structural adequacy of the liner design under a 100 ft hydrostatic head was illustrated in Section 3.

5.1.5 Nominal Tunnel Diameter

The 15 to 21 ft specification covers the diameter range of past and proposed mass transit subway tunnels.

5.1.6 Minimum Radius of Curvature

The minimum turn radius on the WMATA system is 750 ft. Tighter turns would be the exception rather than the rule on future transit tunnels.

5.1.7 Ambient Temperatures

Ambient temperatures at the tunnel heading are of interest since they will influence the concrete set rate. Ambient temperature at the heading can be quite high due to heat generated by the TBM transformers and hydraulic pumps. A temperature range of 55° to $90^{\circ}F$ has been specified. The rock formation will act as a heat sink. Rock temperature for a relatively shallow urban transportation tunnels is expected to be in the 50° to $70^{\circ}F$ range.

5.2 TUNNEL BORING MACHINE

The performance parameters of the TBM will impose operational requirements on the ETLS. These parameter specifications are discussed in the following subsections.

5.2.1 TBM Geometry

There are a number of different TBM designs currently in use, each having been designed to best fit a given tunneling situation. These designs are continually evolving as technology advances. The state of flux in the various TBM designs suggests that the ETLS design be independent of the geometry of a particular TBM. The ETLS design should be compatible with or adaptable to TBM's in general.

5.2.2 Advance Rate

The TBM advance rate is of interest since it will place an upper limit on the required ETLS lining rate.

The statistics on TBM advance rates are presented in a variety of ways which do not lend them to direct application to the question at hand. For example, the maximum daily (three shift) TBM advance rate in the Bureau of Reclamation's 18.5 ft, Navajo No. 3 Tunnel was 260 ft/day. However, the number of hours which the TBM operated during that day is not available, so the average advance rate cannot be determined. To achieve the 260 ft/day advance rate, a maximum TBM advance rate of 20 ft/hr was attained. At that rate, the advance rate was limited by the capacity of the mucking system.

5.2.3 Bore Diameter Variation

Some variation in the bored diameter of the tunnel can be expected due to wear of the gauge cutters. This wear is allowed to reach 1.5 in. before cutter replacement is required. TBM advance between replacements varies with rock hardness, but a minimum advance of 100 ft can be expected.

5.2.4 Bore Surface Roughness

Surface ridges are created by the gouging action of the gauge cutters which cut a spiral groove in the tunnel wall. Maximum TBM advance per revolution of the cutter head is approximately 1 in. Thus, these ridges are expected to have a maximum pitch of 1 in.

5.2.5 Gripper Slip

TBM gripper pads can slip backwards during a cutting shove. This backwards movement must be accommodated by the ETLS components which connect the slipform to the TBM gripper section. The specifications cited in Table 1 were recommended by the Robbins Company.

5.2.6 TBM Alignment Deviation

Current practice is to specify a tolerance for line and grade of the finished tunnel. On the WMATA system this tolerance was typically 1 to 1.5 in. Thus, the longitudinal axis of the finished tunnel must fall within a "bullseye" 2 to 3 in. in diameter. Early TBM designs had difficulty driving the tunnel within specified tolerances, thus contractors typically overbored the tunnel by about 3 in. on radius in order to allow the concrete liner to correct the alignment deviation. The Robbins Company is currently employing an alignment system which utilizes a laser and photo sensitive target. This system enables the TBM to maintain the desired tunnel alignment within a fraction of an inch.

5.2.7 Shove Length

The 2 to 6 ft specification covers the range of shove lengths of current and proposed TBM's.

5.2.8 Routine Shutdown Periods

A range of 15 min to 2 hr covers the time required for many routine periodic shutdowns from occurrences such as cutter changes, muck train derailments, and power or other service interruptions.

5.3 EXTRUDED TUNNEL LINING SYSTEM

5.3.1 Liner Thickness

The results of the analysis presented in Section 3 show that a liner 6 in. thick can accommodate all rock and construction loads anticipated in machine bore rock tunnels. Due to considerations of public safety and potential ground settlement transit tunnel liner design has generally been more conservative than that for other tunnel applications. This conservatism has fostered the continued use of the 1 in. of liner thickness per foot of tunnel diameter thumb rule by tunnel contractors. Recently developed analytical methods are, however, beginning to increase confidence and interest in thin flexible tunnel liners. To encompass this range of thinking, we have specified that the ETLS be able to cast liners 6 to 12 in. thick.

5.3.2 Maximum Lining Rate

To ensure that the ETLS did not limit the TBM advance rate, it would have to place a liner at a rate equal to the TBM's maximum sustained (several shoves in succession) advance rate. But, since advance rates of 20 ft/hr appear to be the exception rather than the rule, there does not seem to be a strong incentive to push the ETLS performance requirements to 20 ft/hr. The Robbins Company states that an ETLS with a maximum advance rate of 10 ft/hr would be attractive for use with a TBM. A compromise rate rate of 12 ft/hr has been specified for the system.

5.3.3 Minimum Lining Rate

There are numerous incidents which can cause the TBM to shut-down for periods ranging from minutes to hours. In order to minimize the number of ETLS shutdowns, the system should be designed to advance at a rate slow enough to permit its continued operation without overtaking the TBM which has been stopped for a prolonged period. This rate will be limited by the maximum allowable residence time of the concrete in the systems slick lines. A design goal of 2 ft/hr has been specified.

5.3.4 Tunnel Alignment Correction

Cast-in-place concrete liner forms are typically positioned to compensate for deviations of the bored tunnel centerline from the specified centerline. The ETLS should have a similar capability. Based on the precision of recently developed TBM control systems, an alignment capability of ± 1 in. should be more than adequate.

5.3.5 ETLS/TBM Relative Motion

The system should accommodate the TBM gripper advance during the regrip cycle (6 ft) and permit the TBM to backup for cutter replacement (2 ft). Thus, a total relative motion of 8 ft must be permitted and the ETLS should not advance to within 2 ft of the trailing end of the TBM.

5.3.6 Slipform Design Pressure

The concrete in the slipform must be maintained above some minimum pressure in order to ensure proper distribution and consolidation. For design purposes, we assume a maximum concrete pressure of 15 lb/in.^{2} at the crown. Adding the 20 ft concrete head yields a maximum form pressure of 35 lb/in.^{2} at the invert.

5.4 LINER CONCRETE

5.4.1 Workability

The liner concrete must maintain adequate workability to permit pumping and distribution to a period of 20 to 25 min after mixing.

5.4.2 Time to Self Support

In order to enable the system to attain the desired advance rates without use of an excessively long slipform, the liner concrete should be able to support itself in the tunnel crown 40 to 60 min after mixing.

5.4.3 Final Concrete Strength

The liner concrete should attain a 28 day compressive strength in excess of 4500 lb/in.^{2} to ensure adequate strength for a 6-in. liner.

6. TECHNOLOGY REQUIREMENTS AND TEST PLAN

The state-of-the-art review answered many questions regarding the feasibility and desirability of various ETLS design alternatives. System requirements for which current tunneling and construction practice offered no acceptable solution also became apparent. These requirements and the research and development test plan followed to develop the necessary information are discussed in the following subsections.

6.1 IDENTIFICATION OF REQUIREMENTS

The following tasks were identified during the state-of-the-art review and definition of system specifications as the major items that should be investigated before detailed design of the ETLS proceeded. The tasks were:

a. Formulation of a concrete for early workability and rapid set

b. Development and evaluation of concrete placement and distribution techniques

c. Evaluation of concrete self-support and required slipform dimensions at proposed system advance rates

d. Evaluation of concrete pressure and drag loads on the slipform and bulkhead.

During the state-of-the-art review it was discovered that currently available supplies of reg-set cement were not suitable for use with the ETLS because of their extremely rapid (~10 min) and erratic set times. The erratic set times resulted from the mixing cements from several burns, with different set times, to facilitate storage. It was thus necessary to formulate a concrete from another commercially available rapid set cement. This concrete formulation was then used in the three remaining research and development tasks.

None of the construction practices reviewed enabled concrete distribution and compaction within a closed slipform to be predicted. It was thus necessary to test the proposed form filling and vibration techniques before proceeding with the detailed design of the slipform and bulkhead.

Current slipforming practice does not impose any shear or flexural loads on the concrete as it leaves the slipform. In the extruded tunnel liner, these loads are imposed on the liner in the crown due to the concrete's own weight, which may make it sluff from the tunnel roof. Therefore, a test that would determine the strength gain required of the concrete before it cleared the slipform was proposed. Because of liner geometry and loading, it is possible that the concrete will, unlike the case of vertical slipforming, tend to remain in contact with the slipform after initial set. Thus, the degree of taper the slipform must have to prevent tearing of the green concrete required investigation as well. Finally, the influence of the pressurized concrete on slipform drag required experimental determination in order to predict the force required to advance and stabilize the slipform.

6.2 TEST PLAN

A test plan was devised to provide the data necessary to proceed with the ETLS design. The three major areas investigated were:

a. Concrete mix design

b. Concrete distribution in a closed form

c. Horizontal slipforming.

The overall test program flow chart is shown in Figure 39. Individual procedures and results for each of the tests are presented in the following sections. The general approach to concrete mix design was to test several commercially available rapid set cements with accelerators or retarders as appropriate, for set time, workability and early strength. These tests are discussed in detail in Section 7.

The questions regarding concrete distribution and slipforming were answered by tests conducted in configurations which are representative of the ETLS. Because of problems modeling systems using concrete, due to size effects of paste and aggregate, it was not desirable to use a scale model as the test device. In order to avoid this problem of size effect, separate tests were conducted to determine the factors effecting distribution and slipforming. The use of separate test facilities, representing partial segments of the liner, permitted full-scale liner thickness and concrete aggregate to be tested without resorting to a full scale system mockup. These tests are discussed in detail in Section 8.

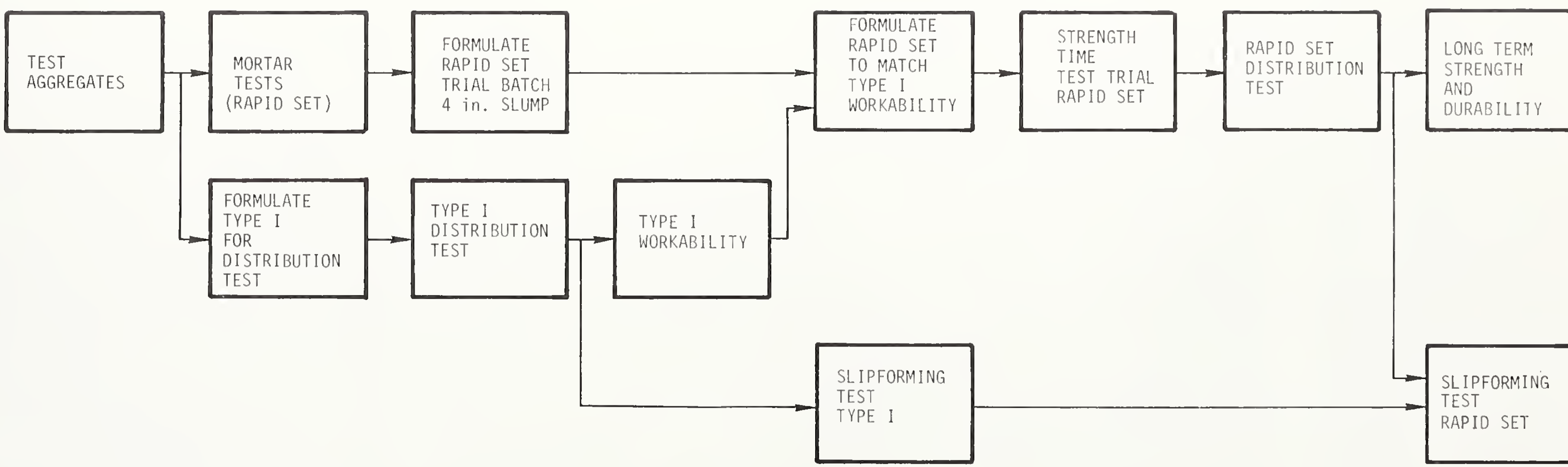


FIGURE 39. OVERALL TEST PLAN

7. CONCRETE FORMULATION TESTS

A laboratory test program was developed to formulate a concrete that would be suitable for ETLS application. There are specific material property requirements that must be met for the material to be useful for extruded tunnel lining application.

The concrete must be pumpable. It must maintain adequate workability for a period of 20 to 25 min in order to permit proper placement and consolidation within the slipform. Once the material is in place it must set rapidly in order to develop high early strength. A compressive strength of about 500 lb/in.^{2} should be reached in 2 hr. On a long-term basis, the concrete must exhibit a 28-day strength of about 5000 lb/in.^{2}. In addition, the material must be durable and resistant to deterioration from prolonged exposure to water and sulfate attack.

Four cement candidates were chosen for testing. They include:

a. Aluminous cement

b. Portland Type III cement

c. A combination of Portland Type I and Aluminous cement

d. Very high early cement.

The concrete formulation method followed an orderly, systematic procedure. The proper aggregate gradation was chosen to insure pumpability. Cement candidates were then tested for time of setting to eliminate those cements that were unable to meet the set time requirements. Workability tests and preliminary strength-time determinations were conducted on the promising candidates. Extensive mix design and long-term testing were performed using the final candidate.

The results of the mortar set time tests showed that only aluminous cement and VHEC formulations could meet the set time requirement without exhibiting undesirable secondary effects.

Both candidates were used to formulate trial batches to study workability and strength-gain. VHEC concrete performed very well and was chosen for advanced mix design and long-term testing.

During the extensive mix design study, a VHEC concrete formulation was developed that exhibited an excellent workability range. In addition, results of the strength-time testing indicated the VHEC formulation surpassed both the 2 hr and 28-day strength requirements.

The results of the durability testing showed that VHEC concrete is resistant to sulfate attack and maintains strength through prolonged exposure to water.

The final VHEC formulation appears to meet all the requirements necessary for tunnel lining application. In addition, VHEC concrete has a reasonable degree of controllability and can be modified to exhibit various workability and strength-gain characteristics for a particular application.

7.1 TIME OF SETTING DETERMINATION

7.1.1 Introduction

The material used for the ETLS should exhibit an initial set time in the range of 20 to 40 min. Four cement candidates were chosen to be tested for conformance to this specification. They include:

a. Aluminous cement

b. Portland/Aluminous cement combinations

c. Portland Type III Cement

d. VHEC.

All cements were tested for time of setting by the Gillmore Needle Method according to ASTM C266.

With the aid of set controlling admixtures, three of the four candidates exhibited an initial set time within the 20 to 40 min time requirement. The three candidates include:

a. Aluminous cement with a combination of lithium carbonate accelerator and Plastiment® retarding densifier

b. VHEC with citric acid retarder

c. Portland Type III cement with $CaCl_2$ accelerator.

The fourth candidate, the Portland/aluminous combination, exhibited very erratic set behavior and was eliminated from further consideration.

Portland Type III cement modified with calcium chloride was eliminated despite its ability to meet the set time requirement. The large amount of calcium chloride added to accelerate set created an excessive amount of heat to be given off during setting. In large batches, this effect may create difficulties in controlling set and may cause thermal distress.

The two remaining candidates were subjected to further testing to study workability characteristics and strength-time development.

7.1.2 Method of Testing

A laboratory program was conducted to determine the initial and final set times of the four cement candidates. Mortars with various combinations of set controlling admixtures, water-cement ratios, and water temperatures were tested. Set times were determined by the Gillmore Needle Method according to ASTM C266. The nature of the fast setting cements required minor variation from the standard method of testing. Most of the variations involved changes in mix quantities and length of mixing time. The application of the Gillmore Needles was performed according to the standard.

Typically, a 150g sample of cement was batched with the appropriate amount of water according to water-cement ratio. Admixtures, when used, were dissolved in the mix water before cement addition. The materials were mixed for 3 min. A 1/2 in. thick pat of mortar was formed on a 4 by 4 in. glass plate. Mortars were tested at 1 min intervals to initial and final set conditions. Between testing times the pat was stored in a portable plastic moisture chamber to prevent evaporation of mix water.

7.1.3 Results of Testing

7.1.3.1 Aluminous Cement - A total of 21 different aluminous cement-admixture combinations were studied. Various amounts of lithium carbonate accelerator, Lomar D superplasticizer, and Plastiment retarding densifier were used. All but one mix were formulated at a 0.35 water-cement ratio.

Lithium carbonate additions in the range of 2.5 to 10 g/sack of cement, or about 0.005 to 0.025 percent by weight of cement were used. Due to the small amounts of lithium carbonate required and because the admixture was not readily soluble in water, a 0.1 percent solution was used for all tests. Admixture quantities and results of the setting time trials appear in Table 4.

Initially, mixes were formulated with only lithium carbonate accelerator, in a range of 2.5 to 10 g/sack of cement. Most

TABLE 4. RESULTS OF THE TIME OF SETTING TESTS FOR ALUMINOUS CEMENT

MIX	WATER-CEMENT RATIO	Li_2CO_3 - GRAMS PER 94 LB SACK CEMENT	PLASTIMENT, LIQUID, % WEIGHT OF CEMENT	LOMAR-D LIQUID % WEIGHT OF CEMENT	WATER TEMP (°F)	INITIAL SET HR: MIN	FINAL SET HR: MIN
Al-1	0.35	10	-	-	72	0:06	0:08
Al-2	0.35	10	-	-	72	False Set	-
Al-3	0.35	5	-	-	72	0:09	0:45
Al-4	0.35	4	-	-	63	0:26	1:47
Al-5	0.35	3.75	-	-	63	0:46	1:03
Al-6	0.35	2.5	-	-	63	1:42	2:04
Al-7	0.30	2.5	-	-	63	0:52	1:49
Al-8	0.35	5	-	1.5	63	0:08	1:33
Al-9	0.35	4	-	1.5	63	0:16	1:45
Al-10	0.35	4	-	1.0	63	0:22	1:10
Al-11	0.35	10	0.0035	-	63	0:08	0:16
Al-12	0.35	10	0.014	-	63	0:17	0:28
Al-13	0.35	10	0.0212	-	63	0:21	0:41
Al-14	0.35	10	0.028	-	63	0:29	0:49
Al-15	0.35	10	0.0353	-	63	0:28	1:00
Al-16	0.35	10	0.0389	-	63	0:36	1:12
Al-17	0.35	10	0.0424	-	63	0:35	1:14
Al-18	0.35	10	0.0460	-	63	0:50	1:36
Al-19	0.35	5	0.0035	-	63	0:15	1:52
Al-20	0.35	5	0.0070	-	63	0:17	1:50
Al-21	0.35	5	0.0106	-	63	0:20	1:40

mixes exhibited either very slow or very rapid setting characteristics. In addition, many mixes became stiff and unworkable prior to initial set and some exhibited false setting characteristics. To counteract the stiffening effect, Lomar D superplasticizer was added to improve workability. Mixes formulated with Lomar D did show improved behavior, however, final set was delayed.

The use of Plastiment retarding densifier was suggested by the cement manufacturer to provide a more consistent, workable batch. Trial mixes formulated showed sensitivity to small variations in both lithium carbonate and Plastiment content. For mixes formulated with less than 10 g/sack, lithium carbonate accelerator, at various Plastiment contents, initial set time was controllable to about 20 min, but final set time occurred at about 2 hr. In mixes with an accelerator content of 10 g/sack, initial set could be controlled within a range from 10 to 30 min with final set at about 1 hr. Figure 40 shows the variation of initial set time with Plastiment content for accelerator contents of both 5 and 10 g/sack of cement. Mixes with accelerator content of 10 g/sack within the range of 0.020 to 0.040 percent Plastiment, produced the most favorable results.

Despite a number of inconsistent and occasionally unexplained setting characteristics, aluminous cement appeared to be a potential candidate.

7.1.3.2 Portland Type III and Calcium Chloride - Eighteen different mix combinations were tested for time of setting. Mixtures with water-cement ratios of 0.30, 0.35 and 0.40 were used. Water temperature was varied from 63° to 75°F. Calcium chloride additions from 3 to 5 percent were used. Lomar D was added to some formulations in anticipation of use in the final mix design.

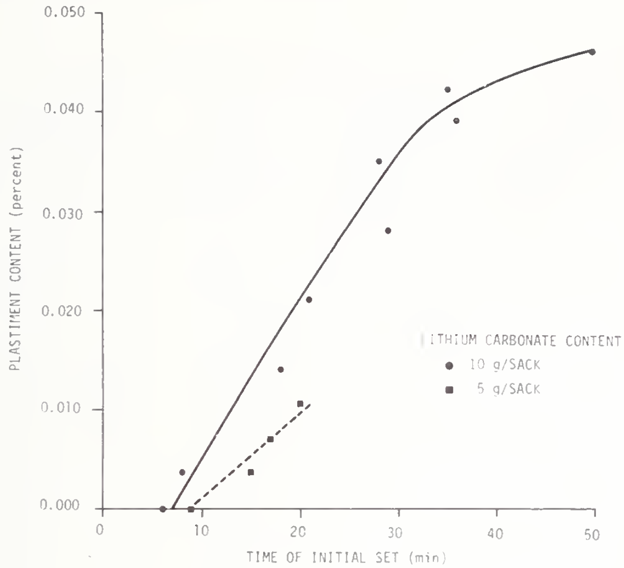


FIGURE 40. VARIATION IN SET TIME OF ALUMINOUS CEMENT MORTARS FOR DIFFERENT LITHIUM CARBONATE CONTENTS

Results of the set time tests appear in Table 5. Mortars formulated at a 0.35 water-cement ratio required calcium chloride additions of 3 to 4 percent, by weight of cement to obtain a rapid initial set. Although the modified mixes exhibited favorable set characteristics, two undesirable properties were developed. A majority of formulations at this water-cement ratio were mealy and somewhat friable at final set. In addition, all of the mixtures became very hot during the setting process although the test pats were generally only 1/2 in. thick.

Mixtures formulated with a 0.40 water-cement ratio required 4 to 5 percent calcium chloride to obtain a rapid set. Although these mixes were more workable than the 0.35 water-cement ratio formulations, excessive heat generation upon setting was still encountered.

TABLE 5. RESULTS OF TIME OF SETTING TESTS FOR PORTLAND TYPE III CEMENT MODIFIED WITH CALCIUM CHLORIDE

MIX	WATER-CEMENT RATIO	$CaCl_2$ % WEIGHT OF CEMENT	LOMAR-D LIQUID, % WEIGHT OF CEMENT	WATER TEMP (°F)	INITIAL SET HR: MIN	FINAL SET HR: MIN
Type III-1	0.30	3	-	63	0:41	1:07
Type III-2	0.35	2	-	63	1:02	1:30
Type III-3	0.35	2	-	75	1:08	1:39
Type III-4	0.35	3	-	63	0:56	1:12
Type III-5	0.35	3	-	75	0:59	1:21
Type III-6	0.35	4	-	63	0:10	1:27
Type III-7	0.35	4.5	-	63	0:04	0:53
Type III-8	0.40	4	-	63	1:04	-
Type III-9	0.40	4.5	-	63	0:41	1:38
Type III-10	0.40	5	-	63	0:15	1:16
Type III-11	0.30	4	1.0	63	0.05	0.33
Type III-12	0.35	3.5	0.0	63	0:46	1:22
Type III-13	0.35	4.0	0.5	63	0:22	1:19
Type III-14	0.35	4.0	0.75	63	0:56	1:27
Type III-15	0.35	4.0	1.0	63	0:53	1:38
Type III-16	0.35	4.0	1.5	63	1:03	1:19
Type III-17	0.35	4.5	1.0	63	0:08	0:57
Type III-18	0.35	4.5	1.5	63	0:15	1:11

In anticipation of poor workability, typical of low water-cement ratio concretes, the effect of Lomar D Superplasticizer on set time was studied. Results of the time of setting tests indicated that Lomar D delayed initial set two to five times that of mixtures without Lomar D at constant calcium chloride content. Final set, however, was not effected.

Although Type III-calcium chloride mixes exhibited definite rapid setting characteristics within the required range, the large amount of heat evolved during setting may cause thermal

TABLE 6. RESULTS OF TIME OF SETTING TESTS FOR PORTLAND TYPE I - ALUMINOUS CEMENT COMBINATIONS

MIX	% PORTLAND/ % ALUMINOUS	WATER-CEMENT RATIO	WATER TEMP (°F)	INITIAL SET HR: MIN	FINAL SET HR: MIN
PORT/Al-1	90/10	0.35	63	1:01	>4:00
PORT/Al-2	90/10	0.40	63	>2:00	-
PORT/Al-3	85/15	0.35	63	0:08	0:13
PORT/Al-4	85/15	0.40	63	0:18	0:56
PORT/Al-5	80/20	0.40	63	0:17	0:28
PORT/Al-6	75/25	0.40	63	0:12	0:18
PORT/Al-7	70/30	0.40	63	0:06	0:11
PORT/Al-8	60/40	0.40	63	0:04	1:41
PORT/Al-9	50/50	0.40	63	>1:30	-
PORT/Al-10	40/60	0.40	63	3:57	-
PORT/Al-11	20/80	0.40	63	Flash Set	-

distress and create a serious problem for set control and workability in large concrete batches. Additionally, it has been found (13) that nominal additions of $CaCl_2$ to concrete reduces sulfate resistance. For concretes modified with large amounts of $CaCl_2$ this effect might make this material undesirable for tunnel lining application.

7.1.3.3 Portland Type I - Aluminous Cement Combinations - Set time tests were performed on a number of Portland Type I/aluminous cement combinations. Portland cement content was varied from 20 to 90 percent of the total cement quantity. Water-cement ratios of 0.35 and 0.40 were used. No admixtures were used in the tests.

Most results were inconsistent and badly scattered, although 27 trials of 11 different combinations were conducted. The results of these tests appear in Table 6. Mixes would either set within a few minutes after mixing or remain plastic for a few hours. Six trials at a combination of 85 percent Portland cement and 15 percent aluminous cement produced average initial and final set times of 18 and 56 min, respectively. These values, however, represent wide variations from test to test.

One trial of a combination of 70 percent Portland and 30 percent aluminous reached initial set in 6 min while another trial showed no signs of setting after 1 hr. This type of erratic set behavior was apparent at other cement combinations and required the elimination of this candidate from further consideration.

7.1.3.4 Very High Early Cement - Set time tests were performed on VHEC mortars formulated with varying amounts of citric acid retarder. Water-cement ratio was varied from 0.35 to 0.50 and water temperature was varied from 32° to 75°F. A majority of the tests were conducted at a 0.40 water-cement ratio with 60° to 65°F temperature water. Results of the set time tests appear in Table 7.

Unretarded VHEC reached initial set in 11 min and final set in 30 min. For citric acid contents of 0.05 to 0.25 percent by weight of cement, VHEC reached initial set in a range from 15 min to 40 min. A plot of initial set time and citric acid content for selected water-cement ratios and water temperatures appears in Figure 41. Chilling the water to 32° to 39°F had a small effect on set time for the 0.40 water-cement ratio mix. Increasing the water temperature to 70° to 75°F for the same mix had a similarly small effect. Variations in water-cement ratio changed setting characteristics. Increasing water-cement ratio decreased initial set time for constant citric acid content and constant water temperature.

In general, VHEC behaved very consistently. No occurrences of flash set or excessive heat evolution were encountered. Due to the fine degree of controlability, VHEC offered the most potential for use above the other cement candidates tested.

TABLE 7. RESULTS OF THE TIME OF SETTING TESTS FOR VERY HIGH EARLY CEMENT

MIX	WATER CEMENT	CITRIC ACID % WT. OF CEMENT	WATER TEMPERATURE °F	INITIAL SET HR: MIN	FINAL SET HR: MIN
VHEC-1	0.35	0.15	63	0:11	0:36
VHEC-2	0.35	0.20	32 to 36	0:25	--
VHEC-3	0.35	0.20	63	0:17	0:54
VHEC-4	0.35	0.25	63	0:25	1:11
VHEC-5	0.40	--	34	0:10	--
VHEC-6	0.40	--	39	0:10	--
VHEC-7	0.40	--	61	0:12	0:32
VHEC-8	0.40	--	68	0:11	0:30
VHEC-9	0.40	0.05	63	0:11	0:34
VHEC-10	0.40	0.10	63	0:23	1:04
VHEC-11	0.40	0.10	72	0:17	0:57
VHEC-12	0.40	0.20	32 to 36	0:35	1:14
VHEC-13	0.40	0.20	61 to 64	0:31	1:21
VHEC-14	0.40	0.20	68	0:22	1:02
VHEC-15	0.40	0.20	72	0:41	1:39
VHEC-16	0.40	0.20	75	0:25	1:21
VHEC-17	0.40	0.25	32 to 36	0:42	1:36
VHEC-18	0.40	0.25	61 to 63	0:44	1:45
VHEC-19	0.40	0.25	63	0:36	1:32
VHEC-20	0.45	0.05	63	0:19	0:42
VHEC-21	0.45	0.01	63	0:35	1:01
VHEC-22	0.45	0.15	63	0:39	1:17
VHEC-23	0.45	0.20	32 to 36	1:09	--
VHEC-24	0.45	0.20	63	0:55	1:37
VHEC-25	0.45	0.25	63	0:48	1:47
VHEC-26	0.47	0.20	68	1:21	2:16
VHEC-27	0.50	0.00	63°	0:24	1:10
VHEC-28	0.50	0.05	63°	0:43	1:10
VHEC-29	0.50	0.10	63°	0.42	1:20
VHEC-30	0.50	0.20	32° to 36°	1:41	--
VHEC-31	0.50	0.25	61°	1:21	2:38
VHEC-32	0.50	0.50	63°	> 1 hr	--

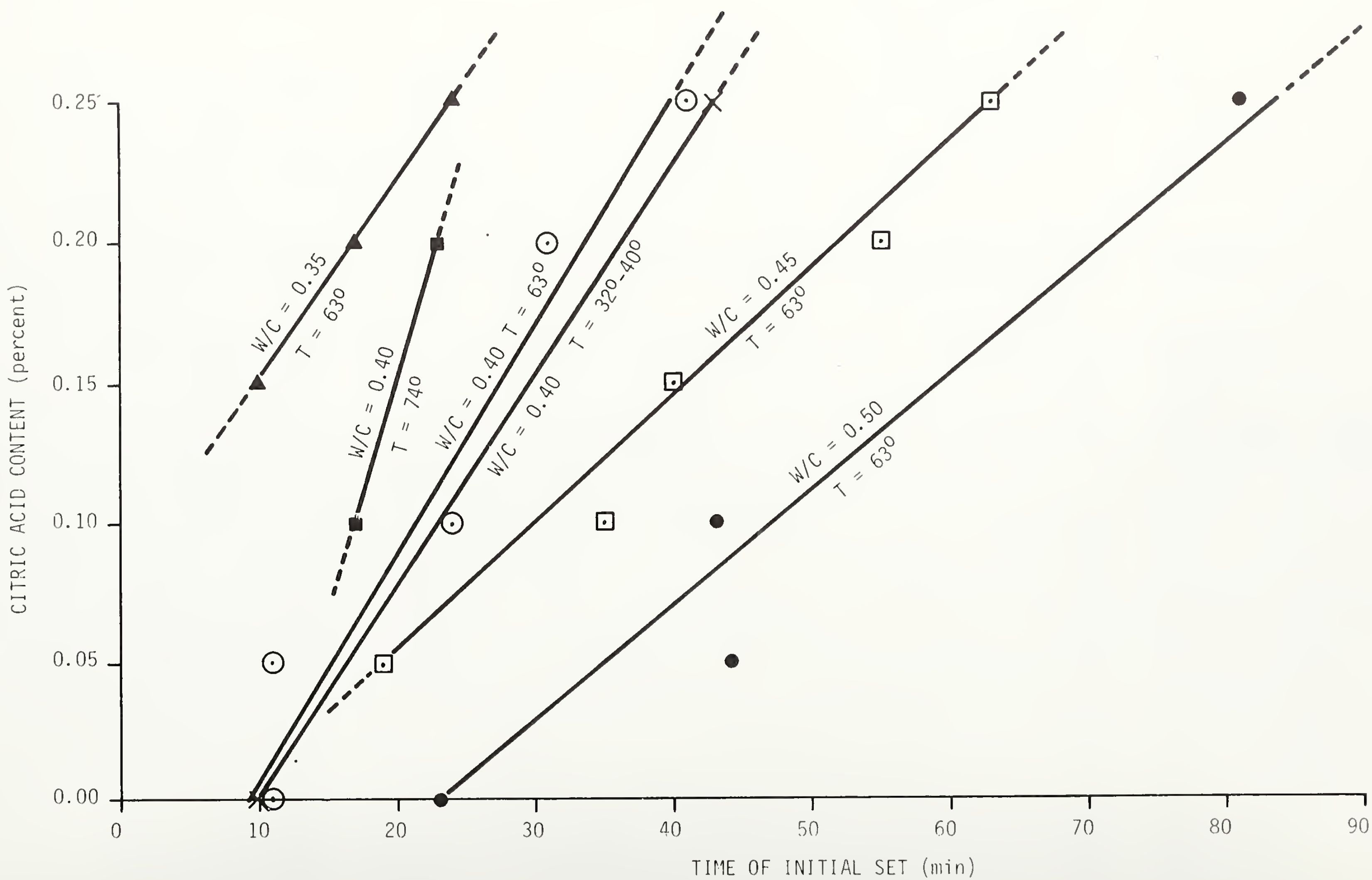


FIGURE 41. VARIATION IN SET TIME OF VHEC MORTARS FOR DIFFERENT WATER-CEMENT RATIOS AND WATER TEMPERATURES

Further testing to measure workability and strength-time development was conducted. The results of these tests follows in Subsection 7.3.3.

7.2 MATERIALS

7.2.1 Aggregates

The coarse and fine aggregates used for the material testing program were obtained from local sources. The coarse aggregate was combined from two separate gradations and the maximum particle size was 3/4 in. The sand was a well-graded, bank run deposit.

A series of tests were performed to obtain aggregate gradations, specific gravities, and absorption contents. The results of these are listed in Tables 8 and 9. Aggregates conform to ASTM C33.

Pumpable concrete can be formulated by choosing the densest combination of aggregates. Unit weight tests (ASTM C29) were conducted to determine the proportions of coarse and fine aggregate that produce the densest mix. A combination of approximately 55 percent coarse aggregate and 45 percent fine aggregate yielded the densest mix for this maximum aggregate size. Figure 42 shows the combined aggregate gradation and the gradation limits for pumpable concrete as recommended by ACI Committee 304. The 55 percent coarse - 45 percent fine aggregate gradation lies within the 3/4-in. maximum aggregate band at all particle sizes. This aggregate gradation was used in all tests throughout the program.

TABLE 8. SIEVE ANALYSIS OF SAND AND GRAVEL

	SAND, PERCENT		COARSE AGGREGATE NOMINAL MAXIMUM-SIZE 3/4 - in. PERCENT	
SIEVE	RETAINED	PASSING	RETAINED	PASSING
1 - in.	0	100	0	100
3/4 - in.	0	100	6	94
1/2 - in.	0	100	44	56
3/8 - in.	0	100	62	38
No. 4	2	98	96	4
No. 8	15	85	98	2
No. 16	31	69	100	0
No. 30	54	46	100	0
No. 50	72	28	100	0
No. 100	92	8	100	0
No. 200	98	2	100	0
Fineness Modulus	2.66		7.06	

TABLE 9. PROPERTIES OF SAND AND GRAVEL

PROPERTIES	SAND	GRAVEL
Unit Weight, lb/ft^3	100.5	100.0
Bulk Specific Gravity, SSD	2.63	2.64
Absorption Capacity, Percent	1.3	1.5

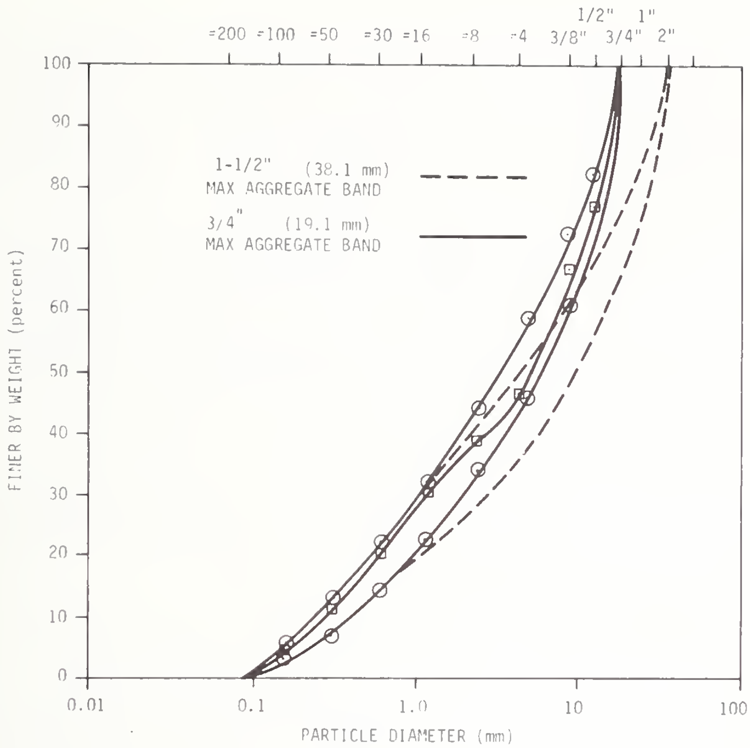


FIGURE 42. RECOMMENDED COMBINED NORMAL WEIGHT AGGREGATE GRADATION FOR PUMPED CONCRETE (ACI-304-74)

7.2.2 Cements and Fly Ash

Portland Types I and III, aluminous, and VHEC were used in the testing program.

Portland Type I and III cements were obtained locally and conform to ASTM C150.

The aluminous cement was obtained from Lone Star Lafarge Company, Norfolk, VA. The specific gravity of this material is approximately 3.2.

VHEC is a proprietary product of the United States Gypsum Company, Chicago, IL. The material is a modified Portland cement. The specific gravity of VHEC is approximately 3.0. VHEC is somewhat finer than the other cements used in the testing program.

The fly ash used for the laboratory work was not locally available in bagged form. The material that was acquired was shipped in small barrels. The fly ash has been tested and conforms to Pozzolan Class F of ASTM C618. The specific gravity is 2.35.

7.3 INITIAL STRENGTH-TIME DEVELOPMENT AND WORKABILITY MEASUREMENTS

7.3.1 Introduction

In addition to meeting the set time requirements, the concrete for use in the ETLS must be pumpable and remain workable for proper distribution in the form. In order to insure adequate pumpability over a range of pumping rates, the concrete should remain workable for about 20 to 25 min. Once distributed in place, the concrete must gain rapid strength over a short period of time. The tunnel lining concrete should develop about 500 $lb/in.^2$ compressive strength in about 2 hr.

A series of concrete batches were tested in the laboratory to assess workability and strength-gain properties. Concrete batches were formulated using the cement candidates that exhibited satisfactory performance in the time of setting tests.

Trial mixes were tested for slump loss with time up to 30 min and for early strength development in compression for ages up to 2 hr.

At the completion of the trial mix formulation tests a single cement candidate concrete was chosen for final mix design formulation.

7.3.2 Testing Procedure

Trial concrete formulations were batched and mixed in a 3.5 ft^3 tumble mixer. The majority of the trials used 1.2 and 1.5 ft^3 test batches.

A standard mixing procedure was adopted for all of the tests. All of the aggregates and one-third of the water were charged into the mixer. After about 30 sec of tumbling, the fly ash, cement, and a second quantity of water were added. Admixtures, when used, were placed into separate portions of the remaining mix water and placed in the mixer. Formulations were mixed for 3 min from time of addition of cement. Mixes were examined to verify a homogeneous mixture. Batches were dumped into a mortar pan, and the mixer was cleaned immediately.

Samples were taken from the mortar pan to conduct slump (ASTM C143), unit weight (ASTM C138), and air content (ASTM C231) tests. In addition, three, 6 in. diam 12 in. compressive cylinder specimens were cast. Upon completion of the slump test, the concrete was returned to the mortar pan and remixed by hand. Concrete used for the unit weight and air content tests was discarded. The slump test was repeated at 15, 20, 25 and 30 min, or until the concrete reached a slump of less than 1-1/2 in.

Compressive test cylinders were stored under polyethylene sheeting until test times of 1, 1-1/2, and 2 hr. Cylinders were stripped just prior to testing. When possible, specimens were capped. Compression testing was performed using a portable testing machine.

Revisions in the mix design changing cement content, water-cement ratio, and admixture content were made to improve concrete quality.

7.3.3 Results of Testing

7.3.3.1 Aluminous Cement - A total of 16 trial batches were tested using a combination of aluminous cement, lithium carbonate accelerator, and plastiment retarding densifier. Lomar D Superplasticizer was used in some formulations. Mixes typically contained an 8-sack cement content at a 0.35 water-cement ratio.

In general, the trial mixtures exhibited either poor workability with reasonable strength-gain, or, good workability with poor strength-gain. In some cases, mixes exhibited both poor workability and poor strength-gain. The various proportions of the mix trials and measured material properties appear in Table 10.

Trial mix No. 4 contains an admixture combination of 12 g/sack lithium carbonate accelerator and 2.0 oz/sack Plastiment. This formulation produced a concrete with poor workability and only moderate strength-gaining ability. The subsequent trial (No. 5) contains twice as much Plastiment at constant accelerator content and yielded an attractive workability range. Strength-gain, however, was very poor. Small variations in admixture quantities in trials No. 6, 7, and 8 offered little improvement.

Some mixes were formulated with Lomar D Superplasticizer, both with and without Plastiment. Only trials No. 13 and 15 exhibited reasonable workability with limited strength-gain. The remainder of the trials generally exhibited poor workability.

Aluminous cement concrete was eliminated at this point in the testing program because of its poor performance. Although there may be some combinations that might be more attractive in terms of workability and strength-gain, a much more extensive testing program with other admixtures and combinations would be

TABLE 10. INITIAL ALUMINOUS CEMENT CONCRETE FORMULATION TRIALS

MIX NO.	NOMINAL BATCH SIZE FT^3	WATER-CEMENT RATIO	CEMENT FACTOR SACKS	LITHIUM CARBONATE CONTENT, GRAMS/SACK	BATCH QUANTITIES, LB.					PLACEMENT, FLUID-OZ	AIR CONTENT, PERCENT	UNIT WEIGHT, LB/FT^3	SLUMP, INCHES					COMPRESSIVE STRENGTH, LB/IN.^{2} AGE			
					WATER	CEMENT	GRAVEL[a]	SAND[a]	LOMAR-D LIQUID				INITIAL	15 MIN	20 MIN	25 MIN	30 MIN	1 HR	1-1/2 HR	[illegible] HR	[illegible] HR
1	1.2	0.44	7	10	12.8	[illegible]	75.6	65.0	--	0.50	[illegible]	144.2	2-1/2	[illegible]	[illegible]	[illegible]	--	--	--	[illegible]	--
2	1.2	0.35	7.6	12	11.1	31.8	76.1	[illegible]	--	1.08	--	151.4	8	--	--	--	[illegible]	--	--	[illegible]	--
3	1.0	0.375	7	10	9.2	[illegible]	62.4	[illegible]	--	0.31	-	--	[illegible]	[illegible]	--	--	--	--	--	--	--
4	1.2	0.35	8	12	11.7	33.4	71.6	[illegible]	--	0.67	[illegible]	144.5	[illegible]	--	--	--	--	[illegible]	--	[illegible]	--
5	1.2	0.35	8	12	11.7	33.4	71.6	[illegible]	--	1.34	[illegible]	[illegible]	[illegible]	4-1/4	3-1/4	[illegible]	[illegible]	--	[illegible]	[illegible]	--
6	1.2	0.38	8	14	12.7	33.4	71.6	[illegible]	--	[illegible]	[illegible]	151.6	[illegible]	1	--	--	--	24	[illegible]	[illegible]	--
7	1.2	0.35	8	14	11.7	33.4	71.6	[illegible]	--	1.28	[illegible]	151.6	[illegible]	1-1/4	--	-	--	24	1	[illegible]	--
8	1.2	0.35	8	16	11.7	33.4	71.6	[illegible]	--	2.11	[illegible]	151.4	6-3/4	6-1/2	[illegible]	[illegible]	4-3/4	-	11	14	--
9	1.2	0.35	8	10	11.7	33.4	71.2	[illegible]	[illegible]	--	--	--	1	--	--	--	--	[illegible]	[illegible]	[illegible]	--
10	1.2	0.35	8	5	11.7	33.4	72.7	[illegible]	[illegible]	--	2.3	149.7	2-1/4	1	--	-	-	[illegible]	[illegible]	[illegible]	--
11	1.2	0.35	8	10	11.7	33.4	72.5	[illegible]	0.33	0.66	2.3	149.7	[illegible]	--	--	--	--	44	[illegible]	[illegible]	--
12	1.2	0.35	8	8	11.7	33.4	72.5	[illegible]	[illegible]	0.85	[illegible]	148.5	2	-	--	--	-		[illegible]	[illegible]	--
13	1.2	0.35	8	8	11.7	33.4	72.5	[illegible]	0.33	[illegible]	2.0	[illegible]	4-3/4	[illegible]	[illegible]	--	[illegible]	14	[illegible]	[illegible]	--
14	1.2	0.35	8	10	11.7	33.4	[illegible]	[illegible]	0.33	1.07	2.6	[illegible]	[illegible]	--	--	-	--	[illegible]	[illegible]	[illegible]	--
15	1.2	0.37	[illegible]	9	12.4	33.4	[illegible]	[illegible]	[illegible]	1.07	[illegible]	149.8	[illegible]	4-1/4	3-3/4	1-3/4	--	[illegible]	17	[illegible]	--
16	1.2	0.36	8	9	12.0	33.4	72.6	[illegible]	[illegible]	1.07	3.3	148.5	6-3/4	[illegible]	--	--	--	[illegible]	42	[illegible]	--

[a]Saturated, Surface Dry

required. If it appeared that aluminous cement concretes exhibited the best potential for use above the other candidates, a more extensive testing program would have been initiated.

7.3.3.2 VHEC - A series of trial batches were formulated with VHEC. Cement contents of 6, 7, and 8 sacks per cubic yard of concrete were used. Typically, mixes were formulated with citric acid additions of 0.275 to 0.40 percent by weight of cement. Water-cement ratio was varied from 0.40 to 0.50. In some formulations, Lomar D Superplasticizer was added to improve workability. Tap water at 60° to 65°F was used for all tests.

Twenty-two trial batches were formulated. Table 11 lists the mix batch quantities, slump loss with time data, and the compressive strength data for the trial formulations.

The first few trials were formulated for workability only. Once a satisfactory workability was obtained, subsequent trials were reformulated with lower citric acid content, increased cement content, lower water-cement ratio, or a combination of one or more to improve early strength-gain. The addition of a superplasticizer was necessary in some instances to maintain adequate workability.

Mixes that contained an 8-sack cement content at a 0.40 water-cement ratio with either 0.275 or 0.300 percent citric acid and 2 percent Lomar D appeared to be the best mix designs. These mixes had an initial slump of about 7 in. and maintained adequate workability for about 25 min.

The strength-gaining characteristics of the VHEC concretes were favorable. In general, VHEC concretes gained most of its early strength between 1-1/2 and 1-3/4 hr after mixing. Figure 43 shows the trend of early strength-gain with time for the typical trial formulations that exhibited satisfactory workability.

TABLE 11. INITIAL VHEC CONCRETE FORMULATION TRIALS

MIX NO.	NOMINAL BATCH SIZE, FT^3	WATER-CEMENT RATIO	CEMENT FACTOR, SACKS	CITRIC ACID CONTENT, PERCENT	BATCH QUANTITIES, LB					AIR CONTENT, PERCENT	UNIT WEIGHT, LB/FT^3	SLUMP, IN.					COMPRESSIVE STRENGTH $LB/IN.^2$ AGE			
					WATER	CEMENT	GRAVEL[a]	SAND[a]	SUPERPLASTICIZER[b]			INITIAL	15 MIN	20 MIN	25 MIN	30 MIN	1 HR	1-1/2 HR	2 HR	3 HR
1	1.2	0.686	6	0.5	17.1	24.9	66.5	66.5	--	--	--	9	8	6-1/2	5	--	--	--	--	--
2	1.2	0.51	6	0.4	12.7	24.9	66.5	66.5	--	--	--	4	2	--	--	--	--	--	--	--
3	1.2	0.50	6	0.4	12.5	24.9	73.5	60.5	--	--	--	3-1/2	2	--	1/2	--	--	--	--	--
4	1.2	0.50	6	0.5	12.5	24.9	73.5	60.5	--	--	--	5-1/2	3-3/4	2-3/4	2-1/4	1-1/2	--	--	--	--
5	1.2	0.50	6	0.4	12.5	24.9	73.5	60.5	0.25	--	--	8-1/2	7-1/2	5	--	3	--	--	--	--
6	1.2	0.50	6	0.4	12.5	24.9	73.5	60.5	--	2.9	147.2	5-3/4	3	2-1/2	1-1/2	--	--	--	37	--
7	1.2	0.45	7	0.4	13.1	29.2	71.0	59.3	--	2.6	147.2	5-1/2	1-3/4	--	--	--	--	--	386	--
8	1.2	0.45	7	0.4	13.0	29.2	71.0	59.3	0.13	4.2	145.1	7-1/2	6	3-3/4	--	2-1/2	--	--	25	--
9	1.2	0.45	8	0.45	15.0	33.4	66.4	55.3	--	2.3	146.9	5-3/4	3-1/2	--	1-3/4	--	--	--	52	--
10	1.2	0.44	8	0.40	15.2	33.4	66.3	55.2	0.08	3.9	144.8	6-1/2	2	--	1	--	--	--	108	--
11	1.2	0.45	8	0.35	15.0	33.4	66.4	55.3	0.08	3.6	145.1	6-1/2	2-1/4	--	1	--	--	--	1441	--
12	1.2	0.45	8	0.30	14.9	33.4	66.4	55.3	0.18	3.9	146.3	6-1/2	--	1-1/4	--	--	--	--	2812	--
13	1.2	0.45	8	0.00	15.0	33.4	66.4	55.3	0.10[c]	--	--	2-1/2	--	--	--	--	--	--	3300	--
14	2.0	0.45	8	0.35	25.0	55.7	111.8	93.2	0.28	4.3	144.0	6-1/2	1-1/2	--	--	--	10	41	1291	3343
15	1.2	0.45	8	0.35	14.8	33.4	66.4	67.1	0.33	4.9	144.0	7-1/4	4	2-3/4	2-1/2	2	--	35	424	--
16	1.2	0.45	8	0.30	14.8	33.4	66.4	55.3	0.33	4.8	143.4	7	3-3/4	--	1-3/4	1-1/4	--	37	1106	--
17	1.2	0.45	8	0.25	14.7	33.4	66.4	55.3	0.50	4.4	144.2	8	4-3/4	--	2-1/4	--	--	171	2264	--
18	1.5	0.40	8	0.25	16.3	41.8	93.5	67.4	0.63	4.3	143.4	5-3/4	2-1/4	1	--	--	27	1203	3378	--
19	1.5	0.41	8	0.25	16.5	41.8	93.4	67.3	0.84	--	--	6-3/4	2-3/4	1-1/4	--	--	33	1537	3042	--
20	1.5	0.40	8	0.30	16.2	41.8	87.9	71.9	0.84	4.9	141.9	6-3/4	3-1/4	2-1/4	1-1/2	--	21	255	3254	--
21	2.0	0.40	8	0.30	21.7	55.7	117.1	95.9	1.11	4.5	142.5	8	4-1/2	3-1/2	2-1/2	2	22	275	2750	--
22	2.0	0.40	8	0.275	21.7	55.7	117.1	95.9	1.11	6.0	140.7	7-1/2	4	3-1/4	2-1/2	2	18	1238	2874	--

[a] Saturated Surface Dry

[b] Lomar D Liquid Unless Otherwise Noted

[c] Plastiment Liquid

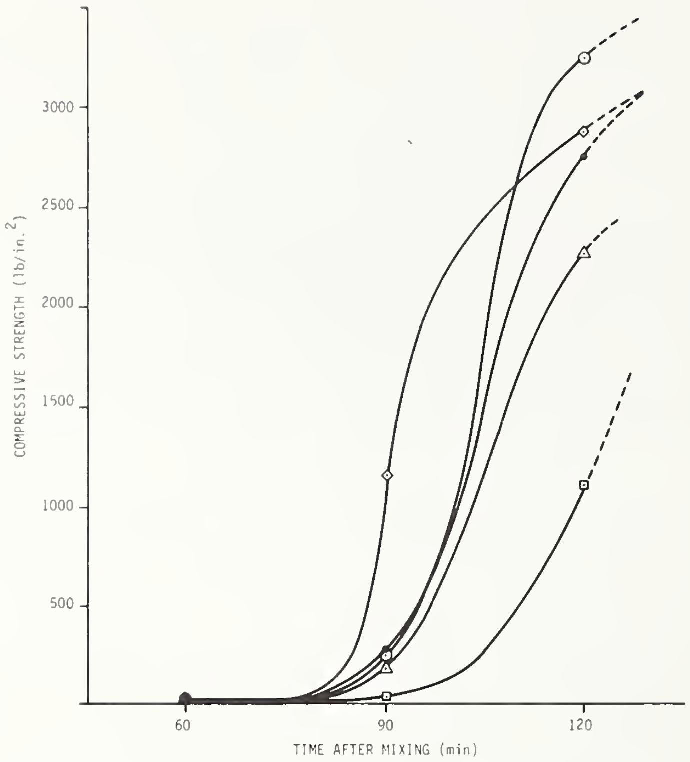


FIGURE 43. EARLY STRENGTH-GAIN WITH TIME FOR TYPICAL FORMULATIONS

Strength-gain was usually very rapid at 1-1/2 hr. Mixes that exhibited less than 25 min workability showed a sharp increase in strength before 1-1/2 hr whereas mixtures with excessive workability did not exhibit rapid increases until at least 1-3/4 hr. Most trial mixes exhibited 2 hr strength greatly in excess of the 500 lb/in.^{2} requirement.

Although these mix designs offered both excellent workability and rapid strength-gain, several undesirable properties were noticed. First, the air content of all of the superplasticized batches was high. Although high air content improves workability, it may be detrimental for pumped placements. High air content may also reduce ultimate strength although the early values seem unaffected. Secondly, an 8-sack cement mix with VHEC was very sticky in consistency. This property may create high pipeline friction forces during pumping. Finally, due to the large amount of cement in the mix, the admixtures were not always effective. On more than one occasion batch to batch variation in workability was significant.

The mix designs developed during this part of the testing program were very promising. Further mix design work was necessary to address some of the problems in mix properties before final application.

7.4 FINAL MIX DESIGN - VHEC CONCRETE

The mix design developed during the preliminary design investigation was an 8-sack, 0.40 water-cement ratio mix with 0.275 to 0.300 percent citric acid and 2 percent Lomar D Superplasticizer. Although this formulation easily met the early strength and workability requirements, the mixes exhibited some undesirable properties as discussed in subsection 7.3.3.2. Basically these properties included: a sticky mix consistency, inconsistent plastic behavior, and excessive air content.

In an attempt to remedy at least the first two problems, mixes were reformulated with fly ash as a partial cement replacement. The batch quantities and results of the workability and strength tests appear in Table 12. Reducing the cement content of a mix generally lowers early strength, however, it was apparent from the preliminary testing that an 8-sack mix was exhibiting more than adequate early strength.

TABLE 12. FINAL VHEC CONCRETE FORMULATION TRIALS

MIX NO.	NOMINAL BATCH SIZE FT^3	CEMENT FACTOR				BATCH QUANTITIES (LB)						AIR CONTENT, PERCENT	UNIT WEIGHT, LB/FT^3	SLUMP (IN.)					COMPRESSIVE STRENGTH ($LB/IN.^2$) AGE			
		WATER-CEMENT RATIO	SACKS CEMENT	SACKS FLY ASH	CITRIC ACID CONTENT, PERCENT	WATER	CEMENT	FLY ASH	GRAVEL[a]	SAND[a]	SUPER-PLASTICIZER[b]			INITIAL	15 MIN	20 MIN	25 MIN	30 MIN	1 HR	1-1/2 HR	2 HR	3 HR
23	2.0	0.39	8	0	0.275	21.2	55.7	-	117.2	95.9	1.11	5.4	141.6	7	2-1/2	2-1/2	2	-	24	416	2750	-
24	1.2	0.42	7	1	0.275	13.7	29.2	4.2	70.3	56.8	0.67	4.3	143.7	7-1/4	3-3/4	2-1/4	1	-	212	2476	2812	-
25	1.2	0.42	7	0	0.275	12.0	29.2	-	72.7	58.7	0.58	4.9	144.0	7	4-1/4	3-1/4	2-3/4	1-1/2	14	42	1397	-
26	1.5	0.41	7	1	0.275	16.5	36.5	5.3	87.9	71.0	0.83	4.6	143.4	8-3/4	6-1/2	6	5-3/4	4-3/4	18	42	1150	-
27	1.5	0.40	7	1	0.275	16.3	36.5	5.3	87.9	71.0	0.83	4.6	143.1	7-1/4	4	3-1/4	2-3/4	2	18	98	2175	-
28	1.5	0.40	6-1/2	1-1/2	0.25	16.1	33.9	7.9	87.9	71.0	0.83	5.0	143.1	6-3/4	4	2-3/4	2	1-1/4	27	672	2264	3166
29	1.5	0.43	6-1/2	1/2	0.25	15.3	33.9	2.6	96.8	79.1	0.73	5.5	143.1	6-1/2	3-3/4	2-1/2	2-1/4	1-1/2	20	205	1999	-
30	1.5	0.40	6-1/2	1	0.225	15.3	33.9	5.2	95.4	78.7	0.78	5.4	143.9	6-1/4	2-1/2	1-3/4	-	-	28	1433	2317	3219
31	1.5	0.40	6-1/2	1	0.25	15.3	33.9	5.2	95.4	78.7	0.78	5.4	142.2	6-1/2	3-1/4	2-1/2	2-1/4	1-1/4	24	654	2175	2388
32	1.5	0.40	6-1/2	1	0.25	15.4	33.9	5.2	95.4	78.7	0.78	5.6	141.9	-	5-3/4	3-1/4	2-1/2	1-3/4	18	126	1733	-
33	1.5	0.40	6-1/2	1	0.25	15.3	33.9	5.2	95.4	78.7	0.78	5.3	142.5	6	2-1/4	1-3/4	3/4	-	37	1645	2600	-
34	1.5	0.40	6-1/2	1	0.25	15.5	33.9	5.2	95.4	78.7	0.78	5.3	143.1	7	4	3	2-1/2	2	20	135	1150[d]	1698[e]
35	1.5	0.43	6-1/2	1	0.25	16.0	33.9	5.2	95.4	78.7	1.17[c]	3.2	144.8	5-3/4	2	1-1/4	-	-	-	796	2122	-
36	1.5	0.44	6-1/2	1	0.25	16.2	33.9	5.2	95.4	78.7	1.57[c]	3.4	144.5	6-3/4	2-3/4	2-1/4	1-1/4	-	-	584	973[d]	-

[a] Saturated, surface dry.
[b] Lomard liquid unless otherwise noted.
[c] Melment superplasticizer
[d] 95 min test age
[e] 100 min test age

By reformulating the mix to include one sack of fly ash and seven sacks of cement, the percentage of citric acid required for constant workability was reduced. This reduction in citric acid, in turn, increased early strength development thus reducing the need for seven sacks of cement. Consequently, the cement content was reduced to six and one half sacks while maintaining a one sack fly ash content. This new formulation produced superior consistency and reduced variations in workability. Strength-gain was still in excess of the requirement.

Although the reformulation produced an improved, more economical mix design, the air content remained high. Two mix trials were formulated with Melment Superplasticizer in place of Lomar D. Both mixes exhibited reduced air content, however, twice as much Melment was required to match the workability of an identical mix formulated with Lomar D. More experimentation with the mix design would be necessary to make a proper evaluation of the potential use of Melment.

The final mix design is presented in Table 13. This mix design was used throughout the remainder of the testing program.

TABLE 13. FINAL MIX DESIGN*

WATER-CEMENT RATIO	MIX MATERIALS, LB/YD^3						
	WATER	CEMENT	FLY ASH	SAND**	GRAVEL**	CITRIC ACID	LOMAR D LIQUID
0.40	273	611	94	1290	1575	1.5	14.1

Notes: *Properties of Fresh Concrete:

Air Content, Percent 4.6
Slump, In. 7
Unit Weight, lb/ft^3 142.5

**Saturated, Surface Dry

7.5 STRENGTH-TIME TESTING

7.5.1 Casting, Curing, and Testing

A series of strength-time tests were performed on the final mix design. The mix design is reported in Table 12. Both compressive and flexural strength tests were conducted at a number of test times.

Compressive strength specimens were 6-in. diam 12-in. long cylinders. Flexural strength specimens were prisms 6- by 6- by 21-in. Flexural specimens were loaded at one-third points on an 18-in. span.

Testing times of 1-1/2, 2, 3, 4 hr and 1, 3, 7, 28, and 90 days were used. For each test time three specimens were cast for both compressive and flexural tests. Samples were randomized so that no two test specimens were cast from the same batch within either test group.

Mixes were batched by the same procedure discussed in subsection 7.3.2. Unit weight and air content was checked on random batches. After casting, specimens were covered with polyethylene sheet until stripping for testing or storage. Short-term specimens were stripped from the molds immediately prior to testing time. Long-term specimens were stripped at 4 hr and placed in a fog cabinet meeting ASTM C511 until test time.

Compressive and flexural strength tests were performed on a portable testing machine. Loading was applied by hand pump. Test procedures were kept as constant as possible.

7.5.2 Results of Testing

The average values of the compressive and flexural strength tests appear in Table 14. Compressive cylinder test results are plotted with time in Figure 44.

TABLE 14. COMPRESSION AND FLEXURAL STRENGTH OF THE VHEC CONCRETE

	STRENGTH, lb/in.^{2}								
	1-1/2 HR	2 HR	3 HR	4 HR	1 DAY	3 DAYS	7 DAYS	28 DAYS	90 DAYS
Compression*	125	1295	2694	3242	3867	4298	4828	5271	6255
Flexural**	74	251	377	444	508	577	644	890	839

*6 in. diam by 12-in. cylinder

**6 by 6 in. by 21-in. beam

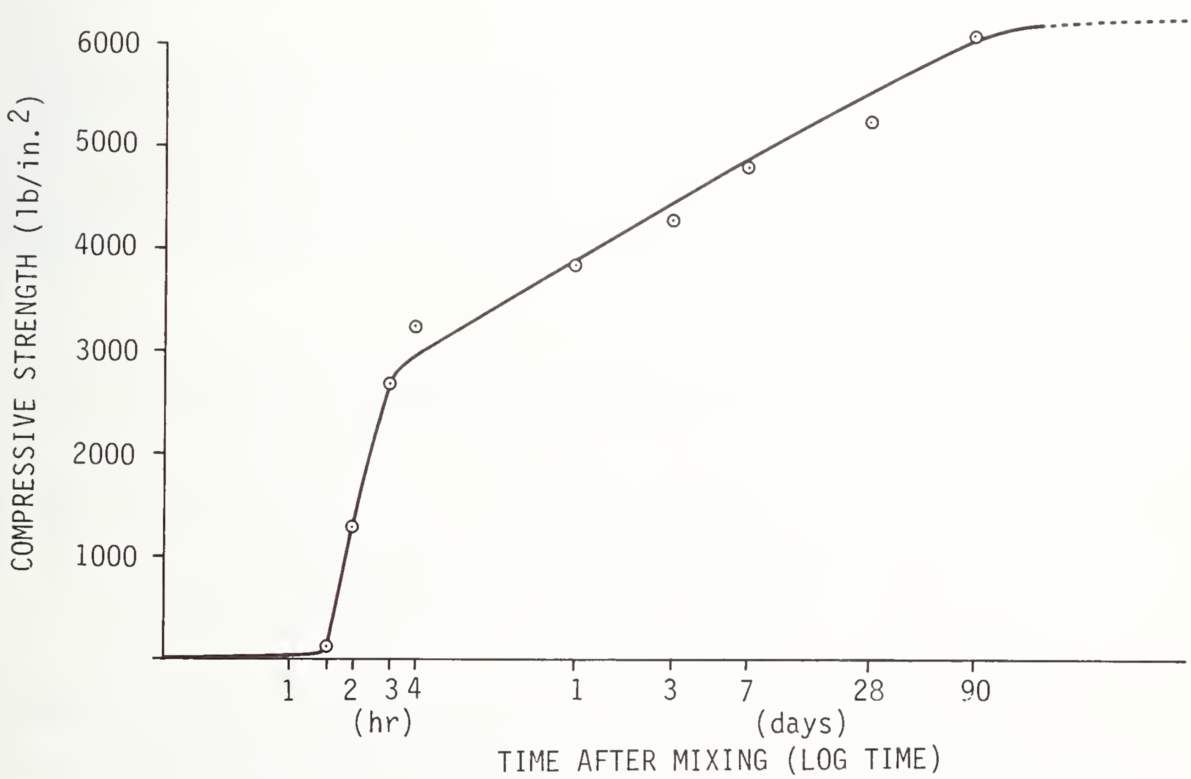


FIGURE 44. VHEC CONCRETE COMPRESSIVE STRENGTH

Strength-gain was very rapid from 1-1/2 through 4 hr. The 2 hr strength was slightly less than 1300 lb/in.^{2}, well above the 500 lb/in.^{2} specification. The 28-day strength was approximately 5300 lb/in.^{2} and somewhat lower than expected. There are two possible explanations. The reactivity of the fly ash in the mix may not be noticeable until after 28-days. In addition, the high air content of the mix may reduce the expected ultimate strength.

The flexural strengths were determined by computing the modulus of rupture values obtained from the third point loading tests. The gain in flexural strength follows the same pattern as the compressive strengths except that some values through the 28-day test are high in relation to companion compressive strengths. Flexural strength is sensitive to excessive or uneven loading rates that can occur when using a portable testing apparatus.

7.6 DURABILITY TESTS

7.6.1 Resistance of VHEC Concrete to Prolonged Water Exposure

VHEC is formulated with an unknown quantity of gypsum. Although the gypsum is not thought to exist in a free state, there may be some question as to the durability of VHEC concretes under continuous exposure to water.

A test series comparing the compressive strength of water-cured and fog-cured specimens was initiated. Five sets of six, 6-in. diam 12-in. long cylinders were cast from the standard mix design, Table 13. From each set of six, three cylinders were fog-cured in the laboratory curing cabinet and three were submerged in a tap water bath at 70°F. The tap water bath was changed weekly.

At intervals of 14, 28, 56, 84, and 112 days a set of three specimens were withdrawn from each curing environment and tested for compressive strength.

Results of the testing through the 84 day test time appear in Table 15. The water-cured specimens exhibit only slightly lower compressive strength than companion fog-cured specimens. At ages up to 28 days there is no noticeable difference. Within the group of water-cured specimens there is a slight increase in strength to 56-days. Beyond 56-days the compressive strength is fairly constant.

Based on the results of the tests to date it does not appear that the final VHEC concrete will undergo any significant deterioration from prolonged exposure to water. A longer term testing program will allow a more definitive analysis.

7.6.2 Sulfate Resistance of VHEC Concrete

A concrete tunnel liner placed below the water table may be subject to attack from sulfate bearing groundwater. A laboratory program was initiated to study the susceptibility of VHEC concrete to sulfate attack.

In conventional Portland cements, hydrated calcium aluminate (C_3A) will react with sulfate ions to form calcium sulfoaluminate. The consequence of this reaction is that the newly formed substance takes up a larger volume than the reactants causing

TABLE 15. COMPRESSIVE STRENGTH OF THE VHEC CONCRETE DURABILITY SPECIMENS

	COMPRESSION[a] STRENGTH, LB/IN.^{2}				
	14 DAYS	28 DAYS	56 DAYS	84 DAYS	112 DAYS
Fog-Cured	5011	5182	5654	5884	6792
Water-Cured	5100	5153	5365	5288	5736

[a]6-in. diam by 12-in. cylinder

expansion and cracking. Long-term exposure causes continual expansion leading to extensive deterioration.

The testing program monitored the expansion and weight change of specimens continuously submerged in a 10 percent solution of sodium sulfate. Control specimens were placed in a plain tap water solution. Specimen geometry was approximately 3 by 3 by 10 in. Specimens were cast using the final mix design concrete.

After initial length and weight measurements were taken, the specimens were placed in the sulfate and tap water solutions. Specimens were withdrawn for measurements every 7-days.

Expansion and weight change of the specimens through 91 days are shown in Figures 45 and 46, respectively. No noticeable distress was observed on any of the specimens. The results indicate that the specimens stored in the sulfate solution exhibit only slightly more expansion than the control specimens. Weight is increasing slowly for both specimen sets.

The results through 91 days indicate that VHEC concrete is fairly resistant to sulfate attack. Since deterioration due to sulfate is a long-term process, the test specimens will continue to be submerged and periodically measured.

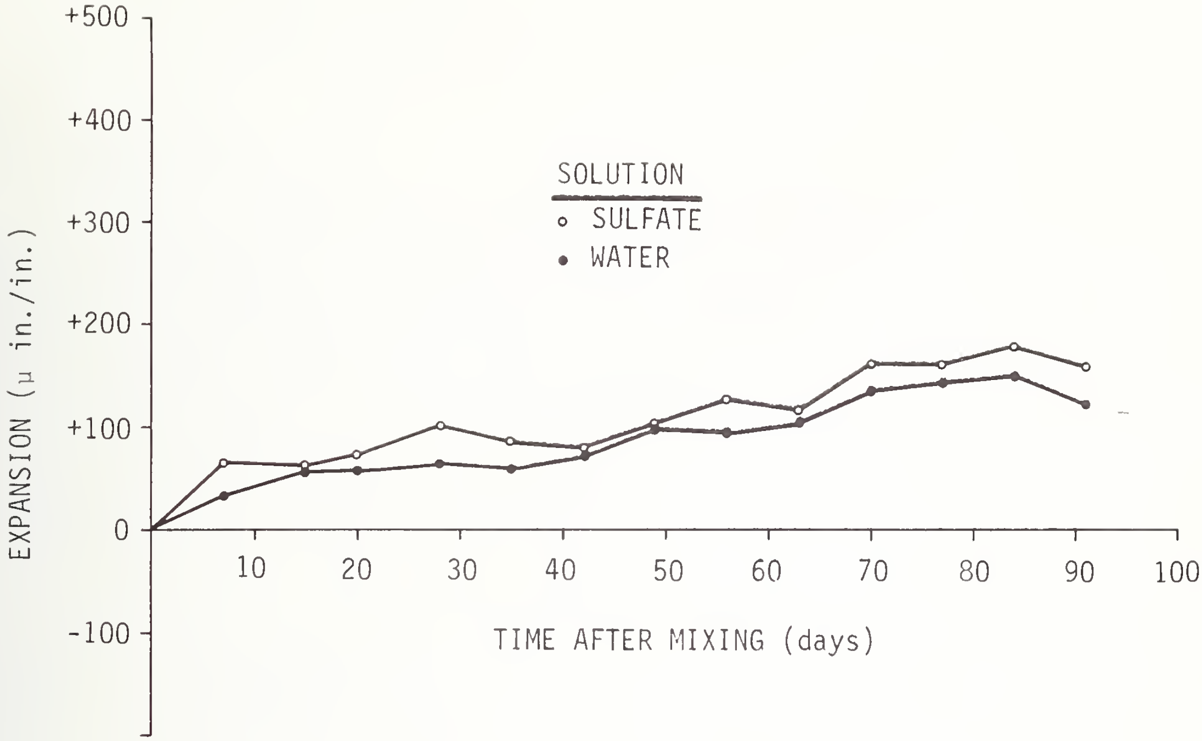


FIGURE 45. EXPANSION OF VHEC CONCRETE

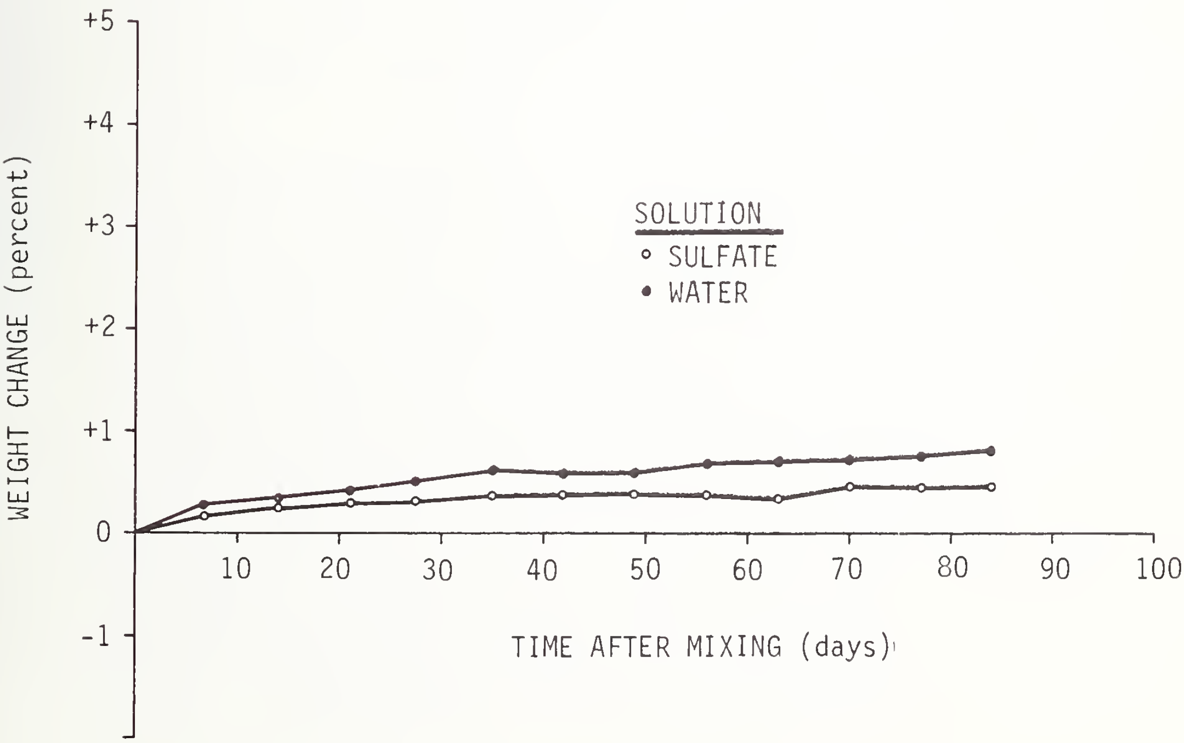


FIGURE 46. WEIGHT CHANGE FOR VHEC CONCRETE

8. CONCRETE DISTRIBUTION AND SLIPFORMING TESTS

8.1 INTRODUCTION

The distribution and slipforming tests were devised to answer questions regarding the feasibility of the extruded tunnel liner concept for which the state-of-the-art review offered no answers. The test plans, test rigs, and results for each of these tests are discussed in detail in the following subsections. In addition to the test rigs fabricated specifically for each test, two pieces of commercial construction equipment, a concrete pump and a proportioning mixer, were modified for use in these tests.

The concrete pump was a standard Challenge-Cook Squeeze-Crete-200 shown in Figure 47. The controls furnished with the pump enabled the pumping rate to be varied in three discrete steps from 3 to 20 yd^3/hr. In order to permit the pump output to match the concrete delivery rates required for the distribution and slipforming tests, the pump's controls were modified to permit the operator to vary the pumping rate continuously over the 0 to 20 yd^3/hr range. The pump was tested at pumping rates as low as 0.25 yd^3/hr.

In addition to the modification of the pump's hydraulic controls, the size of its remix hopper was reduced from 12 ft^3 to 3 ft^3 in order to minimize the system's inventory of rapid setting concrete. The reduced inventory permitted closer control of the age of the concrete delivered to the test rigs.

The proportioning mixer was used to continuously batch the rapid setting concrete mix at a rate equal to the pumping rate. This procedure enabled the operator to control the age of the concrete delivered to the test rigs and to maintain a constant inventory in the pump remix hopper.

The proportioning mixer used as a B&B Fabrication Reliable Proportionate Mixer Model 8000 shown in Figure 48. As delivered, the mixer could proportion sand, aggregate, cement and water at the rates required to batch 7 to 14 yd^3/hr. A variable frequency power supply was added to the mixer to enable the operator to continuously vary the batching rate over the 0.25 to 2 yd^3/hr range needed for the tests. A fly ash hopper and feeder and the tanks and valving necessary to control the addition of citric acid retarder and superplasticizer to the mix were also added to the mixer.

The general equipment arrangement used during the distribution and slipforming test is shown in Figure 49.

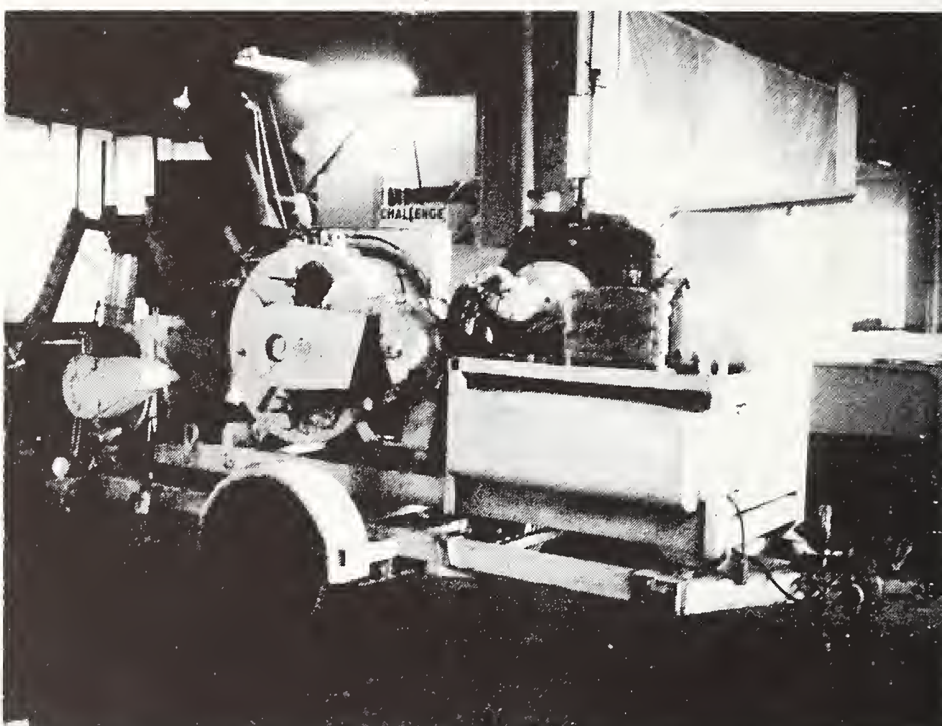


FIGURE 47. CONCRETE PUMP

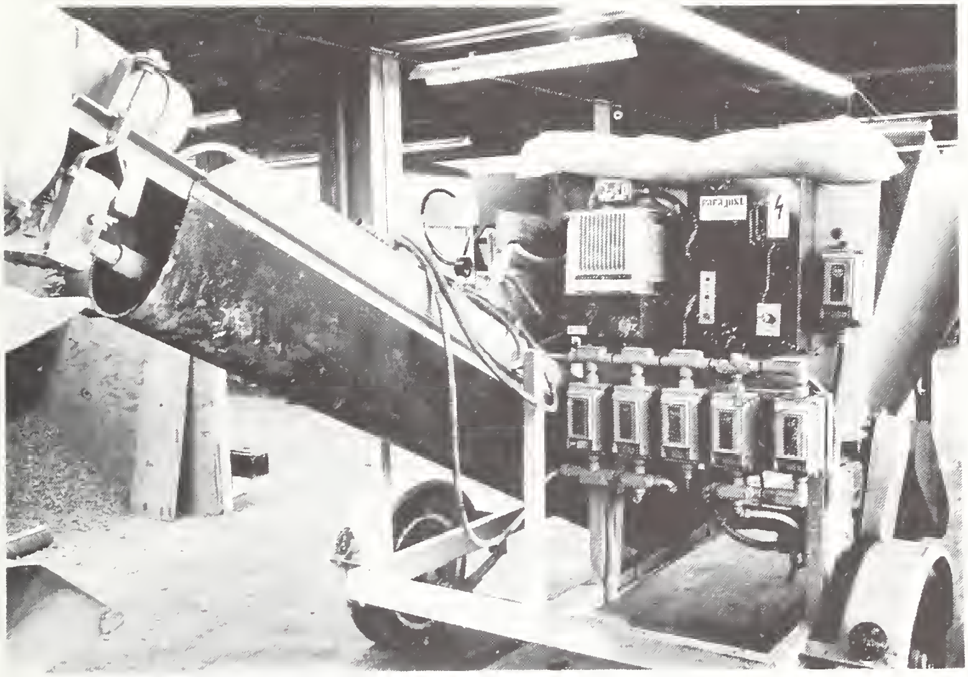


FIGURE 48. PROPORTIONING MIXER

FIGURE 49. GENERAL EQUIPMENT ARRANGEMENT

8.2 CONCRETE DISTRIBUTION TESTS

8.2.1 Distribution Test Plan

The state-of-the-art review did not answer all of our questions relating to concrete distribution within a closed form. This test plan was devised to study the manner in which concrete was distributed within the slipform behind an advancing bulkhead. The effect of concrete mix workability, vibration, pumping pressure, and bulkhead advance rate on concrete distribution were studied. In addition, the test plan, shown in Figure 50, was formulated to determine the minimum concrete workability required for complete form filling.

The test plan called for tests to start with a pumpable Type I concrete mix with 4 in. slump and to progressively test mixes with more fines, greater slump, and superplasticizers until the form was successfully filled at bulkhead advance rates of 5 and 10 ft/hr. The test path actually followed is denoted by the heavy solid line in Figure 50. The successful Type I mix was tested for workability, slump loss versus time, and those results were used as a basis for determining the acceptability of various rapid set concrete mixes.

Once a rapid set concrete mix which met both the workability and early strength requirements was found, it was tested in the distribution test rig to ensure its acceptability.

8.2.2 Distribution Test Rig

The concrete distribution test rig, shown in Figures 51, 52, and 53, was designed to simulate the placement of concrete in a closed form behind a moving bulkhead. The form cavity is 6 in. wide, 10 ft high, and 50 in. long. The cavity is sealed on one end by a fixed steel plate and on the other by a moveable bulkhead.

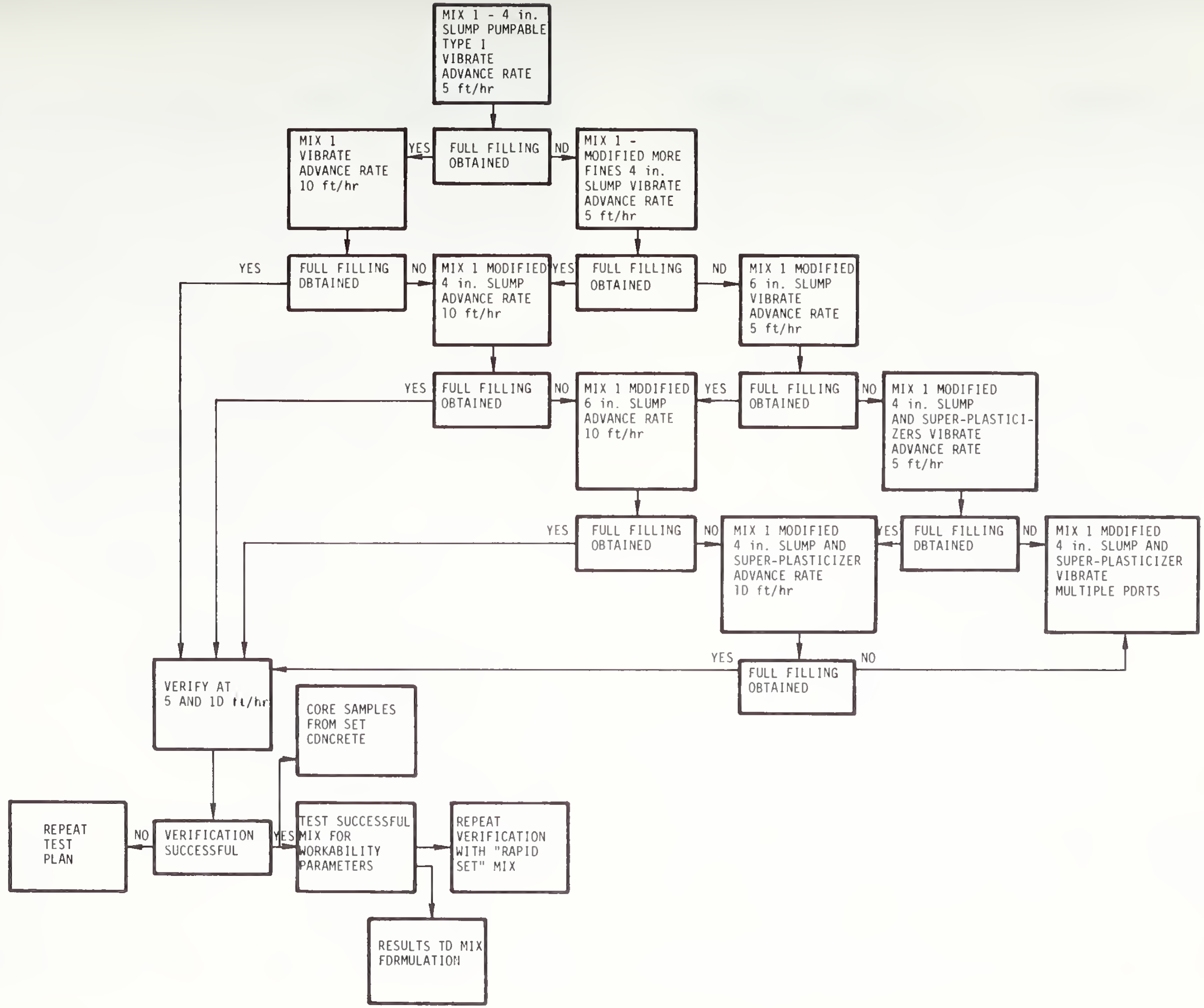


FIGURE 50. DISTRIBUTION TEST PLAN

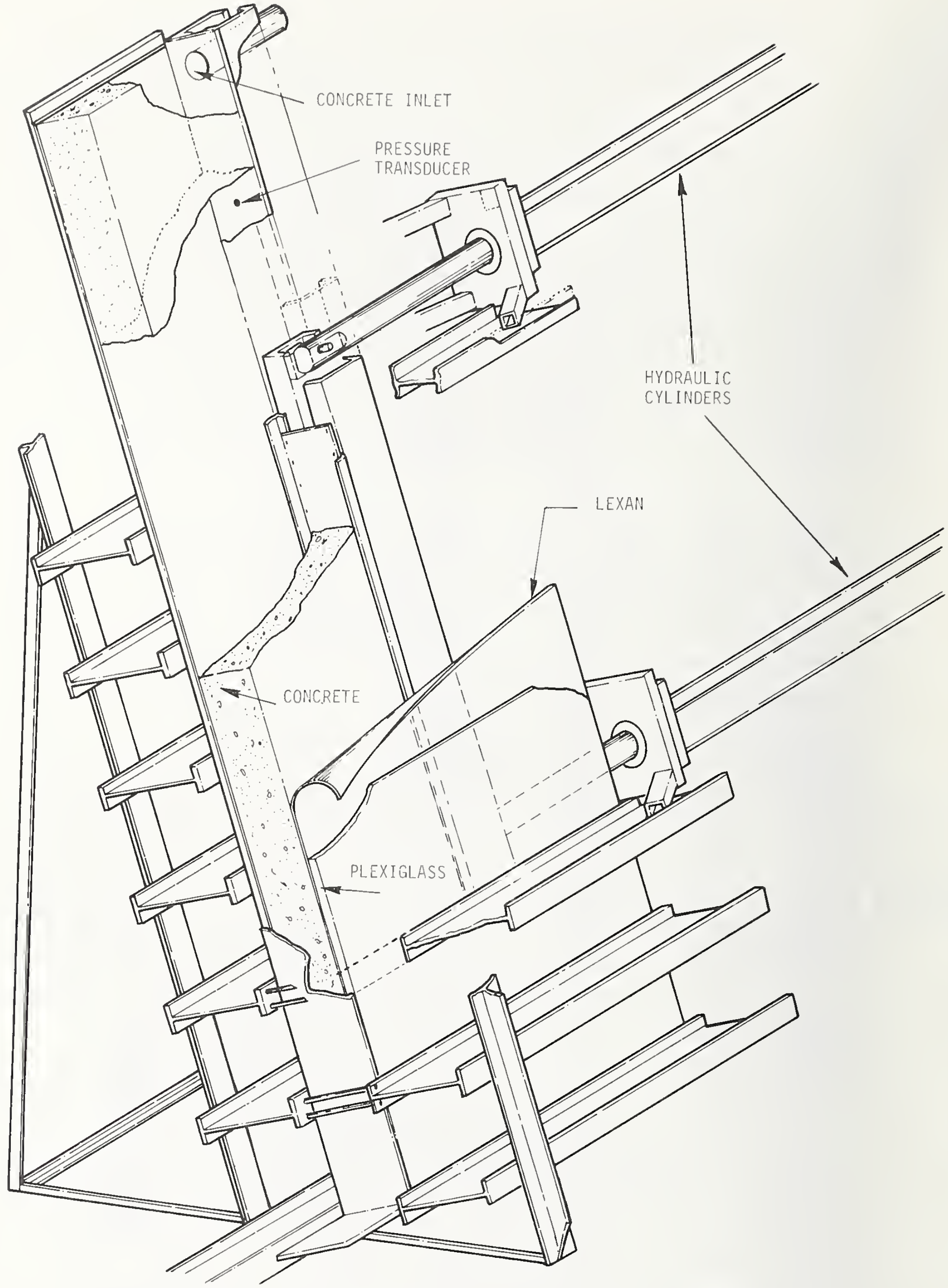


FIGURE 51. DISTRIBUTION TEST RIG - CUTAWAY

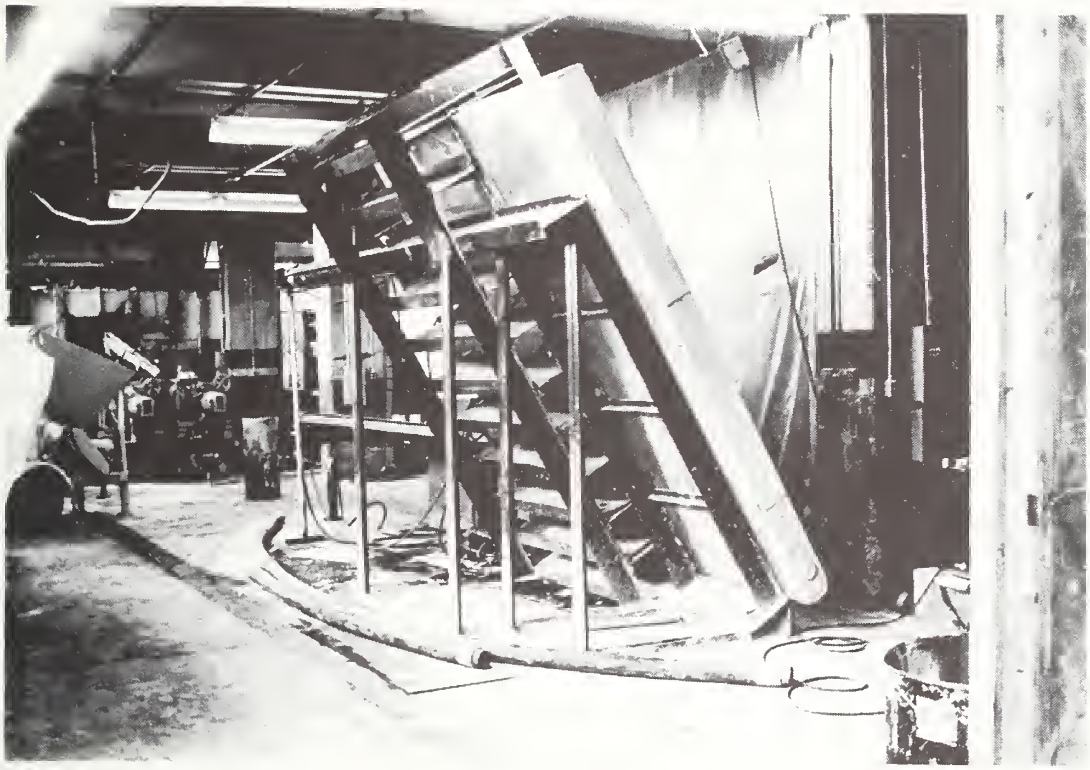


FIGURE 52. DISTRIBUTION TEST RIG (back view)

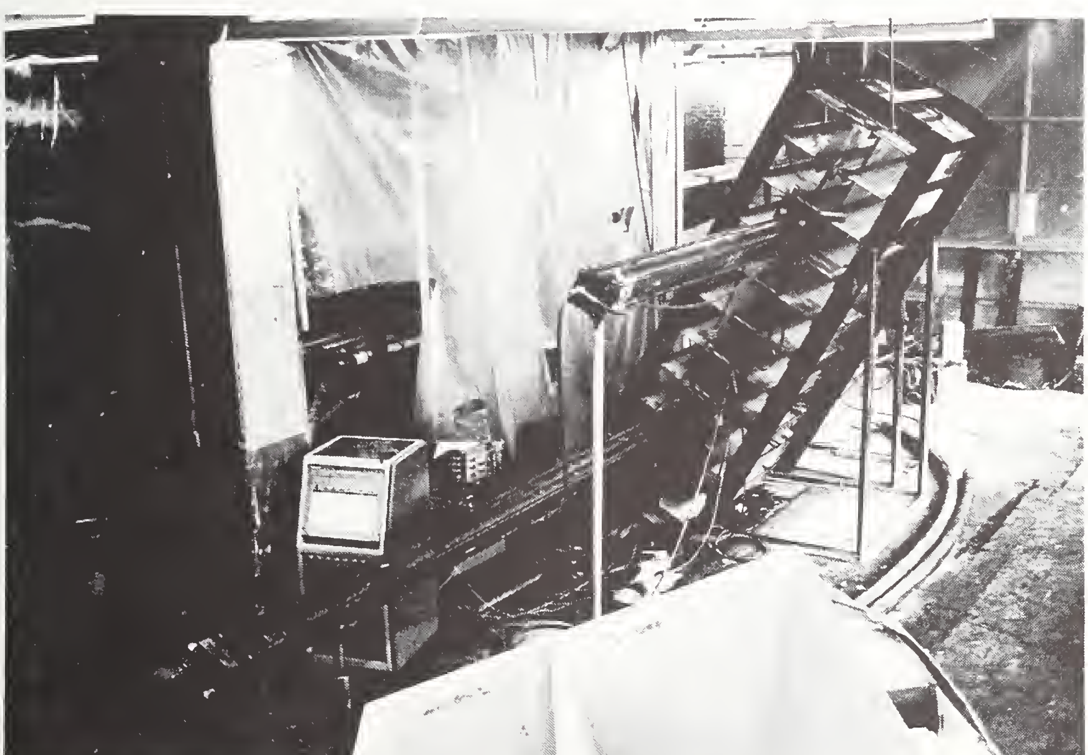


FIGURE 53. DISTRIBUTION TEST RIG (side view)

The bulkhead is fabricated from steel channel sections. The bulkhead surface, which contacts the concrete, is a steel plate which had three form vibrators attached to it. It has two concrete injection ports, one at the top and the other at the bottom; only the top injection port was used for these tests. Three pressure transducers were flush mounted at a uniform vertical spacing on the bulkhead in order to monitor the concrete pressure within the form. The bulkhead's horizontal movement was controlled by two hydraulic cylinders which also served to restrain any vertical cocking due to the vertical pressure gradient.

The form was made from 3/4 in. transparent lucite braced with steel I-beam sections to withstand 60 $lb/in.^2$ internal form pressure. The lucite form, steel bulkhead and hydraulic cylinders were all mounted on a free standing steel frame which tilted the form 30 deg from the vertical.

8.2.3 Distribution Test Results

A total of three distribution tests were conducted. No difficulty was encountered filling the form and maintaining it completely filled as the bulkhead advanced. Type I concrete was used for the first two tests and a VHEC concrete mix was used for the third test.

In all tests concrete was pumped through the top injection port in the test rig bulkhead. At the start of each test, the bulkhead was positioned 6 in. from the form's fixed end plate. Once this initial cavity was filled, the bulkhead was advanced (that is, pushed away from the fixed end plate, by the concrete pressure). The bleed rate of hydraulic fluid from the two bulkhead cylinders was regulated in order to control the concrete pressure within the form. Two mechanical hydraulic servo valves, mounted on the bulkhead, were used to maintain its vertical alignment. The servo valves adjusted the hydraulic fluid bleed rate to compensate for the concrete pressure gradient within the form. The bulkhead advance rate was controlled by varying the speed of the concrete pump. The results of the three tests are presented in the following:

a. Test 1 - A Type I concrete mix with 4 in. slump was used for this test. It was brought to the test site by a local supplier in a transit mix truck. The concrete was transferred to the pump remix hopper and pumped into the test rig at rates varying from 0.55 to 1.1 yd^3/hr. Correspondingly the bulkhead advance rate varied from 3 to 6 ft/hr.

The form was maintained completely full and no distribution problems were noted. Bulkhead vibration was not necessary, however, the vibrators were operated for a brief period. No change in pump discharge pressure or form pressure was noted during vibrator operation.

The form pressure, as monitored on the center bulkhead pressure transducer, was varied from 10 to 20 $lb/in.^2$ during the test. At 20 $lb/in.^2$ the wiper seals failed, permitting concrete paste to leak by the bulkhead.

b. Test 2 - The same procedures and mix used for test 1 were used for this test. The bulkhead seals were modified prior to the start of the test and operated successfully at form pressures up to 25 $lb/in.^2$. The results of test 1 were duplicated at bulkhead advance rates of 7 to 9 ft/hr during this test.

c. Test 3 - VHEC rapid setting concrete batched with the proportioning mixer was the material evaluated during this test. Form pressure was maintained at 15 $lb/in.^2$ and the bulkhead was advanced at 7 ft/hr. No vibration was used and no concrete distribution problems were noted. The VHEC concrete slab was removed in mass from the test rig approximately 2 hr after the start of the test.

A concrete dye was added to the VHEC mix part way through the test to make the flow pattern of concrete more visible. Figure 54 shows the distribution pattern observed. The concrete enters at the top of the form and slowly works its way to the bottom.

8.3 CONCRETE SLIPFORMING TESTS

8.3.1 Slipforming Test Plan

The slipforming test plan was devised to investigate the following areas in which no data was found during the state-of-the-art review.

a. Test the independent slipform/bulkhead concept for horizontal slipforming.

b. Demonstrate self-support of rapid set concrete after it clears the slipform.

c. Study the effect of form pressure on concrete compaction and finished liner quality.

d. Provide data on form drag variation with advance rate and concrete pressure.

The test plan is shown in Figure 55. The test path followed is denoted by the heavy solid line.

The test plan called for the initial test to be conducted using the Type I concrete mix successfully used in the distribution test. The test with Type I concrete was used to debug the test apparatus and demonstrate the horizontal slipforming concept. Since the Type I concrete would not set during the course of a test, it was used only in the tunnel invert configuration.

More extensive tests were conducted with the rapid set concrete mix which met the workability and early strength requirements.

FIGURE 54. VHEC DISTRIBUTION TEST SLAB

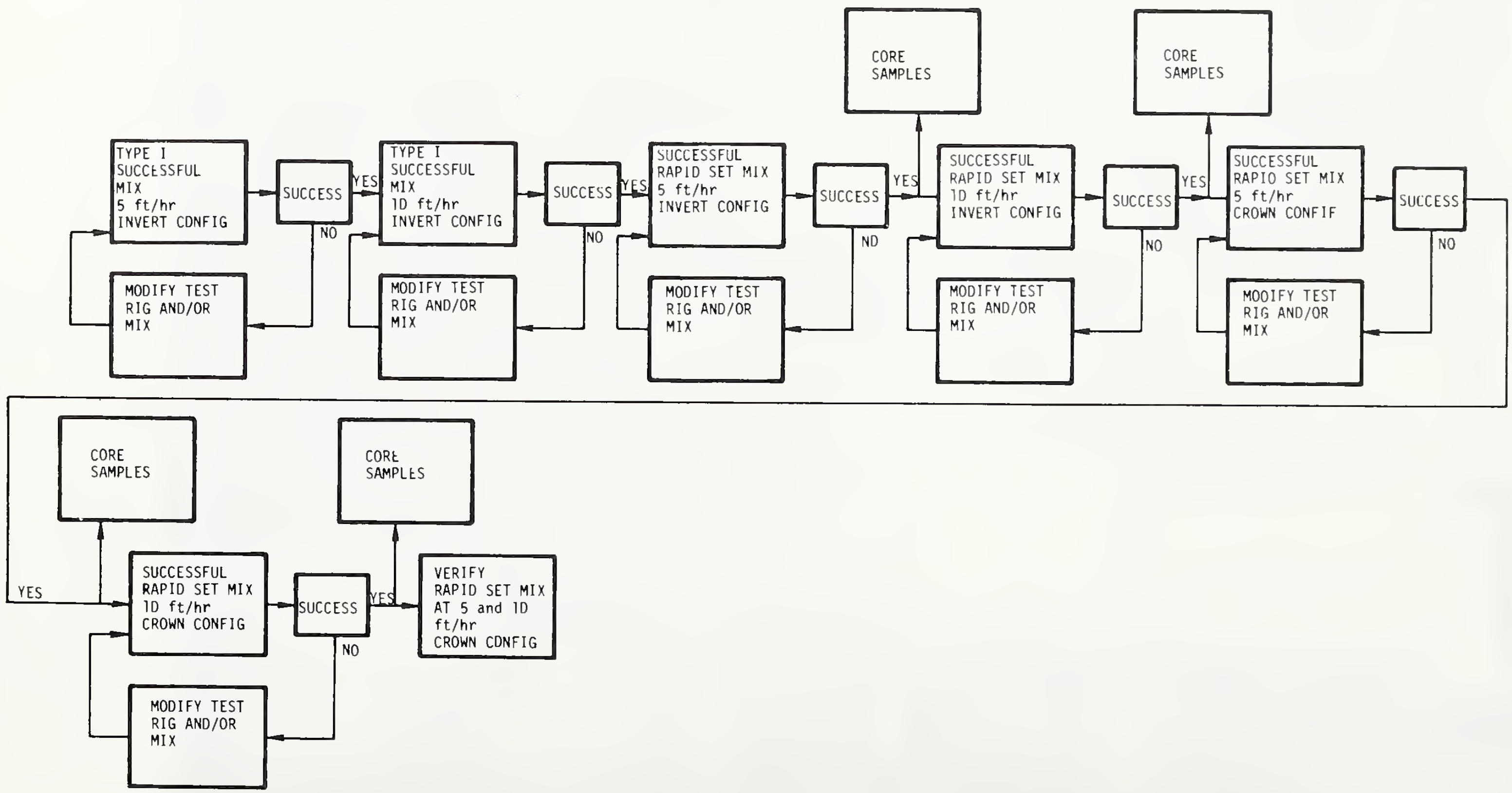


FIGURE 55. SLIPFORMING TEST PLAN

These tests were conducted in both the tunnel invert and crown configurations. In addition to demonstrating the slipforming concept with rapid set concrete these tests provided information on form drag and demonstrated the self-support capability of the rapid set concrete. Core samples were taken to determine the degree of concrete compaction and the quality of the finished liner.

8.3.2 Slipforming Test Rig

The slipforming test rig is shown in Figures 56 and 57. A 4 ft circumferential segment is simulated by casting a trapezoidal cross section encompassing a 24 deg arc of the 20 ft tunnel liner, as shown in Figure 58. The length of the cast segment is 12 ft. The entire test rig can be rotated to simulate five circumferential positions of the cast tunnel liner.

The slipform is simulated by a steel plate with a 3 ft parallel section and 4 ft tapered section. The taper can be varied from 0 to 0.5 in./ft. The pressure on the slipform is reacted by overhead tracks which are coated with a material having a low coefficient of sliding friction so as not to significantly effect the measurements of form to concrete friction. The slipform is moved parallel to the bed of the test rig by a hydraulic cylinder. The rate of flow of hydraulic fluid to the cylinder is regulated so that the desired form advance speed can be maintained.

The bulkhead is similar in construction to that used in the distribution test rig and is contoured to match the slab cross section shown in Figure 58. The bulkhead is connected to the slipform by a hydraulic cylinder which enables it to move independently of the slipform.

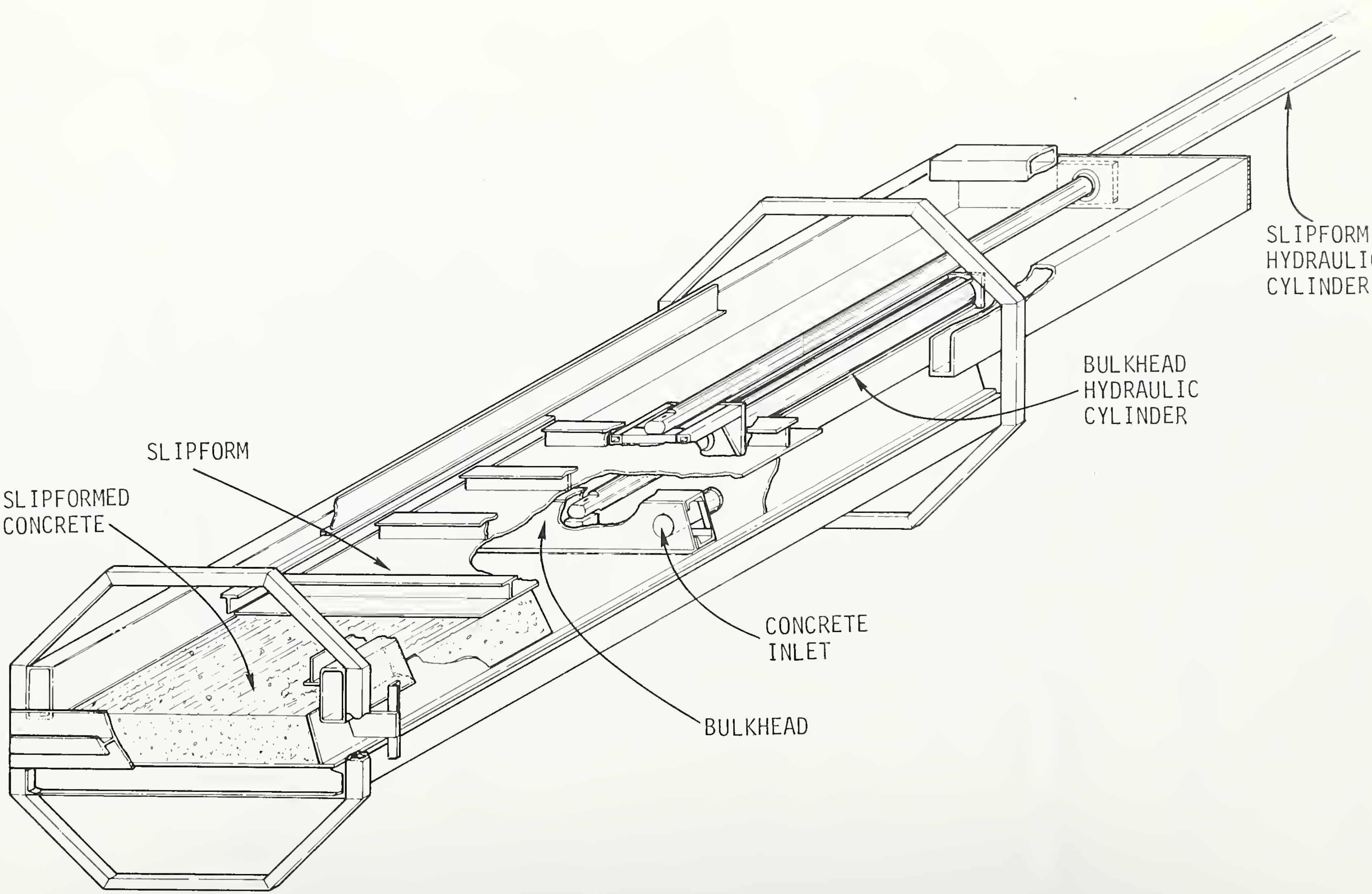


FIGURE 56. SLIPFORMING TEST RIG - CUTAWAY

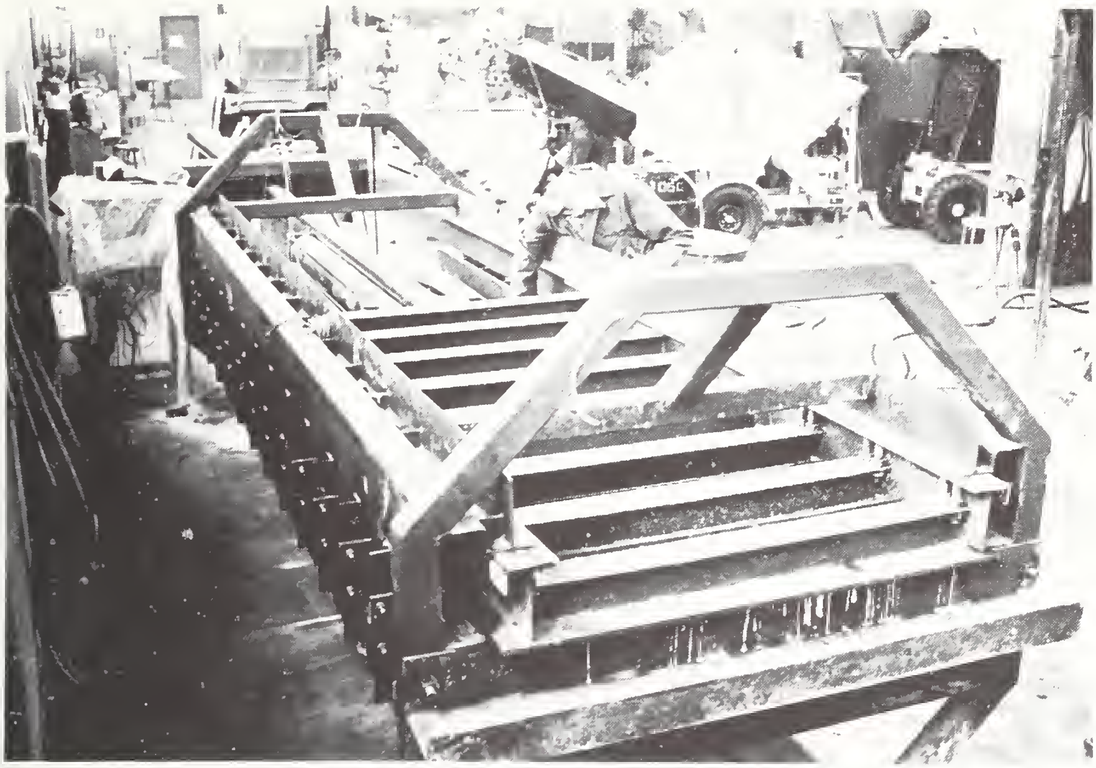


FIGURE 57. SLIPFORMING TEST RIG

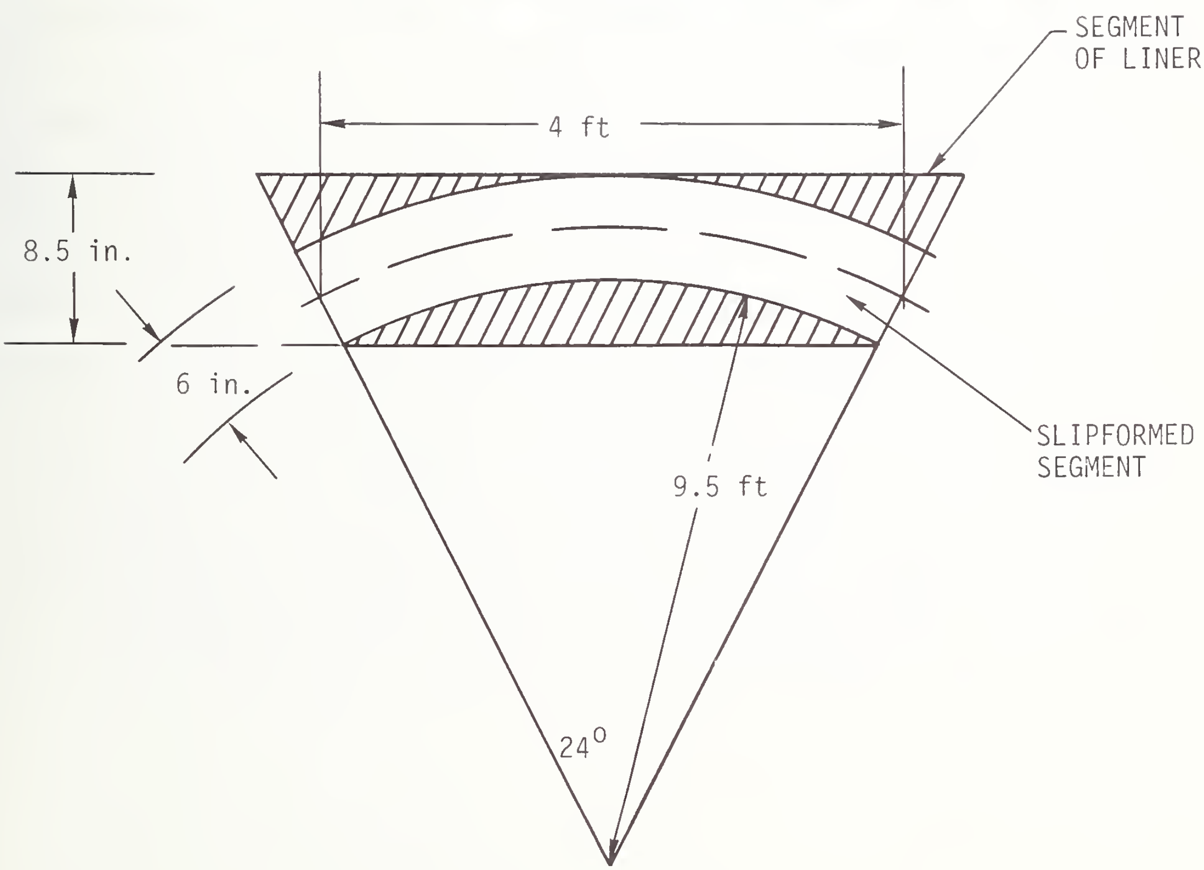


FIGURE 58. CROSS SECTION OF SIMULATED SEGMENT

The bulkhead and slipform advance rates are variable between 1 and 10 ft/hr. The test rig is designed to withstand a 60 $lb/in.^2$ internal pressure over the entire surface of the slipform and bulkhead. Both the sides and the stationary end of the test rig are removable to simplify extraction of the cast concrete.

8.3.3 Slipforming Test Results

A total of seven slipforming tests was conducted, three in the invert configuration and four in the crown configuration. The first test was conducted with the test rig in the invert configuration using Type I concrete. The remainder of the tests were conducted using the rapid setting VHEC concrete mix formulated during the course of the test program discussed in Section 7 of this report. The tests demonstrated the feasibility of slipforming a well consolidated section of rapid setting concrete. The ability of the rapid setting concrete to support itself in the crown of the tunnel approximately 1 hr after mixing was also demonstrated.

a. Test 1 - This test was conducted with the test rig in the invert configuration using Type I concrete. The main purpose of this test was to exercise and debug the test equipment to ensure proper operation prior to its use with rapid setting concrete. During this test the slipforming rate was varied between 5 and 9 ft/hr. The concrete pressure within the form was varied from 3 to 40 $lb/in.^2$. Neither the variation in slipforming rate nor the variation in concrete pressure had any apparent effect on the degree of consolidation or surface appearance of the slipformed slab. However, the higher pressure did cause the concrete pump to seize and caused the concrete to bulge as it cleared the trailing edge of the slipform.

The bulkhead vibrators were energized for a period of 10 min. During this period the slipform traveled about 1.5 ft. The concrete placed during this period was badly segregated, with a paste rich mix being placed at the side of the slab most distant from the concrete injection port. No segregation was observed in portions of the slab not subjected to vibration.

In summary, this test demonstrated the feasibility of the independent slipform concept and the fact that a well consolidated slab could be placed at low form pressures with little or no vibration.

b. Test 2 - As in test 1, this test was conducted with the test rig in the invert configuration, but in this case the rapid setting VHEC concrete mix was used. The slipform taper was set at 1/16th in./ft, approximately the same taper used in conventional vertical slipforming. The concrete was placed at a rate of 1 yd/hr which required the slipform to advance at a rate of 9.4 ft/hr. This rate was held constant throughout the test in order to minimize the need to change the production rate of the proportioning mixer. The concrete pressure in the form was varied from 6 to 13 lb/in.^{2} during the startup portion of the test where the concrete was being placed with the slipform stationary. Once slipform movement was started it was necessary, because of limitations in the hydraulic system, to increase the concrete pressure to the 20 to 25 lb/in.^{2} range. No differences in consolidation were observed between the concrete placed at 6 lb/in.^{2} and that placed at 25 lb/in.^{2}. The concrete clearing the slipform had no tendency to bulge even at the highest pressure. Penetrometer tests of the concrete as it cleared the slipform indicated that it was well past initial set. The penetrometer was driven off scale (>500 lb/in.^{2}) with no indentation of the concrete surface (see Figure 59).

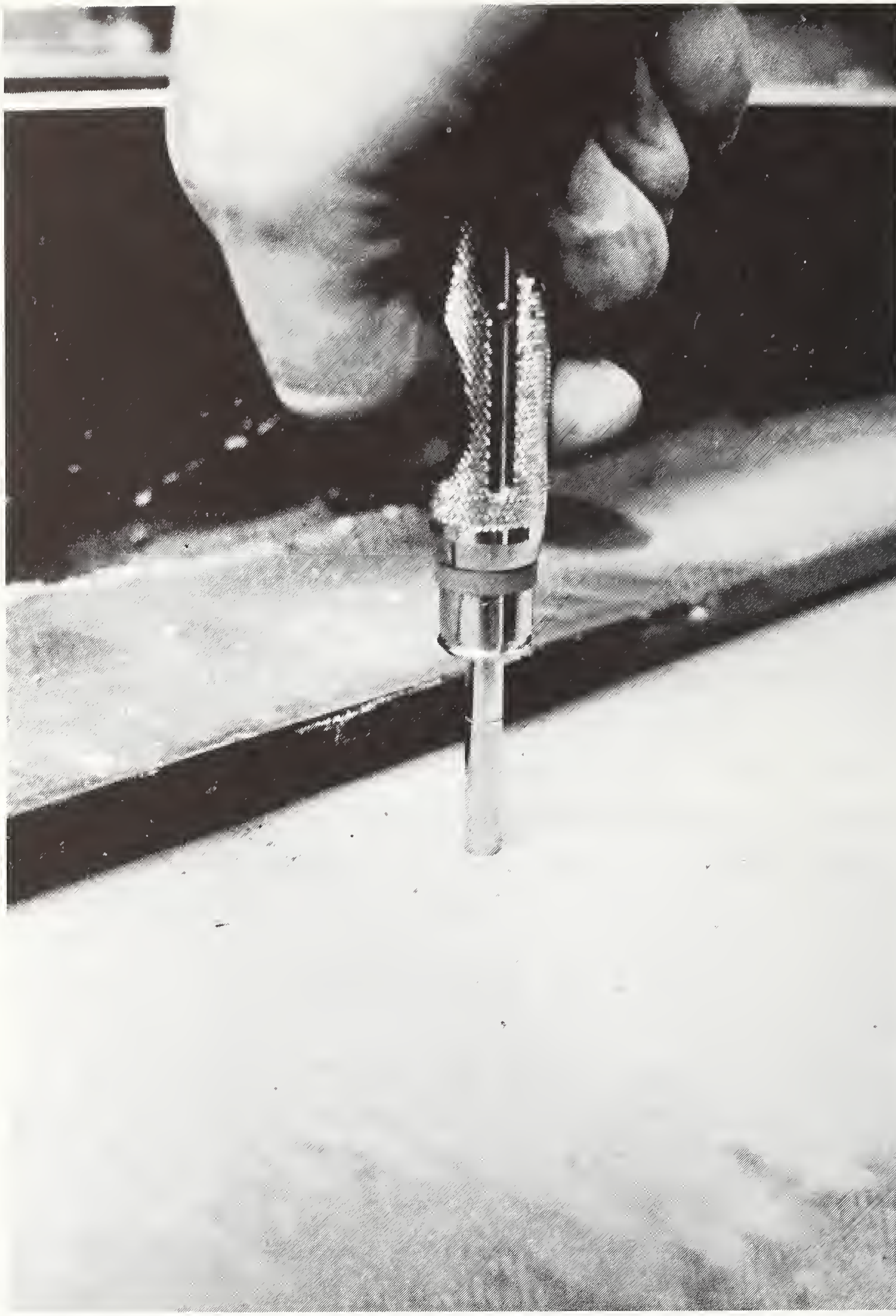


FIGURE 59. PENETROMETER TEST OF SLIPFORMED VHEC CONCRETE

A 12 ft long concrete slab was slipformed during this test. The bulkhead vibrators were employed intermittently (10 sec every 30 sec) for a period of 10 min. The segregation caused by the continuous vibration used in test 1 was not caused by the intermittent vibration used in this test. The test showed that vibration was not required since no difference in the degree of consolidation of the vibrated and non-vibrated concrete was observed.

c. Test 3 - The purpose of this test was to verify the results of test 2. The test procedure of test 2 was followed until a plug developed in the concrete line used to deliver concrete from the pump to the slipform. It was necessary to retract the bulkhead and disconnect the concrete line. With the line disconnected, the plug cleared, the line purged and reconnected to the bulkhead. Since this operation took approximately 30 min, the concrete placed before the plug developed had set. The bulkhead was positioned approximately 1 ft behind the set concrete and concrete flow was initiated. Due to a concern that high pumping pressure might cause another line blockage, concrete pressure was not increased above the minimum required to move the bulkhead ($\sim$3 lb/in.^{2}) until the bulkhead had traveled about 1 ft. If this concern did not exist, the pressure would have been increased to ensure full contact between the concrete of the two placements. Even without this precaution an adequate bond, as shown in Figures 60 and 61, was attained.

No further pumping problems were encountered during this test which was continued until a 12 ft long slab was cast. No vibration was used during this test; as in previous tests, the concrete slab was well consolidated and showed no signs of segregation. The absence of voids and even aggregate distribution can be seen in

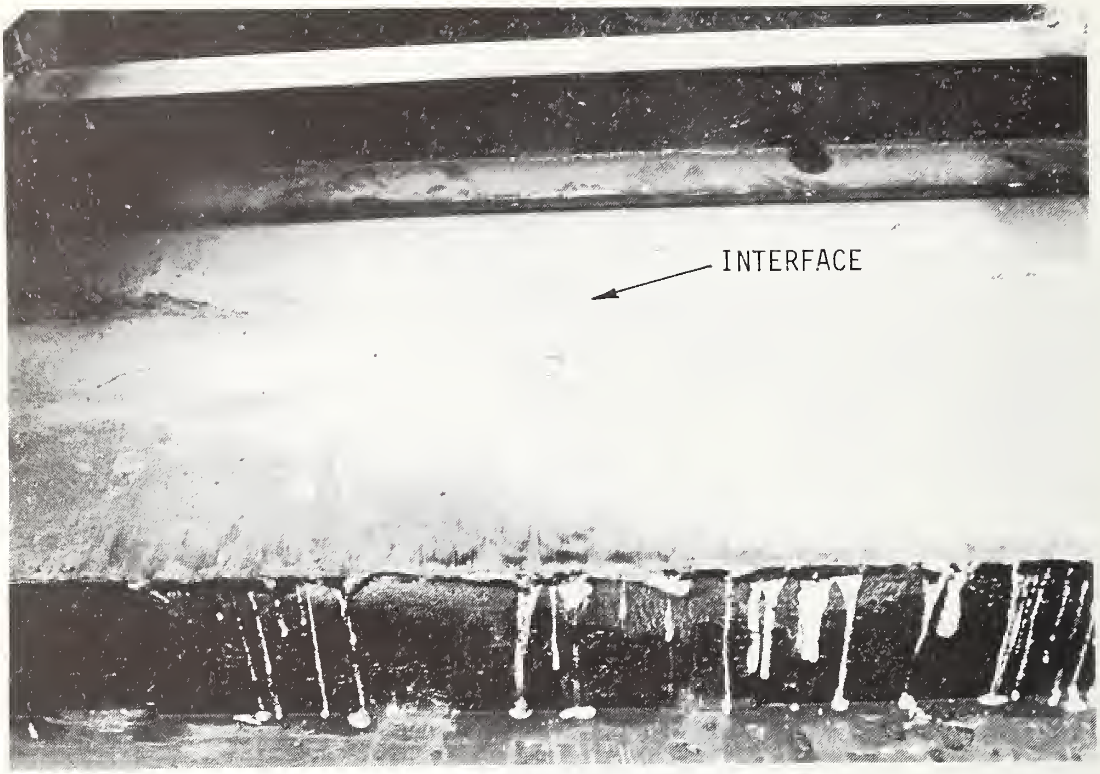


FIGURE 60. INTERFACE BETWEEN TWO CONCRETE PLACEMENTS

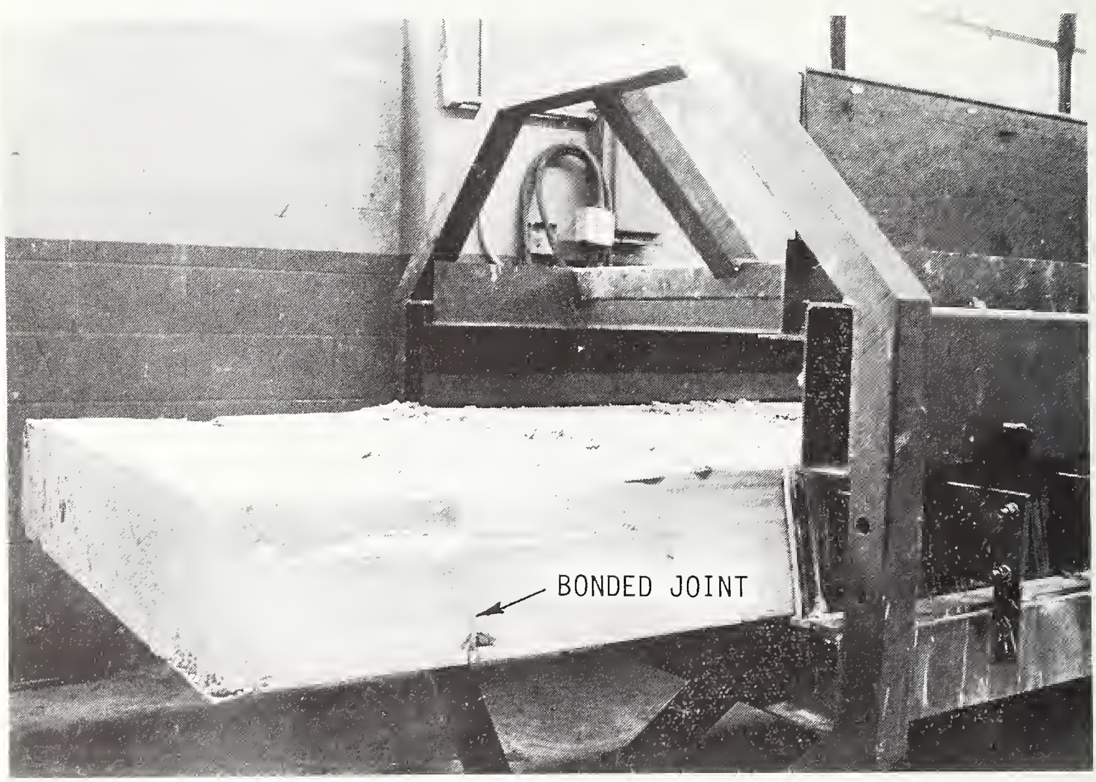


FIGURE 61. BOND BETWEEN TWO CONCRETE PLACEMENTS

the slab cross section shown in Figures 62 and 63. The large discontinuities seen in the slab are not caused by placement problems but are penetrations made with an impact chisel, see Figure 64. The discontinuities form a fracture plane, which facilitates removal of the concrete from the test rig.

d. <u>Test 4</u> - This test was the first test conducted with the test rig in the tunnel crown configuration. This test was designed to determine the age at which the concrete could support itself in the crown of the tunnel. With the test rig in this configuration, access to the surface of the slab was restricted and it was not possible to place a fracture plane in the slab. Therefore, the test plan called for placement of a slab with a 6 ft maximum length in order to facilitate its removal from the test rig. The concrete was placed at a rate of 10 ft/hr and the slipform initially cleared the concrete when the concrete was 52 min old. Once exposed, the concrete slab sagged with a deflection at the center of approximately 3/8 in.

After the slipform had cleared 1.5 ft of concrete, a solenoid valve in the hydraulic control system failed in the open position. This failure caused the slipform advance rate to increase significantly and the remaining 4.5 ft of concrete slab was exposed within 5 min. Thus, the major portion of the slab was exposed when it was only 35 to 40 min old. This concrete sagged significantly and developed a tensile crack along the longitudinal center line and after 10 to 15 min fell out of the form. The first 1.5 ft of slab exposed did not fall out.

e. <u>Test 5</u> - The test procedure used in test 4 was repeated for this test except that the slipform advance rate was adjusted so that the concrete would not be exposed until it was 60 min old.

FIGURE 62. CROSS SECTION OF SLIPFORMED SLAB

FIGURE 63. CROSS SECTION OF SLIPFORMED SLAB

FIGURE 64. FRACTURE PLANES PLACED IN SLIPFORMED SLAB

A slab approximately 3 ft long had been placed when a blockage occurred in the concrete line. This blockage necessitated the removal of all concrete hose, dumping of the pump hopper and flushing of the pump to prevent concrete from setting up in the system.

The slipforming portion of the test continued. The slipform cleared the concrete when it was approximately 60 min old. The concrete was self-supporting, however it did sag with center deflection reaching 1/4 in., see Figure 65. The concrete did sag further and, as can be seen in Figure 66, no tensile cracks developed in the slab.

It was felt that the 60 min old concrete might be self-supporting with no sag if the 1/4 in. form taper was removed.

f. Test 6 - This test was essentially a repeat of the previous two tests with the following exceptions:

1. The slipform was adjusted so no taper was provided.

2. In an attempt to prevent concrete line blockages, the line was primed with a highly plasticized cement-sand mortar rather than the cement slurry used in all previous tests.*

The concrete was placed at an advance rate of 9 ft/hr, form pressure was maintained in the 8 to 10 $lb/in.^2$ range. The slipform exposed the concrete when the concrete was 80 min old. The concrete remained in contact with the bed of the slipform test rig and showed no sign of sagging, see Figure 67. The initial slipform advance rate was 5 ft/hr; once a few inches of concrete had been exposed, the advance rate was increased to 12 ft/hr. Thus,

*This procedure was used in all remaining tests and no line blockages were experienced.

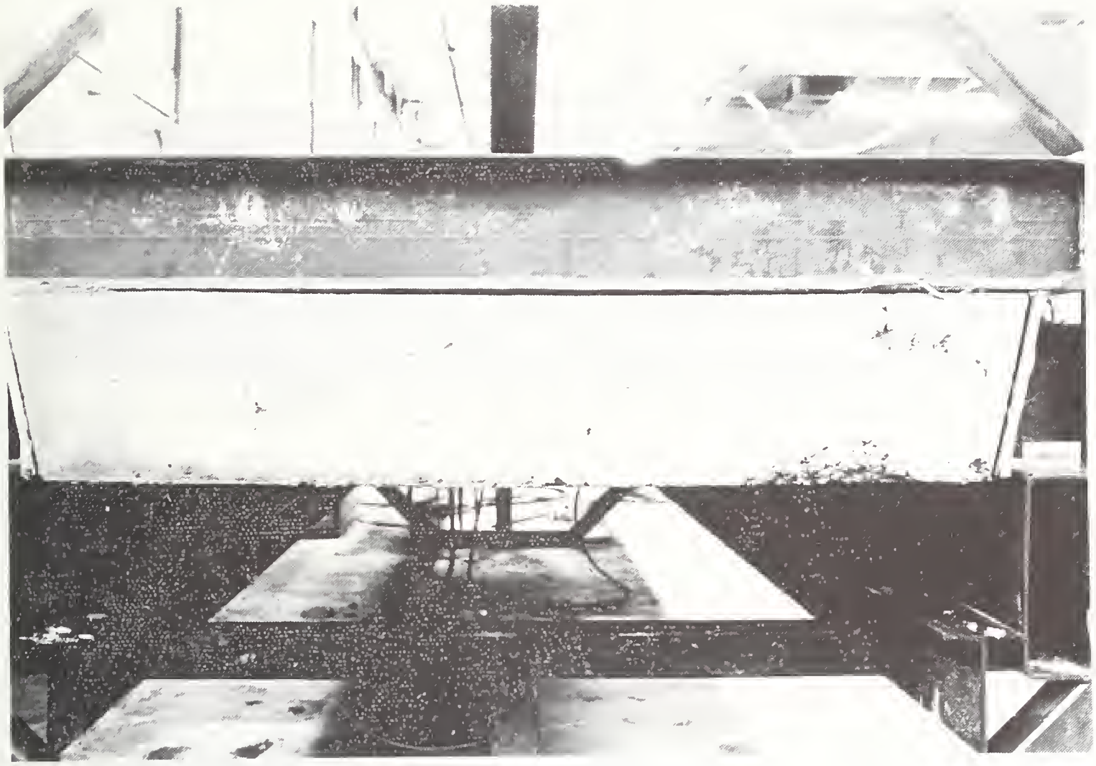


FIGURE 65. SLIPFORMED SLAB SLUFFED FROM TEST BED SURFACE

FIGURE 66. UNDERSIDE OF SLIPFORMED SLAB

FIGURE 67. SELF-SUPPORTING SLIPFORMED SLAB

the concrete near the end of the slab cleared the slipform when 70 min old. The 70 min old concrete was able to support itself without sagging.

During this test it was noted that the heat of hydration of the concrete placed first accelerated the set of the younger concrete. Because of this accelerating effect it can be expected that the concrete placed during steady state operations will gain the strength required for self-support more rapidly than these tests indicate. In addition, the concrete temperature appears to be an easily measurable parameter that could be used to determine the allowable rate of slipform advance.

g. Test 7 - This test was conducted to verify the results of test 6. The same test procedure was repeated. The slipform initially cleared the concrete when the concrete was 80 min old, the slipform advance rate was increased so that the concrete near the end of the slab was only 60 min old when the slipform cleared. The 60 min old concrete was self-supporting and showed no tendency to sag.

8.3.4 Forces on Slipform Advancing Mechanism

In addition to demonstrating the feasibility of slipforming a self-supporting slab of rapid setting concrete, the slipforming tests were used to evaluate the net loads that must be overcome or resisted by the slipform advancing mechanism. It was thought that the relatively high concrete pressure within the slipform and the expected higher coefficient of friction of the rapid setting concrete would increase form drag significantly above that experienced with vertical slipforming. On the other hand, the concrete pressure acting on the bulkhead, which in turn reacts off the slipform, would tend to counteract the drag force.

During the state-of-the-art review, an attempt was made to calculate slipform drag. The assumptions made regarding friction coefficients, degree of plasticity of the concrete, and the actual concrete pressure required to distribute the concrete, caused the calculated loads which the slipform advancing mechanism would have to overcome to vary over a wide range. For example, in a 20 ft diam ETLS system, these loads could vary from a load of 400 tons to advance the system, to a restraining load of 100 tons. The data from the slipforming tests indicate that the slipform advancing mechanism would have to provide a force of about 100 tons in order to advance the system.

9. CONCEPTUAL SYSTEM DESIGN

9.1 GENERAL SYSTEM DESCRIPTION

9.1.1 System Configuration

The ETLS equipment will be designed to fit into a normal tunneling machinery arrangement with as little interference as possible. The conventional tunneling machinery arrangement will be lengthened about 20 ft by the installation of the ETLS at a position just behind the rear gripper section of the TBM. Components of the ETLS surround this segment of the tunnel like a ring permitting normal services and muck to pass through this region. This conceptual system arrangement is shown in Figure 1. The major components of the ETLS which are depicted are:

a. Slipform - a cylindrical steel form which holds the concrete liner in position until the concrete has gained sufficient strength to be self-supporting.

b. Bulkhead - a circular steel ring which contains the forward edge of the plastic concrete and maintains sufficient pressure behind the slipform to ensure proper compaction of the concrete.

c. Concrete Handling System - a system which includes all equipment for transporting, batching, mixing and pumping the rapid setting concrete mix used for the extruded tunnel liner.

A typical cross section of the slipform and bulkhead is shown in Figure 2; these same components are shown in exploded view in Figure 68.

A brief description of the system operation is given in the next subsection. The individual components and system controls are described in detail in following subsections.

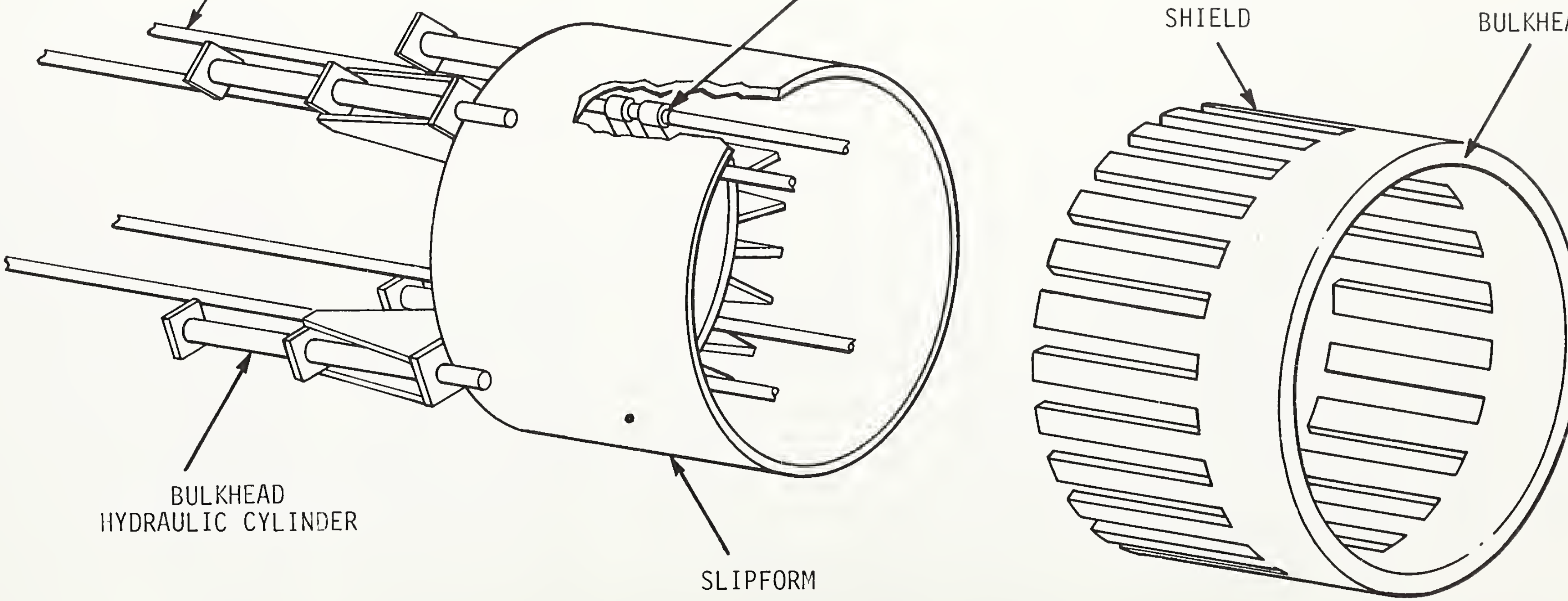


FIGURE 68. ETLS COMPONENTS

9.1.2 System Operation

A typical operating sequence is shown in Figure 69. At startup the trailing edge of the slipform overlaps the finished concrete surface of the previously placed liner forming a seal. The bulkhead is positioned just forward of the trailing edge of the slipform. As concrete is pumped into the cavity between the slipform and the tunnel wall, the bulkhead is advanced at the rate necessary to control the concrete pressure within the form at the desired level. When the concrete within the form attains the strength necessary for self-support, the slipform is advanced at a speed equal to the rate of advance of the bulkhead. Thus the slipform exposes the concrete tunnel liner shortly after the liner has become self-supporting. The concrete placement rate will be varied in order to maintain the desired spacing between the slipform and the TBM.

At shutdown the concrete pump is stopped, the concrete lines are purged, and the bulkhead is held stationary to maintain the desired concrete pressure. Slipform advance continues at the previous rate until its trailing edge reaches the bulkhead. At this time all concrete which has been placed is self-supporting and both the bulkhead and the slipform are moved forward to prevent adhesion and permit joint preparation.

The effective slipform length is defined as the distance from the bulkhead face to the trailing edge of the slipform. It is a function of both the concrete placement rate and the time required for the liner to become self-supporting. The independent motion of the bulkhead and slipform permits the proper effective length to be maintained regardless of the variations in concrete placement rate required to maintain the desired ETLS/TBM spacing.

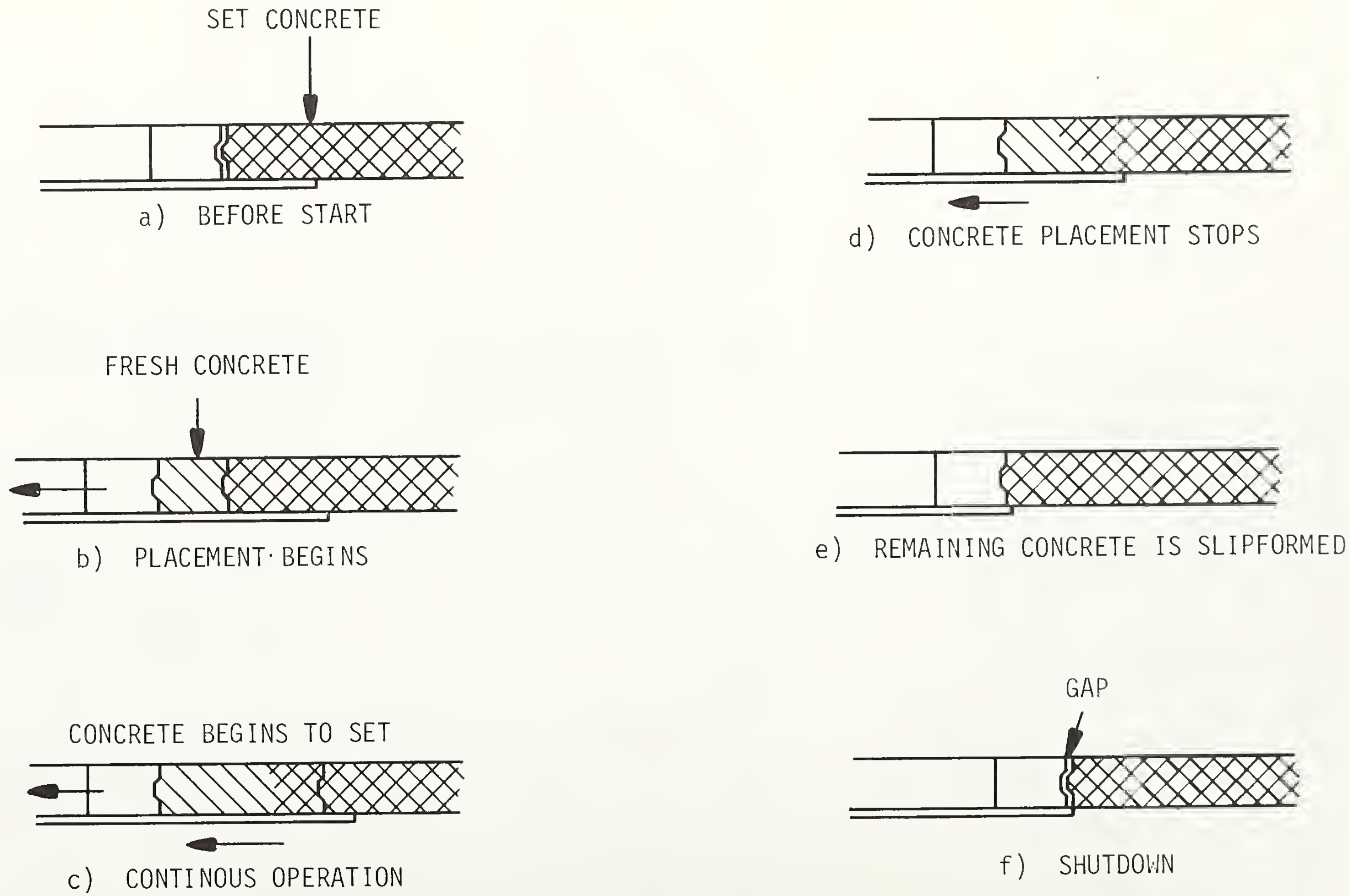


FIGURE 69. SLIPFORMING SEQUENCE

9.2 COMPONENT DESCRIPTIONS

9.2.1 Independent Slipform

The independent slipform is a means of adapting the slip-forming process, which demands continuity and uniformity, to the discontinuous and irregular aspects of tunnel boring. The name derives from the relative motion permitted between the slipform, the bulkhead, and the TBM. It was specifically conceived to handle the frequent stops, starts, and variations in advance rate of a TBM.

The slipform design is shown conceptually in Figure 70. It is essentially a cylindrical steel form incorporating mounting brackets for attachment of the bulkhead control cylinders and the slipform advancing jacks. These jacks advance the slipform by climbing along jacking rods which are fixed to the gripper section of the TBM. The leading edge of the form is stiffened to enable it to resist the loads generated by the bulkhead cylinders and advancing jacks and to react the locating pad loads.

The locating pads, see Figure 71, will maintain the proper alignment of the slipform's leading edge. These pads will react forces on the slipform caused by form bouyancy and nonuniform concrete drag. They could also be used to steer the slipform and correct for TBM misalignment.

The slipforming tests conducted in Phase I indicated that the originally proposed tapered slipform design could not be used since it would permit the liner to sluff from the crown of the tunnel. This problem resulted from the fact that the rapid setting concrete lost its ability to be remolded long before it gained sufficient strength to be self-supporting. Thus, the

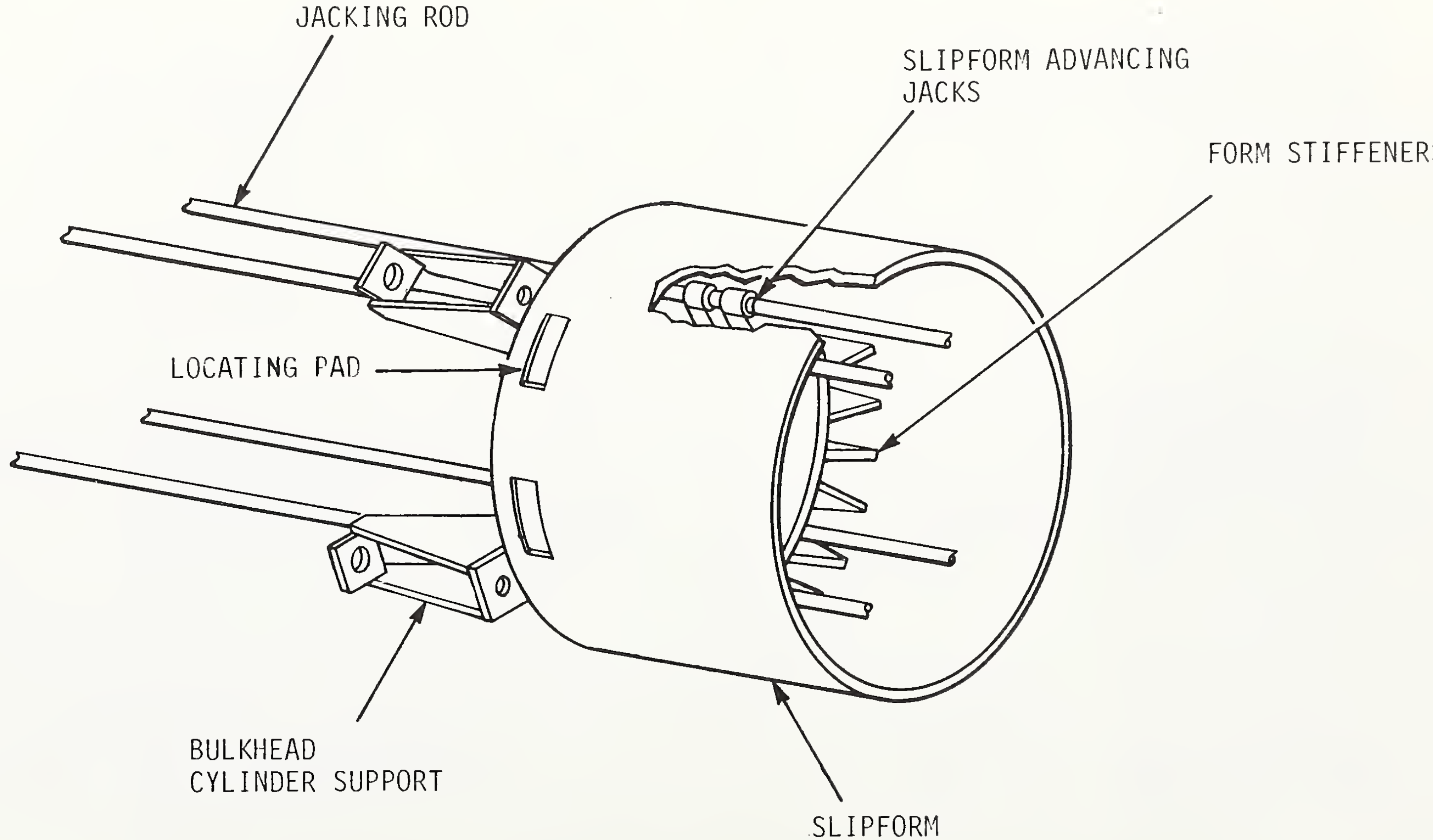


FIGURE 70. SLIPFORM

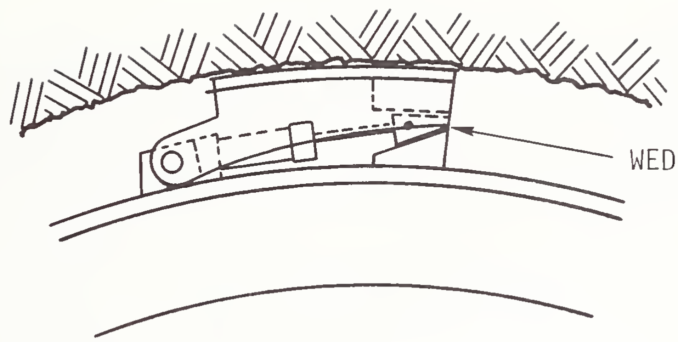


FIGURE 71. END VIEW OF LOCATING PAD

concrete could not fill the gap formed as the concrete liner sluffed from the tunnel crown and maintained contact with the slipform taper.

The taper was adapted from conventional vertical slipforming design where it is used to ensure the concrete does not contact the form once initial set has occurred. The taper would have permitted the ETLS slipform to negotiate a turn without disrupting the set concrete. In order to enable the slipform to negotiate a turn while still providing the necessary support for the liner, the trailing section of the form may have to be designed with some compliance. One approach would be a compliant fluid filled mat. This compliant mat could deform sufficiently to enable the slipform to turn without disrupting the set concrete which has yet to clear the form. This problem will be studied in further detail during the design review task of Phase II.

9.2.2 Bulkhead

The bulkhead is the component of the ETLS that contains the forward surface of the freshly placed plastic concrete. A typical cross section of the bulkhead is shown in Figure 72. It is essentially a steel ring beam fitted with a resiliently mounted steel plate and two rubber seals. The steel plate can be vibrated to facilitate concrete distribution within the form. The seals prevent cement paste from leaking between the bulkhead and the tunnel wall and slipform. Concrete is pumped through a penetration in the steel plate located near the top of the bulkhead ring beam.

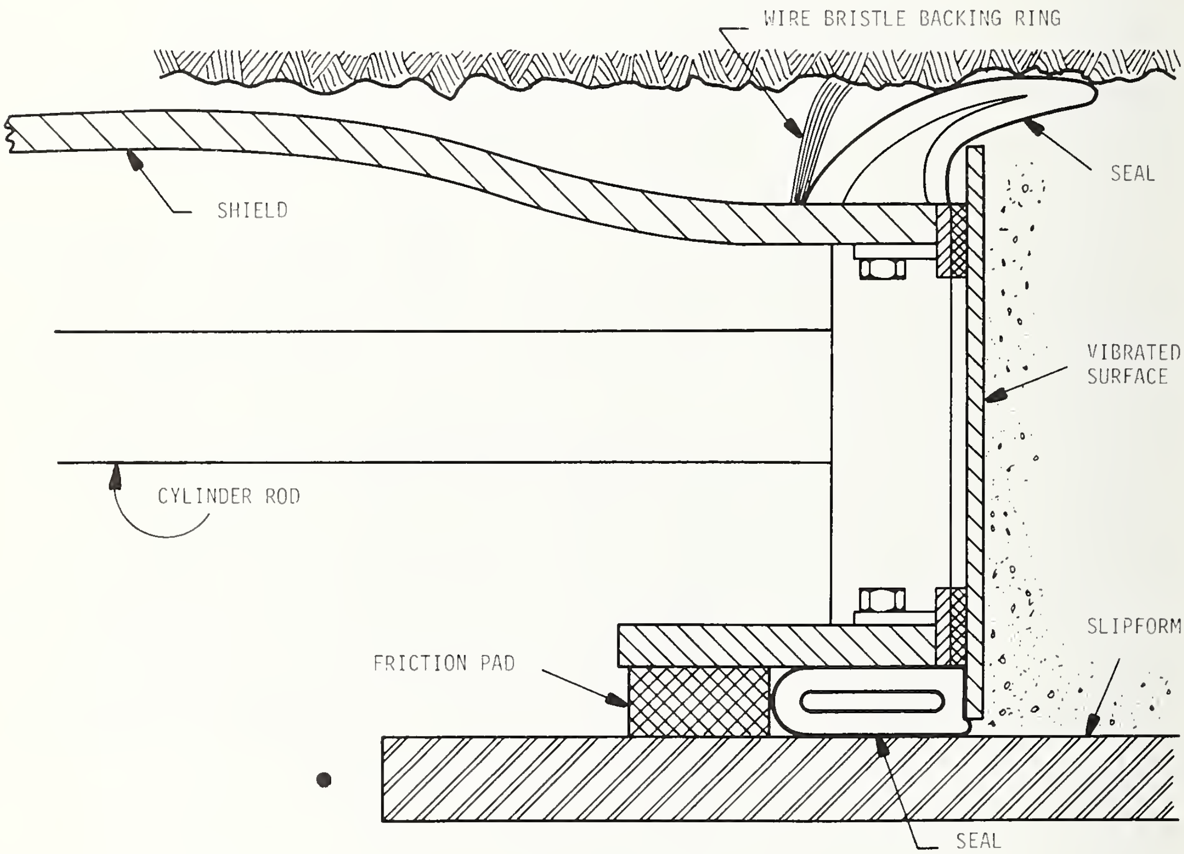


FIGURE 72. BULKHEAD CONFIGURATION

The bulkhead is positioned by a set of hydraulic cylinders which are mounted on the slipform structure. The cylinders control the vertical alignment of the bulkhead and in addition maintain the required concrete pressure within the form to ensure adequate compaction and proper distribution of the concrete.

A protective shield is attached to the forward edge of the bulkhead. This shield is designed to interleave with the trailing shield of the TBM and provide continuous ground support between the TBM and the extruded tunnel lining.

9.2.3 Concrete Handling System

Because of the rapid setting nature of the concrete used for the extruded tunnel liner, it is necessary to batch and mix the concrete components at the tunnel face. The concrete components will be brought to the face in transit supply cars designed to run with the muck train. The various components will be transferred to the storage hoppers of a proportioning mixer. These storage hoppers will be sized to permit the system to operate for several minutes without replenishment permitting the mixer to continue operation during changeover from one supply car to another. The mixer will batch and mix the cement, sand, aggregate, water and admixtures in the proper proportions to produce a rapid setting cement with the desired characteristics.

The concrete pump will be a standard positive displacement concrete pump design adapted for use in the tunnel environment. The proportioning mixer will deliver freshly mixed concrete to the pump. The pump will inject the concrete into the cavity behind the bulkhead between the slipform and the tunnel wall. The pumping rate will be dictated by the bulkhead advance rate. The concrete production rate will be adjusted to match the pumping rate.

In addition to the transit supply cars, the proportioning mixer, and the concrete pump, the concrete handling system will provide the concrete slick lines and purge valves necessary to deliver the rapid setting mix to the slipform. This portion of the system will include a specially designed diverter valve mounted at the bulkhead injection port that will permit the entire slick line run from the pump to the bulkhead to be rapidly purged following a shutdown.

9.3 ETLS CONTROL SYSTEM

The control system will be designed to enable two operators to perform all functions required for operation of the ETLS. The controls will be grouped in two locations, one station for the slipform-bulkhead operator and the other for the concrete mixer and pump operator.

The slipform-bulkhead operator will control the slipform advance rate and maintain concrete form pressure by controlling the bulkhead hydraulic cylinder pressures. The slipform advance rate will be set considering the TBM advance rate and the concrete set point.

The concrete mixer and pump operator will control the rate of concrete production. He will control the rate at which the concrete will set by adjusting the citric acid admixture addition rate. The rate of concrete production will be based on the desired bulkhead advance rate while the admixture addition rate will be based on the desired slipform advance rate. He will also control the concrete purge system.

The present concept incorporates a passive steering system. The bulkhead will follow the contour of the tunnel wall. The slipform will be positioned by the bulkhead, its own locating pads which also follow the tunnel wall, and its trailing edge which is positioned by the set liner. This method of steering is desirable since it involves no operator or control system action. It is made possible by the improved accuracy of new TBM guidance systems which are able to maintain tunnel alignment within tolerances which require no compensation by the ETLS. If it were desirable to use the system with a TBM which did not incorporate the new guidance system, the ETLS could be steered in chords by positioning the locating pads. The slipform operator will control the locating pad position.

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APPENDIX

REPORT OF NEW TECHNOLOGY

No new technology was developed during this contract. However, a significant amount of experimental data was collected which broadens our understanding of the effect of admixtures, such as citric acid and superplasticizers, on the early strength gain and workability of rapid setting concretes. These data are presented in Section 7. In addition, state-of-the-art advances in batching and placing rapid setting concretes were made in conjunction with the test program which is discussed in Section 8.