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16. Abstract

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Steel connections to maintain the continuity of steel reinforcement in CRCP is a critical aspect of the repair. These connections can be made by welding, clamping, or use of commercial rebar connectors. Polymer concrete is a fast setting material which has excellent properties as a repair material for use around these connections. The set time of the material to be placed around the steel connections will, to a large extent, govern the overall repair time.

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PRECAST REPAIR OF CONTINUOUSLY REINFORCED CONCRETE PAVEMENT

by

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B. Frank McCullough

W. Ronald Hudson

Research Report 177-15

Development and Implementation of the Design, Construction
and Rehabilitation of Rigid Pavements

Research Project 3-8-75-177

conducted for

Texas
State Department of Highways and Public Transportation

in cooperation with the
U. S. Department of Transportation
Federal Highway Administration

by the

CENTER FOR HIGHWAY RESEARCH

THE UNIVERSITY OF TEXAS AT AUSTIN

May 1979

PREFACE

The development of a rapid means of repairing continuously reinforced concrete pavement (CRCP) is essential, considering the rapid increase of heavily loaded vehicles on our highway system. Furthermore, since the design life of most of our pavements is reaching its terminal point, this issue is a very important one.

This report provides an introduction for the maintenance and design engineer to use in developing precast repair slabs for rapid repair of CRCP. In addition to a survey of the Texas State Department of Highways and Public Transportation maintenance practices on CRCP, a complete literature review of existing techniques was conducted to establish the current state-of-the-art. Thus, with this background, the factors affecting precast slabs are discussed, then a design method is presented along with specific recommendations on design construction, installation, and experimental evaluation of precast slabs. The report also covers the concepts for the mechanistic stress analysis for installation and in-place performance of the precast slabs.

LIST OF REPORTS

Report No. 177-1, "Drying Shrinkage and Temperature Drop Stresses in Jointed Reinforced Concrete Pavement," by Felipe R. Vallejo, B. Frank McCullough, and W. Ronald Hudson, describes the development of a computerized system capable of analysis and design of a concrete pavement slab for drying shrinkage and temperature drop. August 1975.

Report No. 177-2, "A Sensitivity Analysis of Continuously Reinforced Concrete Pavement Model CRCP-1 for Highways," by Chypin Chiang, B. Frank McCullough, and W. Ronald Hudson, describes the overall importance of this model, the relative importance of the input variables of the model and recommendations for efficient use of the computer program. August 1975.

Report No. 177-3, "A Study of the Performance of the Mays Ride Meter," by Yi Chin Hu, Hugh J. Williamson, B. Frank McCullough, and W. Ronald Hudson, discusses the accuracy of measurements made by the Mays Ride Meter and their relationship to roughness measurements made with the Surface Dynamics Profilometer. January 1977.

Report No. 177-4, "Laboratory Study of the Effect of Non-Uniform Foundation Support on CRC Pavements," by Enrique Jiminez, B. Frank McCullough, and W. Ronald Hudson, describes the laboratory tests of CRC slab models with voids beneath them. Deflection, crack width, load transfer, spalling, and cracking are considered. Also used is the SLAB 49 computer program that models the CRC laboratory slab as a theoretical approach. The physical laboratory results and the theoretical solutions are compared and analyzed, and the accuracy is determined. August 1977.

Report No. 177-6, "Sixteenth Year Progress Report on Experimental Continuously Reinforced Concrete Pavement in Walker County," by Thomas P. Chesney, and B. Frank McCullough, presents a summary of data collection and analysis over a 16-year period. During that period, numerous findings resulted in changes in specifications and design standards. These data will be valuable for shaping guidelines and for future construction. April 1976.

Report No. 177-7, "Continuously Reinforced Concrete Pavement: Structural Performance and Design/Construction Variables," by Pieter J. Strauss, B. Frank McCullough, and W. Ronald Hudson, describes a detailed analysis of design, construction, and environmental variables that may have an effect on the structural performance of a CRCP. May 1977.

Report No. 177-9, "CRCP-2, An Improved Computer Program for the Analysis of Continuously Reinforced Concrete Pavements," by James Ma and B. Frank McCullough, describes the modification of a computerized system capable of analysis of a continuously reinforced concrete pavement based on drying shrinkage and temperature drop. August 1977.

Report No. 177-10, "Development of Photographic Techniques for Performance Condition Surveys," by Pieter J. Strauss, James Long, and B. Frank McCullough, discusses the development of a technique for surveying heavily trafficked highways without interrupting the flow of traffic. May 1977.

Report No. 177-11, "A Sensitivity Analysis of Rigid Pavement-Overlay Design Procedure," by B. C. Nayak, B. Frank McCullough, and W. Ronald Hudson, gives a sensitivity analysis of input variables of Federal Highway Administration computer-based overlay design procedure RPOD1. June 1977.

Report No. 177-12, "A Study of CRCP Performance: New Construction versus Overlay," by James I. Daniel, B. Frank McCullough, and W. Ronald Hudson, documents the performance of several continuously reinforced concrete pavements (CRCP) in Texas. April 1978.

Report No. 177-13, "A Rigid Pavement Overlay Design Procedure for Texas SDHPT," by Otto Schnitter, B. Frank McCullough, and W. Ronald Hudson, describes a procedure recommended for use by the Texas SDHPT for designing both rigid and flexible overlays on existing rigid pavements. The procedure incorporates the results of condition surveys to predict the existing pavement remaining life, field and lab testing to determine material properties, and elastic layer theory to predict the critical stresses in the pavement structure. May 1978.

Report No. 177-14, "A Methodology to Determine an Optimum Time to Overlay," by James I. Daniel, B. Frank McCullough, and W. Ronald Hudson, describes the development of a mathematical model for predicting the optimum time to overlay an existing rigid pavement. May 1979.

Report No. 177-15, "Precast Repair of Continuously Reinforced Concrete Pavement," by Gary E. Elkins, B. Frank McCullough, and W. Ronald Hudson, describes an investigation into the applicability of using precast slabs to repair CRCP, presents alternate repair strategies, and makes new recommendations on installation and field testing procedures. May 1979.

ABSTRACT

An investigation into the applicability of precast slabs for rapid repair of CRCP is presented. Analytical techniques are used to study loading of repair slabs due to volume change, wheel loads, and lifting, with detailed results presented in several of the appendices.

Rapid repair of CRCP with precast slabs appears feasible. Calculations indicate that slabs longer than 7 feet (2.13 m) have potential problems with excessive steel stress at the construction joints. These stresses can be controlled with a weakened plane, which causes the concrete to crack at mid-span, thus lowering the stresses. A precast slab can be satisfactorily lifted from four lifting points secured into the slab and situated between 1/5 and 1/4 the span length from each edge of the slab.

Steel connections to maintain the continuity of steel reinforcement in CRCP is a critical aspect of the repair. These connections can be made by welding, clamping, or use of commercial rebar connectors. Polymer concrete is a fast setting material which has excellent properties as a repair material for use around these connections. The set time of the material to be placed around the steel connections will, to a large extent, govern the overall repair time.

SUMMARY

It is important that repair of punchouts, spalls, and other severe defects in continuously reinforced concrete pavements (CRCP) be performed in a minimal time, use materials readily available, be structurally sound and long lasting, and be economical. This is of particular importance on CRCP which is used primarily for high volume roads where hazards due both to the defect and to repair of the defect are great. This report is an initial investigation into the design and construction of precast slabs as an innovative approach to the repair of CRC pavements.

In this report, the most important of the factors affecting the design of precast repair slabs are established first. A series of alternate strategies are discussed next. Finally, the structural analysis and the design procedure are outlined along with a set of guidelines for experimental installation in Texas.

IMPLEMENTATION STATEMENT

A study has been completed which establishes a procedure for the use of precast slabs for the rapid repair of CRCP. It is strongly recommended that the procedure be implemented by the Texas SDHPT on appropriate repair projects as soon as possible. Also, the procedure should be incorporated into the relevant section of the Texas SDHPT operations and Procedures Manual. The actual procedure for installation is described in Chapter 6 of this report.

TABLE OF CONTENTS

CHAPTER 1. INTRODUCTION

It is important that repair of punchouts, spalls, and other severe defects in continuously reinforced concrete pavements (CRCP) be performed in a minimal time, use materials readily available, be structurally sound and long lasting, and be economical. This is of particular importance on CRCP which is used primarily for high volume roadways where hazards due both to the defect and to repair of the defect are great. This report is an initial investigation into the design and construction of precast slabs as an innovative approach to the repair of CRC pavements.

BACKGROUND

Continuously reinforced concrete pavements were developed to reduce many maintenance problems encountered princially at joints in concrete pavements. Longitudinal reinforcing steel was incorporated throughout the length of the pavement to keep transverse cracks tightly closed. Thus, CRCP is constructed with no transverse joints for contraction or expansion. However, it was found through experience, that CRCP exhibited a wide variety of structural responses and that maintenance on some sections was required sooner than anticipated.

From past surveys of in-service CRC pavements, six distinct distress patterns have been identified. These are transverse cracking, radial cracking, longitudinal cracking, spalling, pumping, and punchouts (Ref 1). The most serious manifestations of these distresses are punchouts, where portions of the pavement become detached and ejected or depressed, leaving a pothole. In addition to posing a hazard to vehicular operation, punchouts may lead to further deterioration of the surrounding concrete and subbase through impact loading and water penetration. Thus, repair of distress on CRCP warrants prompt attention.

CAST-IN-PLACE REPAIR OF CRCP

In Texas, repairs of punchouts are the most common repairs performed on CRCP. A survey of the maintenance practices of district maintenance personnel of the Texas State Department of Highways and Public Transportation (SDHPT) revealed a wide variation in repair techniques, materials, and successes. This survey is presented in Appendix 1. Several of the more common techniques are described below.

Asphaltic mixtures have been used to a great extent to repair punchouts in CRCP. Cold laid asphaltic mixtures are commonly used as a temporary measure to promptly restore service to a portion of pavement. Hot-mixed hot-laid asphaltic concrete was reported as a temporary repair material in most districts; however, several districts considered repairs with this material as permanent. Simply stated, the repair of punchouts with asphaltic mixtures consists of removal of deteriorated concrete, application of a bituminous tack coat to the sides of the hole, placement and compaction of the asphaltic repair material, and then opening the repair to traffic.

The majority of districts considered only repairs constructed with portland cement concrete as permanent repairs on CRCP. The preferred method with these cast-in-place repairs was to replace the deteriorated portion of the pavement with a sound portion identical in design to the original CRCP. The repairs were performed by removing the deteriorated concrete, leaving the longitudinal steel at the ends of the repair exposed. Steel reinforcement for the repair was either welded or spliced to the exposed steel. The concrete was then placed and cured. Curing times as long as a week were reported.

Several negative aspects of these repair practices have been long lane closure times, faulting, shoving, and deterioration of surrounding pavement. In some instances, results of the use of portland cement concrete patches have been so poor that some SDHPT districts in Texas no longer use portland cement concrete as a repair material for CRCP (Appendix 1). Several of these problems are shown in Figs 1.1 through 1.3.

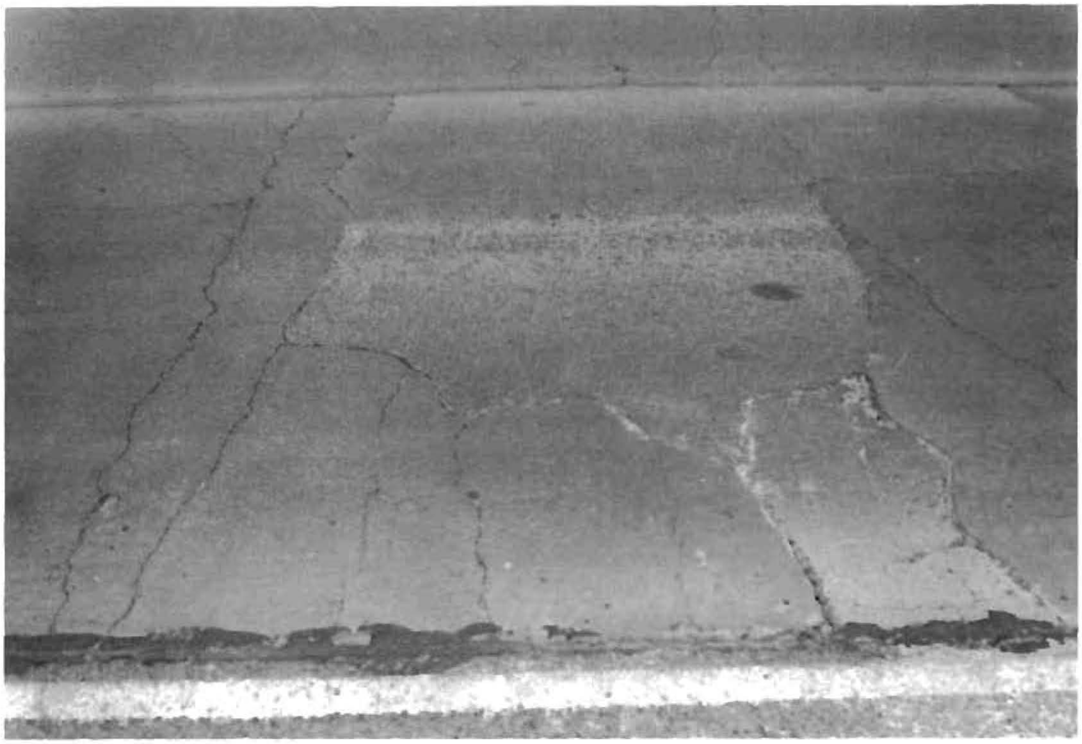


Fig 1.1. Portland cement concrete patch with further deterioration adjacent to patch.

Fig 1.2. Asphaltic concrete patch exhibiting shoving.

Fig 1.3. Multiple asphaltic patches; it appears there are five patches on top of each other in this location.

PRECAST CONCRETE PAVEMENT REPAIR

The repair of concrete pavement in the United States using precast concrete slabs is not a new idea. Previous precast repair work has been performed on jointed concrete pavements. Several of the repair methods reported in the literature are briefly summarized below.

A test program to evaluate precast and cast-in-place repair of joints in jointed reinforced concrete pavement was begun in Michigan in 1971 (Ref 2). The precast slabs used in this program were 8 or 9 inches (203 and 229 mm) thick, 12 feet (3.66 m) wide, and between 6 and 12 feet (1.83 to 3.66 m) long. The slabs were cast to standard sizes at a maintenance yard and then transported to the repair sites. The slabs were installed on grout beds struck off to the proper height. The repairs were made with and without dowel bars along the transverse edges for load transfer. When dowel bars were used they were installed by drilling holes into the vertical faces of the adjacent pavements at middepth. The dowel bars were greased and then inserted into the holes prior to slab placement. An asphaltic filler board was also placed along the transverse edges. After the slab was placed, the dowel bars were welded to a steel plate cast into the end of the slab. The joints at the edges of the slab were then sealed with a hot-poured rubber-asphalt sealant. The procedure used in these precast repairs resulted in average lane closure times of two hours and 40 minutes for doweled slabs and one hour and 25 minutes for undoweled slabs. Repair costs in 1971 and 1972 for these precast slab repairs were $41 and $52 per square yard, respectively.

In 1972 the Florida Department of Transportation used precast slabs to replace fractures jointed concrete pavement slabs (Ref 3). The 12 X 20-foot (3.66 X 6.10-m) slabs were cast and cured at a maintenance yard, transported to the repair site, placed in the prepared area one-half inch (12.7 mm) below the elevation of the pavement surface, and raised to grade by slab jacking with pressurized grout, and then pressed fiberboard with a rubberized asphalt sealant was placed around the edges. This work was performed at night and as many as four slabs were replaced during an eight-hour worknight. The cost of the repair in 1972 was $35 per square yard.

The California Department of Transportation repaired a 30-foot (9.14-m) long portion of concrete pavement on the Ventura freeway with two precast slabs in 1974 (Ref 4). The two slabs were 12.3 and 17.4 feet (3.75 and 5.31 m) long, 11.4 feet (3.47 m) wide, and 8 inches (203 mm) thick. The slabs were cast in an area close to the repair site and placed on a prepared grout bed, and the remaining spaces at the edges of the slabs were filled with concrete grout. The total lane closure time was seven hours, 15 minutes. The total cost of the repair was under $3,000, or, approximately, $75 per square yard.

An experiment was begun in 1974 by the Virginia Department of Highways and Transportation to evaluate repair of concrete pavement with partial-depth precast slabs (Ref 5). The slabs were 2 inches (51 mm) thick, ranging from 1 X 1 foot to 2 X 3 feet (.3 X .3 m to .61 X .91 m). Some of the slabs were formed with a hydraulic press and others were, more conventionally, formed by casting with metal fibers. Deteriorated areas were prepared by cutting a 2.5-inch (64-mm) deep portion of pavement with a Klarcrete cutting machine. Some of the precast panels were cut to size at the repair site with a bench type masonary saw. The slabs were seated in a bed of epoxy grout. The average installation time per patch on this project was 77 minutes. This does not account for lane closure time. The average cost of the hydraulic pressed precast slab repairs was $25 per square foot.

The New York State Thruway Authority used prestressed precast concrete slabs as large as 13 X 30 X .75 feet (3.96 X 9.14 X .23 m) to replace deteriorated portions of a jointed concrete pavement in 1974 (Ref 6). The steam cured slabs developed a 5,000 psi (34,500 kPa) compressive strength in three days. The slabs were transported to the site and installed overnight. As much as 120 square yards of replacement were completed per night.

PRECAST METHOD APPLIED TO CRCP

The application of the precast method to repair of CRCP has an additional complicating factor when compared to its use in the repair of jointed concrete pavements. This complication arises from the necessity to maintain steel continuity in the CRC pavement. In terms of volume change stress, restraint of the steel at the end of a reinforced concrete slab induces significant stress increases over the unrestrained condition. These stresses may become destructively excessive and must be accounted for in design. The precast repair methodology as applied to CRCP, illustrated in Fig 1.4, consists of replacing the deteriorated pavement with a precast slab, anchoring the steel at the ends of the repair, and then filling the space around the steel connections with a fast-setting cast-in-place material.

OBJECTIVES AND SCOPE OF STUDY

This report is the presentation of a study into the development of the precast methodology for the repair of CRC pavements. The objectives of this study are

(1) formulation of precast repair strategies,
(2) development of analytical techniques,
(3) development of a design procedure, and
(4) presentation of recommendations on precast repairs.

This study is restricted to consideration of repair of CRCP with full-depth, full-lane-width, reinforced precast concrete slabs. The elements of precast repair strategies discussed include fabrication, reinforcement, steel connection, cast-in-place concrete, preparation of deteriorated areas, and installation.

The analytical techniques developed in this study deal with modeling loading conditions with computer programs. These programs are used to estimate a precast slab's response to volume change, lifting, and vehicle loads. Computer programs are used to simplify the very involved calculations required to predict the above responses.

The sequential design procedure developed in this report spans from identification and investigation of the pavement problem to monitoring of the constructed repair. A generalized design approach is adopted and is detailed for precast slab repairs.

Recommendations are made concerning alternate repair strategies and development of the precast repair method through field trials.

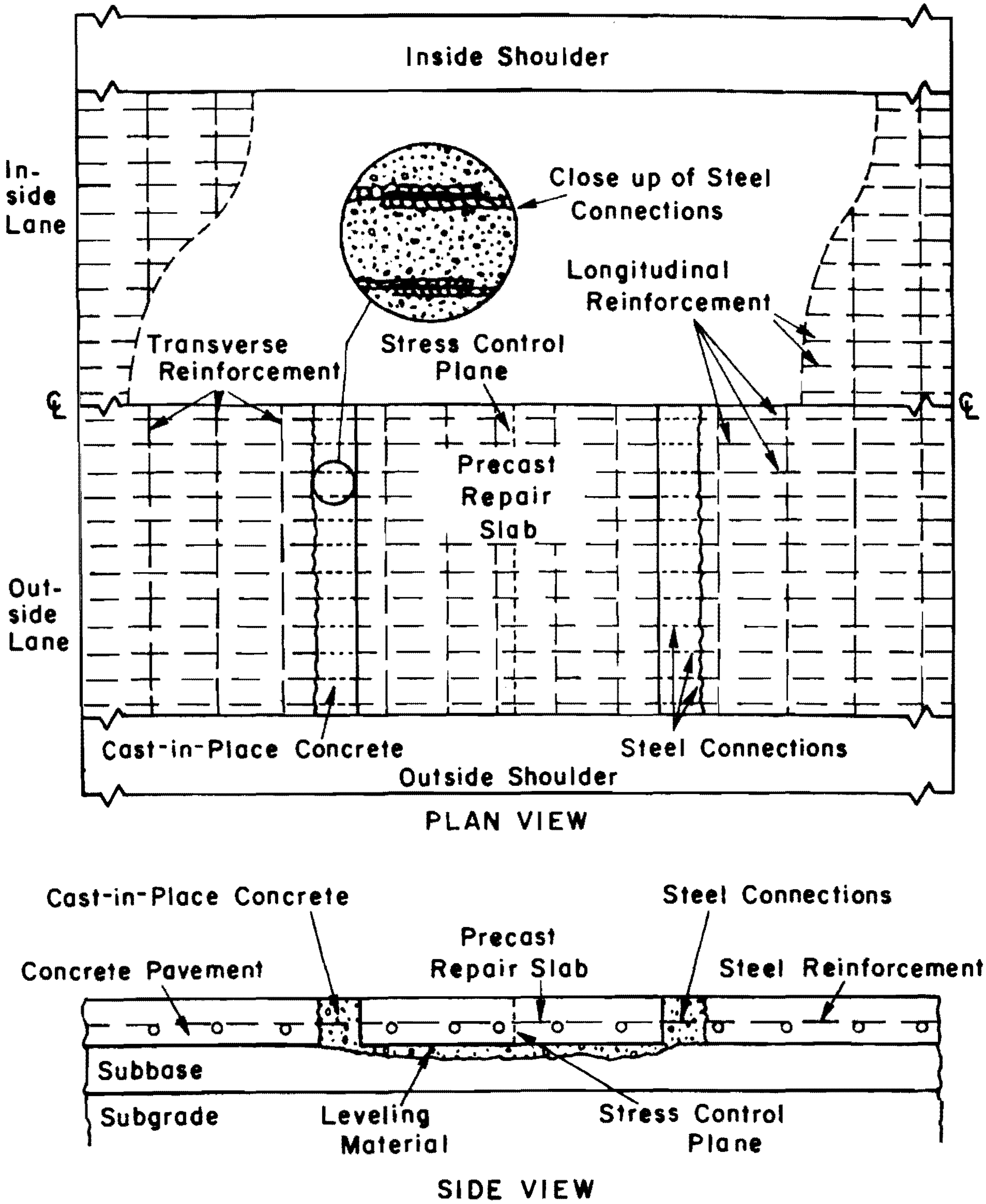


Fig 1.4. Precast repair methodology applied to CRCP.

REPORT ORGANIZATION

This report is organized into eight chapters and six appendices. Chapter 1 is the introduction. Chapter 2 is a discussion of the various factors affecting a precast repair slab. Chapter 3 presents alternate strategies, methods, and elements that might be used for precast repair of CRCP. Chapter 4 presents the analysis of loads on precast repair slabs. Chapter 5 presents a sequential design procedure for precast repairs. In Chapter 6, procedures for the design, preparation, and installation of precast slabs in Texas are presented. In Chapter 7, guidelines for experimental installation in Texas are detailed. Chapter 8 concludes the text with presentation of significant conclusions and recommendations. The six appendices at the end of this report contain detailed investigations into various aspects of precast repair of CRCP. The topics of the appendices are

(1) survey of current maintenance practices on CRCP in Texas,
(2) analysis of wheel load stresses at steel connections,
(3) lifting analysis for precast slabs,
(4) investigation of the effect of voids beneath precast repair slabs,
(5) volume change analysis, and
(6) test of weld and cable clamp steel connections.

CHAPTER 2. FACTORS AFFECTING DESIGN OF PRECAST REPAIRS

The structural response of CRCP is composed of a complex set of interacting elements. When the normal continuity of this pavement type is disrupted, as is the case when full depth repairs are performed, the resulting discontinuities add to the complexity. To rationally design a repair, these factors and their influence must be anticipated and analyzed. Structural response is not the only factor affecting design of precast repair slabs. Other factors, such as fabrication, transportation, and installation, should also be considered. A cognitive method of anticipating the combination of factors which may affect a precast repair slab is to follow the history of a hypothetical slab from fabrication to a period after completion of installation.

The following steps illustrate the history of a hypothetical precast slab. A slab is cast at a convenient location. This includes construction of form work, placement of steel reinforcement, and placement of concrete. After the setting of the concrete, the forms must be removed and the concrete prepared for further curing. After the concrete has achieved sufficient strength, it is ready for transportation to the job site for installation. Prior to installation of the precast slab, deteriorated portions of the existing pavement structure must be removed and the area prepared for placement of the slab. After installation is completed, the slab must resist the effects of repetitive wheel loads and volume change.

This sequence of events illustrates most of the items which influence a precast repair slab. Also influencing the design of precast repair slabs, not specifically stated above, are magnitudes of the various dimensions of CRC pavement slabs. All the items which affect the design, construction, and performance of a precast repair slab will be discussed under the following topics:

(1) dimensions of CRCP,
(2) fabrication,
(3) transportation,
(4) preparation of deteriorated area,
(5) installation, and
(6) loading.

DIMENSIONS OF CRCP

Size compatibility of separately constructed structural elements is an important consideration in all engineered construction. This is of prime importance to precast repair slabs, which must form a smooth riding surface with adjacent pavement. The dimensions of CRCP which influence prefabrication of repair slabs include overall size and shape, reinforcement dimensions, and dimensions of deteriorated areas.

Overall Size and Shape

The overall size of CRCP refers to the pavement's width, thickness, crack spacing, and the thickness of the subbase material. The shape of a pavement refers to the type of crown section.

Most CRC pavements are constructed with 12-foot (3.66-m) lane widths because that width is used almost exclusively on high volume highways. The lane widths of high volume type highways may vary between 11 and 13 feet (3.55 and 3.96 m). Widths of adjacent lanes may not be the same. The width may also vary on turning roadways.

Design and construction data from CRCP highways in the United States, collected under NCHRP research project 1-15 (Ref 1), indicates that pavements have been constructed with thicknesses ranging from 6 to 10 inches (152 to 254 mm). An 8-inch (203 mm) pavement was the most common design thickness. Thickness measurements taken from construction records on several selected sections of CRCP in Texas indicate a maximum variation of less than a half-inch (12.7 mm) from the specified depth (Ref 7). The average depth was slightly in excess of the specified. The maximum depth variation across the pavement was 3/8 inch (9.52 mm), with an average variation of approximately 0.1 inch (2.56 mm).

Some CRC pavements have been constructed with thicknened edges. This type of pavement was used on I-45 in Houston, Texas. This 8-inch (203-mm) pavement has edges 9 inches (229 mm) thick. Thickened-edge CRC pavements are an exception due to the unusually wide lanes required and increased cost of construction.

The crack spacing of a CRC pavement is its effective slab length. The average crack spacing for the sample of CRC pavements surveyed and reported in Ref 1 varied from 1.7 to 12 feet (.518 to 3.66 m). This spacing was found to vary significantly with the steel reinforcement ratio. A typical distribution of crack spacing taken from a section of CRCP is shown in Fig 2.1.

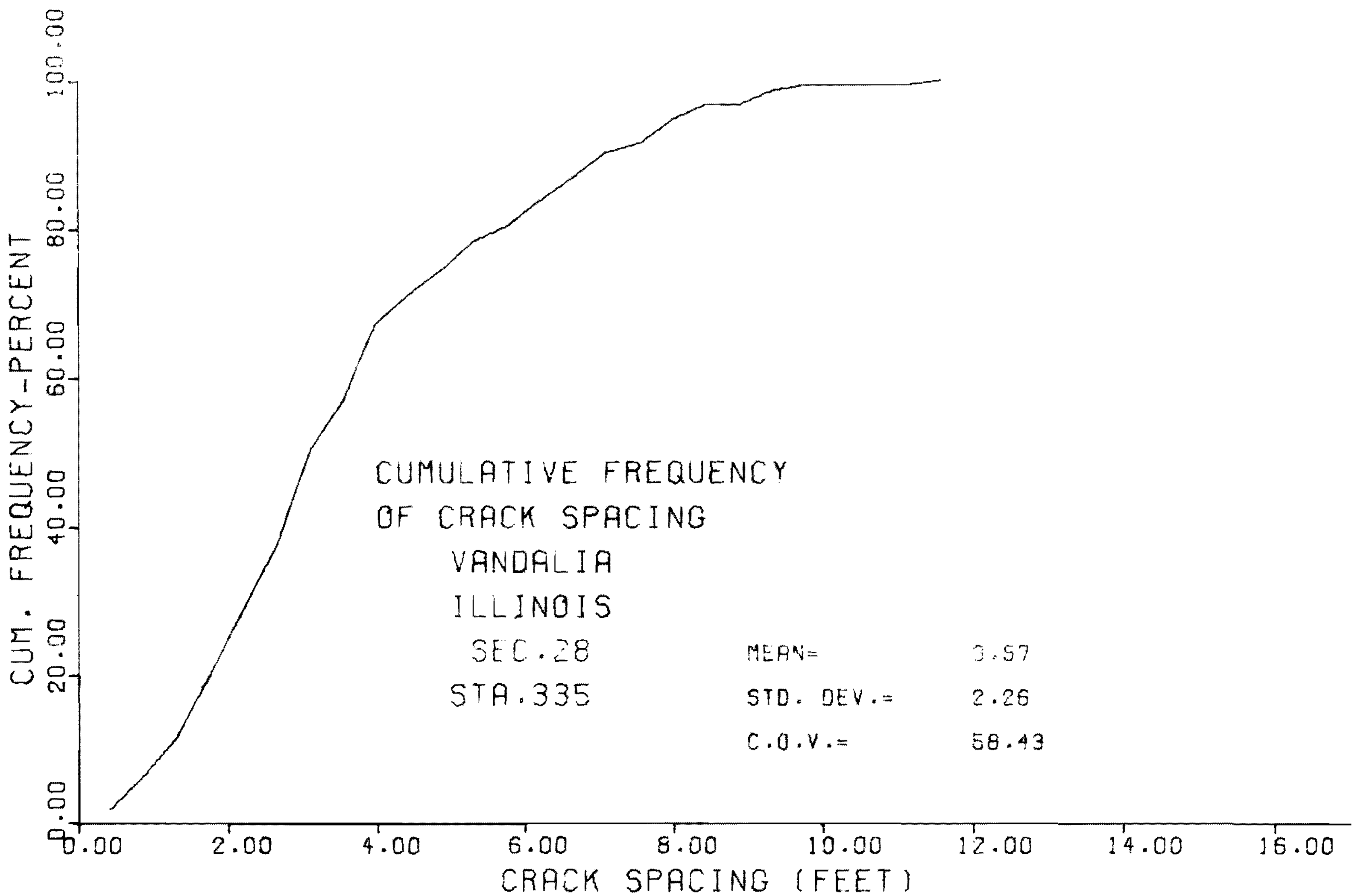
Thickness of a subbase varies with materials, subgrade, and design practice. Six types of subbase materials were reported used for CRCP in 12 states (Ref 1). These included granular gravel, crushed stone, sandy gravel, bituminous treated granular gravel, lime treated granular gravel, and cement treated granular gravel. Thicknesses ranged from 2 inches (50.8 mm) for bituminous treated to 12 inches (304 mm) for granular gravel. Subbase thickness within states tended to be more uniform with respect to different subbase types.

Three types of crown sections are used in pavement construction; plane, curved, and a combination of plane and curved sections. Plane surfaces are used the greatest, most likely due to simpler construction. However, some states use a parabolic or curved crown, which must be accounted for in design. Specifications for cross-slope rates for high type pavement surfaces are between 1/8 to 1/4 inch per foot or 0.01 to 0.02 feet per foot (Ref 8).

Reinforcement Dimensions

The principal reinforcing element in CRCP is the longitudinal steel which is essentially continuous throughout the length of the pavement. Transverse reinforcement is provided to a lesser degree.

The total area of reinforcement is generally specified as a percentage of the concrete cross sectional area. The amount of longitudinal reinforcement in CRCP varies from about 0.4 percent up to a maximum of 1.0 percent (Ref 1). A range from 0.5 to 0.6 percent is typical. At transverse construction joints, the amount of longitudinal reinforcement may be increased



1 ft = .305 m

Fig 2.1. Example crack spacing distribution on CRCP.

as much as 100 percent (Ref 9). The amount of transverse steel varies from 0.08 to 0.25 percent.

Three types of steel reinforcement have been used in CRCP, deformed bars, deformed wire fabric, and welded wire fabric. Diameters of this steel have ranged from 0.33 to 0.75 inch (8.38 to 19.0 mm), with yield points from 33 to 85 ksi (228 to 586 mPa) (Ref 1).

Measurement of steel depths on in-service pavements indicates that the reinforcement is generally located slightly above mid-depth of the slab. An average ratio of steel depth to pavement thickness, derived from data presented in Ref 1, was 0.42, with a standard deviation of 0.059. In terms of an 8-inch (203-m) thick pavement, this is a depth of 3.4 inches (86.4 mm) and a standard deviation of 0.47 inches (11.9 mm).

Steel spacing varies in relationship to the type, size, and amount of steel reinforcement. Spacing of longitudinal steel in pavements reinforced with welded wire fabric tends to be closer than in those reinforced with deformed bars. This is primarily due to the smaller diameter of the welded wire fabric. The same is generally true for deformed wire fabric, which can be obtained in the exact sizes called for in design. Spacing of longitudinal steel in CRCP has been specified from 3 to 12 inches (76.2 to 305 mm) (Ref 1). The steel spacing along the edges of the pavement may be closer than that in the interior of the pavement. Figure 2.2 shows examples of steel spacing specifications with closer spacing of longitudinal steel along the pavement edges.

Spacing of transverse steel in CRCP in the United States has been specified between 12 and 48 inches (30.5 and 122 cm) (Ref 1).

Dimensions of Deteriorated Areas

Deteriorated portions of CRCP requiring replacement may be characterized by one or more of the following: irregular crack pattern, wide cracks, broken and spalled pieces of concrete, percolation of water into the pavement structure, presence of subbase fines on the surface, steel corrosion, and general nonuniformity. The dimensions of these characteristics vary greatly and in some cases are difficult to quantify. Two types of pavement deterioration which require replacement are punchouts and repair patches.

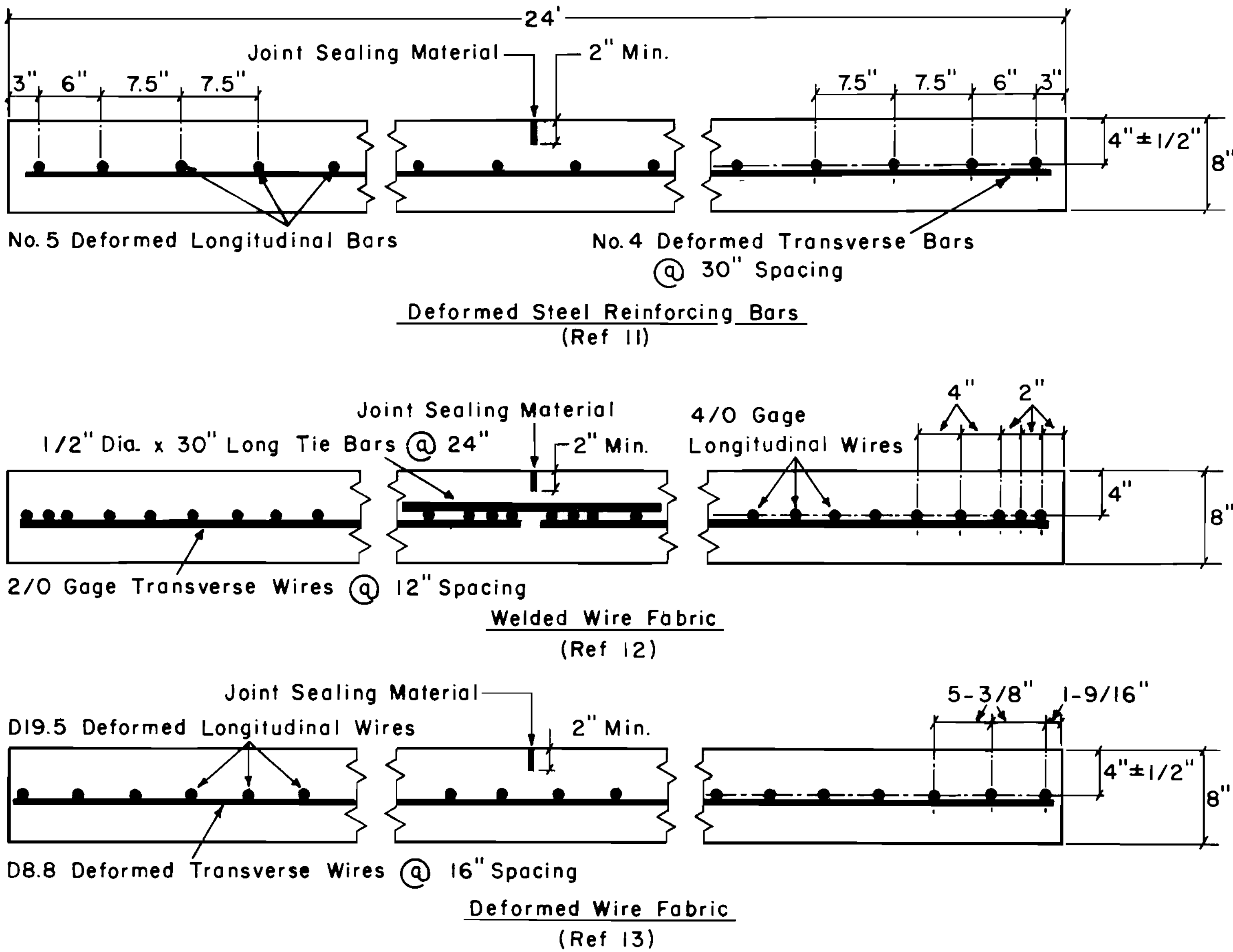
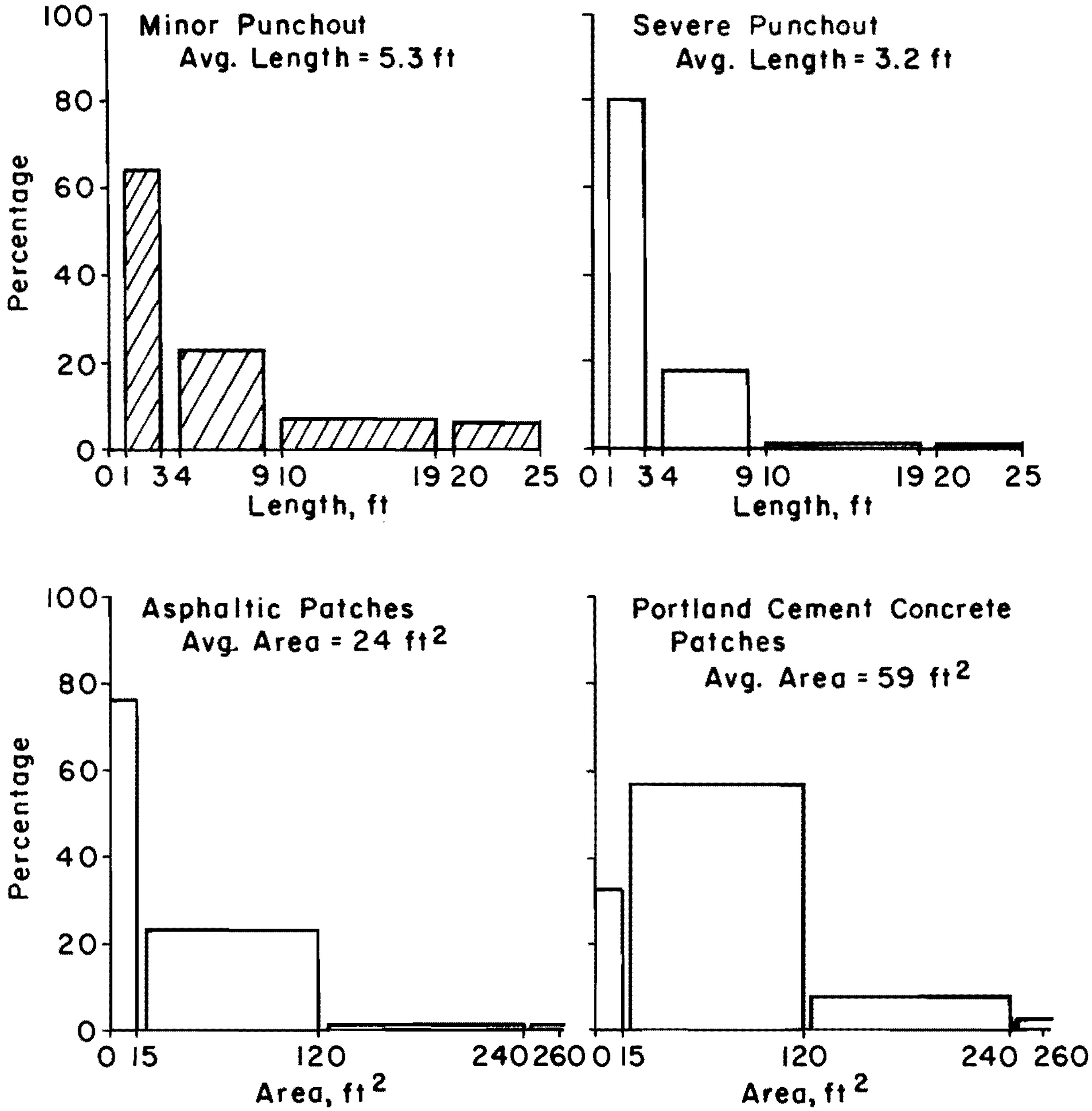


Fig 2.2. Example CRCP cross sections. 1 ft = .305 m, In. = 25.4 mm



1 ft = .305 m, 1 ft ^{2} = .093 m^2

Fig 2.3. Distribution of sizes of punchouts and patches on CRCP in Texas in 1974.

The condition survey of CRC pavements in Texas (Ref 10) collected information on the size of punchouts and repair patches. Punchouts were divided into minor and severe and classified by longitudinal length. Repair patches were divided into those constructed with bituminous materials and with portland cement concrete and classified by area. The raw data collected during this survey were reworked to produce the frequency distribution detailing size shown in Fig 2.3.

The average dimensions indicated in Fig 2.3 are weighted averages. The weighting factors consisted of the midpoints of the inclusive classification intervals and maximum dimensions on the abscissa of each graph for the unbounded intervals.

FABRICATION

Precast slabs may be cast at some convenient location. This location should have facilities suitable for casting and curing concrete and be close to sources of labor and materials and the repair site to minimize transportation. The site should be flat, stable, level, and compatible with the forms to be used. It should be easily accessible to workmen, concrete trucks, and lifting equipment. In certain circumstances, it may be desirable that this location be an area where protection from the elements during casting and curing is available, or near an area convenient for stockpiling of slabs. Convenience and transportation requirements are the main factors influencing the fabrication site selection.

The concrete forms directly affect the finished precast slab. The forms should not move or deflect during placement of the fresh concrete. They should form right angles with adjoining forms and also the casting surface. The forms should be convenient for placement of steel reinforcement, lifting hardware, and steel connection hardware. They should be of uniform size so that opposite inside faces are parallel and the tops form a reference plane for leveling the surface of the slab. They should permit easy disassembly and facilitate removal of the finished slab. A provision should be made for preventing bonding between the concrete and the forms and casting bed. The forms should be made out of a durable material if reuse is desired.

Size is an important consideration in any type of precast work. The length, width, and thickness of the precast slab and the pavement repair location should be compatible. Due to the geographical proximity of CRC pavements for which a particular maintenance group is responsible, it is likely that these pavements were constructed with similar widths, thicknesses, and crowns. Length is expected to vary the greatest. Thus, some provision for varying slab lengths of reuseable forms may be desired for precast slabs.

The steel reinforcement strategy should be considered in designing forms. Provisions for supporting the reinforcing steel might be incorporated into the forms. The steel connections mechanism will affect the forms along the transverse edges of the slab. Provisions for supporting connection hardware or protruding reinforcing steel may be necessary for these pieces. The forms along the longitudinal edges will be influenced by the transverse steel and possible tie bars.

Proper positioning of the reinforcing steel is important. The steel should be supported at the proper elevation. It should not move during placement of the concrete. Adequate spacing should be allowed for passage and vibration of the concrete. Cover of reinforcement along the sides of the slab should be maintained.

The placing of the concrete should assure a solid and dense mass. The reinforcing steel and other details of a precast repair slab should not be disturbed during placement or consolidation.

The surface of a precast repair slab should be textured for skid resistance

The concrete should attain sufficient strength before removal of forms and movement to another location. Cracking of the concrete should be avoided. Curing of the concrete is important to strength gain. Concrete quality control using destructible specimens is another consideration associated with the above factors.

TRANSPORTATION

Preventing damage to precast repair slabs is the main consideration during transportation. A suitable mechanism for lifting and positioning slabs will be required. Large reinforced concrete slabs may be very heavy and awkward to handle. Capacities of both lifting equipment and transporting vehicles may be an important consideration with this size slabs. Movement of the slabs while in the transporting vehicle during transportation to the site should be avoided. Attention should be paid to the safety of workmen during lifting and transporting operations.

PREPARATION OF DETERIORATED AREA

Preparation of the deteriorated area includes removal of concrete, removal of damaged subbase, and other work necessary to prepare an area for installation of a precast slab.

The removal of concrete should include all deteriorated or damaged concrete, be performed as quickly as possible, minimize disturbance to the surrounding concrete and subbase while the dimensions of the removed sections should be within design tolerances. It is important that the concrete be excavated back to sound material. Experience with cast-in-place portland cement concrete patches has shown that there is a tendency for the areas adjacent to the patches to fracture, rock, become dislodged, and require replacement. Figure 2.4 shows such an area, where patches have been placed side by side.

Time is an important element on highway repairs. The quicker the removal of the concrete, the more time there is for installation and curing of the precast repair before traffic must be returned to the area.

Detrimental effects to the surrounding pavement structure should be minimized during removal of concrete. With some of the impact tools used for this purpose, damage to the surrounding concrete may be unavoidable. In this case, the repair should be designed to lessen the effects of the concrete removal equipment.

As with all precast work, the deteriorated area should be excavated to the size of the precast slab. In general, an area excavated too small is better than too large an excavation. An area too small can be enlarged as required. However, an area too large for a precast slab will require greater amounts of cast-in-place materials, which cost more than precast concrete.

It is important that steel continuity in CRC pavements be maintained. The concrete removal equipment should be capable of removing the concrete from around the steel reinforcement to provide for steel connections. The

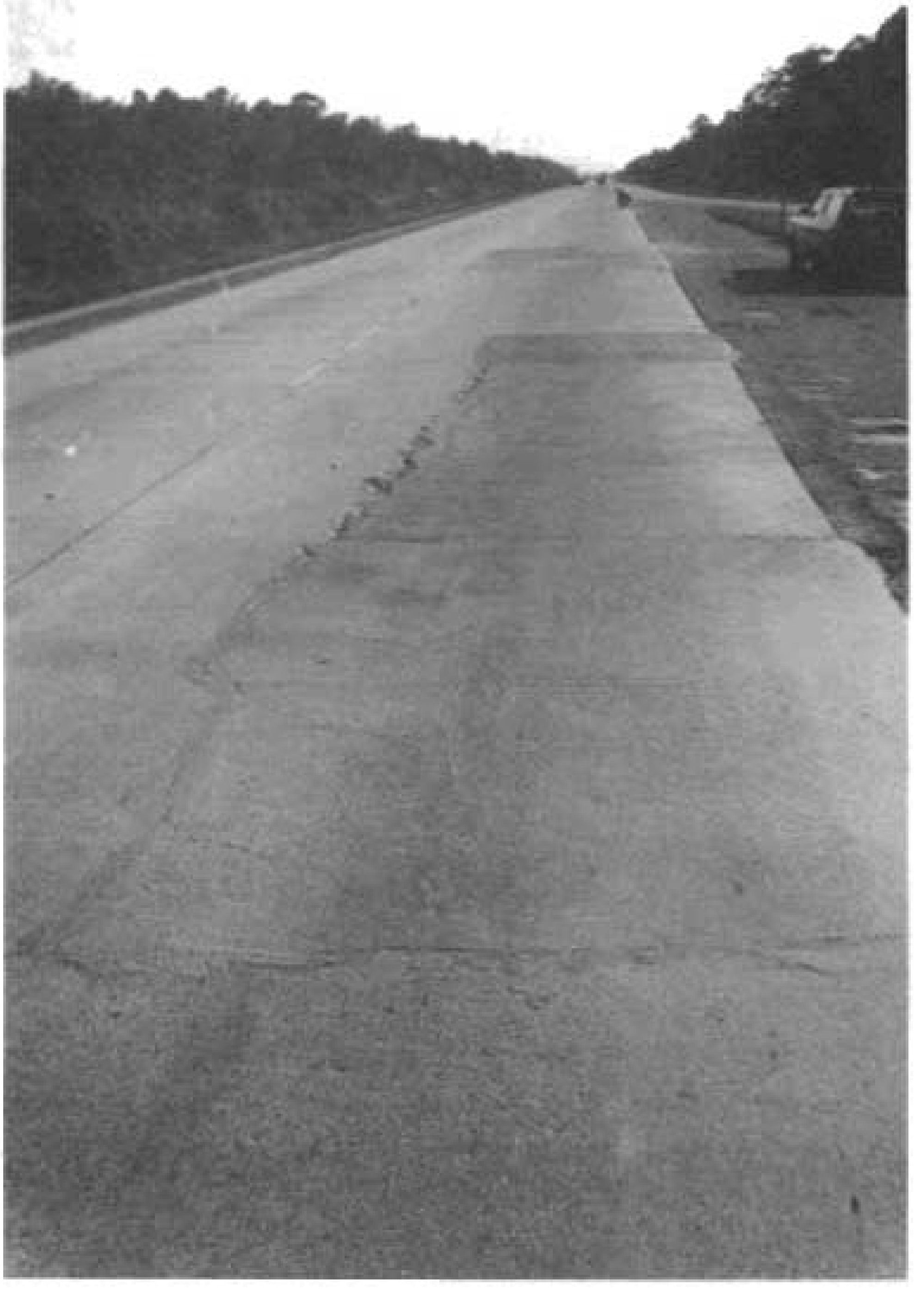


Fig 2.4. Portland cement concrete patches placed side by side.

disruption of steel continuity by cutting of the steel should be maintained for only a short period. Disruption of continuity over a period of temperature variation will create movements in the slab ends which may be destructive to the adjacent lane and also the repair.

The cause of the pavement failure should be determined. Many times it is not possible to determine the cause from inspection of the surface. Observation of the area during concrete removal operations may help in the identification. If possible, corrective steps should be taken to prevent similar failure of the repair.

During the preparation of the area to be repaired, the lane being repaired should be blocked off in a positive manner to protect workmen from vehicular traffic.

INSTALLATION

Installation is a critical step in the repair of pavements. Even with good planning and execution of all the other construction steps and a well designed repair slab, success depends greatly upon installation procedures and methods as well as execution of the same. Elements of installation which should be considered in the design of a precast repair include lifting and positioning, elevation alignment, steel connections, and cast-in-place concrete.

Lifting and Positioning

Slabs must be lifted from the transport vehicle and positioned over the repair. The lifting device should have adequate clearance over the transport vehicle. This device must also be movable to accommodate positioning and lowering of the slab into the repair. During placement, the slab should remain level to achieve a uniform seating. Positioning and maneuvering requirements of the lift device should also be considered.

Elevation Alignment

The surfaces of a repair slab and the pavement must align to produce a smooth transition across the repair. Proper elevation of the surface of a precast repair slab will be aided if the slab is of uniform thickness. A mechanism for elevation adjustment of the slab through the supporting medium which is in direct contact with the bottom of the slab may be required. The support medium should provide uniform support, be easy to place, have adequate strength characteristics, be compatable with the underlying subbase, and be appropriate for use as a subbase. Frictional resistance provided by this material should be accounted for in design.

Steel Connections

Anchorage of the reinforcing steel at the ends of the repair slab is necessary for maintaining steel continuity in CRC pavements. The anchorage mechanism should, ideally, develop the ultimate tensile strength of the steel. In practice, however, a mechanism which only develops the yield strength of the reinforcing steel may be quite suitable for precast repair slabs. When these connections are encased in concrete, they are aided by the bond and bearing of this material.

Some types of steel connection strategies may require positionable connections due to the variation in the position of steel in the pavement. Specifically, it may be necessary to allow for positioning in the vertical, transverse and logitudinal directions.

The anchorage mechanism should not interfere with positioning and placement of a slab into a repair.

Cast-In-Place Concrete

A cast-in-place concrete will be required in the steel anchorage zones to fill the space between the repair slab and pavement and to aid in anchorage of the steel. This material must gain strength rapidly to minimize lane closure time. The material should show good bond to steel and concrete. It should be compatable with the leveling materials used beneath the slab. The material should be convenient to place and show good properties as a pavement surface.

LOADING

A precast repair slab is subjected to different types of loading with varying conditions of restraint. Loading of slabs is caused by concrete shrinkage, thermal volume change, lifting, and wheel loads. Concrete restraint is provided by its bond to the steel reinforcement, friction along the bottom, and lift supports. Steel restraint is provided by its bond to the concrete and connection to steel in the adjacent pavement. A summary of load and restraint conditions expected for precast repair slabs is shown in Table 2.1.

In addition to a verying type of loading, the concrete strength varies with time. Thus, its resistance to the various stress producing agents should be evaluated as a function of time from casting.

TABLE 2.1. EXPECTED LOAD AND RESTRAINT CONDITIONS FOR PRECAST REPAIR SLABS

Description	Loading	Restraint Conditions
Initial casting and curing	Z, ΔT	c: bond to steel, friction with forms and casting bed s: bond to concrete
Form removal and transport to storage	g	c: bond to steel, lift supports s: bond to concrete
Further curing	Z, ΔT	c: bond to steel, friction on surfaces s: bond to concrete
Transportation to repair site and positioning over repair	g	c: bond to steel, lift supports s: bond to concrete
Connection of steel in pavement and slab	Z, ΔT	c: bond to steel, friction along bottom, bond to adjacent pavement s: bond to concrete, connection to steel in adjacent pavement
Open repair to traffic	Z, ΔT, P	Same as above
Period after installation	Z, ΔT, Pn	Same as above

Key:

Z = concrete shrinkage
ΔT = temperature change
g = gravity
P = wheel load
Pn = repetitive wheel loads

c: concrete
s: steel

CHAPTER 3. ALTERNATE STRATEGIES

This chapter presents and discusses several possible alternatives for precast slab repair strategies. The alternatives presented here are examples of possible strategies and should not be interpreted as representing all possible methods and strategies. The designer should formulate his own ideas into strategies. Feasibility and practicality should not be allowed to inhibit the development of imaginative or innovative ideas, even though, in the final analysis, they will, in part, be judged on those merits.

In the design procedure presented in Chapter 5, the formulation of alternate strategies comes after investigation of the problem and prior to analysis of the strategies. Investigation of the problem includes factors such as those discussed in the previous chapter. The insight and information gained in the investigation step can be used as a guide in preliminary evaluation of each strategy. However, it should not inhibit the formulation of ideas. A preliminary evaluation can be used prior to a formal evaluation to reduce the number of alternatives rigorously inspected, as discussed in the next chapter.

The range of solutions to the problem of precast repair of CRCP investigated in this report has been restricted to full-width, full-depth portland cement concrete slabs. This restriction was used here to limit the problem range so that analytic techniques, presented in the next chapter, could be developed.

The elements of a precast slab repair discussed in this chapter are organized under the following topics:

(1) fabrication,
(2) weakened plane,
(3) reinforcement,
(4) steel connections,
(5) cast-in-place concrete,
(6) preparation of deteriorated areas, and
(7) installation.

Various schemes and options are presented under the above headings and implications of these strategies are discussed.

FABRICATION

Fabrication of precast slabs covers the overall casting strategy up to the placing and curing of the concrete. Although casting of precast slabs is the first construction step in this repair technique, it is not the first design step. Fabrication cannot proceed until all dimensions and details have been worked out. The elements of fabrication considered here are overall casting strategy, casting location, concrete forms, concrete placement, and curing.

Overall Casting Strategy

Two overall casting strategies can be adopted for the manufacture of precast slabs. One strategy is to cast slabs to specific dimensions on an individual repair basis. The other alternative is to cast a number of slabs at the same time to a variety of standard dimensions. These slabs may then be stockpiled until required.

On a small scale, casting slabs for specific repairs may result in material savings because they are cast to the required dimensions. Casting standard sizes, however, necessitates using slabs that are larger than repair requirements and must be trimmed to fit. However, mass production of slabs on a somewhat larger scale might prove more economical, due to the generally lower unit price charged for greater quantities of materials.

Unless repairs are planned far enough in advance, the curing requirements for individually cast slabs may delay repairs. Depending on the type of cement and of curing which are used, portland cement concrete can generally achieve sufficient strength in a week. In general, the longer the cure time for precast slabs, the better. The longer the curing period, the less the concrete shrinkage incurred after the steel at the ends of the slab has been restrained. This lessens the magnitude of volume change stresses, which leads to longer fatigue life due to lower stress levels. Multiple casting of slabs avoids possible delays due to curing through continuous availability of a stockpile. Repairs may be performed when conditions warrant.

During development of this repair method, either in general or by specific maintenance organizations for their particular conditions and problems, the individual casting strategy is more appropriate. After designs have been proven through field installations, standarization required for multiple casting strategies can be made.

Casting Location

Precast slabs can be cast at a convenient location. The construction yard of a highway department's maintenance office may be such a location. Many of the materials, equipment facilities, and labor requirements are available. This site is compatible with each overall casting strategy. Use of a central location will require transportation of fabricated slabs to repair sites.

Alternatively, slabs may be cast somewhere near the repair site. This alternative requires transportation of fabrication materials and labor to the casting site, but will reduce transportation requirements for completed slabs. This alternative is principally suited for individual slab casting.

Forms

The slab casting strategy will influence the overall form plan. Multiple casting schemes may be facilitated with a set of continuous side forms with insertable end pieces (Fig 3.1). With this form plan, slab widths are held constant and lengths can be varied with the moveable end pieces. The continuous form set may also be used for individual "tailor-fit" slabs. A variation on this scheme, using a shorter length of reuseable parallel forms of variable lengths, may also be used for individually sized slabs. The other option is to construct individual forms of specific dimensions for each slab.

Materials for form work will depend upon the casting strategy, availability, and cost. Reuseable forms should, preferably, be made with metal. This material generally has good durability, strength, stiffness, and bonding characteristics for use as concrete forms. The American Society for Testing and Materials (ASTM) recommends use of watertight molds made of

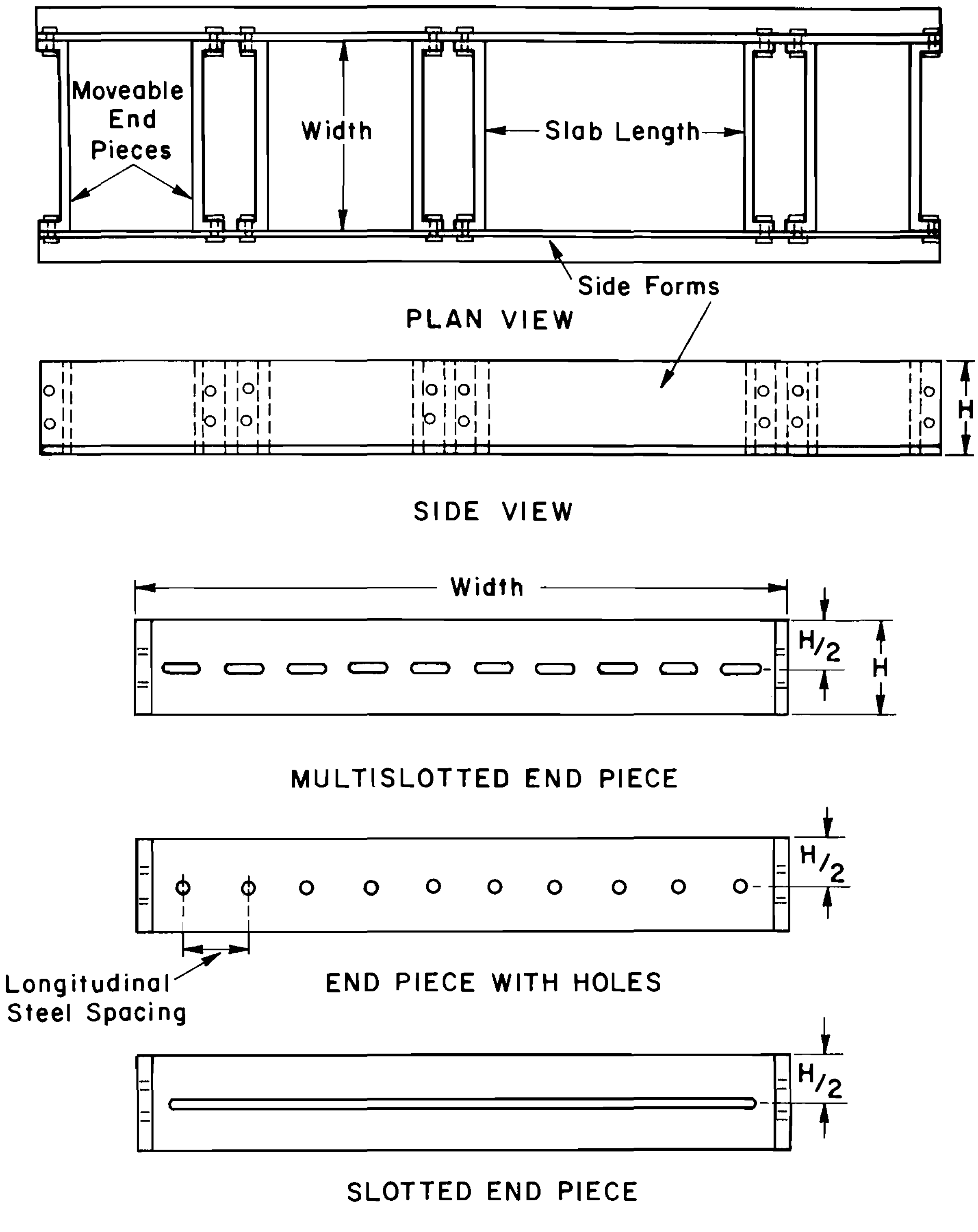


Fig 3.1. Multi-casting slab forms with alternate types of end pieces.

materials which do not react with constituents of portland cement and are resistant to tearing, crushing, and deformation for concrete testing purposes (Ref 14). Iron, brass, steel, and plastic are suggested for reuseable portland cement concrete molds by ASTM. Wood is a commonly used material for concrete forms. Wood is generally less expensive than metals and is readily available. However, it is not as durable, is subject to damage from handling, has undesirable moisture absorption characteristics, and has less uniformity.

Form height should be uniform. For reuseable forms, this height should be equal to the thickest slab expected. Slabs of lesser thicknesses can be cast by constructing a screed of proper dimensions to fit on top of the forms to strike the fresh concrete off at the desired elevation.

The end pieces of the form, which form the transverse edges of the slab, are influenced by the longitudinal reinforcement design and connection strategy. These end pieces occur at the junction of the slab's reinforcement and steel connection hardware, if special connecting hardware is used. In some cases, the steel reinforcement can be extended past the ends of the slab with no special connecting hardware attached. Holes or slots in the end pieces at mid-height, as shown in Fig 3.1, can be used for supporting connection hardware or extending reinforcing steel. The spacing of the openings in the end pieces will depend upon the slab's steel spacing.

Provisions for tie bars along the edge of the slab that will be adjacent to the center of the pavement may be desired. Tie bars, if provided, would only be placed on one edge of the slab. The edge adjacent to the shoulder does not require tie steel. The tie bars can protrude through holes, as in the end pieces, or, as in construction of CRC pavements, the tie bars can be wrapped with nonbonding material and bent against the side form. When the slab is removed from the forms, the tie bars are simply bent into place.

Concrete Pavement

It is important that fresh concrete be adequately consolidated into a dense mass. To achieve this, attention should be given to environmental conditions and vibration. Placement of concrete in an unprotected location during periods of high temperature, low humidity, or high winds should be avoided or preventive measures, such as, using cold water, dampening the forms and casting bed, or sheltering from the wind, should be taken to avoid a rapid initial set. Surface or internal vibrators may be used to assure proper consolidation. It is of paramount importance that concrete beneath the steel receive adequate vibration. Surface vibrators loose effectiveness as the slab thickness increases.

During concrete placement several types of quality control tests can be performed. These include slump tests, Kelly ball tests, and destruction testing of precast cylinder and beam specimens.

Curing

Prevention of evaporation of water from the concrete until it has achieved the desired strength is the essence of concrete curing. Evaporation may be retarded with special pigmented membranes, wet coverings, or a supply of excess water on the surface.

Steam curing is possible with precast slabs. Steam curing accelerates the concrete's strength gain and allows form removal at an early age. It also provides protection during cold weather and makes it possible for higher strength concrete to be attained.

WEAKENED PLANE

The volume change analysis presented in Appendix 5 indicates that precast repair slabs longer than about 6 feet (1.83 m) may have problems with excessive steel stress at the ends of the slab. It is shown in this analysis that these stresses can be controlled by a weakened plane incorporated into the center of the slab. The weakened plane, which is simply the result of a reduction in the effective resisting area of concrete, acts as a safety valve by causing the concrete to fracture at low levels of steel stress. Two shorter length slabs which have lower stress levels are formed.

The weakened plane can be formed by casting a bond breaker into the slab or by cutting a groove in the surface. The first method has two advantages over the second. One advantage is that a greater reduction in concrete area is possible. Saw cuts can be made only to the depth of the reinforcing steel and even then adequate cover over the steel must be maintained. The bond breaker can be placed around the reinforcing steel and is limited only by cover requirements at the surfaces of the slab. The major advantage of a bond breaker is that a defect is not induced onto the surface of the slab until it is required. It is possible that stresses will not become great enough to fracture the concrete and, therefore, that stress control will not be necessary. In this situation, a grooved cut in the surface would become a permanet defect and source of potential problems, whereas the bond breaker would remain encased within the concrete slab and would not affect the slab's performance.

Bond breakers can be a flat plane or an irregular shape. Irregularly shaped bond breakers, such as a corregated metal shape or a zigzag pattern, may aid load transfer by direct bearing on the overlapping surfaces, but it may be difficult to properly consolidate the concrete around the irregular surfaces. A flat plane bond breaker does not add to the load transfer characteristics of the weak plane. Dowel bars may be added to aid the steel reinforcement in the slab with load transfer across this section. Several weakened plane alternatives using bond breakers are shown in Fig 3.2.

The bond breakers should be positioned so that they will not cause the slab to crack during lifting. The bond breaker should be oriented away from the side of the slab which is in tension. Another alternative is to make slots in the bond breaker corresponding to the steel spacing and orient the slotted side of the breaker towards the tension face of the slab. This increases the available concrete area resisting the tension. The lifting mechanism may be designed to induce very little or no tensile stress across the weak plane.

Bond breakers can be made with galvanized metal, aluminum, plastic, bituminous filler board, wood, or steel sheets. The material should not react chemically with the portland cement.

REINFORCEMENT

The principal reinforcement in a precast repair slab to be used for CRCP is the longitudinal and transverse steel. This reinforcement is for volume change control. Other types of reinforcement which may be incorporated into precast slabs include fibers and steel wire.

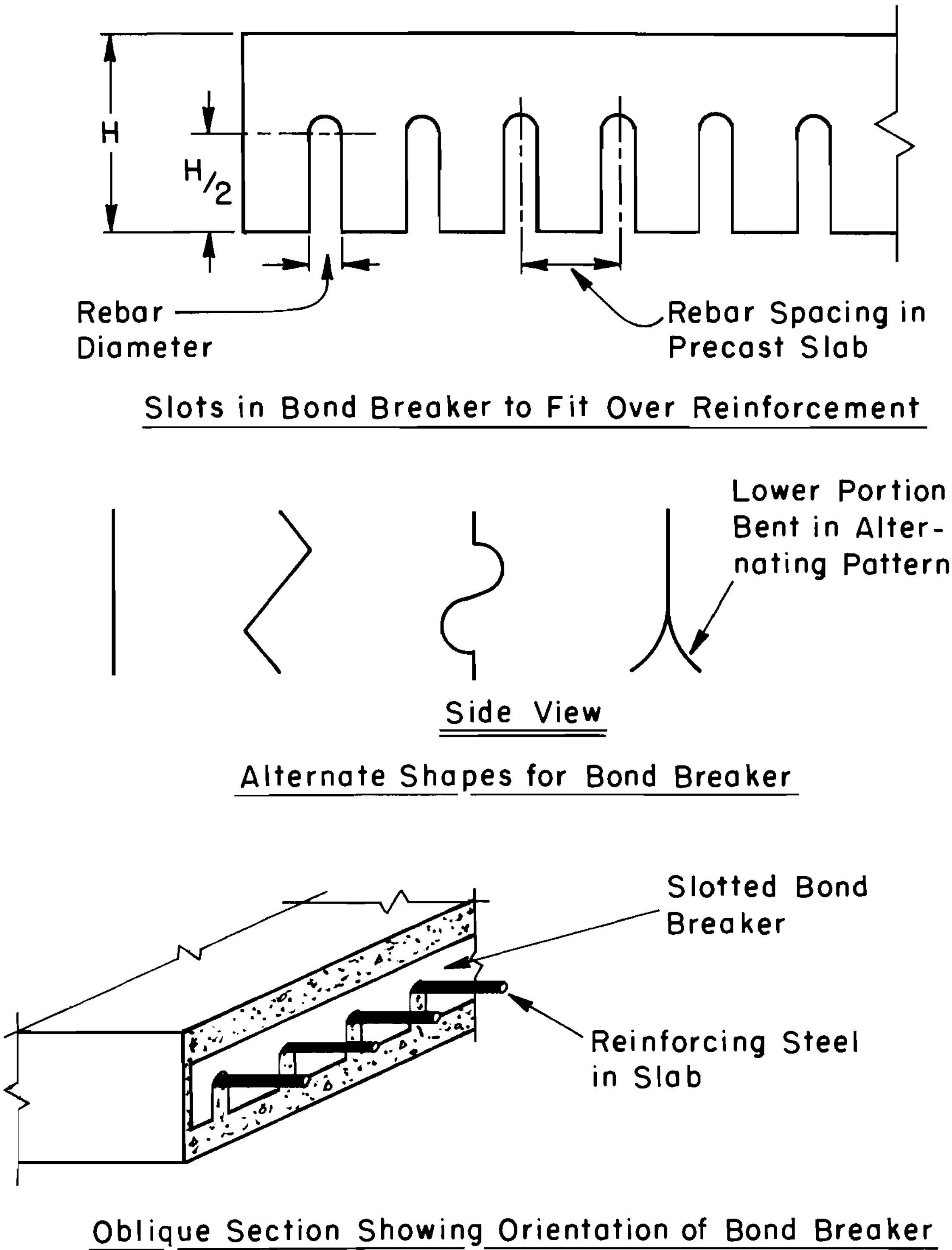


Fig 3.2 Weakened plane configurations.

The longitudinal and transverse reinforcements are described by the amount, type, size, depth, spacing, length, and grade. The simplest way to start slab reinforcement design is to reproduce the reinforcement design in the pavement to be repaired. This design may then be altered to conform to the requirements for precast repair slabs. In general, deformed steel reinforcement should be used because of its bond properties. Two-inch surface cover and one to two-inch side cover over and around the rebars should be maintained (Ref 9). The steel grade should be selected to facilitate possible bending or welding.

Fibers can be added to portland cement concrete to increase its strength and toughness. These short length fibers can be composed of steel, glass, synthetic polymers, or cotton. The amount of fibers required is from 1 to 2 percent (Ref 15). This is a greater percentage of reinforcement than the percentage of steel bars in CRCP. Special provisions are required for the mixing of concrete with this type of reinforcement.

Steel wires can be used around elements subjected to stress concentrations to help control cracking. The high bond provided by the small diameter wire assists in prevention of cracking and maintenance of small crack widths if cracking does occur. These wires may be spiraled around steel connection parts and lift points. They may also be used at plane surfaces used to form a weakened plane.

STEEL CONNECTIONS

To maintain continuity in the longitudinal reinforcement of CRCP, a steel connection mechanism to use with precast repair slabs will be required. The connections may be positive, passive, or a combination of positive and passive. A positive connection is defined here as a connection in which the principal connection is achieved through direct bonding of rebars. A passive connection is achieved by anchoring the bars without direct contact. Some of these connection devices will also require a positioning mechanism to allow alignment of bars to be connected.

Positive Connection

Alternatives for positive steel connections include bonding of bars by welding, commercial splicer, and cable clamps. To adjust to variations in the alignment of reinforcement, these types of connections may require a positioning mechanism. Each of these alternatives is discussed in the following subsections.

Weld Connection. A weld connection can be made for each rebar in the CRC pavement or groups of bars can be connected. Examples of these alternatives are shown in Fig 3.3.

Perhaps the simplest type weld connection is the single bar lap weld. A separate connection is provided for each rebar in the pavement. There can be as many as 35 bars in one lane of a CRC pavement. Pavements which have this many bars are usually constructed with either deformed wire fabric or welded wire fabric. CRCP constructed with deformed bars has fewer bars due to the larger size. Thus, individual connections appear to be more suitable for CRC pavements with deformed bars because of the large number of connections required for the wire mats.

Conversely, multiple connections appear suited for CRCP with wire mats. The multiple connection of bars in the pavement will result in a reduction in the number of connectors to the precast slab. It may be possible to use stationary connection bars attached to the precast slab with this connection option. As shown in Fig 3.3, the stationary bars are cast in positions so that they will not interfere with slab placement. A steel plate can then be welded to the connection bars and pavement rebars. It is expected that the depth of steel in wire mats across the transverse profile of the pavement is more uniform than in individually placed rebars. This is due to the closer spacing and welded connections of the prefabricated wire mats.

Commercial Connectors. Several types of commercially available rebar connectors have been developed for structural application in buildings and nuclear power plant construction. These connectors generally develop the ultimate strength of the reinforcing steel over a short length.

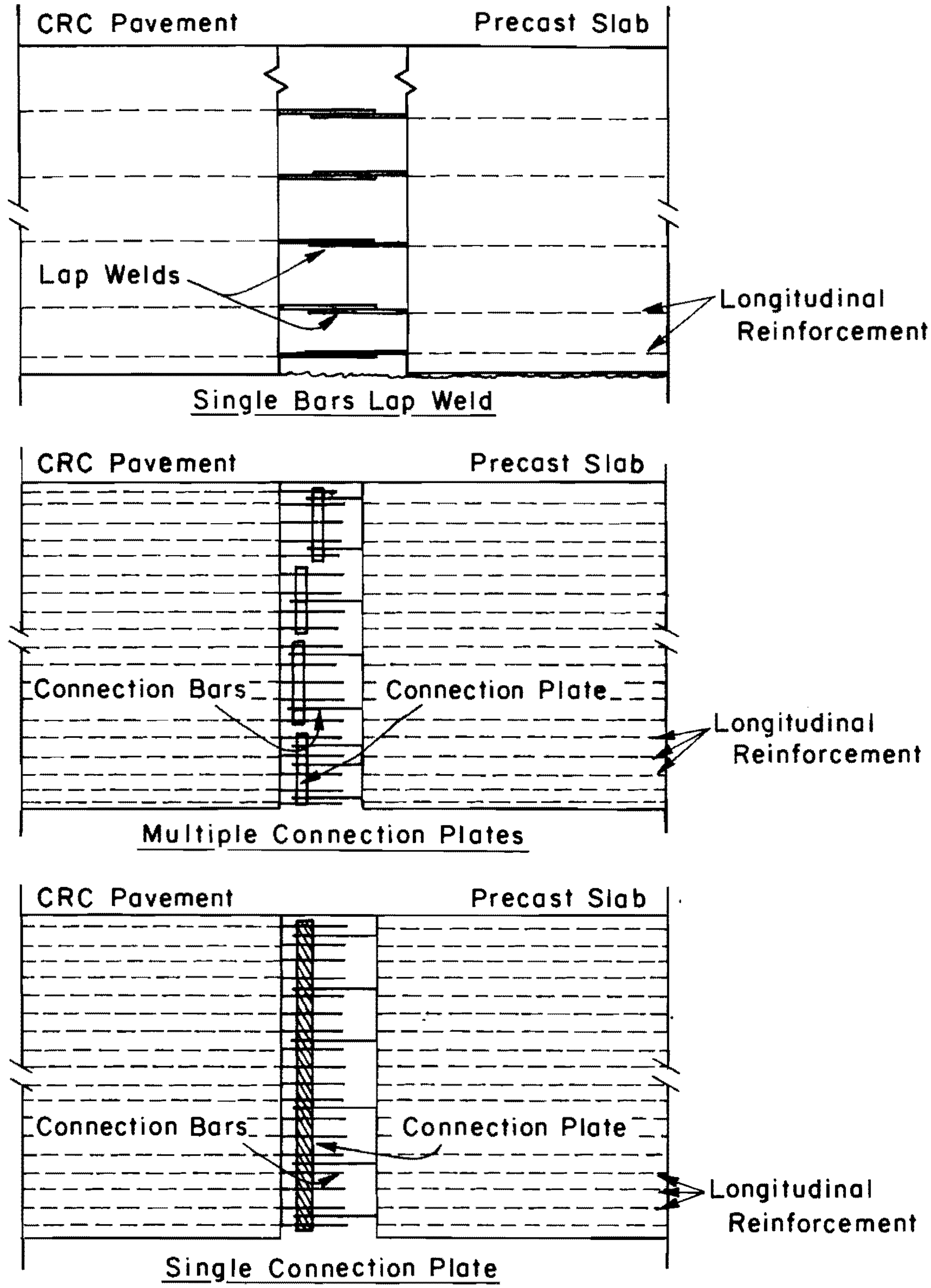


Fig 3.3. Welded steel connection alternatives.

Erico Products, Fox-Howlett Industries, Splice Sleeve Japan, Sylgab Steel and Wire, and Williams Form Engineering Corporation are companies which market connectors which use a sleeve to form connections (Refs 16, 17, 18, 19, and 20). The connection sleeve is either filled with molten metal, filled with a fast setting mortar, heated and compressed around the bars, or screwed into position through threads on the ends of the bars. For the most part, the connected rebars must be aligned end to end. With some of these connectors, some variation in alignment is permissible.

Other connection mechanisms commercially available through many companies in the United States are cable clamps. These clamps consist of a u-bolt and a curved seat. They are designed for connecting cable for applications such as telephone pole stays. A simple series of tension tests of rebars connected with these clamps, reported in Appendix 6, indicates that the clamps can develop forces in excess of the steel's yield point.

Cable clamp connections are lap type splices. This allows for considerable freedom in bar alignment since the bars may be joined in any orientation as long as they are parallel to each other and touching. It may be possible to provide connections with cable clamps, if the steel spacing is known with sufficient accuracy, by simply extending the rebars in the precast slab past the end of the slab. Positioning devices allowing movement in the transverse direction or the transverse plus vertical directions, if steel spacing varys greatly, may also be used with these clamps.

Positioning Device. Positive steel connections may require a positioning device for steel alignment. The steel reinforcement in the CRC pavement may vary in the vertical and horizontal (transverse) directions. Depending on the connection device, it may be required that the connecting bar be positionable in the longitudinal direction as well as the horizontal and vertical directions. Those types of connectors requiring an end-to-end alignment of bars would require longitudinal adjustment. Lap connectors require only horizontal and vertical alignment.

A drawing of the positioning device is shown in Fig 3.4. The device consists of a slotted metal track cast across the transverse edge of the precast slab with insertable nuts which attach to positioning hardware. The nut

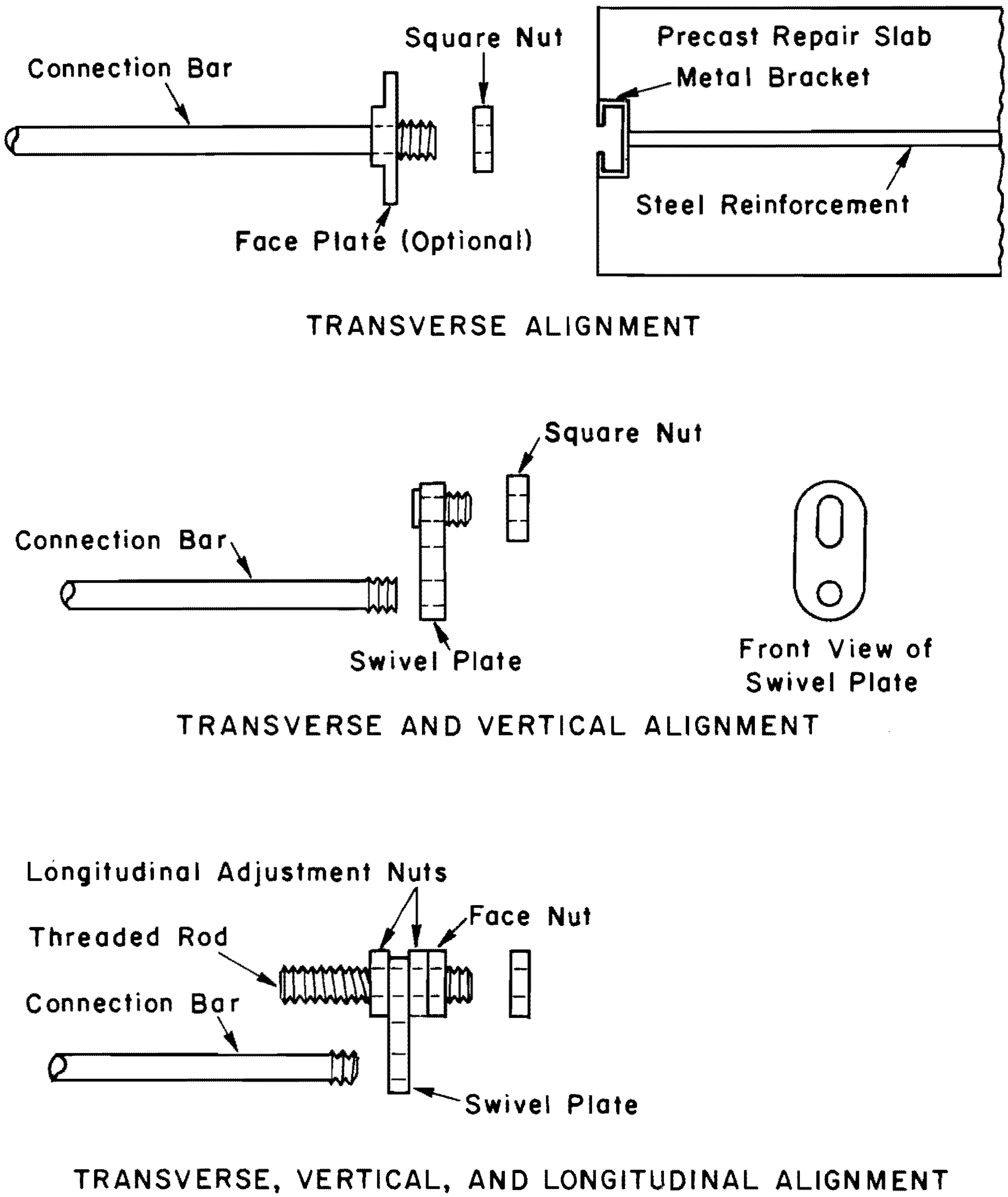


Fig 3.4. Steel connection positioning devices.

and track are of appropriate size so that the nut will both slide through the track and be restrained from turning by the track. The track allows positioning in the transverse direction. The other connection hardware shown in Fig 3.4 allows further positioning.

Passive Connection

The purpose of steel connections is to provide continuity to the reinforcing steel. This continuity acts to anchor the reinforcing steel at the cracks in CRCP, which restrains the concrete slabs in between. Thus, the important physical phenomenon of continuity is anchorage of the reinforcing steel relative to the adjacent concrete slabs. At discontinuities in the normal continuity of CRCP, it may be possible to anchor the reinforcing steel without direct connection of the bars with each other.

The development length of deformed reinforcing bars is influenced by a number of factors. Development length is directly proportional to bar size and steel grade and indirectly proportional to concrete strength. Bond development is also influenced by steel spacing and spiral reinforcement around the bars (Ref 21). Slip characteristics of anchorage bars are significantly improved with a symmetric bearing surface at the ends of the bars (Ref 22).

The development length of steel in the CRCP and the precast slab is the important consideration for use of the passive connection. The development lengths of Numbers 4, 5, and 6 grade 60 deformed rebars, common rebars used in CRCP, encased in normal portland cement concrete, according to the American Concrete Institute (ACI) building code (Ref 21), are 12, 15, and 18 inches (305, 381, and 457 mm), respectively. These lengths are for simple embedment. Development lengths could be reduced through the use of spherical bearing surfaces at the ends of the bars, spiral reinforcement around the bars, or high strength concrete.

A high-strength fast-set concrete which possesses excellent qualities for use in pavement repair is polymer concrete. No data on the development length or bond characteristics of this particular material could be located. However, inference of its properties can be drawn from pull-out tests on polymer impregnated concrete. The data shown in Table 3.1 were collected

TABLE 3.1. ULTIMATE PULLOUT STRENGTH OF NO. 4 DEFORMED BAR EMBEDDED IN POLYMER-IMPREGNATED CONCRETE (Ref 23)

Test Temperature, °C	Unimpregnated Concrete		DAP-MMA Impregnated Concrete			Styrene-TMPTMA Impregnated Concrete		
	Specimen No.	Ultimate Strength, psi	Specimen No.	Polymer Loading, %Wt	Ultimate Strength, psi	Specimen No.	Polymer Loading, %Wt	Ultimate Strength, psi
21	7103	1,510	7001	6.8	1,970	7006	5.8	1,990
	7104	1,570	7004	5.8	1,960	7019	5.3	1,930
	7108	1,410	7008	7.8	1,900	7026	5.8	1,970
	7110	1,580	7009	7.4	*1,980			
			7014	7.3	1,950			
	Average	1,520	Average	7.0	1,950	Average	5.5	1,960
121	7101	1,230	7007	7.4	1,740	7005	5.9	1,620
	7106	1,290	7012	7.9	1,700	7013	5.4	1,540
	7109	1,040	7021	7.4	1,860	7023	4.4	1,640
	7112	1,170	7029	7.6	1,680	7030	5.9	1,570
	Average	1,180	Average	7.6	1,740	Average	5.4	1,590
143	7102	1,110	7002	6.9	*1,860	7016	5.4	1,480
	7105	1,170	7003	7.8	1,770	7018	5.3	1,500
	7107	1,090	7015	6.3	1,720	7025	4.9	1,520
	7111	1,220	7024	7.4	*1,820	7028	6.8	1,700
	Average	1,150	Average	7.1	1,790	Average	5.6	1,550

*On these specimens, the reinforcement rod failed in tension.
DAP-MMA = (90 - 10) diallyl phthalate-methyl methacrylate.
Styrene-TMPTMA = (60 - 40) styrene-trimethylolpropane trimethacrylate.

$$\text{Ultimate strength} = \frac{\text{total load}}{\text{embedded surface area of rod}}$$

psi = 6.89 kPa

for Number 4 rebars embeded in 6-inch-diameter x 6-inch-long (152 by 152 mm) cylinders (Ref 23). In comparison to the unimpregnated concrete, the impregnated specimens developed between 30 and 55 percent greater ultimate strengths. Three of the 13 specimens failed through tensile failure of the steel.

The above results are significant because the bond properties of polymer concrete are thought to be greater than those of polymer impregnated concrete. The average bond strengths in Table 3.1 in terms of stress on the cross-sectional area of the rebar ranged from 84 to 94 ksi (579 to 648 mPa). This stress level is greatly in excess of the yield strength. These results strongly suggest that the development lengths for steel bars commonly used in CRCP embeded in polymer concrete are much less than 12 inches (305 mm).

The principal reason for using passive steel connections for precast repairs is to reduce installation time. A time savings is realized by not providing connections for each rebar. In addition, positioning mechanisms and extraneous hardware are not required. Several passive connections are shown in Fig 3.5.

Combination of Positive/Passive Connection

Positive and passive connection types could be combined to provide steel anchorage. The essence of this combination connection is to simply extend the rebars past the end of the precast slab. Where the rebars in the CRCP and the precast slab align, positive connections can be made. Bars not aligned could be provided with bearing surfaces at their ends. A combination connection, not strictly positive nor passive, could consist of a transverse rebar, positioned transversely across the longitudinal steel extending from the CRCP and precast slab and tied with wire to each rebar it crosses. Combination connections are illustrated in Fig 3.6.

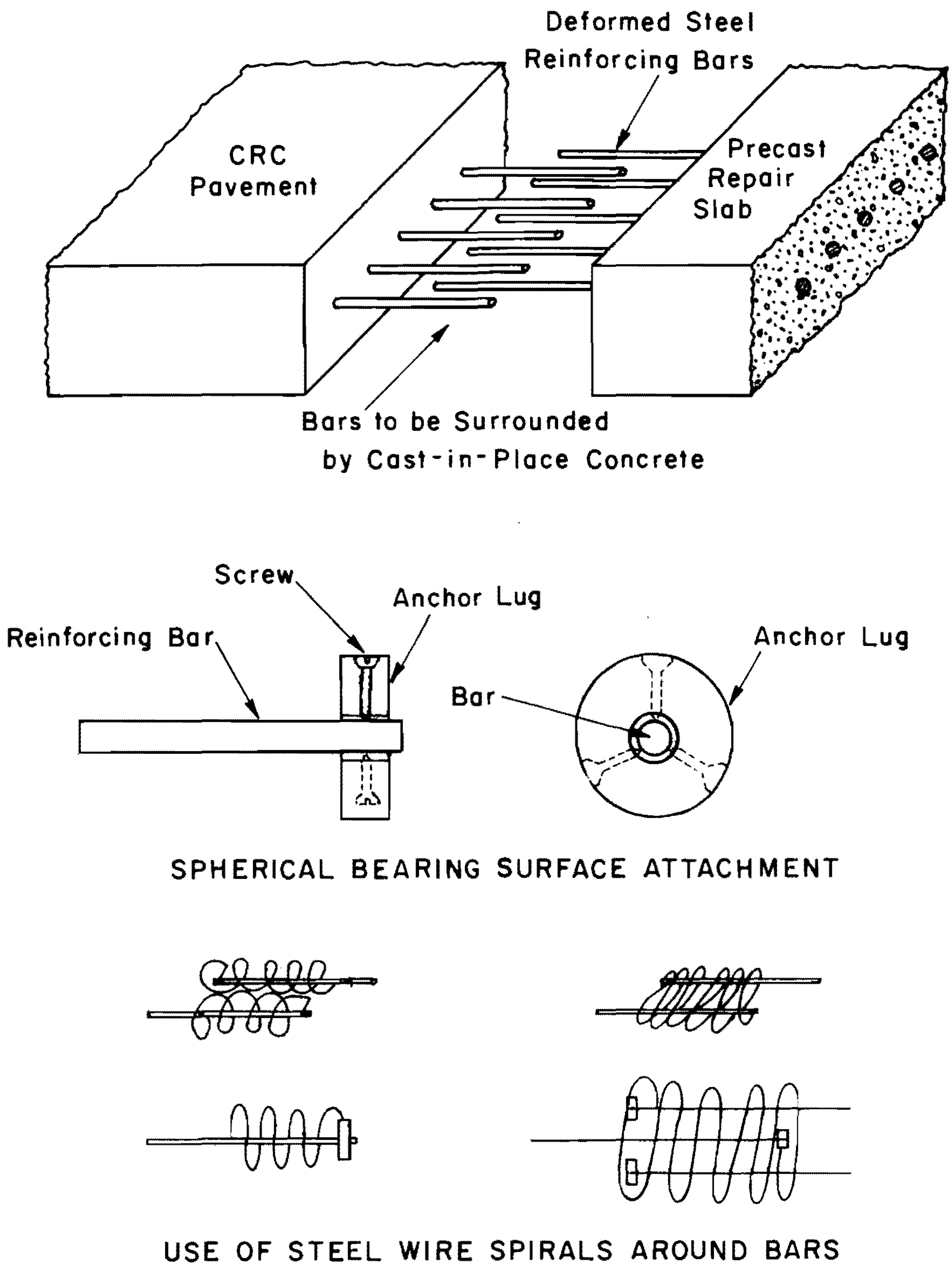


Fig 3.5. Examples of passive steel connection devices.

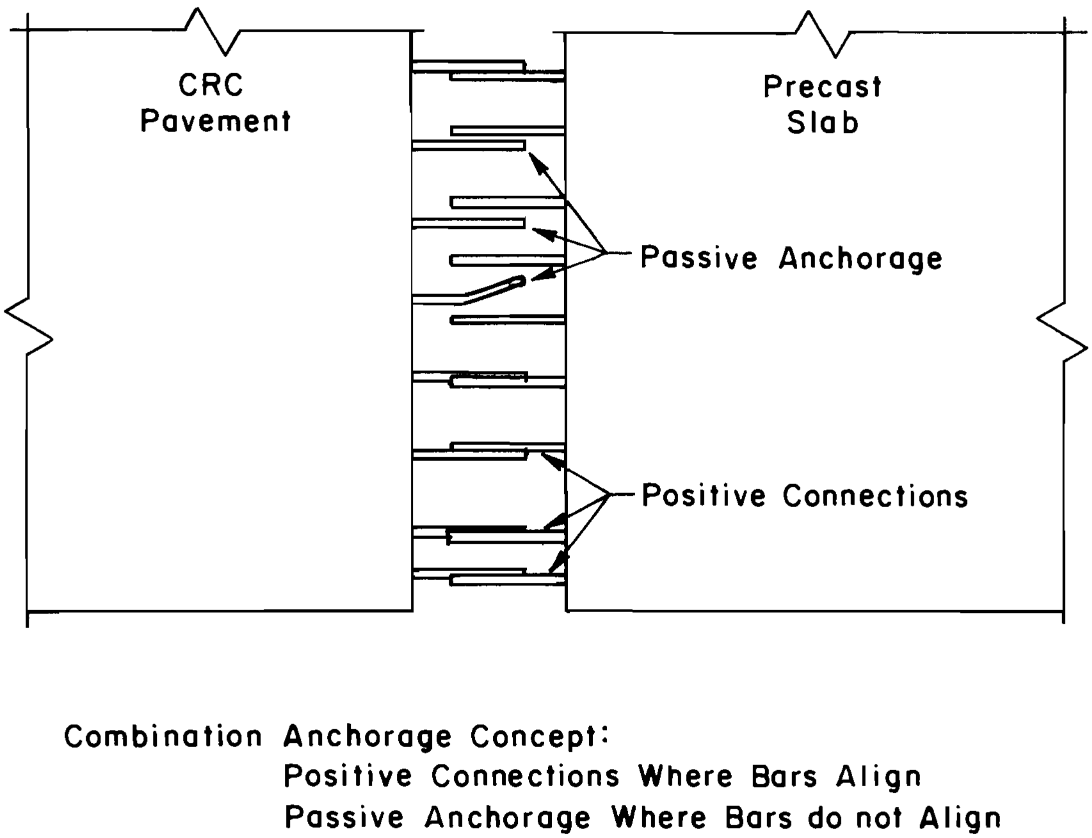


Fig 3.6. Combination positive and passive steel connections for precast repair slabs.

CAST-IN-PLACE CONCRETE

The void left at the steel connections between the CRC pavement and precast repair slab must be filled with a cast-in-place material. This material must develop sufficient strength over a short period of time to obtain acceptable repair times.

A test of rapid setting cement mortars conducted in Texas concluded that a mortar consisting of type II portland cement, high strength gypsum, fine sand, powdered saponified vinsol resin air entraining agent, water reducer, and plastic retarder offered the best overall performance that could be obtained with rapid setting materials based on combinations of portland cement and gypsum (Ref 24). This material purportedly sets in less than an hour, exhibits strengths comparable with normal portland cement concrete, and performs well in field trials. The durability of the mix was thought to be less than that of normal portland cement.

One of the best known accelerators of setting of portland cement is calcium chloride. Small amounts of calcium chloride, generally less than 2 percent, added to concrete with high cement content, greater than seven sacks/cubic yard (5.35 sacks/m^3), will produce acceptable strengths in less than three hours. However, corrosion of reinforcing steel and increased shrinkage have been attributed to calcium chloride. Many state highway departments including Texas have prohibited its use for these reasons.

Polymer concrete is a material which possesses qualities of a good cast-in-place repair material. Polymer concrete is a mixture of polymerized monomers and aggregate. Prior to polymerization, the monomers have a characteristically low viscosity, which allows them to easily penetrate into the voids in cured concrete. After polymerization, which can be regulated to occur in less than 15 minutes, the polymer concrete generally exhibits greater strength than ordinary portland cement concrete, excellent freeze-thaw resistance, exceptionally strong bond to exposed concrete and steel, and low water absorption. Research and development work on this material performed by the United States Bureau of Reclamation (Refs 23, 25, 26, and 27) and the University of Texas at Austin (Refs 28, 29, and 30) has demonstrated the applicability of polymer concrete for repairing reinforced concrete in bridge decks, highway pavements, beams, and airfield pavement.

The properties of polymer concrete which make it attractive as a cast-in-place concrete for precast repairs are its set time, concrete impregnation properties, and steel bond. Figure 3.7 shows compressive strength versus polymerization time for a common monomer formulation. Note that lower strengths are produced for shorter polymerization times. These lower strengths are in excess of 5,000 psi (34 mPa). This strength is generally greater than that of concrete used in pavement construction. Because of the penetration of monomers into adjacent concrete prior to polymerization, fractures and discontinuities induced into the concrete during removal operations are repaired and a strong bond between the polymer concrete and the pavement is developed. As stated in the previous section, development of bond with steel reinforcing bars may be great enough to anchor the steel over a short length without positive connections.

Other types of fast-setting materials which may have applicability for use as cast-in-place concrete include magnesia-phosphate, calcium sulfate, epoxy resin, polyester resin, and high alumina cement. These materials are marketed under many brand names. Results of field tests are limited. Long-term observations are particularly scarce. Results of short-term trials have been mixed. Materials proposed for use should be evaluated. Additional information on these materials can be found in Refs 31, 32, and 33.

PREPARATION OF DETERIORATED AREA

Preparation of a deteriorated area of concrete pavement for repair includes limit marking , concrete removal, steel preparation, and subbase preparation. Methods used for these purposes are in some ways dependent upon pavement condition, type of subbase, precast repair strategy, repair materials, and availability of equipment.

Limit Marking

It is necessary to mark the limits of concrete removal to satisfy requirements for size compatability between a precast repair slab and pavement. The length of the repair will vary with the overall casting strategy. The use

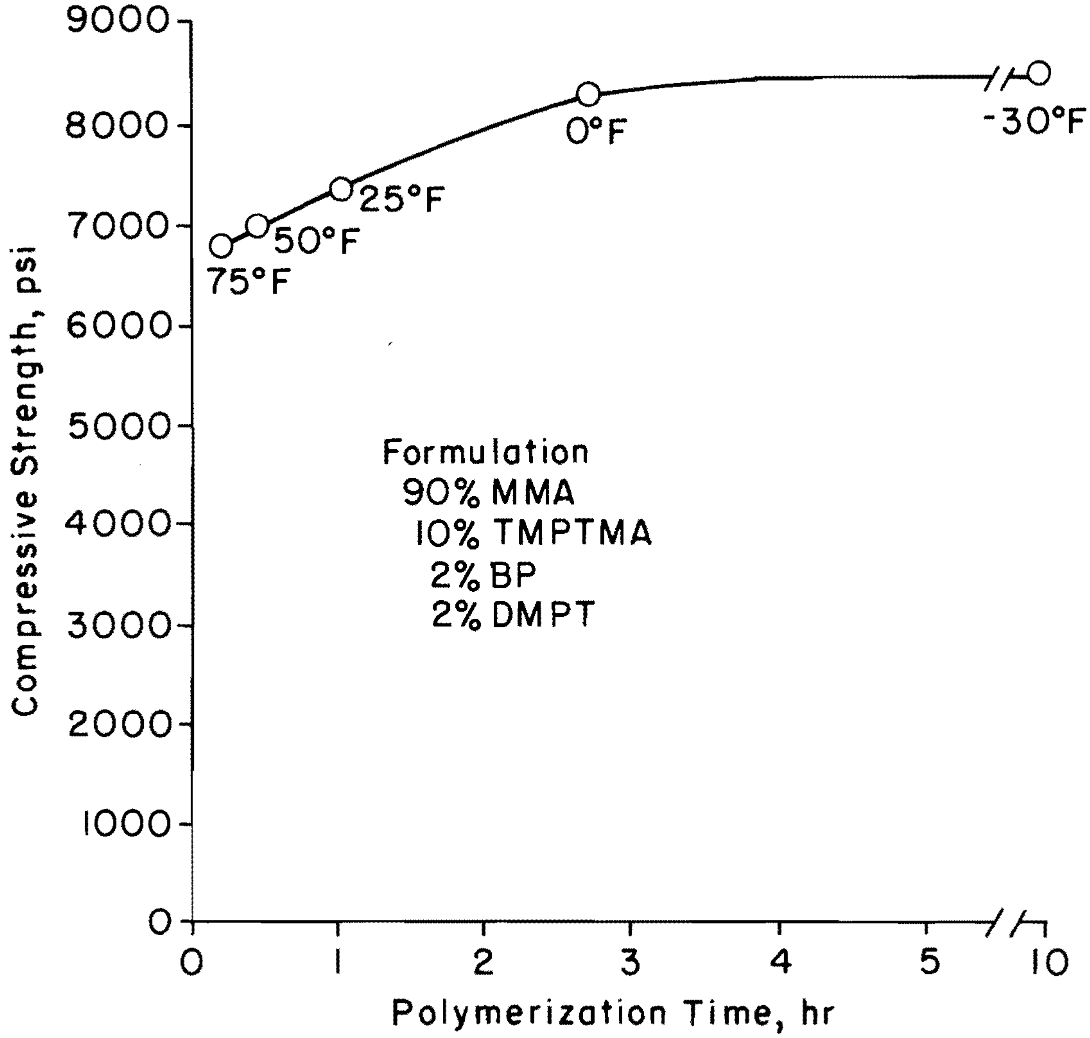


Fig 3.7. Effect of polymerization time on compressive strength of polymer concrete (Ref 29).

of standardized slab dimensions will limit repair lengths to these sizes. Repair limits may be placed where required for cast-to-size strategies. The limits should emcompass all broken and deteriorated concrete.

The end limits of a repair should be perpendicular with the edge of the pavement. A simple procedure for doing this with a 3-4-5 right triangle was used in Michigan (Ref 2). As shown in Fig 3.8, point C is located by striking arcs of 4 and 5 feet (1.22 and 1.52 m) from points A and B, respectively. Points A and B are 3 feet (.91 m) apart. A 5-12-13 right triangle can also be used to construct the perpendicular. The 5-12-13 right triangle is particularly suited to full lane width repairs due to the 12 foot (3.66 m) measurement across the pavement. Other transverse lines are laid out parallel to line A-C by making equidistant marks along the inside and outside lane edges.

Lines should be marked with an easily distinguishable, permanent material. Lines might become erased or disturbed during removal operations. Water used in concrete sawing can also remove non-permanent lines.

Concrete Removal

Deteriorated concrete may be removed from a pavement in many ways. Removal is a two-step process. Concrete must be dislodged and then lifted from the pavement. Material in the deteriorated area can be broken into small chunks or the entire section can be dislodged and removed intact. Many types of equipment which are designed for this task are available to road maintenance authorities.

The tool which is probably used the most for concrete removal is the hand-held compressed air jackhammer. Jackhammers are easily manipulated and current maintenance crews have a significant amount of experience with them. These hammers can be used to remove concrete from around reinforcing steel to provide steel connections. By shattering the concrete through momentary impacts on a small surface area, a rough face is left on all exposed concrete. Possible damage to adjacent concrete may be sustained. Noise and physical wear on construction personnel are other potential negative aspects of jackhammers.

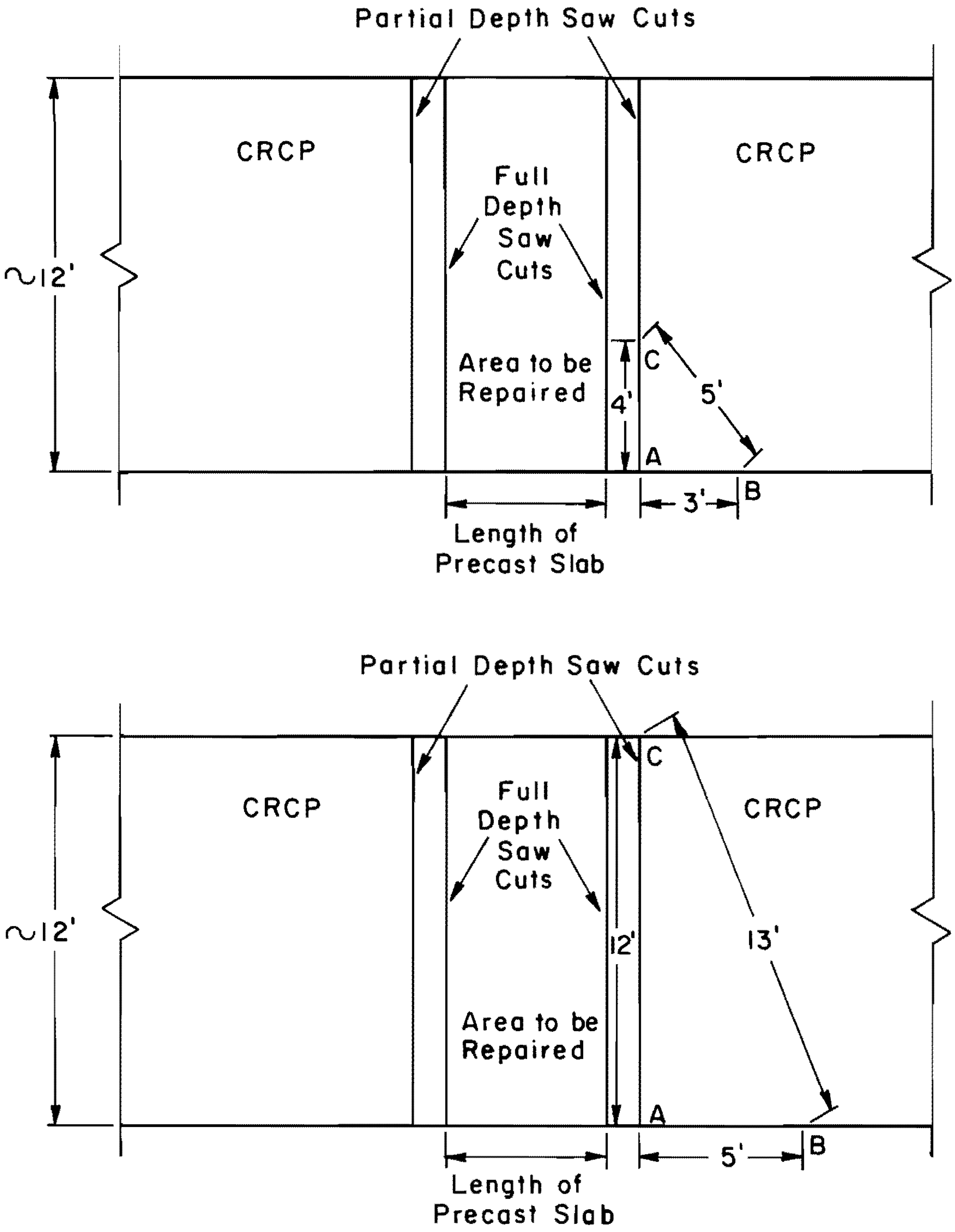


Fig 3.8. Right triangle techniques for laying out saw cuts.

A variation of the jackhammer is a pavement breaker attachment on a backhoe. This is essentially a jackhammer mounted on the end of the positionable arm of a backhoe. Its advantage is that a worker is not required to hold and manipulate a vibrating hammer. The other aspects of the jackhammer apply to this type of equipment.

Diamond bladed concrete saws are useful in dislodging concrete. Cuts can be performed at a relatively rapid rate and can be full or partial depth. Full-depth cuts can be made through the reinforcing steel to free the concrete. Partial depth cuts can be made at the edge of areas where concrete is to be removed from around the steel. This will produce a smooth lip at the surface of the pavement and reduce detrimental effects of destructive concrete removal.

Data from full-depth sawing of 9-inch (229 mm) reinforced concrete pavements using a 26-inch (660 mm) diameter blade, in Michigan, indicate a range from 200 to 450 linear feet (61 to 137 m) per blade with an average blade productivity of 295 feet (90 m) (Ref 2). The aggregate type, saw condition, blade quality, and reinforcing steel in the pavement are thought to affect cutting speed and blade life.

Advantages of concrete sawing are speed and non-detrimental effects on the concrete. Sawing operations can be performed at a faster rate than most destructive methods. Little physical damage is imposed upon the surrounding concrete. The cooling water needed can be a disadvantage of cutting operations. Some fast-set concrete materials are adversely affected by the presence of moisture.

Cutting operations can be performed in advance of installation. Traffic can aid in fracturing concrete cut in advance. Advance cutting also allows excess water to dissipate from the area. Cutting operations on a number of repair sites can be executed by a crew prior to removal operations.

Once the deteriorated portion of the concrete has been dislodged it must be lifted from the roadbed. The key to removing concrete pieces is to get a good grip on them. One method is to chip away concrete down to the reinforcing steel. Lifting attachments may be made at these locations. Another method, using a keyed lift pin, Fig 3.9, was used in Michigan (Ref 2). A hole is drilled into the concrete, the pin inserted, the key

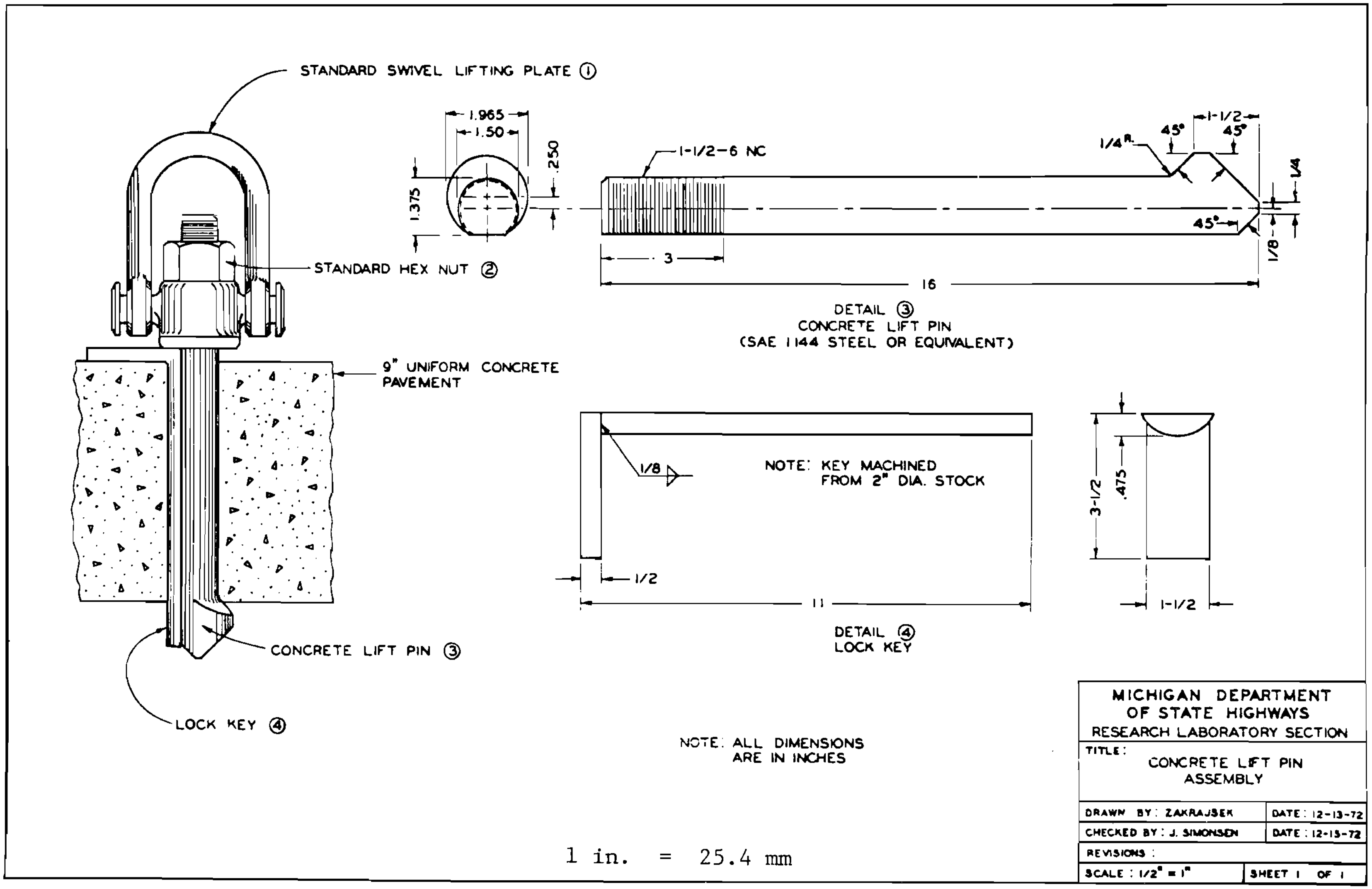


Fig 3.9. Concrete lift pin assembly (Ref 2).

inserted, the swivel plate attached, and a lift point created. Large portions of concrete can be removed rapidly from the roadway by this method.

Steel Preparation

Steel preparation for the repair of CRCP is mostly limited to preparation of the longitudinal reinforcement for anchorage of the precast slab. Preparation is influenced by the conrete removal operation, steel condition, and connection strategy.

If full-depth saw cuts are made in the concrete at the ends of the precast slab, the longitudinal steel in the CRCP will be cut. If the cutting operation is not used or not performed full depth, the steel at the end limits of the precast slab should be cut with an appropriate tool, such as a blow torch, bolt cutter, or hacksaw. A jackhammer should not be used to fracture steel reinforcement. There should be as little disturbance as possible in the steel during separation from the conrete.

Unless the steel is badly corroded or the effective diameter of a steel bar is significantly reduced, generally no surface treatment of the steel is necessary. Small amounts of rust are not detrimental to the bond between concrete and steel. Excessive rust can be removed with a wire brush. Steel which is badly corroded can be reinforced by welding an extra amount of steel onto the bar.

Depending on the steel connecting mechanism, it may be possible to perform some of the connection work prior to installation of the slab. Positive connecting devices, which fit around the steel extending from the CRCP, can be placed, positioned, and/or secured prior to slab placement. Anchor lugs for passive connections or a combination of passive-positive connections, can be installed at the ends of the bars. In all cases, these connections should not interfere with placement of the precast repair slab.

Subbase Preparation

In general, when the subbase is disturbed during concrete removal operations or is in poor condition, it should be repaired. If sufficiently large quantities of subbase are disturbed, addition and compaction of a similar material may be used. A compressed air hand-held tamp may be used for compaction. Materials which are used for subbases should be compatible with the leveling material. Small voids and undulations in the surface of the base course can be filled with the leveling material. If pumping is evident, drains might be installed beneath the repair.

INSTALLATION

Past experience with portland cement materials in pavement construction has shown that its success is very dependent upon proper construction. Installation of a precast repair slab is a crucial step. Careful attention should be paid to details to guard against problems resulting from improper or poor construction.

The installation process consists of four steps. The slab must be lifted to transport and position it. Once positioned, it must be leveled with the adjacent pavement surface. The next step is to make steel connections. The final step is sealing the voids around the edges of the precast slab.

Lifting

After casting and curing, prior to transportation to the construction site and final placement into a pavement, a precast slab must be lifted. During lifting, the weight of the slab induces stresses into the slab. These stresses can possibly crack a slab, especially if care is not taken to guard against rupture.

A simple lifting scheme using four lift points near the corners of the slab was investigated in Appendix 3. Lift points are made by casting threaded inserts, Fig 3.10, into the concrete. Swivel lifting plates are then bolted into these inserts. Chains or cables may then be attached to the swivel plates and the lifting device.

Another lifting scheme is to use a combination of threaded inserts and steel I-beams. The steel I-beams can be attached to the slab with bolts through the threaded inserts. The I-beams act to stiffen the slab during lifting.

A study of stresses induced into a slab due to lifting with the simple lifting scheme above is presented in Appendix 3. This study demonstrated that slabs as large as 20 by 12 feet and 8 inches thick, with a unit weight

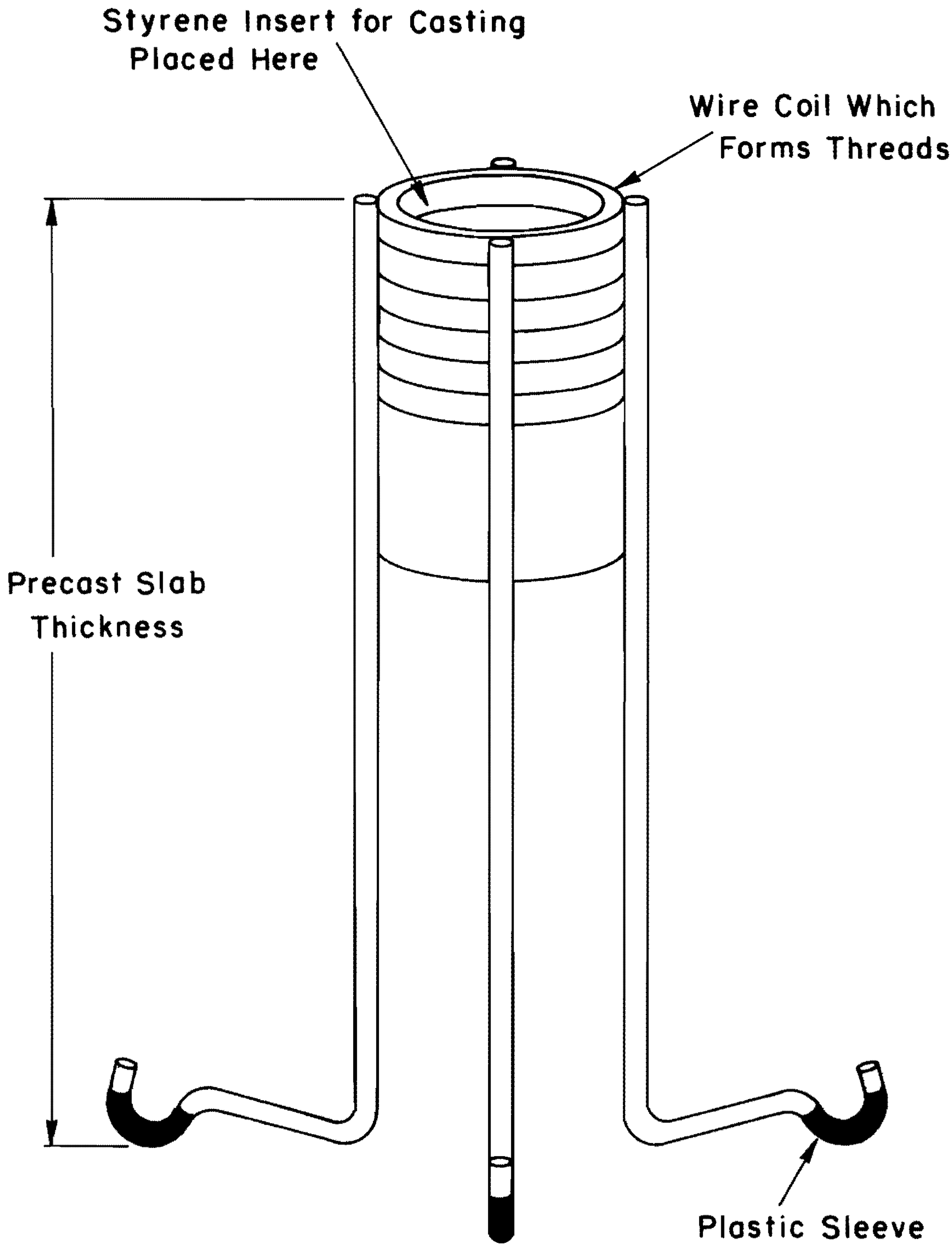


Fig 3.10 Sketch of threaded inserts for lift connections (Ref 2).

of 150 lb/ft^3 (2403kg/m^3), can be lifted with the lift points positioned between 1/4 to 1/5 the span length away from the edges of the slab, without inducing destructive stresses into the concrete.

Lifting and positioning of slabs can be accomplished with a front end loader or moveable crane. The weight of the slab, clearance of the lifting device, and availability of equipment are the significant factors. A 20 x 12 x .67-foot (6.1 x 3.7 x .2-m) reinforced portland cement concrete slab weighs approximately 24,000 pounds (107 kN). The lifting device must clear the height of the transporting vehicle with the slab.

Leveling

A smooth transition from the pavement to the patch would minimize ride roughness and dynamic loading of the patch. Three techniques to align the surfaces of the precast repair slab and the adjacent pavement surface are described here.

The first technique is to prepare a flat leveling course struck-off at the proper elevation. To strike the leveling course off at the proper elevation, the screed-track configuration shown in Fig 3.11 could be used. For plane crown sections, the track is positioned transversely across the pavement while the wood striker is manipulated longitudinally. For parabolic crowns, the track should be positioned longitudinally so that the precast slab aligns most closely in the wheel paths. The frame assembly shown in Fig 3.12 was used in Michigan to align precast slab surfaces (Ref 2).

The leveling layer may be a bound or unbound granular material. Unbound materials should be compacted before being struck-off. Cement, lime, and bituminous treated materials should also be compacted. Cement mortar can also be used. Mortar should be consolidated well.

Another leveling technique is to mudjack or force grout beneath a positioned slab. The precast slab can be supported with the surface aligned with the pavement and material forced beneath it, or it can be placed on the surface of the subbase and raised by forcing material beneath it. Holes in the slab for pumping material beneath the slab should be combined with lifting inserts.

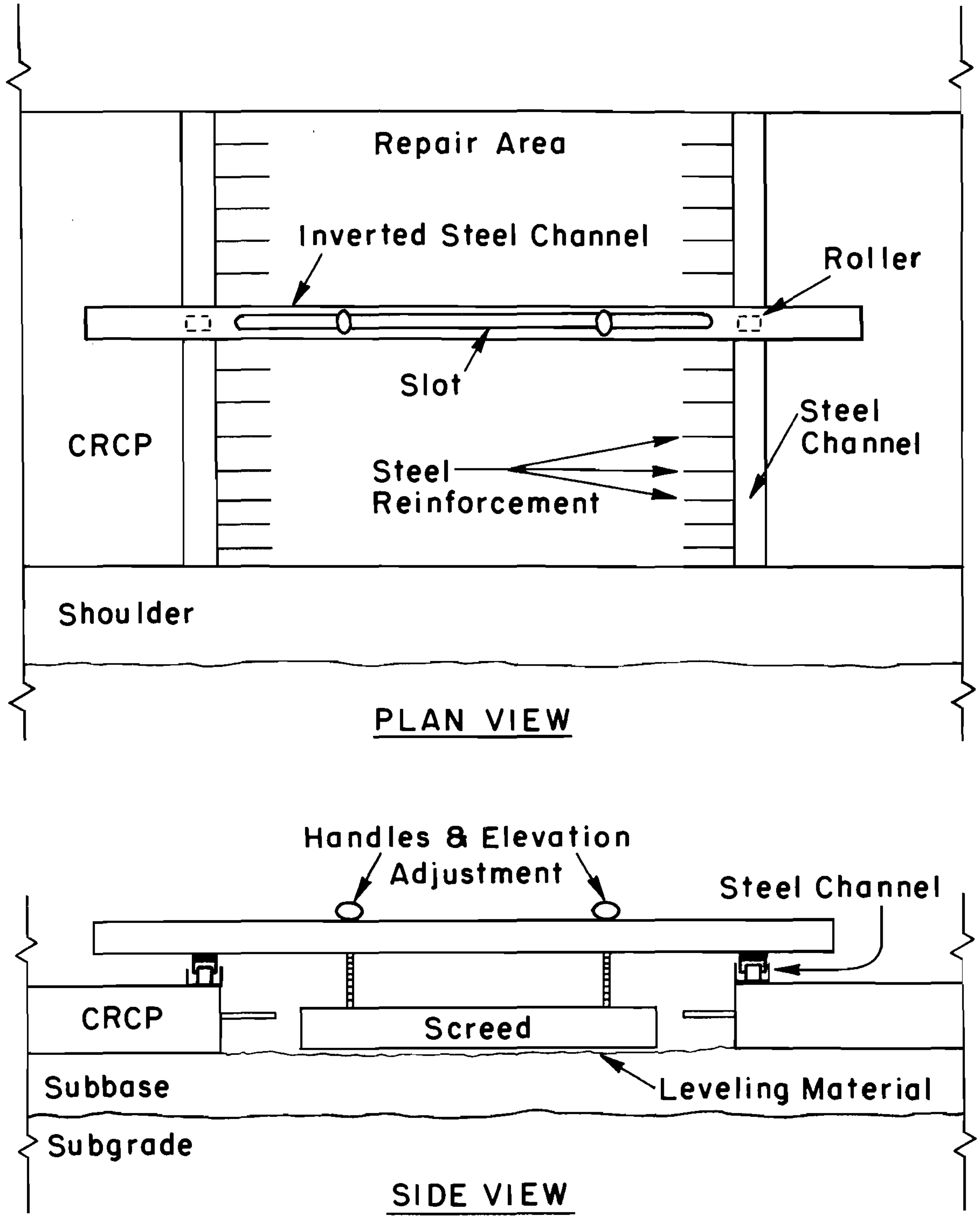


Fig 3.11. Conceptual diagram of tracked screed for striking leveling material.

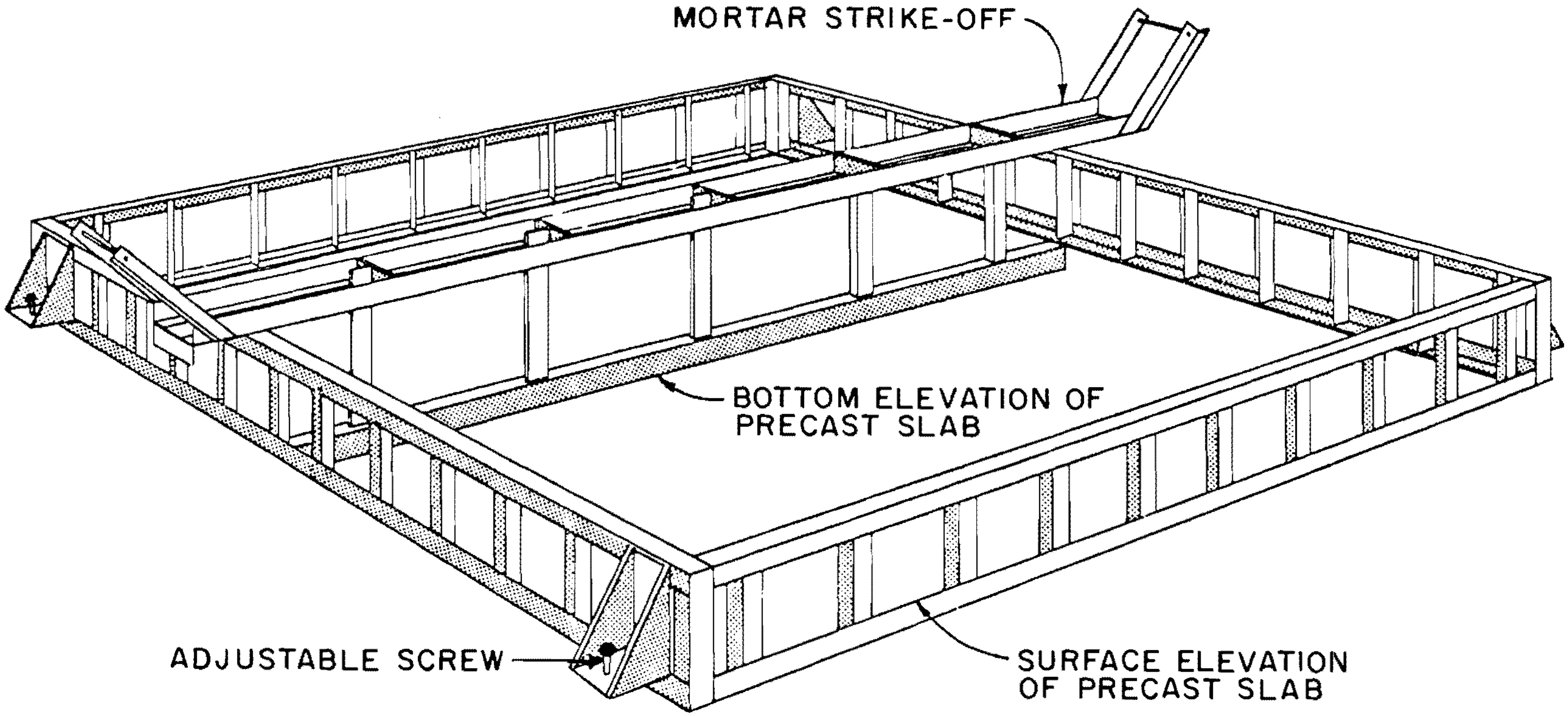


Fig 3.12. Frame for setting precast slab elevation (Ref 2).

The third leveling technique is a combination of the techniques previously discussed. The leveling layer is placed and struck-off to the proper elevation. The slab is placed on top of this layer. Fine adjustments can then be made by forcing more material through holes positioned about the slab.

Steel Connections

After the slab has been positioned and aligned with the pavement surface, steel anchorage is performed. Discussion of alternate strategies for accomplishing this are included earlier in this chapter.

Sealing Voids Around Slab

After provisions for steel anchorage have been made, the voids in these areas are filled with a cast-in-place concrete. Placement of this material will allow for minor elevation differences between the surfaces to be smoothed out. These alternatives are discussed earlier in this chapter.

The void along the inside longitudinal edge of the repair can be filled with a concrete grout or sealed with bituminous materials. Concrete grout has been used in this manner on precast repairs of jointed concrete pavements in California (Ref 4). Placement of a partial depth bituminous filler board with a rubberized asphalt sealant placed on top, such as used on precast repairs in Michigan (Ref 2), can also be used.

The addition of material along the outside edge of the repair slab will depend on the type of shoulder. A bituminous shoulder will require placement of an asphaltic mixture. For a concrete shoulder, this void could be handled in the same manner as the one along the inside edge. This also holds for repairs bounded by traffic lanes on each side.

SUMMARY

A variety of alternate strategies for the repair of CRCP have been presented in this chapter. These strategies are summarized below.

Fabrication

Slabs can be cast to a variety of standard dimensions or they can be cast on an individual repair basis. Two convenient locations are a highway department construction yard or an open area near the repair site. Forms can be designed to facilitate multiple casting of slabs at the same time or designed on an individual basis. Forms can be constructed from steel, iron, brass, wood, or plastic. The concrete should be placed so that adequate consolidation is achieved and rapid initial set is avoided. Quality control tests that can be performed on the concrete include slump test, compressive strenth, and beam strength. Curing can be enhanced by pigmented membranes, wet coverings, ponded surface water, and steam exposure.

Weakened Plane

Analysis shows that stresses in precast concrete slabs longer than 6 feet (1.83 m) can be controlled with the use of a weakened plane across the middle of the slab. The weakened plane can be formed by reducing the cross-sectional area of the concrete at this point by means of a bond breaker made with galvanized metal, aluminum, bituminous filler board, plastic, wood, or steel sheets. The weakened plane can also be constructed by either forming or cutting a groove in the surface of the slab.

Reinforcement

Longitudinal and transverse reinforcement should consist of deformed steel bars or deformed wire mats. Fibrous reinforcing is another alternate type of reinforcement that can be incorporated into a slab in addition to the longitudinal and transverse reinforcement. Steel wire spirals around stress concentration areas, such as lift points, can be used as added protection against cracking.

Steel Connections

The continuity in steel reinforcement must be maintained in CRCP. Steel connections can be made by welding or splicing with commercial connectors the longitudinal steel in the pavement and repair slab. Depending on the length of the connection zone and the type of cast-in-place material to be placed there, the longitudinal steel can be simply anchored in this zone, without direct connection. Spherical bearing surfaces at the ends of these embeded bars will aid their anchorage. A combination of anchored bars and connected bars can be used.

Cast-in-place Concrete

Fast-setting cast-in-place concrete must be placed in the steel connection/anchorage areas. One promising material for this application is methyl methacrylate based polymer concrete. Other potentially applicable concrete materials include portland cement concrete with special admixtures, magnesia-phosphate mixtures, expoxy resins, polyester resins, and calcium sulfate mixtures.

Preparation of Deteriorated Area

Preparation of the deteriorated area for repair includes marking limits of the repair, concrete removal, steel preparation, and subbase preparation. The repair limits should encompass all unsound deteriorated portions of the pavement. Use of the 3-4-5 and 5-12-13 right triangle procedures can be made to construct lines across the pavement perpendicular to the edge. Concrete removal can be facilitated with use of concrete saw; jack hammer; pavement breaker attachments on machinery, such as a backhoe; and keyed insert lift pins. It may be possible to remove the concrete in large sections or it may be necessary to break the concrete into smaller blocks for removal. The concrete must be removed from around the longitudinal steel at each end of the repair. Care must be taken to minimize damage to the steel during removal operations. After the concrete has been removed from around the reinforcing steel, the steel should be cleaned of any rust or debris and steel connection hardware should be positioned if convenient. Damaged areas of the subbase should be repaired. Detrimental conditions which may have helped create the distress being repaired should also be corrected.

Installation

Installation consists of four steps. First the slab must be lifted to be transported and positioned over the repair. Slabs may be lifted from four lift points on the surface of the slab. These lift points should be positioned between 1/4 and 1/5 of the length of the slab from each end in each direction. The next step is alignment of the repair slab and adjacent pavement. This may be performed by striking off a grout bed to the proper height or by mudjacking a grout beneath the repair slab. The steel connections are then made. The final step is to seal the voids around the slab. A fast-set cast-in-place concrete is placed in the steel anchorage area and concrete grout or bituminous joint sealing material is placed in the voids along the remaining edges.

Keeping the alternate methods of precast repair in mind, the next step in the design of precast repair slabs is the analysis of these alternatives. This is discussed in the next chapter.

CHAPTER 4. ANALYSIS

The analysis presented in this chapter will focus upon estimating the structural response of precast repair slabs to various loading conditions. Analysis of other factors pertaining to a precast slab, such as forms, casting strategy, etc., lie in the field of traditional economic analysis. These incidentals have less overall effect on the design of precast repair slabs and can be manipulated to meet design requirements stemming from structural considerations.

First, a summary of the expected history of loading will be presented in this chapter. This is followed by descriptions of the physical models and calculation techniques used to estimate the precast slab's response to the various load conditions. These load conditions include volume change, wheel load, and lifting analysis.

LOADING HISTORY

An expansion of the expected load and restraint conditions presented in Table 2.1 is shown in Table 4.1. This table details the loading causes, restraints, stresses, and concrete strengths which occur at various times in the life of a precast repair slab. Variable symbols are assigned to these quantities and will be used in this discussion.

The process begins with casting the concrete at time t_0. The temperature at t_0 is T_0. Curing occurs between times t_0 and t_1. During this period, shrinkage, Z_1, and temperature drop, $\Delta T_1 = T_0 - T_{min}$, occur. T_{min} is the lowest temperature between time t_0 and t_1. Shrinkage and temperature drop produce stresses in the steel and concrete. The largest stresses occur in the center of the slab. Concrete stress, σ_{cm1}, and steel stress, σ_{sm1}, are produced by the restraint to volume

TABLE 4.1. LOAD HISTORY FOR PRECAST REPAIR SLABS

Description	Time	Loading	Restraint Conditions		Concrete Strength	Stress
			Concrete	Steel		
Casting	t_0	---	---	---	---	---
Curing	t_0-t_1	Z_1 ΔT_1	Bond with steel Friction with form and casting surface	Bond with concrete	f_1	σ_{cm1} σ_{sm1}
Removal from form and transportation to storage area	t_1	Z_1 ΔT_1 γ_{conc}	Bond with steel Lift supports	Bond with concrete	f_1	σ_{cv1} σ_{cL1} $\sigma_{c1} = \sigma_{cv1} + \sigma_{cL1}$
Further curing	t_1-t_2	Z_1-Z_2 $\Delta T_1-\Delta T_2$	Bond with steel Friction on slab surfaces	Bond with concrete	f_1-f_2	σ_{cm2} σ_{sm2}
Transportation to repair site and placement in repair	t_2	Z_2 ΔT_2 γ_{conc}	Bond with steel Lift supports	Bond with concrete	f_2	σ_{cv2} σ_{cL2} $\sigma_{c2} = \sigma_{cv2} + \sigma_{cL2}$
Anchorage of steel at the end of the slab	t_2	Z_2 ΔT_2	Bond with steel Friction along bottom	Bond with concrete Anchorage at end of slab	f_2	σ_{cv2}
Open repair to traffic	t_2	Z_2 ΔT_2 P_{wL}	Bond with steel Friction along bottom	Bond with concrete Anchorage at end of slab	f_2	σ_{cv2} σ_{pe} $\sigma_{c3} = \sigma_{cv2} + \sigma_{pe}$
Period after installation	t_3	Z_2, Z_3 ΔT_2, ΔT_3 P_{wL}	Bond with steel Friction along bottom	Bond with concrete Anchorage at end of slab	f_3	σ_{cv2}, σ_{cv3} σ_{pe} $\sigma_{c4} = \sigma_{cv2} + \sigma_{cv3} + \sigma_{pe}$ σ_s

change created by friction along the bottom and sides of forms and the bond between the concrete and steel. Time t_1 is chosen to represent the time the slab is removed from the forms to be stockpiled or moved to another curing environment. Lifting of the slab during removal induces stresses into the concrete. The largest stresses due to lifting, σ_{cL1}, will occur either at the lift points or in the center of the slab. Friction is lost when the slab is lifted. Calculation of volume change stress in the concrete during lifting, σ_{vc1}, is based upon the frictionless model. The critical concrete stress during lifting at t_1, σ_{c1}, is the sum of σ_{cL1} and σ_{vc1}. The stress σ_{c1} is compared to the tensile strength of the concrete at t_1, f_1.

At time t_2, the slab is lifted for transportation to the repair site. During the period from t_1 to t_2, the concrete undergoes further curing and volume change. Concrete shrinkage increases from Z_1 to Z_2. The temperature drop varies during this period with $\Delta T_2 = T_0 - T_{min}$, where T_{min} is the minimum temperature which occurs during this period. The concrete strength increases from f_1 to f_2. The critical stress during lifting, σ_{c2}, is the sum of frictionless volume change stress, σ_{cv2}, and the maximum lifting stress, σ_{cL2}.

In terms of concrete strength gain, concrete shrinkage, and temperature change, the slab may be considered as being installed into the pavement at time t_2 also. Although, technically, lifting and installation occur at different times, the changes in magnitudes of the above factors are so slight that for practical purposes they may be considered as occurring at the same time. Installation itself does not induce significant stresses into the slab. However, the restraint conditions are altered by restraining the steel at the ends of the slab. The temperature, T_2, and concrete shrinkage, Z_2, at the time of steel restraint become the base values for any further volume change calculations that may be due to the change in restraint conditions.

After installation is complete, traffic is returned to this portion of the pavement. Stress in the concrete due to the design wheel load at the edge of the slab is σ_{pe}. Stress in the concrete at this point, σ_{c3}, is the sum of σ_{pe} and σ_{cv2}. This should be compared with the concrete strength f_2.

After the precast repair slab has been in service for a period of time, volume change stresses due to the new restraint conditions are developed. At time t_3 , the lowest temperature, T_3 , is encountered. Concrete shrinkage at time t_3 is Z_3 . Volume change stresses are calculated with the temperature drop, $\Delta T_3 = T_2 - T_3$, and shrinkage, $Z = Z_2 - Z_3$. A restrained friction model is used for calculating concrete stress in the center of the slab, σ_{cv3} , and steel stress at the end of the slab, σ_s . The design concrete stress, σ_{c4} , is the sum of frictionless volume change stress, σ_{cv2} , restrained friction volume change stress, σ_{cv3} , and design wheel load stress, σ_{pe} . The concrete stress σ_{c4} is evaluated against concrete fatigue strength f_3 . Steel stress at the end of the slab, σ_s, is compared against the steel's yield point, f_y .

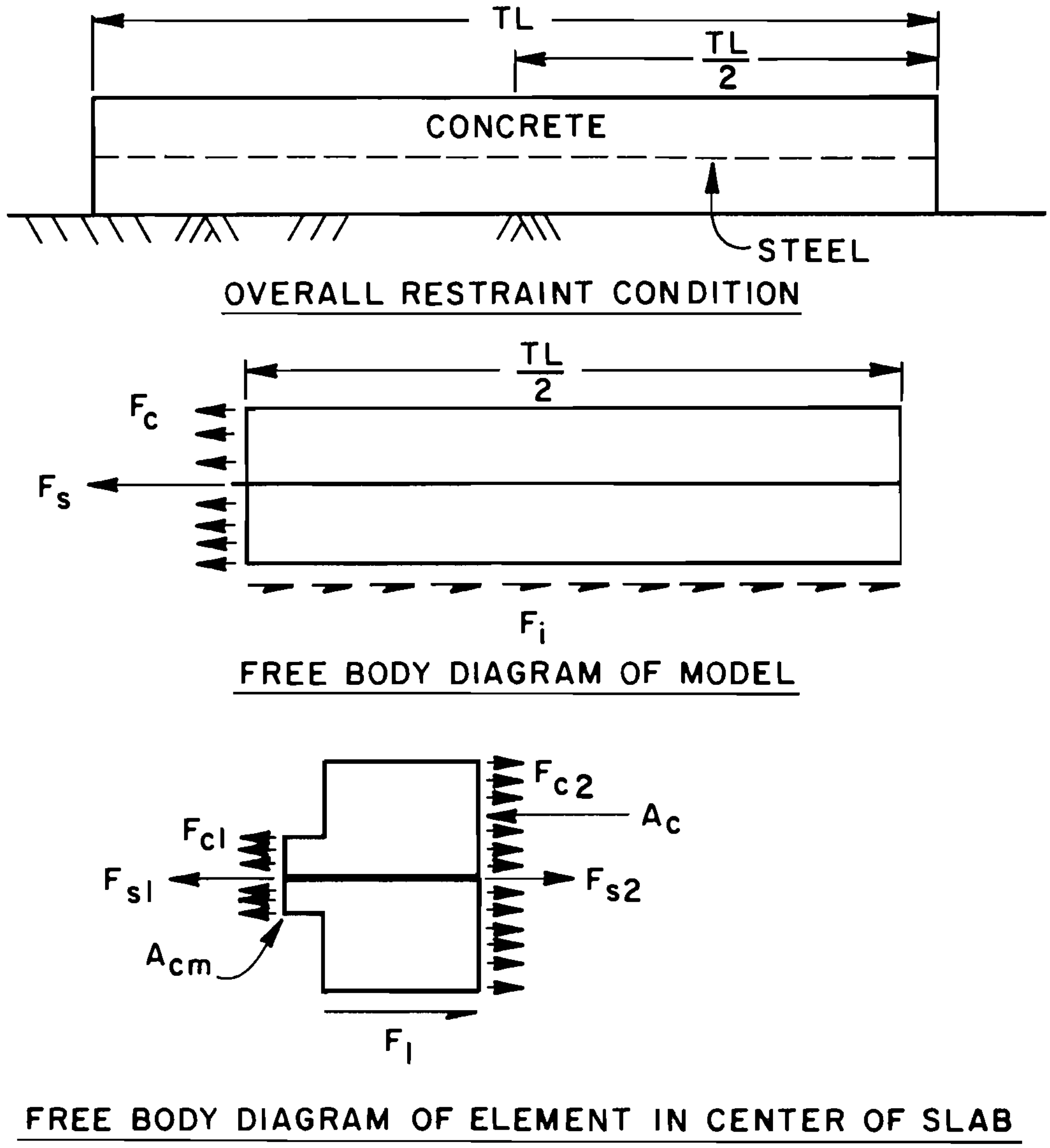
VOLUME CHANGE ANALYSIS

Volume change in reinforced concrete is caused by two factors, concrete shrinkage and temperature change. The analysis of volume change is conveniently separated by the steel restraint condition at the end of the slab. Volume change analysis without and with steel restraint at the end of the slab will be discussed in separate portions of this chapter. Volume change calculations using the methods described below are presented in Appendix 5.

No Steel Restraint

The physical model along with the free body diagram used to calculate stresses and movement in a reinforced concrete slab with no steel restraint at the end is shown in Fig 4.1. To solve for forces in this model a compatability model for the steel and concrete interaction is required. This model is shown in Fig 4.2. Derivation of the length change equation is also shown in this figure.

Calculation of stresses and concrete movement is an iterative procedure. The iterative nature of these calculations makes them unsuitable for hand solution. A computer program called VCACS, Volume Change Analysis for Concrete Slabs, was written to perform these calculations. This program was written as an interactive program to make its use much simpler.



A_c : Concrete Area

A_{cm}: Concrete Area In Center of Slab

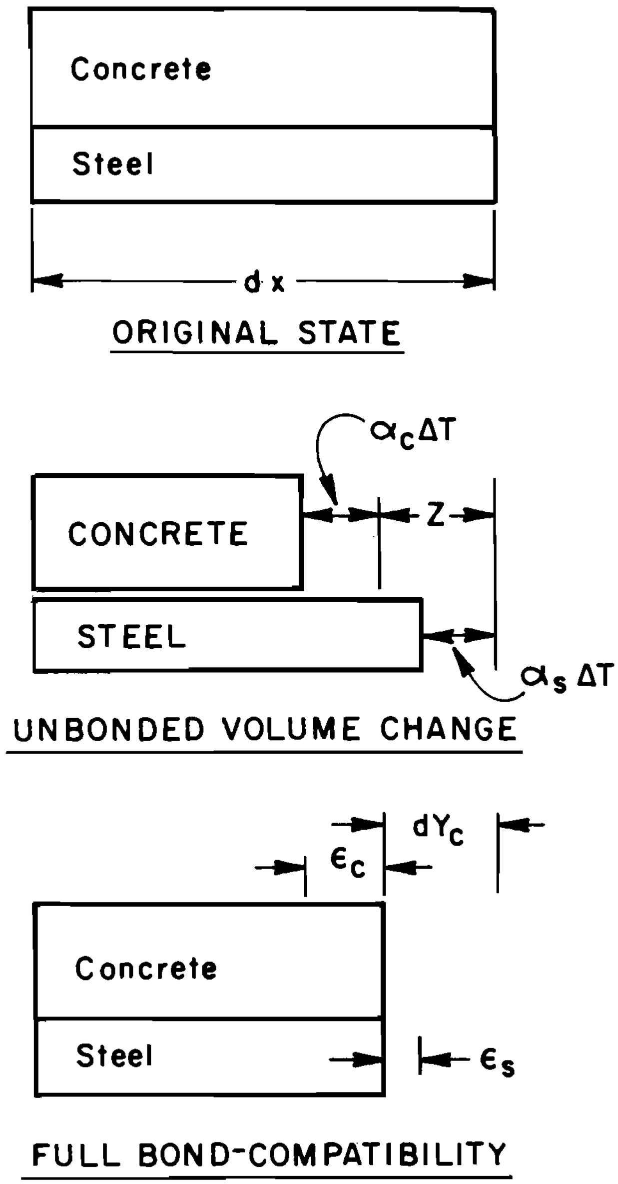
F_{ci} : Concrete Force at i

F_{si} : Steel Force at i

F_i : Friction Force

TL : Total Length of Slab

Fig 4.1. VCACS equilibrium models for concrete slabs with unrestrained steel at the end of the slab.



CONRETE-STEEL COMPATIBILITY

$$(\alpha_s \Delta T + \epsilon_s + \epsilon_c)dx = (Z + \alpha_c \Delta T)dx$$

$$\epsilon_s + \epsilon_c = Z + \Delta T(\alpha_c - \alpha_s)$$

CONCRETE MOVEMENT

$$dY = -\epsilon_c dx + (Z + \alpha_c \Delta T)dx$$

$$Y_c = \int_0^x dY_c = -\int_0^x \epsilon_c dx + (Z + \alpha_c \Delta T)X$$

Fig 4.2. Concrete-steel volume change compatibility and concrete movement model.

The VCACS program is capable of calculating stresses in a slab with a reduced concrete area in the center. This feature was incorporated into the program to allow it to analyze slabs with a preformed weak plane in the center. Calculations for slabs without weakened planes are accomplished by specifying the full concrete area in the center and zero thickness for the bond breaker.

The analysis of unrestrained concrete slab performed with program VCACS, presented in Appendix 5, demonstrates effects of not anchoring precast repair slab steel reinforcement at the end of the slab and the expected volume change stresses prior to installation. Comparison of concrete movements at the end of the slab, for restrained and unrestrained slabs, shown in Figs A5.3 and A5.4, shows larger crack widths for the unrestrained condition. The volume change calculations indicate that inclusion of a bond breaker will not cause the slab to crack prior to installation. The largest concrete stress calculated was less than 100 psi (689 kPa).

Steel Restraint

Restraint of the steel at the end of the slab introduces further complexities into calculation of volume change effects in a concrete slab. The physical model and free body diagram of the restrained concrete slab are shown in Fig 4.3. Introduction of the steel force at the end of the slab necessitates consideration of both the bond stress between concrete and steel in the bond slip zone, and of the steel boundary condition in the calculations.

An interactive computer program was also written to perform volume change calculations in restrained concrete slabs. This program is called VCARCS, Volume Change Analysis in Restrained Concrete Slabs.

Calculations for slabs with reduced concrete area in the center of the slab can be performed with VCARCS. These calculations can be performed only for the frictionless condition. Except for very high friction forces or very long slabs, the results of the frictionless model are very close to those of the friction model. For the calculations with reduced concrete area, the frictionless model gives adequate solutions (Appendix 5).

The volume change calculations for restrained concrete slabs indicates that slabs longer than 7 feet (2.13 m) have potential problems with steel stress at the end of the slab. Further analysis demonstrated that inclusion

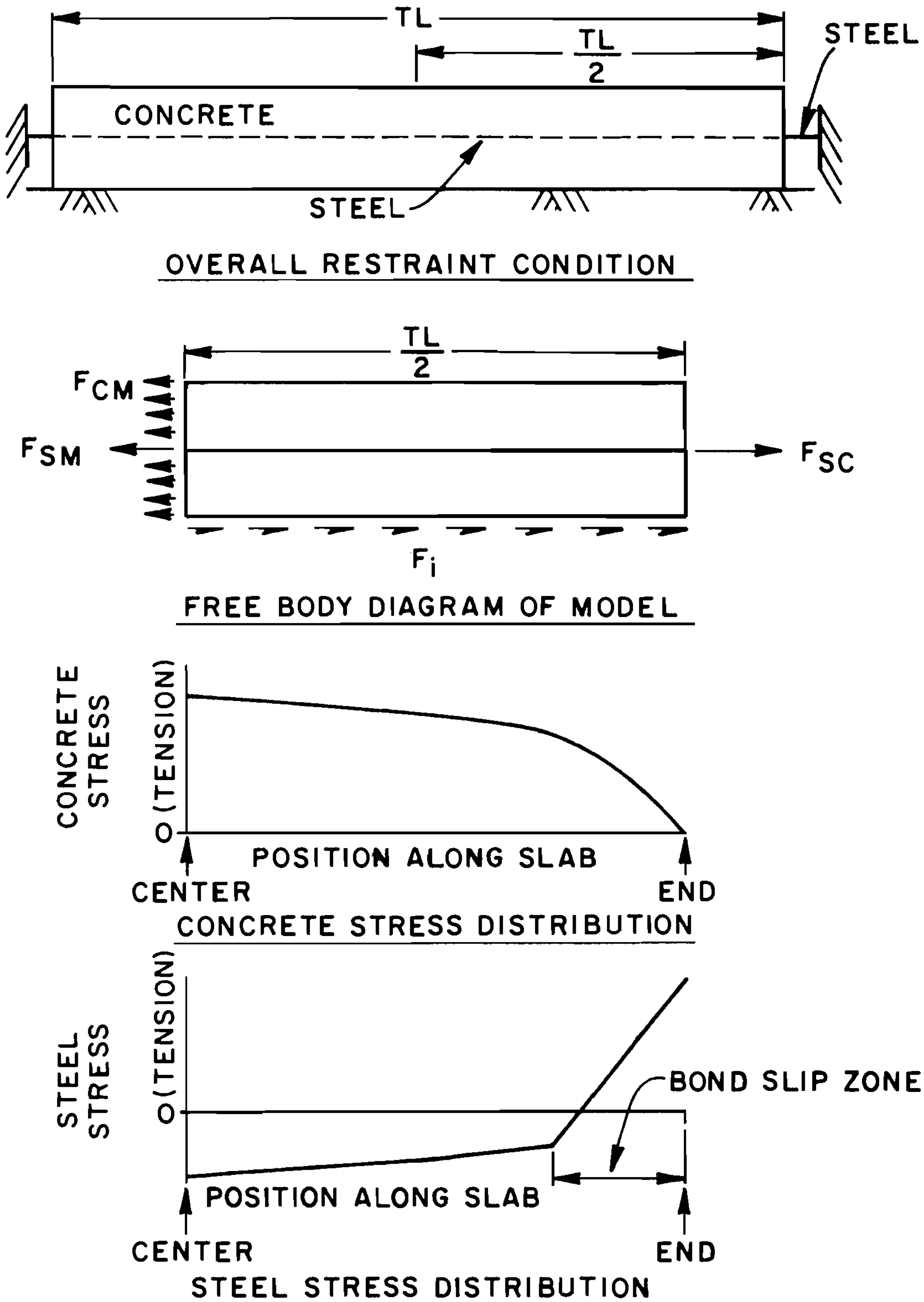


Fig 4.3. VCARCS equilibrium model and stress distribution in restrained concrete slab.

of a bond breaker to form a weakened plane in the middle of the slab would increase the concrete stresses over this area so that the concrete would be forced to fracture before steel stresses become excessive. Calculations indicate that an area reduction of 50 percent is sufficient for the weakened plane to act as a steel stress safety valve. A method to determine the required area reduction more accurately for each situation also is presented in Appendix 5.

Several other conclusions also resulted from this analysis. For equal areas of steel reinforcement, smaller rebars result in higher concrete and steel stresses and lower concrete movements. For a constant rebar size, increasing the area of steel reinforcement increases the concrete stress, decreases the steel stress, and decreases the concrete movement. Increasing concrete strength and modulus of elasticity results in increases in steel stress, concrete stress, and concrete movement. These results may be used by a designer to help modify designs to achieve the desired responses.

WHEEL LOAD ANALYSIS

A variety of methods have been developed to estimate a pavement's response to wheel loads. Each has its own assumptions and limitations. Most methods are not capable of modeling and analyzing discontinuities in pavements, such as cracks, holes, and voids. Discrete element and finite element programs have the capability of modeling these discontinuities in pavements. Requirements of finite element programs to model simple pavement problems often become excessive. The SLAB49 computer program (Refs 34 and 35) is a discrete element program whose computational requirements are more reasonable than those of most finite element programs.

Appendix 2 presents a study of wheel loads on a precast repair with the SLAB49 program. Appendix 4 presents a study of the effect of wheel loads on a precast slab with voids beneath it. The analysis presented in these appendices demonstrates methods of modeling CRCP with the SLAB49 program. Briefly, the pavement stiffness is represented by orthogonal bending stiffness and torsional twisting stiffness between nodes. Spring support is specified at the nodes to represent the foundation support. By varying these properties at selected nodes, discontinuities in the pavement can be modeled.

The wheel load analysis presented in Appendix 2 investigates the effects of loads on the discontinuity at the steel connections and the length of precast repair slabs. An increase in stiffness of the steel connection zone results in a small stress concentration of a magnitude that is probably not a destructive element within the pavement structure. A decrease in stiffness of the steel connection zone results in potentially harmful stresses being induced into the subbase. Slightly higher stress levels for slabs less than 6 feet (1.83 m) long than for longer slabs suggest that the construction of shorter slabs is more critical due to their lower fatigue potential.

The effect of wheel loads on voids beneath precast repair slabs was investigated in Appendix 4. The most critical location of a void was located beneath the steel connection zone. The concrete stress at this location is 13 percent higher than for the full support condition. A precast repair slab should always be constructed with the bottom of the slab in full contact with the supporting layer.

In short, the SLAB49 program has proven to be useful for studying particular discontinuities in a precast slab repair. For very extensive investigations of particular features within a rigid pavement structure, finite element programs may be used. For estimation of wheel load stresses for design purposes, use of simpler computation methods, such as Westergaard analysis, may be expedient.

LIFTING ANALYSIS

Analysis of stresses induced into concrete slabs during lifting was also performed with the SLAB49 program. This analysis is presented in Appendix 3. The structural elements of a concrete slab were modeled, as usual for slabs, with SLAB49. Lifting was simulated by specifying subbase support only at the lift points. The subbase support was set to an arbitrarily high value so that resulting deflections were negligible at these points. The slab weight was distributed proportionately over the surface of the slab. This modeling technique is shown in Fig 4.4.

The simple lifting mechanism investigated in Appendix 3 is suitable for lifting precast repair slabs. The distribution of stresses is quite sensitive to the location of the lift points. It is recommended that the lift points be located 1/5 to 1/4 the span length away from the edge of the slab.

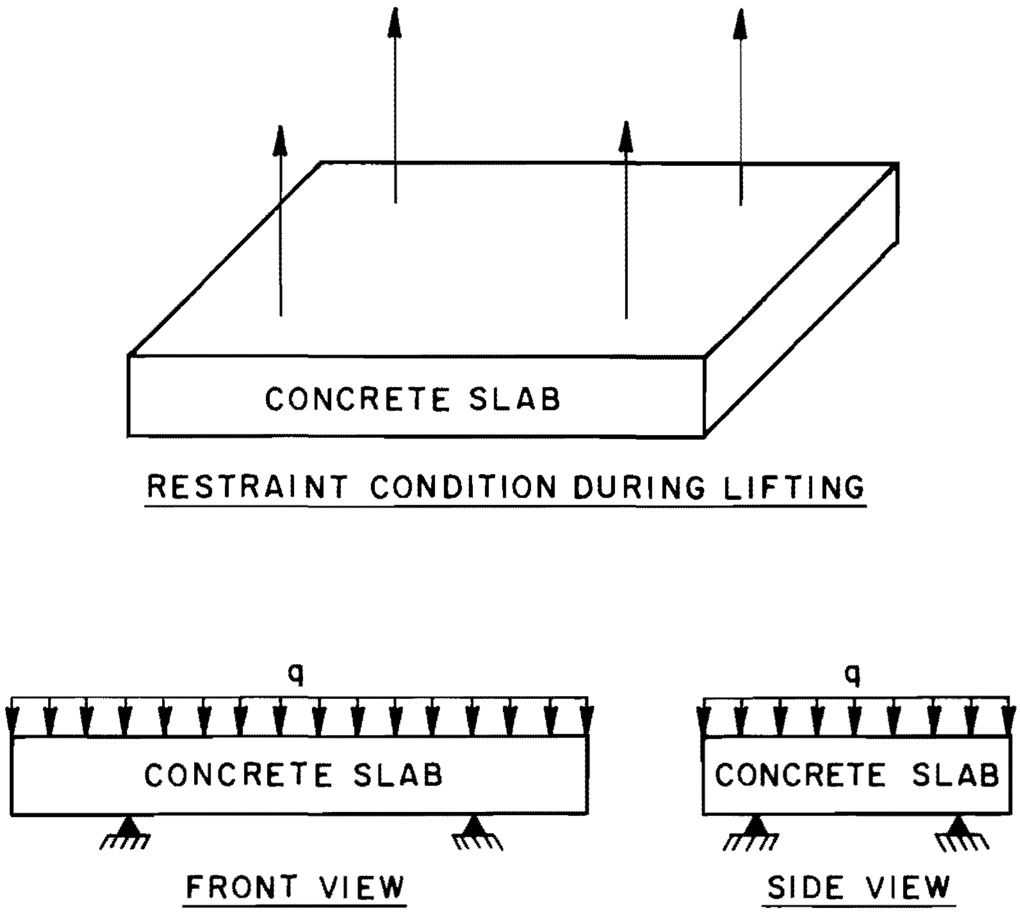


Fig 4.4. SLAB49 Lifting model for precast slabs.

The required mean beam and compressive concrete strengths which should be attained to insure a high degree of reliability against cracking during lifting are presented in Tables 4.2 and 4.3.

SUMMARY

The structural analysis of precast repair slabs was covered in this chapter. This consisted of modeling and calculating the response of precast slabs to loading resulting from volume change, wheel loads, and lifting. Detailed study of these topics is presented in the appendices. The important point of this chapter is the application of mechanistic theories and engineering principles to predict the behavior of precast slabs which can be used in a design process, such as presented in the next chapter, so that evaluations and decisions are made on a rational level.

TABLE 4.2. REQUIRED MEAN FLEXURAL STRENGTHS FOR LIFTING SLABS (psi) (THIRD POINT LOADING).

Slab Size, ft	Standard Deviation of Concrete Strength, psi	Lift Points	
		$\frac{1}{5}$	$\frac{1}{4}$
12 X 6	20	150	178
	40	186	214
	60	223	250
12 X 12	20	245	270
	40	281	307
	60	318	344
12 X 14	20	303	343
	40	339	379
	60	376	416
12 X 20	20	407	502
	40	443	539
	60	480	576

1 ft = .305 m　　1 psi = 6.89 kPa

TABLE 4.3. REQUIRED MEAN COMPRESSIVE STRENGTHS FOR LIFTING SLABS (psi).

Slab Size, ft	Standard Deviation of Concrete Strength, psi	Lift Points	
		$\frac{1}{5}$	$\frac{1}{4}$
12 X 6	20	400	560
	40	615	815
	60	885	1100
12 X 12	20	1100	1300
	40	1400	1700
	60	1800	2100
12 X 14	20	1650	2100
	40	2050	2550
	60	2500	3100
12 X 20	20	2900	4500
	40	3500	5200
	60	4100	5900

1 ft = .305 m 1 psi = 6.89 kPa

CHAPTER 5. DESIGN PROCEDURE FOR PRECAST REPAIR SLABS

Experience has shown that repair of CRCP is not an easy task. Performance of many traditional portland cement concrete cast-in-place patches has been so poor that some maintenance engineers have gone to asphaltic mixtures as their permanent repair material. The need to approach the repair of CRCP as an engineering design, instead of as a routine maintenance procedure, is here.

In the previous three chapters, factors affecting design, alternate strategies, and analysis were discussed. These topics form a part of the design process presented in this report. This chapter presents a sequential procedure for the design of precast slab pavement repairs. The procedure is broken down into specific steps under generalized headings. These headings can apply equally to most engineering design problems. The approach taken is to present a very detailed and thorough approach to the design of repairs for CRCP. In actual design, segments of this procedure may not always be needed or desired by each individual designer. It is recommended that for development and trial installations the procedures be adhered to as closely as possible. The generalized headings for design used in this chapter are

(1) identification of problem,
(2) investigation of problem,
(3) formulation of alternate strategies,
(4) analysis of strategies,
(5) selection of a strategy,
(6) implementation, and
(7) monitoring.

A summary flowchart of this process is shown in Fig 5.1.

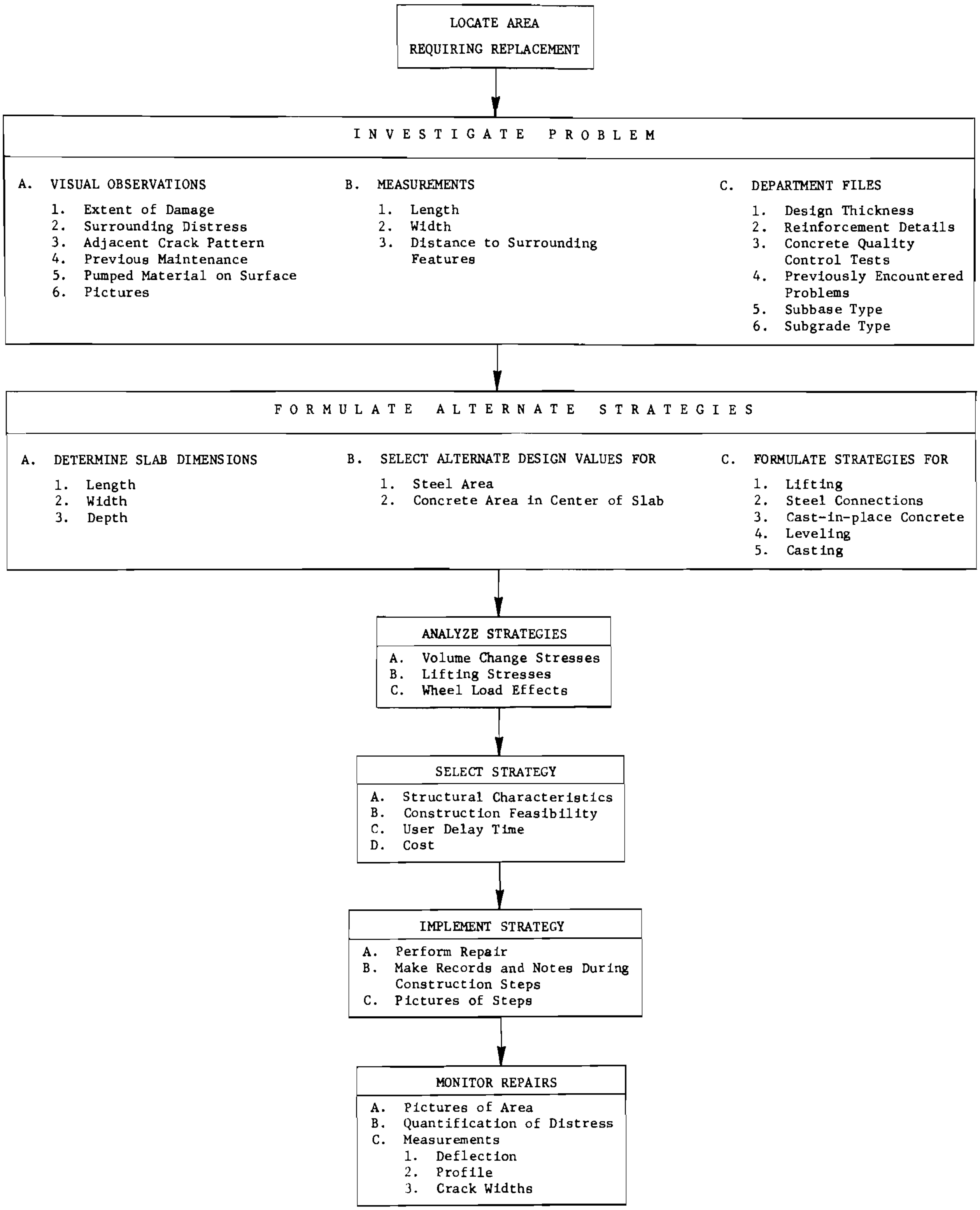


Fig 5.1. Flowchart of precast slab repair design process.

IDENTIFICATION OF PROBLEM

The first step in the design process is the identification of a problem. In this case, it is the location of deteriorated portions of CRCP whose condition warrants replacement. A survey should be performed by qualified personnel to identify and locate potential areas requiring replacement. This survey should include an indication of the type of distress and its relative severity. A priority ranking of repair needs can then be established based on this information.

INVESTIGATION OF PROBLEM

After an area requiring replacement has been located, the next step involves collection of information pertaining to the repair area to determine dimensions, constraints, and other facts leading to an understanding of the problem. Information may be collected at the site and in the records of the highway authority.

Information Collected at the Site

A visual observation of the area is the first step. Items to look for and note are the cause of the failure, extent of damage, surrounding distress, adjacent crack pattern, drainage pattern of the immediate area, location with respect to grade, evidence of previous maintenance, and the presence of pumped material on the surface. This information may be recorded on a form such as the one shown in Fig 5.2. A completed example of the form shown in Fig 5.2 is shown in Fig 5.3.

Pictures of areas scheduled for repair are helpful for both planning and studying repair techniques. The information recorded from the above visual observations are reinforced with photographs of the proposed repair area. It is suggested that a photograph which shows both the pavement edge and center line be taken from the shoulder. This picture will normally be taken within 10 feet (3.05 m) of the pavement edge. A photograph of the longitudinal view of the repair area from the center of the lane to be repaired is also suggested. Any additional pictures of the area should also be made. The more information collected at the site, the better. Each picture should have an identification marker in it which specifies the highway, direction of traffic, date, and location.

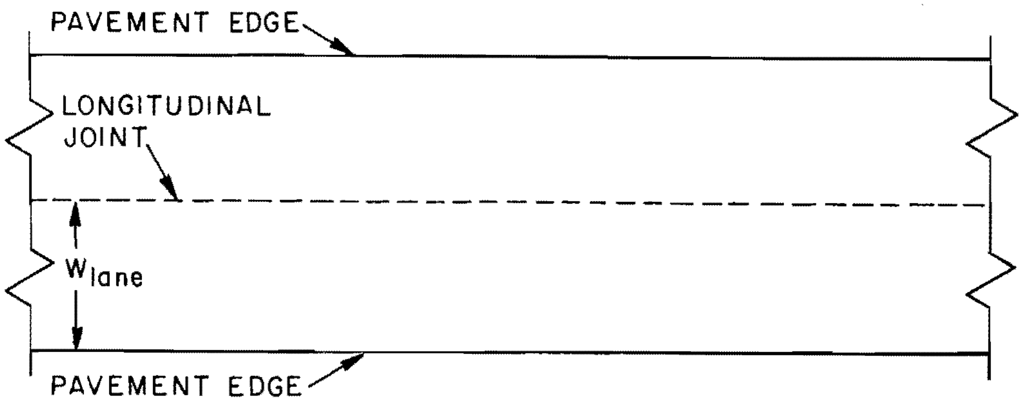
District ______ Control ______ Section ______ Job ______

Highway and Direction ______

Location: Mile Post ______ Mile Point ______

Location Description ______

______



Record the following information on the figure above.

1. Sketch crack pattern in failure area.
2. Sketch any edge pumping present.
3. Sketch and mark the adjacent crack spacing.
4. Sketch and indicate previous repair patches.
5. Sketch limits of proposed repair.

Measure W_{lane} at several locations in the proposed repair area.

Measurement No.	1	2	3	4	5	6
W_{lane}						

Length of proposed repair: ______

Condition of Shoulder ______

______

General Comments ______

______

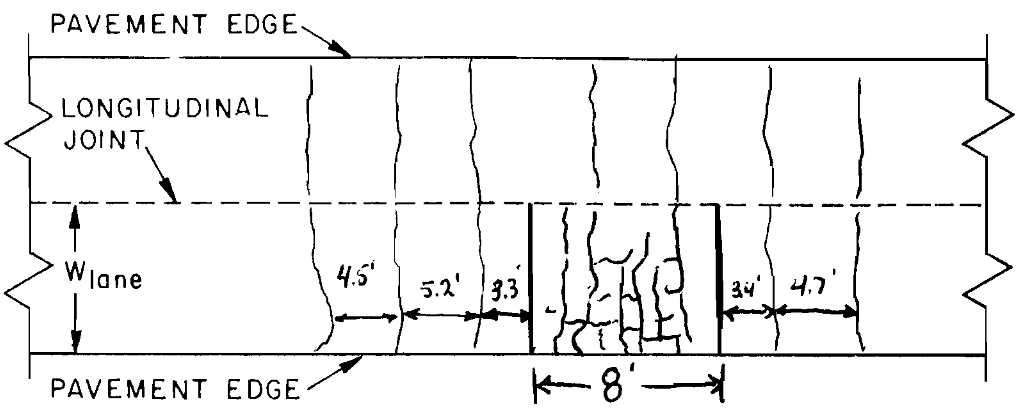
Fig 5.2. Example form for information collected at the site.

District 27 Control 375 Section 12 Job 2

Highway and Direction I-100 South Bound

Location: Mile Post 134.2 Mile Point 12.135

Location Description 300' South of River Creek Bridge



Record the following information on the figure above.

1. Sketch crack pattern in failure area.
2. Sketch any edge pumping present.
3. Sketch and mark the adjacent crack spacing.
4. Sketch and indicate previous repair patches.
5. Sketch limits of proposed repair.

Measure W_{lane} at several locations in the proposed repair area.

Measurement No.	1	2	3	4	5	6
W_{lane}	11.91'	11.95'	12.0'	12.0'	12.0'	12.0'

Length of proposed repair: 8'

Condition of Shoulder Good, Side Joint Open, No Pumping Present

General Comments Area badly fractured, moving blocks begining to come loose, No previous repairs present

Fig 5.3. Example completed form for information collected at the site.

The required measurements of the deteriorated area are the width and length. Highway pavements are not always constructed to a standard width of 12 feet (3.66 m). The distance from the longitudinal joint to the edge of the pavement should be measured at several locations in the proposed repair area. Not only are the lengths of deteriorated portions important, but measurements of the distances to surrounding features, such as transverse cracks, may also be significant. The length of the proposed repair should also be recorded. These measurements may be recorded as shown in Fig 5.2.

Other measurements of the deteriorated area, which are not necessary for design purposes but provide additional information, include steel depth, deflection, profile, and various measurements on cores. Steel depths may be adequately measured with a James Pachometer (Ref 1). This magnetic device was found to produce poor results for depth measurement of welded wire fabric reinforcement in pavements.

Deflection and profile measurements are useful monitoring tools for evaluation of repairs. Deflection measurements may be performed with a Dynaflect (Ref 36), Benkelman Beam, or Falling Weight Deflectometer (Ref 37) devices. Deflection measurements can also be used to characterize the subgrade material for further analysis (Ref 39). Profile measurements may be conveniently measured with the Surface Dynamics Profilometer (Ref 45). These measurements should be performed before and after the repair, to obtain deflection and profile histories.

Cores of the deteriorated area may be taken to help determine the extent of failure, condition of all layers of the pavement structure, severity of deterioration, pavement depth, depth of reinforcing steel, material properties of the pavement layers, and possible causes of the problem.

Information Found in Highway Records

All highway authorities, such as the Texas SDHPT, keep records on roadways in their jurisdiction. Information on the original pavement design can usually be found. Items from the original pavement design which are of concern for precast repairs include concrete thickness, type and size of steel reinforcement, steel spacing, steel depth, subbase type, subbase thickness, and subgrade type. The Texas SDHPT also keeps quality control records of pavement construction. These records are helpful in providing information on the date a particular section was constructed, concrete strength tests, aggregate gradation analysis, air temperature during placement, and thickness measurements of the pavement. Information on previous problems encountered and maintenance performed on particular pavement sections may also be found in a highway department's files. An example of a form to record this information on is shown in Fig 5.4. A completed form is shown in Fig 5.5.

With the detailed information above, a picture of the many aspects of the repair problem should appear. Constraints, restrictions, and requirements can be inferred from this information. Such knowledge can aid in the formulation and selection of correction strategies.

FORMULATION OF ALTERNATE STRATEGIES

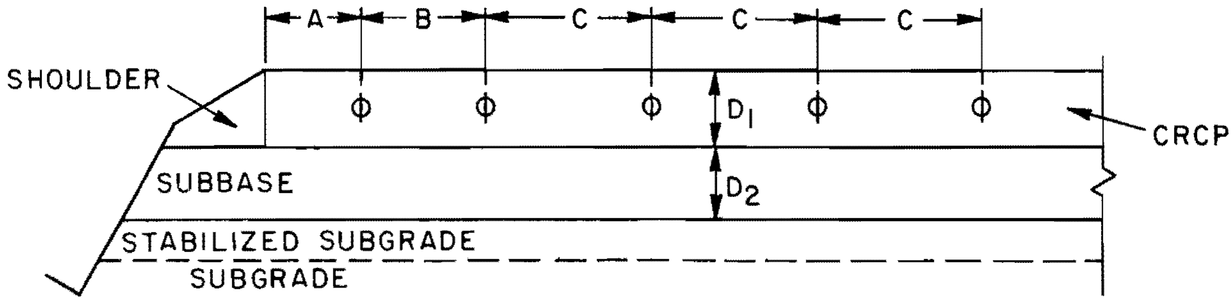
Formulation of alternate repair strategies is the next design step. An alternate precast slab repair strategy should include the details of steel reinforcement, steel anchorage, slab dimensions, cast-in-place concrete, lift provisions, and installation procedures. Help in the formulation of strategies can be found throughout this report. Although the formulation of strategies should be limited to those outlined in this report, the design engineer should feel free to use his own ideas, imagination, initiative, and experience in the formulation process.

Either of two overall formulation schemes can be employed. One scheme is to develop the design through iteration with analysis. A starting strategy is formulated and analyzed and then altered and reanalyzed. This procedure is repeated until an acceptable design is achieved. The other scheme is to formulate a number of strategies to be analyzed at the same time. An optimum strategy, based on cost, for example, is then selected from among the strategies satisfying design requirements.

A simple way to start the formulation of a repair strategy is to reproduce the repaired CRC pavement's initial design. This includes use of the same steel type, size, and spacing. This follows the logic of replacing the deteriorated portion of pavement with a sound portion which matches the rest of the pavement. However, due to the differences between a precast slab and CRCP, this identical design may not satisfy design requirements. In the first scheme, this design is altered and reanalyzed. In the second scheme, District _____ Control __________ Section ____________ Job ____________

Highway and Direction ____

Location: Mile Post ____________________ Mile Point ________________



CONCRETE SLAB

Thickness*, D_1 ____________ Mean Conc. Strength @ 7 days ____________

Type of Coarse Aggregate ____

STEEL REINFORCEMENT

Type ______________ Design Specification ___________________________

Steel Grade ____

Longitudinal Steel:

Diameter __________ Spacing*, A _______ B _______ C ________

Transverse Steel:

Diameter __________ Spacing ____________________

SUBBASE

Type ____________________ Type of Stabilization ____________________

Thickness*, D_2 _________________________________

SUBGRADE

Type of Stabilization ____

ADDITIONAL INFORMATION

____

____

____

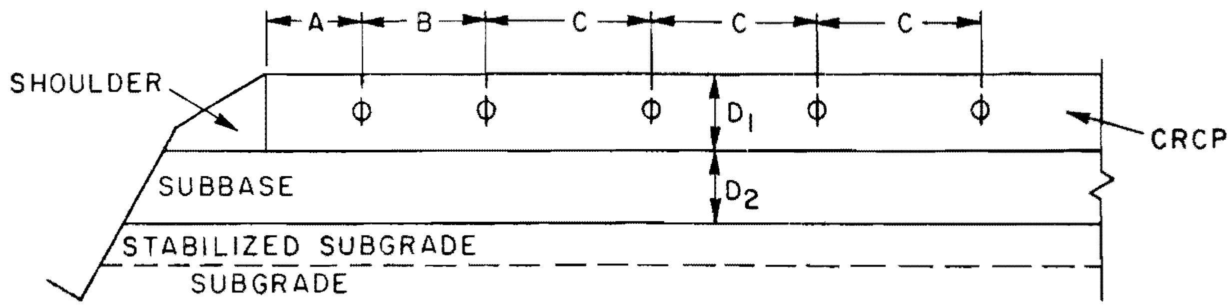
*Refer to the above figure.

Fig 5.4. Example form for information from highway department records.

District 27 Control 375 Section 12 Job 2

Highway and Direction I-100 South Bound

Location: Mile Post 134.2 Mile Point 12.135



CONCRETE SLAB

Thickness*, D_1 8" Mean Conc. Strength @ 7 days 550 psi

Type of Coarse Aggregate ½" max. limestone

STEEL REINFORCEMENT

Type Deformed Bars Design Specification CPCR(B)-62

Steel Grade 60 ksi

Longitudinal Steel:

Diameter 5/8" Spacing*, A 3" B 6" C 7.5"

Transverse Steel:

Diameter ½" Spacing 30"

SUBBASE

Type Stabilized Type of Stabilization cement

Thickness*, D_2 6"

SUBGRADE

Type of Stabilization None

ADDITIONAL INFORMATION

Steel placement by double strike-off method

*Refer to the above figure.

Fig 5.5. Example completed form for information from highway department records.

alternate strategies can be produced by altering certain elements of the identical design, such as amount of reinforcing steel, and these strategies can be sequentially analyzed.

ANALYSIS OF ALTERNATE STRATEGIES

Analysis of precast repair strategies can be divided into three components. These components are modeling of the strategy, estimating responses with the model, and interpretation of the results. This type of analysis is applicable to most engineering problems. It is applied here to the design of the structural elements of a precast repair slab.

Appendices 2, 3, 4, and 5 demonstrate the modeling, estimation, and interpretation of volume change, wheel load, and lifting analysis of precast repair slabs. In brief, the process consists of modeling the slab with the appropriate computer program, running the program, and investigating the results. This process may be used upon a number of alternate strategies at once with the aim of selecting an optimal solution, or it may be used in an iterative process where an alternative is sequentially altered until an acceptable solution is achieved. Analysis is covered in Chapter 4.

SELECTION OF A STRATEGY

The purpose of analyzing alternate strategies is to characterize their respective suitabilities as repair methods. This information is then used in the selection process. Ideally, the selection should be based upon optimization of performance and cost. Performance of a precast repair for CRCP cannot be accurately determined. No field trials have been performed. Also, manipulating those elements of a pavement structure which are thought to influence performance, such as thickness, is constrained by compatibility with the pavement structure. However, performance is related to factors, such as stress levels, which can be estimated with current theories.

The other consideration for selection of a strategy is cost. The initial cost of a technique is perhaps the most important consideration. This is due to the nature of funding of a repair. Funds to perform a repair must come from current resources. Often, future costs, such as maintenance and road user, are used to justify a particular type of strategy over another in terms of total costs. These future costs, however, are difficult to quantify and are, at best, educated guesses.

Selection of a strategy is then based upon predicted response to volume change, lifting, and wheel loading and the cost of the repair.

IMPLEMENTATION

The design of a precast repair does not stop after selection of a strategy. Design should be an ongoing exercise to be updated and improved as information is gathered through experience. To this end, it is important that once a strategy has been selected that it be implemented. This is a natural result since this is the aim of the design process. However, implementation should also be considered as a part of the design procedure.

Construction is a very critical step for the proper functioning of any structure. It is important that all methods and procedures contribute to a sound repair and conform to accepted engineering methods. Records and notes taken during construction steps on significant departures from procedures or designs and significant events can be helpful in further evaluation of the process. Pictures are also useful for this purpose and for communicating to others what has been done.

MONITORING

Monitoring of the condition of a repair is a vitally important step in the design process. Observations and measurements taken at periodic intervals are used to check behavior predictions, determine detrimental elements, and develop a data base for further design improvements. The observations should be recorded in an organized fashion. Pictures of the area over time are also useful for recording condition information. Measurements which might be taken on the repair are deflection, profile, and steel and concrete strain measurements. These measurements may correspond to those taken prior to performing the repair for comparison.

CHAPTER 6. DESIGN, PREPARATION, AND INSTALLATION PROCEDURES FOR TEXAS

The first five chapters of this report have presented a general approach and an introduction to the application of precast techniques to the repair of CRCP. This information may be applicable anywhere in the United States. In this chapter, recommendations for the design, preparation, transportation, and installation of precast repair slabs are presented for Texas conditions.

The design and construction of precast repair slabs, as indicated in Chapter 3, may be performed from two approaches. Ideally, a precast repair slab should be designed and constructed on an individual basis for each proposed repair. However, this individualized approach may prove to be uneconomical and the alternate approach of using a standardized section may have to be adopted. The procedures for each methodology are equally applicable.

DESIGN RECOMMENDATIONS

Recommendations on the design of precast slab elements are presented in this section. The elements of precast repair slab discussed here are slab dimensions, steel reinforcement, lift connections, weakened plane, and steel anchorage. The recommendations presented here are composite for Texas conditions. Adaptations to specific conditions for some districts may be necessary.

Slab Dimensions

The slab thickness should be reduced from 0.10 to 0.25 inch (2.54 to 6.35 mm) thinner than the thickness of the existing CRCP pavement (CRCP depth) unless prior knowledge indicates otherwise.

The width of the slab should be cast 1/2 to 1 inch (12.7 to 25.4 mm) less than the full lane width. This will depend upon the treatment of the edge adjacent to the next lane and the edge next to the shoulder.

The required slab dimensions for a precast repair are shown in Fig 6.1. The length of a precast repair slab may be determined with the following equation:

$$L_S = L_R - 2L_{AZ} \tag{6.1}$$

where

L_S = length of precast repair slab, in.,

L_R = total length of repair, in., and

L_{AZ} = length of steel anchorage zone, 8 to 10 in. (203 to 254 mm).

The width of the precast slab may be determined by

$$W_S = W_R - X \tag{6.2}$$

where

W_S = width of precast repair slab, in.,

W_R = width of repair = lane width, in., and

X = varies according to edge treatment, 1/2 to 1 in. (25.4 mm).

The recommended thickness of a precast repair slab, shown in Fig 6.2, is

$$D_S = D_P - 0.25 \tag{6.3}$$

where

D_S = depth of precast repair slab, in., and

D_P = CRCP depth, in.

Steel Reinforcement

The same type of steel reinforcement as that used in the pavement to be repaired should be used in the repair slab. The exception to this rule is for CRCP constructed with smooth welded wire fabric. In this case, the use of deformed wire fabric should be considered. Deformed reinforcing steel should be used for all precast repair slabs. Guidelines to the steel reinforcement used in 70 to 90 percent of the CRC pavements constructed in Texas between 1959 and 1970 are listed in Table 6.1.

The recommended dimensions and placement of reinforcement in a precast repair slab are shown in Figure 6.1. The spacing, grade, and diameter of longitudinal steel reinforcement are controlled by the steel present in the existing CRCP. The length of the longitudinal reinforcing steel is

$$L_{RS} = L_R - Y \tag{6.4}$$

where

L_{RS} = length of longitudinal steel reinforcement, in.,

L_R = length of repair, in.,

Y = 6 in. (152 mm) to $2L_{AZ}$, in., depending on steel anchorage, and

L_{AZ} = length of steel anchorage zone, in.

The depth of the longitudinal steel reinforcement should be the same as that in the pavement being repaired. In Texas this distance should be

$$D_R = D_P/2 \tag{6.5}$$

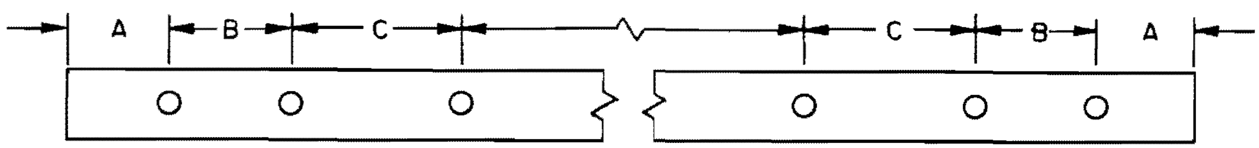
where

D_R = distance from the surface of the slab to the centroid of the longitudinal steel reinforcement, in., and

D_P = depth of pavement to be repaired, in.

TABLE 6.1. GUIDELINES FOR CRCP CONSTRUCTED IN TEXAS BETWEEN 1959 AND 1970[1]

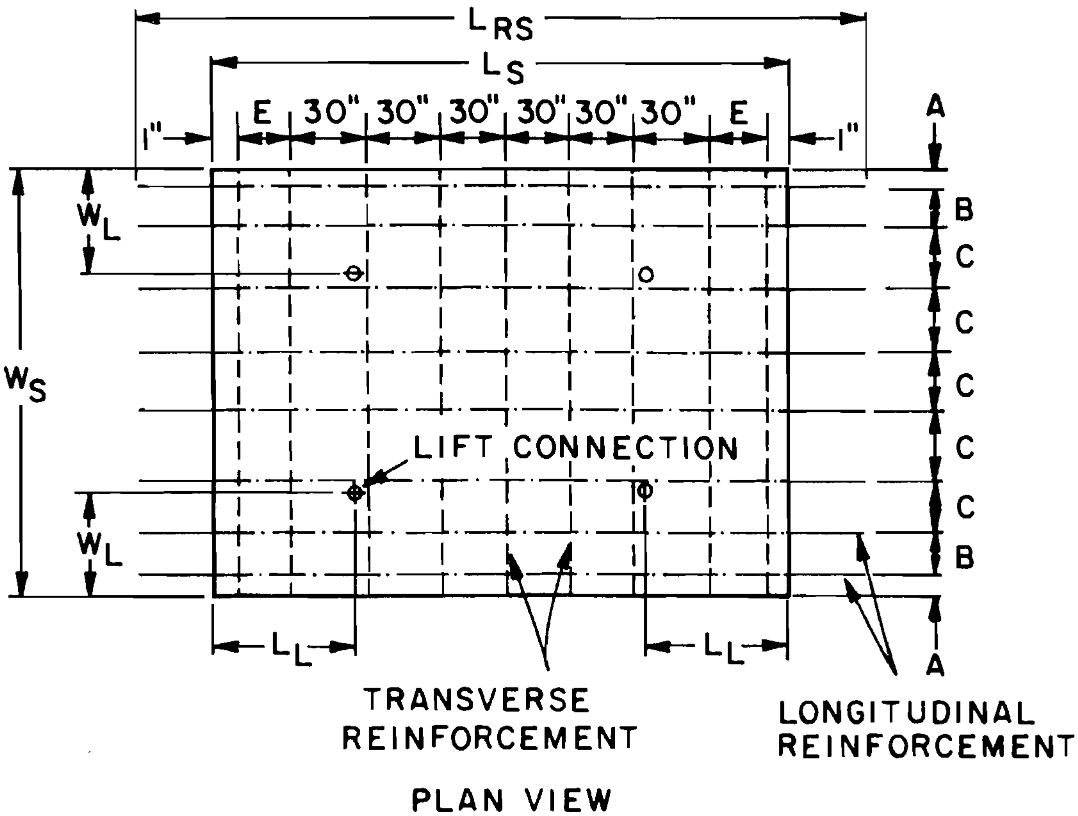
Pavement Thickness, in.	0.5 Percent Longitudinal Steel High Yield, 60 ksi, Deformed Steel Bars							0.6 Percent Longitudinal Steel Hard Grade, 50 ksi, Deformed Steel Bars						
	24-Foot Placement				12-Foot Placement			24-Foot Placement				12-Foot Placement		
	Bar No.	A*, in.	B*, in.	C*, in.	A, in.	B, in.	C, in.	Bar No.	A, in.	B, in.	C, in.	A, in.	B, in.	C, in.
8	5	3	6	7.5	3	5.25	7.5	5	3.5	4	6.5	3	4	6.5
7	5	3	5	8.5	4	8.5	8.5	5	3	6	7.5	3	5.25	7.5
6	4	3	4.5	7	3	6	7	5	3	5	8.5	4	8.5	8.5



* A, B AND C ARE STEEL SPACING DIMENSIONS, AS SHOWN ABOVE.

[1]These dimensions will vary slightly for projects designed and constructed by the Houston Urban Office

1 in. = 25.4 mm   1 psi = 6.89 kPa   1 ft = .305 m



SLAB DIMENSIONS

L_S: Precast Slab Length = $L_R - 2L_{AZ}$

L_R: Length of Repair (not shown)

L_{AZ}: Length of Steel Anchorage Zone (not shown)

L_L: Distance to Lift Connection = $L_S/4$

W_S: Precast Slab Width = W_R - X

W_R: Width of Repair = Lane Width (not shown)

X: Varies for Edge Treatment, 0 to 1 inch

W_L: Distance to Lift Connection = $W_S/4$

LONGITUDINAL REINFORCEMENT

L_{RS}: Length of Longitudinal Reinforcement = L_R - 6 inches

A,B,C: Longitudinal Steel Spacing of Pavement, see Table 6.1

TRANSVERSE REINFORCEMENT

E: Variable Spacing to Accommodate the Transverse Steel Spacing

Fig 6.1. Guidelines for dimensions of precast repair slabs for Texas.

The transverse steel reinforcement should consist of No. 4, grade 60, steel reinforcing bars unless deformed wire fabric is used. The transverse spacing should be 30 in. (76.2 cm), varying at the ends of the slab as required. The length of the transverse steel reinforcement should be

$$L_{TR} = W_S - 3.0 \tag{6.6}$$

where

L_{TR} = length of transverse reinforcement, in., and

W_S = width of precast repair slab, in.

The transverse reinforcement should be positioned on top of the longitudinal reinforcement.

Lift Connections

The positions of threaded inserts for lifting connections are shown in Fig 6.1. The distance from the edge of the slab to the lift points should be

$$W_L = W_S/4 \tag{6.7}$$

where

W_L = distance to lift point along transverse edge, in., and

W_S = width of precast repair slab, in.

and

$$L_L = L_S/4 \tag{6.8}$$

where

L_L = distance to lift point along longitudinal edge, in., and

L_S = length of precast repair slab, in.

Weakened Plane

For precast repair slabs longer than 7 feet (2.13 m), a weakened plane, formed with a bond breaker made of 28-gaǵe galvanized metal, should be oriented in the middle of the slab. The dimensions and details of bond breakers for varying pavement thicknesses are shown in Fig 6.3. The distance from the end of the slab to the bond breaker is

$$L_B = L_S/2 \tag{6.9}$$

where

L_B = distance from end of slab to bond breaker, in., and

L_S = length of precast repair slab, in.

Precast slabs longer than 14 feet (4.26 m) will require two additional weakened planes. These bond breakers are positioned near the quarter points of the slab length and should not interfere with the lifting hardware positioned there. The distance from the end of the slab to these bond breakers is

$$L_{BL} = L_S/4 - 1 \tag{6.10}$$

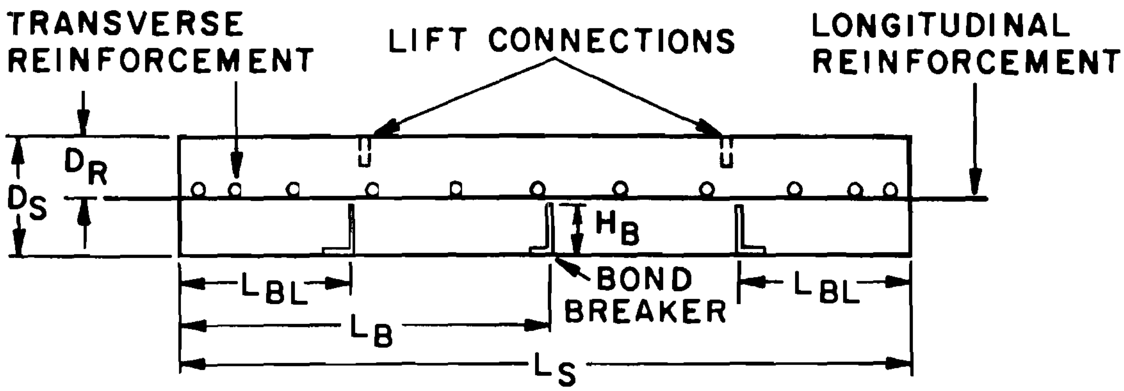
where

L_{BL} = distance from end of slab to additional bond breakers for slabs longer than 14 feet (4.26 m), in., and

L_S = length of precast repair slab, in.

Steel Anchorage

The use of prepositioned steel reinforcement in the precast slab aligned with the reinforcement in the CRC pavement is recommended. Positive steel connections made with individual lap welds, weld transfer plates, or cable clamps appear to offer the best anchorage regardless of the cast-in-place concrete to be placed around them. The length of the steel anchorage zone



D_S: Slab Depth = D_P - 0.25 inch

D_R: Depth of Longitudinal Reinforcement = $D_P/2$

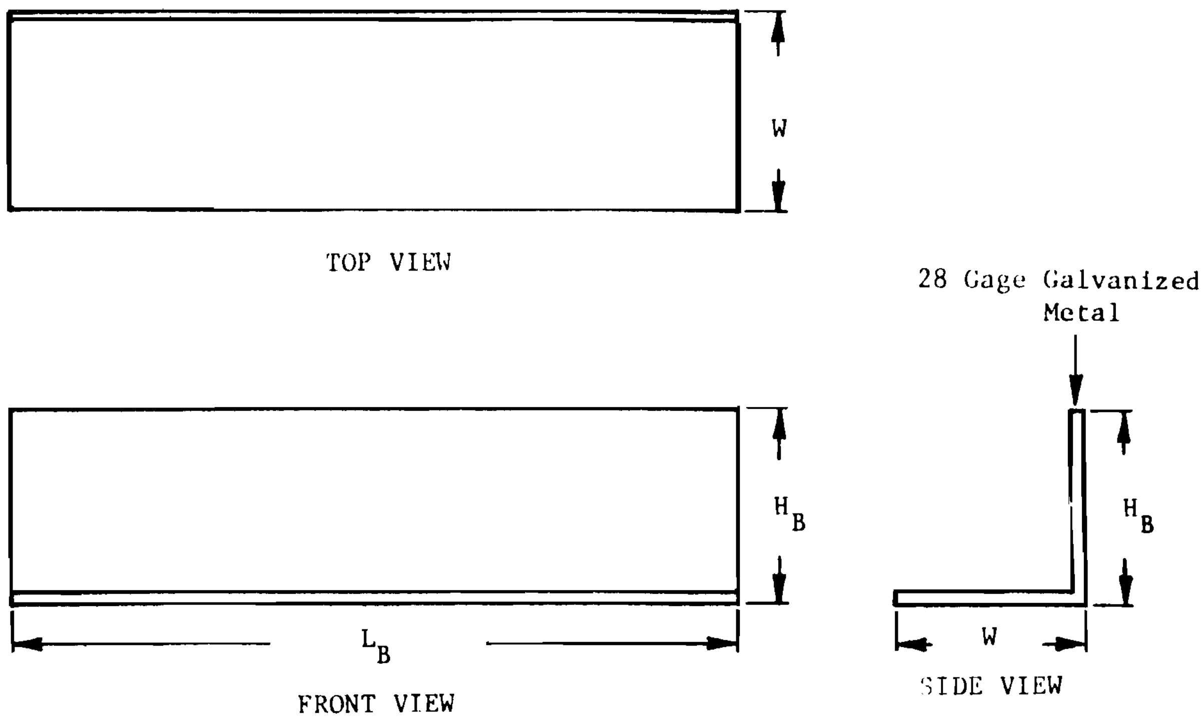
L_S: Precast Slab Length, Fig 6.1

L_B: Distance to Bond Breaker = $L_S/2$ when L_S > 7 feet

L_{BL}: Distance to Bond Breaker = $(L_S/4)$ - 1 inch when L_S > 14 feet

H_B: Height of Bond Breaker, Fig 6.3

Fig 6.2. Guidelines for dimensions of precast repair slabs for Texas.



Dimensions:

Pavement Thickness, in.	Height of Bond Breaker, H_B, in.	Width of Bond Breaker, W, in.
6	2	2
7	2.50	2.50
8	2.75	2.75
10	3.25	3.25

L_B: Length of Bond Breaker, in.

$$L_B = W_S - 3.0$$ in. = 25.4 mm

where W_S = width of precast repair slab, in.

Fig 6.3. Recommended dimension for bond breakers in precast repair slabs longer than 7 ft (2.13 m) should be greater than 6 inches (152 mm) and less than 12 inches (305 mm). A 10-inch (254 mm) long anchorage zone should be adequate for the above positive connections.

The use of polymer concrete in the steel anchorage zones is highly recommended. This material not only offers excellent bond characteristics with steel, it is capable of penetrating, filling the voids, and strengthening the concrete in the adjacent pavement. The use of a combination positive/passive steel connection is possible with this material due to its superior bond with steel. In addition, the length of the achorage zone can be reduced. Positive steel connections may be made as short as 6 inches (152 mm). Passive connections should be made 8 to 10 inches (203 to 254 mm) long. Guidelines for the steel anchorage zone are shown in Fig 6.4.

SLAB PREPARATION

The alternate techniques of constructing precast slabs were detailed in Chapter 3. This chapter should be referenced for more information on the alternatives suggested below. The following steps are recommended for the preparation of precast repair slabs for CRCP.

(1) Select convenient area in which to cast slabs.

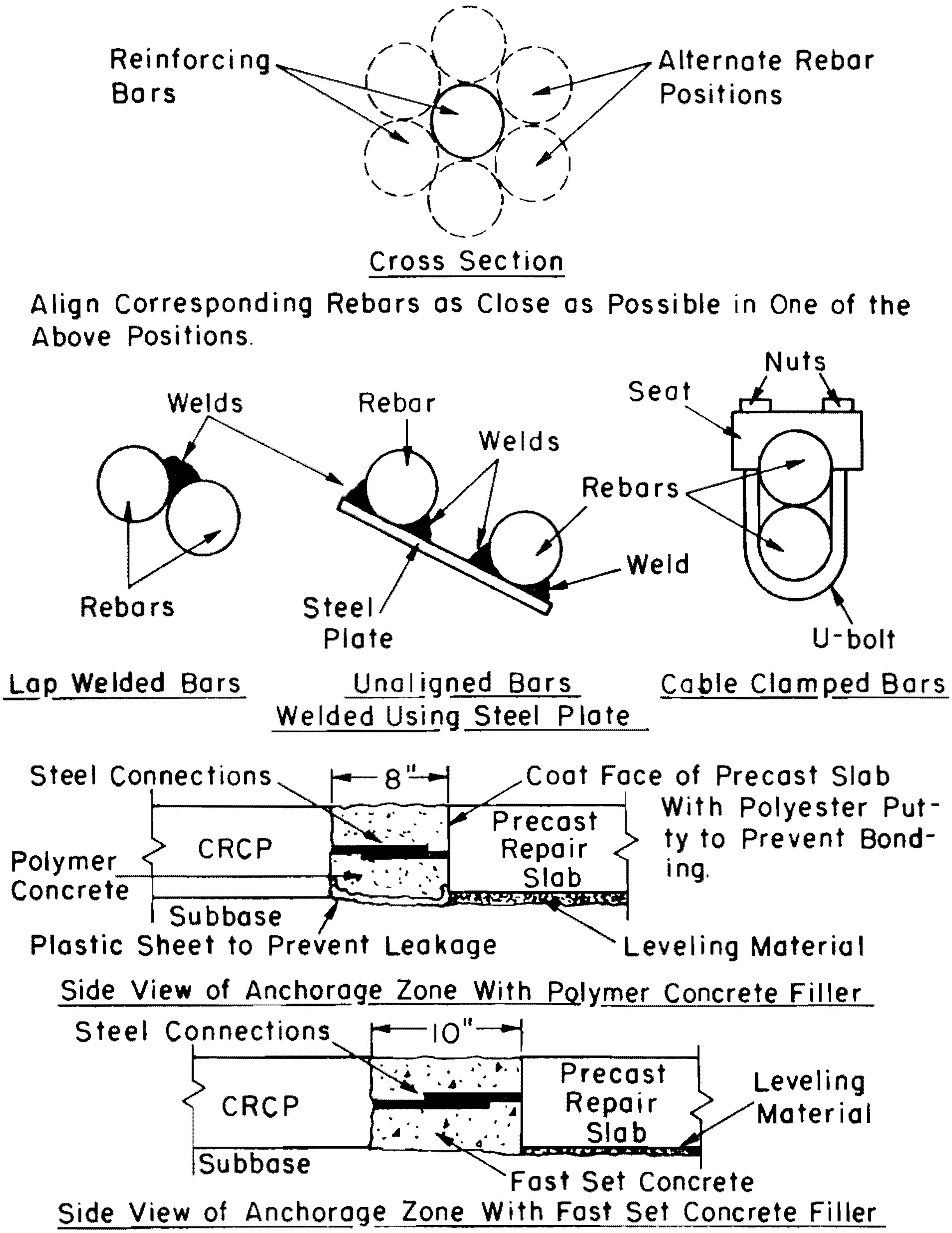
(2) Construct forms:

- (a) use form plan shown in Fig 3.1 with slotted end piece;
- (b) Make the width between the parallel side forms, Fig 3.1, equal to W_S determined from Eq 6.2 with $x = 1.0$;
- (c) determine the length between end pieces using Eq 6.1 with $L_{AZ} = 10$; and
- (d) Construct forms in accordance with Item 420.9 of the Standard Specifications (Ref 47).

(3) Place polyethylene sheets on the ground over the casting bed. The sheets should have a minimum thickness of 4 mils (0.004 in.).

(4) Assemble forms over the polyethylene sheets.

(5) For slabs longer than 7 feet (2.13 m), insert and secure bond breakers as shown in Figs 6.2 and 6.3.



1 in. = 25.4

Fig 6.4. Guidelines for Steel Anchorage.

(6) Place longitudinal reinforcement:

(a) determine length of rebars with Eq 6.4 with $Y = 5$;

(b) cut rebars;

(c) place rebars in forms and position according to steel spacing in CRCP, Table 6.1; and

(d) support rebars on metal or plastic chairs (do not use wood or cinder blocks).

(7) Place transverse reinforcement:

(a) position transverse reinforcement on top of the longitudinal reinforcement according to spacing shown in Fig 6.1 and

(b) tie steel together at intersections.

(8) Place lift connections:

(a) position lift connections at the positions shown in Fig 6.1; positions may be calculated with Eqs 6.7 and 6.8; inserts are shown in Fig 3.10;

(b) align top of lift connection with surface of precast slab; and

(c) insert removable plugs into threads to prevent entrance of foreign material.

(9) Concrete mix should conform to Item 366 (Ref 47); use same coarse aggregate as existing concrete.

(10) Cast concrete beam specimens as outlined in Texas Highway Department Bulletin C-11 (Ref 48).

(11) Place and vibrate concrete in accordance with Item 366 (Ref 47).

(12) Strike concrete off to the proper elevation. Do not overwork.

(13) Create a skid resistant surface texture with a burlap drag or stiff-bristled hand broom as specified in Item 360.8 (Ref 47).

(14) Cure the concrete with a curing membrane or wet mat as specified in Item 360.9 (Ref 47).

(16) Remove forms:

(a) allow concrete to cure a minimum of 72 hours before removing forms; and

(b) do not damage concrete surfaces or edges.

(17) Do not lift slabs until concrete has reached the strengths listed in Table 6.2.

SLAB TRANSPORTATION

Handling and transportation of large concrete slabs may result in chipping of edges or scuffing of the surface texture. Prevention of damage to precast slabs and safety of workmen are the principle considerations for the transport operation.

Precast repair slabs may be lifted with a simple mechanism consisting of four swivel lift plates attached to the slab with threaded inserts. The analysis reported in Appendix 3 shows that stress levels for the 1/4 span placement of the lift points resulted in low stresses and compression in the bottom fiber. Compression in the lower slab fiber is important to prevent slabs with weakened planes from prematurely cracking due to the loss of bond.

The following procedures are recommended for transportation of precast repair slabs to the repair site.

(1) Allow concrete to attain the mean concrete strengths listed in Table 6.2.

(2) Bolt swivel lift plates to lift connections.

(3) Lift slab with movable lifting equipment. The approximate weight for a range of precast concrete slabs is presented in Table 6.3.

(4) Place slab on transport vehicle:

   (a) use a flat bed truck so that the bottom surface of the slab is in full contact with the truck bed;

   (b) for more than one slab, place 2" x 8" wood runners spaced 24" center to center between the slabs;

   (c) secure the slabs to the truck to prevent movement during transportation.

(5) Remove lifting hardware.

(6) Drive to repair location.

(7) Reinsert lifting hardware and remove slabs from vehicle.

INSTALLATION PROCEDURE

The following procedure is recommended for the removal of deteriorated pavement and placement of a precast repair slab.

(1) Block off traffic in lane to be repaired. A positive protection device, such as a portable crash cushion (Ref 39), should be used in addition to common signing practices.

TABLE 6.2. REQUIRED MEAN FLEXURAL STRENGTH FOR LIFTING PRECAST SLAB WITH LIFT POINT PLACED AT 1/4 SPAN

Slab Size, ft	Standard Deviation of Concrete Strength, psi	Required Mean Flexural Strength, psi
12 x 6	20	180
	40	215
	60	250
12 x 12	20	270
	40	310
	60	345
12 x 14	20	345
	40	380
	60	420
12 x 20	20	500
	40	540
	60	580

1 ft = 0.305 m 1 psi = 6.89 kPa

TABLE 6.3. APPROXIMATE WEIGHT OF PRECAST CONCRETE SLABS

Slab Width, ft	Slab Length, ft	Slab Thickness, in.	Approximate Weight,* lb
12	4	6	3,600
		8	4,800
		10	6,000
12	6	6	5,400
		8	7,200
		10	9,000
12	12	6	10,800
		8	14,400
		10	18,000
12	14	6	12,600
		8	16,800
		10	21,000
12	20	6	18,000
		8	24,000
		10	30,000

*Based on unit weight of 150 lb/ft^3 (2403 kg/m^3)

1 ft = 0.305 m   1 in. = 25.4 mm   1 lb = 2.205 kg

(2) Determine length of repair:

(a) estimate length of unsound pavement;

(b) select length based upon lengths of prepared slabs or based upon deterioration and cast repair slab to size; and

(c) include length of steel anchorage zones.

(3) Mark saw cut lines:

(a) mark length of precast repair slab on outside edge of pavement;

(b) measure and mark length of steel anchorage zones along edge of pavement; and

(c) construct a line across the pavement perpendicular to the edge using the 3-4-5 or 5-12-13 right triangle methods described in Fig 3.8;

(d) mark parallel lines at other marks on outside of pavement; and

(e) use a permanent type water resistant marker.

(4) Perform saw cuts:

(a) make cuts on end lines 2 1/2 in. (63.5 mm) deep; avoid cutting longitudinal reinforcing steel;

(b) make cuts to within 1/2 in. (12.7 mm) of concrete depth along the two inside transverse lines, cutting the steel; and

(c) make other cuts as required to dislodge deteriorated concrete.

(5) Remove concrete from around steel in steel anchorage zone:

(a) use jackhammer or pavement breaker attachment on backhoe;

(b) do not damage reinforcing steel;

(c) leave vertical face on concrete at ends of the repair; and

(d) remove any transverse steel in anchorage zone.

(6) Dislodge remaining deteriorated concrete from shoulder and adjacent slab with air hammer.

(7) Remove deteriorated pavement:

(a) expose reinforcing steel on deteriorated portions by chipping away concrete, attach lifting chains at these points, and lift sections from pavement, or drill holes into deteriorated concrete and place key lifting bolts, as shown in Chapter 3, to remove concrete;

(b) place debris in dump truck;

(c) minimize damage to subbase course; and

(d) remove all broken concrete and debris from area.

(8) Repair subbase course as required.

(9) Prepare leveling course:

(a) set up track-screed assembly shown in Chapter 3;

(b) adjust screed to align precast slab with adjacent CRCP surface;

(c) place concrete highly workable grout on surface of subbase; and

(d) strike grout off with transverse and longitudinal movement of screed.

(10) Position precast slab into repair:

(a) attach lifting devices to slab;

(b) lower slab into postion;

(c) keep slab level while lowering into position; and

(d) adjust position while lowering with hands or prying tools.

(11) Anchor steel at end of slab following anchorage strategy.

(12) Place cast-in-place concrete in anchorage zones.

(13) Fill sides around precast slab:

(a) place grout along inside edge of slab and

(b) place bituminous mixture along outside edge of slab between bituminous shoulder, use grout if repair is adjacent to portland cement concrete shoulder or another lane.

FIELD TESTING

The field testing of precast slabs should be designed so that comparisons with present repair techniques and precast repair slab strategies may be made. Comparisons based upon lane closure time, cost, or performance can be made. Well kept records of all costs and periodic condition ratings are the essential information upon which evaluations will be based. Construction of an adequate number of repairs from each experimental population, to provide for proper statistical treatment, will result in greater reliability of conclusions.

For making statistical comparisons and for hypothesis testing purposes, a sample of six observations from each population being evaluated is the smallest allowable sample. The larger the sample taken from a population, the more confidence can be placed on statistical descriptions of that population. For a sample of size less than six, the F and t distributions become increasingly less accurate and sensitive. The sensitivity of these distributions, which are used for comparison and hypothesis testing, increases at a slow rate for samples larger than six (Ref 7). This is for the usual loss of one degree of freedom.

Six samples of each type of repair strategy are desirable for statistical comparison. For a comparison of precast versus cast-in-place repair, 12 repairs would be required, 6 precast and 6 cast-in-place. Each of the six repairs forming a particular population must ideally be identical in design and installation technique with each other. To properly evaluate other factors, such as type of steel anchorage, six more identical repairs with only the particular factor of interest varied would be required. It is not feasible to construct such a large number of experimental repairs. To this end, it is suggested that a basic design be chosen for comparison against cast-in-place repairs and a limited number of other experimental designs also be included into field testing. This is elaborated in the next chapter.

CHAPTER 7. GUIDELINES FOR EXPERIMENTAL INSTALLATIONS IN TEXAS

At the present time in Texas, CRCP in rural areas is experiencing distress requiring repair. It is expected that CRC pavements in urban areas will soon begin to require more repair as these pavements near the end of their design lives. The numerous approaches to repair of CRCP need to be quantitatively evaluated for their suitability. Field testing in the rural areas now, where user costs are generally lower, will provide an opportunity to further develop these methods before repair in high density urban zones is required.

EXPERIMENTAL DESIGN VARIABLES

Ideally, evaluation of repair strategies should be spread over several districts and be designed in advance to assure proper coordination. These strategies need to be tested under a variety of conditions and by a variety of highway personnel. The cost to each district will also be lessened, while a broad data base to evaluate the possible approaches is obtained.

The two basic types of comparisons which can be made from field tests are: current cast-in-place repair versus precast repair and comparison of precast repair strategies against themselves. Comparison against cast-in-place repairs will demonstrate the relative performance of the precast method with respect to the traditional cast-in-place repair method. The comparison of precast repair strategies against each other will test the variety of precast options and aid in optimizing the elements of precast repairs.

Elements of the precast repair technique which may be used as experimental design variables include: casting strategy, steel connection technique, and fast set concrete placed around steel connections. To simplify the factorial combinations of the above experimental variables, only two strategies of each variable should be selected for comparative purposes. The recommended alternate casting strategies for comparison are the standard size slab design and the individually sized slab design. Two steel connection techniques recommended for field testing are welded connections and cable clamp connectors. Polymer concrete (methyl methacrylate based) and a fast setting mixture of portland cement concrete are two options for fast set concrete to be placed around the steel connections. A factorial combination of these alternatives is shown in Table 7.1.

For making statistical comparisons and for hypothesis testing purposes, six samples of each population being evaluated are the least desirable number of samples. The more samples of a population, the more confidence can be placed in statistical descriptions of that population. For sample sizes less than six, the F and t distributions become increasingly less accurate and sensitivity of these distributions, which are used for comparison and hypothesis testing, increases at a slow rate for sample sizes larger than six, for the usual loss of one degree of freedom. (Ref 7).

Six repairs of each alternate repair strategy are desirable for statistical comparisons. The repairs performed representing each technique should be constructed to be as identical to each other as possible. For the complete factorial of combinations shown in Table 7.1, 54 repairs would be required. It may prove infeasible to construct this large a number of experimental repairs. To this end, it is suggested that the primary comparison of cast-in-place repair technique versus precast repair technique be designed with a sample size of six repairs each. A limited number of other alternative precast repairs can be constructed as desired.

The principal comparison of cast-in-place versus precast methodology should be performed with designs which possess the highest potential for success. For the cast-in-place repairs, this translates into the current repair practices of the SDHPT. The precast repair method which appears to have the highest potential for success is the alternative using a prepared grout bed struck off to the proper elevation and welded steel connections surrounded with polymer concrete. The other precast repair designs to be included in initial field tests should be those which improve the economy of the above method. In this way, a basic comparison of methodologies is provided along with a limited study of improvements to the precast technique to determine if further testing is warranted.

TABLE 7.1. FACTORIAL COMBINATION OF ALTERNATE REPAIR STRATEGIES PROPOSED FOR FIELD TESTING

EXPERIMENTAL DESIGN VARIABLES			REPAIR TECHNIQUE	
Casting Strategy	Steel Connection	Concrete in Steel Anchorage Zone	Precast	Cast-in-Place
Cast to size of repair	Weld	Polymer		
		Fast set pcc		
	Clamp	Polymer		
		Fast set pcc		
Standard size	Weld	Polymer		
		Fast set pcc		
	Clamp	Polymer		
		Fast set pcc		

EVALUATION

The purpose of field tests is to evaluate repair strategies. For comparative purposes, evaluations need to be performed so that quantitative statistical comparisons can be performed. The two primary criteria for repairs are that they perform well and be economically feasible. These two criteria form the basis for evaluation.

Performance

The performance of a repair is an overall measure of the changes in its characteristics over a period of time. It is important to note that performance is a function of change over a period of time, whereas condition is a measurement taken at one particular point in time. Thus, measurement of performance is made up of a series of condition measurements. The characteristics which can be used to evaluate performance can be grouped into the following categories: deflection, profile, and physical description.

Deflection measurements give an indication of the structural response of a pavement to wheel loads. The Dynaflect is a commonly employed device for making rapid non-destructive deflection measurements (Ref 36). Measurements at five different locations on and around a repair are recommended. All measurements should be performed in the outer wheel path of the lane to be repaired. Two measurements should be made on a sound portion of pavement within 25 feet (7.62 m) of each end of the repair. The other three measurements should be taken with one at each end of the repair and one in the middle of the repair. Measurements in these approximate locations should be performed prior to the repair and at spaced intervals over several years after the repair. Deflection measurements should be performed at approximately the same time of the year, from year to year, so that seasonal effects on the readings are minimized.

Profile measurements give an indication of the pavement roughness and changes in the surface elevation. The Profilometer is a vehicle designed to make rapid profile measurements. If possible, measurement of the repair area should encompass at least a 200-foot (61 m) length, centered about the repair. Measurements should be taken at the same time as deflection measurements: before and for several years after the repair.

A description of the physical condition of significant features of the repair should also be prepared. This is also referred to as a condition survey. Items such as joint widths, amount of cracking in the repair, surrounding crack pattern, spalling, faulting, pumping, drainage, and deterioration present should be included. In addition to written notes, pictures from the edge of the roadway looking across the pavement and repair, and from the center of the repaired lane looking along the pavement, are very beneficial for this purpose. These condition descriptions should also be performed at spaced intervals over several years, concurrently with deflection and profile measurements.

Economic

Economic evaluation should include all costs associated with construction and include user costs. Construction costs include labor, materials, and transportation. A measurement of user costs is lane closure time. A basic cost rate per minute of lane closure can be used to incorporate lane closure costs into overall costs for comparative purposes. Thus, it is important to keep detailed records of construction costs and lane closure times for each repair performed.

CHAPTER 8. CONCLUSIONS AND RECOMMENDATIONS

The research and information presented in this report provide an introduction to the repair of CRCP with precast slabs. This subject has received a thorough treatment, spanning from suggested alternate strategies, analytical methods, and design procedure to specific recommendations for Texas conditions. The following conclusions and recommendations can be drawn from this study.

CONCLUSIONS

A systematic evaluation of the current maintenance techniques of the SDHPT needs to be performed to further study the repair of CRCP and to improve these techniques.

The primary conclusion which can be drawn from this study is that repair of CRCP with precast slabs appears structurally feasible. Economic feasibility needs to be evaluated through field testing.

The detailed analytical work performed during this study resulted in the following conclusions concerning the behavior and design of precast repair slabs:

(1) A high bending stiffness at the steel anchorage zone does not produce excessive stress concentrations; however, a low bending stiffness should be avoided.

(2) The length of a precast repair slab does not significantly affect wheel load stresses; however, slabs shorter than 6 feet (1.83 m) are more sensitive to design and construction errors due to their slightly higher stress levels.

(3) The most critical location for a void beneath a precast slab is beneath the steel connections. Small voids beneath the interior of a precast slab do not produce detrimental stress concentrations but should be avoided.

(4) Prior to installation, a precast slab which contains a stress controlling bond breaker should not crack due to volume change effects.

(5) Precast repair slabs longer than 7 feet (2.13 m) have potential problems with excessive steel stress at the end of the slab.

These stresses can be controlled with the inclusion of a bond breaker into the slab.

(6) For equal areas of steel reinforcement, smaller reinforcing bars result in higher concrete and steel stresses and smaller concrete movements.

(7) For equal reinforcement bar sizes, greater area of steel reinforcement causes an increase in concrete stress and decreases in steel stress and concrete movement.

(8) Increasing the concrete strength produces increased concrete stress, steel stress, and concrete movement.

(9) A precast slab can be satisfactorily lifted by using four lifting points which should be secured into the slab and situated between 1/5 and 1/4 the span length of each side from the edge.

(10) Slabs with bond breakers situated in the bottom fiber of the concrete should be lifted with 1/4 span placement of lift points to keep this region in compression during lifting.

(11) Welded or cable clamped steel connections are capable of developing the yield strength of grade 60 steel and appear feasible for use at steel connections irrespective of the filler material.

RECOMMENDATIONS

Implementation studies and field experiments as outlined in Chapter 7 should be conducted. Personnel from the D-8, D-10, and D-18 divisions of the SDHPT should coordinate with CFHR personnel to finalize a working experiment.

Further development of the precast repair strategies presented in this report is dependent upon field installations and performance monitoring over time. An effort should be made to optimize components and techniques leading to standardization in the future.

Improvement of the analytical techniques used in this report can be made. An area which needs more research is the bond stress in the steel slip zone in the VCARCS program. Presently, an average bond stress based upon an empirical equation is used in calculations. Additional research into the bond stress, its effect on volume change calculations, and the bond stress of welded wire fabric is needed.

The addition of analyzing slabs with reduced area of concrete in the center in the friction model of VCARCS is another improvement which can be made. In the present version of the program, slabs with reduced concrete areas can be analyzed for only the frictionless condition.

Another improvement to the VCARCS program is to reestimate the length of the bond slip zone after each pass in the friction model. The length of bond slip zone is presently set equal to that in the frictionless model. This appears to be only a marginal improvement. Calculations tend to show a very slight effect of the current estimation technique.

The use of finite element programs to analyze particular elements of precast repair slabs could be introduced to expand the analytical techniques used in this report.

The development of partial-depth and/or partial-lane-width precast repair slabs should be researched. This report was restricted to full-depth, full-width precast repair slabs. Development of precast methods to make smaller isolated repairs appears warranted. The use of polymer concrete in these types of repairs should also be investigated.

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APPENDIX 1a

SURVEY OF CURRENT PRACTICES FOR MAINTENANCE OF CRCP IN TEXAS

APPENDIX 1a. SURVEY OF CURRENT PRACTICES FOR MAINTENANCE OF CRCP IN TEXAS

Logically, the first step in any attempt to improve current maintenance practices or development of new repair techniques requires review of current practices. Maintenance of pavements in Texas is performed by the 25 regional districts of the State Department of Highways and Public Transportation (SDHPT), and these maintenance practices vary from district to district. In order to tap the wealth of information and experience on repair of continuously reinforced concrete pavement (CRCP), a survey of 13 of the 16 SDHPT districts which have significant lengths of CRCP was conducted. The districts surveyed include 1, 2, 3, 4, 9, 10, 12, 13, 15, 17, 18, 20, and 25. The other districts in Texas with CRCP, which were either not surveyed or whose survey is not reported, are 6, 19, and 24.

The survey was performed by interviews with district maintenance personnel conducted by personnel from the Center for Highway Research at The University of Texas at Austin and personnel from the Design Division of the SDHPT and by completion of the prepared survey form by some of the district maintenance engineers. The survey method was not carried out under strictly uniform conditions. The interviewing team as well as the data collection method, varied. Therefore, differences in interpretation of questions and terms used in the survey may exist. However, the categorical breakdown is in generally accepted terminology and questions are specific so that misinterpretation is held to a minimum.

A copy of the survey form is provided in Appendix 1b. The items of information covered in the survey are warrant for repair of a punchout related distress, repair techniques for a punchout related distress, repair techniques of spalling, correction methods of pumping, and a miscellaneous category to include other types of distress maintenance personnel have had to deal with other than those covered above.

The information collected from this survey is first reported in this appendix as a general summary of distresses being repaired and the general methods being employed. This is followed by a series of tables giving a detailed summary of the responses to the particular items covered in the questionnaire. The tables are then followed by a few closing comments on the survey.

GENERAL SUMMARY

The following generalizations are derived from the completed survey forms. These generalizations are felt to be representative of the current CRCP maintenance practices in Texas. They also serve as an introduction to the detailed listing of responses to the survey. This section has been divided into the types of distresses being repaired in Texas and the techniques used in these repairs.

Repairs Being Made on CRCP in Texas

Punchouts. Most districts report repair of a punchout related distress when it reaches the moving block stage. This is perhaps the most common repair performed on CRCP.

Spalling. Three of the thirteen districts surveyed reported no problems with spalling. Seven of the districts reported repairing severe spalling while the remaining three districts reported repairing both minor and severe.

Pumping. Three districts reported they did not correct pumping either because the extent of pumping was very minor or because they questioned the value of such a correction. Ten districts reported they did correct severe pumping, two of them correcting it in association with other failures (Punchouts).

Miscellaneous. Other pavement distresses that were reported to be repaired or corrected are heaving (distortion), buckling, and problems with approach slabs to bridges. These distresses are isolated cases and are not universally experienced by all districts. Perhaps the most common repair of these types is the repair of a distortion with a level up.

Repair Techniques for a Punchout Related Distress

Methods used to repair punchouts are usually categorized as temporary or permanent. Temporary measures are used as stop-gap techniques to promptly return service to a deteriorated portion of pavement. Permanent repairs generally require more time and care and are intended to remain as part of the pavement.

Temporary Repair

The temporary repair method generally consists of removing the deteriorated concrete in the immediate area of the punchout. The steel is either completely removed, left in place, or cut and removed so that adequate length is left for splicing new reinforcement when a permanent type repair is made. Usually, the base course is just cleaned as the deteriorated concrete is removed (left in existing condition). Hot mixed cold-laid (HMCL) asphaltic mixtures are used the most for this type repair. These materials are placed in the hole and compacted. Some districts add portland cement to their asphaltic mixtures. Other materials reported in use are hot-mixed hot-laid asphaltic concrete and some fast setting concretes, such as "Fast Fix," "Custom Crete," "Dura Cal," etc.

Permanent Repair

The majority of districts surveyed indicated that they felt that a repair made with PCC was a permanent repair. Two districts reported problems with PCC repairs and are using asphaltic materials. Excavation of concrete usually extends, at the most, 1 to 3 feet past the limits of the unsound concrete. Rectangular patches are used almost universally; only two districts reported using skewed patches. Either the steel is left in place or it is removed and new steel is added. On smaller repairs the steel was generally reported as being left in place. On larger repairs, the steel was removed with the concrete to expedite concrete removal, and then steel was added and spliced (tie or weld) to the existing pavement reinforcement. The base course was reported prepared simply by excavating to firm material. The hole was then filled with PCC and cured. Type I PC was reported in use by the majority of districts and Type III PC by some. Air entrained concrete was not specified, but when available it was generally preferred. Curing varied greatly depending on material and district practice.

Repair Techniques of Spalling

Spalling is repaired by first cleaning the spalled area, removing all pieces of loose concrete. Generally HMCL asphaltic mixture is placed in in the hole and compacted. Epoxy has also been used with satisfactory results. Most districts reported that they were eager to try some polymer materials for repair of this distress type.

Correction of Pumping

Correction of pumping has been attacked in three ways: Sealing the longitudinal joint at the edge of the pavement is common in some districts. Mudjacking underneath the subbase layer or between pavement and subbase was used by some. Also, longitudinal drains have been installed to help drain water from beneath the pavement. The performances of these procedures were reportedly mixed, varying from very good to poor.

Miscellaneous

The pavement distress types included in this category are more the exception than the rule. Distortion correction is probably the most common maintenance procedure apart from those previously mentioned. Distortion is corrected with a thin level-up course of an asphaltic mixture.

DETAILED SUMMARY OF RESPONSES

Eleven tables detailing the responses collected in the survey were prepared from the individual questionnaires. These tables summarize the responses the districts surveyed gave to particular questions. A reference to the question in the questionnaire is provided in the title of each table. The order of the tables follows the order of questions on the questionnaire.

SUMMARY

It is obvious from the survey that a wide variation in repair techniques exists in Texas. Some of this variation is due to regional differences in climate, soils, construction techniques, materials used in CRCP, and availability of maintenance materials. What may work well in one district on a particular project may not be feasible in another district or even on another project within the same district. The repair of this pavement type is complicated and success of a repair is controlled by a large number of unquantifiable variables which are constantly interacting. Perhaps no universally optimum repair technique exists. It is, however, obvious that improvements and innovations over current techniques can be achieved through further study of this topic.

TABLE A1.1. WARRANT FOR REPAIR OF PUNCHOUT RELATED DISTRESS - Questions A. 1 and 2.

District	Level of Distress Warranting Repair	Comments - Other factors
1	Moving block	Movement associated with pumping
2	Moving block	Movement associated with pumping
3	Moving block	
4	Moving block	Signs of pumping, localized cracking
9	Moving block	Localized cracking and initiation of fracture
10	Initial stage	Repair all failures while barracades are in place
12	Moving block	Ride quality
13	Moving block	
15	Moving block	Excessive movement
17	Moving block	Deterioration is so extensive it is impossible to repair initial stages. Primarily areas in wheel path
18	Moving block	Transverse cracking (distress type) and pumping
20	Moving block	
25	Initial stage	Rideability if possible

TABLE A1.2. REPAIR OF PUNCHOUT RELATED DISTRESS - Questions B. 1 and 2; LIMITS OF CONCRETE EXCAVATION

District	Method to Determine Limits*	Amount of Excavation	Comments
1	Visual	0 to 2 in. past limits	Excavate till solid concrete is encountered
2	Visual	>12 in. past limits	3 to 4 feet into sound concrete
3	Visual	2 to 6 in. past limits	
4	Visual and Tapping	2 to 6 in. past limits	
9	Visual and Tapping	>12 in. past limits	Usually a lane width patch a minimum of 6 feet in length
10	Visual	>12 in. past limits	
12	Visual and Tapping	6 to 12 in. past limits	Between cracks
13	Visual	2 to 6 in. past limits	Halfway to next crack, stable piece of pavement
15	Visual	0 to 2 in. past limits	
17	Visual	2 to 6 in. past limits	
18	Visual	To the limits	Not many problems with poor concrete
20	Visual	>12 in. past limits	Locate area with sound concrete
25	Visual and Tapping	0 to 2 in. past limits	

* "limits" refers to limits between unsound and sound concrete

1 inch = 25.4 mm
1 foot = .305 m

TABLE A1.3. REPAIR OF PUNCHOUT RELATED DISTRESS - Question B.3; EXCAVATION OF CONCRETE

District	Shape of Excavation	Equipment used in Excavation
1	Rectangle	Jack hammer
2	Skew	Jack hammer, diamond saw
3	Rectangle, skew	Jack hammer, diamond saw
4	Rectangle	Jack hammer
9	Rectangle	Diamond saw, machine operated-hydraulic hammer
10	Rectangle, skew	Jack hammer, diamond saw
12	Rectangle	Jack hammer, diamond .
13	Rectangle	Jack hammer, mini-hoe ram
15	Rectangle	Jack hammer
17	Rectangle	Jack hammer, jack-hoe with air hammer attachment
18	Skew	Jack hammer, diamond saw, hop-tow with pavement breaker attachment
20	Rectangle	Jack hammer, dropping weight
25	Rectangle	Jack hammer, diamond saw

TABLE A1.4. REPAIR OF PUNCHOUT RELATED DISTRESS - QUESTION B.4
BASE COURSE PREPARATION

District	Base Coarse Preparation	Backfill	Comments on Base Coarse Preparation
1	Excavate to firm material	Cement-Stabilized base material	Fill repair with stabilized material to within about 2 to 3 in. of concrete surface. Tamp with mechanical tamper.
2	Excavate to firm material	Base material (black base when available)	Backfill with previous base material; stabilized base or black base. Tamp with mechanical or hand tamp.
3	Excavate to firm material	Stabilized base material	
4	Excavate to firm material	Base material	Mechanical tamp
9	Excavate to firm material	Portland cement concrete (PCC)	
10	Excavate to firm material	Rock asphalt, PCC	
12	Excavate to firm material	Base material	Hand tamp base material problems with saturated base.
13	Excavate to firm material	PCC	Wet base material before backfill.
15	Left in existing condition		
17	Excavate to firm material	PCC	
18	Excavate to firm material	PCC	
20	Left in existing condition	Base material	Backfill with stabilized base material. Hand tamp.
25	Left in existing condition		

1 inch = 25.4 mm

TABLE A1.5. REPAIR OF PUNCHOUT RELATED DISTRESS - QUESTION B.5; PREPARATION OF REINFORCING STEEL

District	Steel Preparation
1	Completely remove steel
2	Use existing steel; on larger areas, steel is removed sometimes; tie splice minimum, 20 diameters.
3	Replace steel; splice length, 6 to 12 inches; weld and tie.
4	Use existing steel if in good condition; if in bad condition, weld new steel at one end and tie steel at the other.
9	Use existing steel.
10	Replace steel; splice length, 18 inches.
12	Replace steel; 6-inch weld length.
13	Use existing steel in smaller holes; for larger areas, replace steel; splice length, 36 in., 20 diameter
15	Use existing steel.
17	Use existing steel on small patches; on larger areas replace steel; 20 diameters splice length.
18	Replace steel; splice length, 12-in. minimum, at least 20 diameters.
20	Replace steel; splice length, 12 inches.
25	Use existing steel if possible; when replaced, use 6-inch splice length.

1 inch = 25.4 mm

TABLE A1.6. REPAIR OF PUNCHOUT RELATED DISTRESS - QUESTION B.6, ASPHALTIC MATERIAL USED AS TEMPORARY REPAIR

District	Temporary Material	Replacement Material	Performance Evaluation	Comments
1				
2	HMHL-AC* HMCL	PCC		
3	HMHL-AC	PCC	Satisfactory	
4	HMHL-AC HMCL	PCC	Poor	No site preparation, remove loose material and fill, asphaltic materials used only as temporary repair.
9	HMHC-AC HMCL CM	PCC	Very Good	Asphaltic material used only as temporary patch, it is economical and little inconvenience to the travelling public
10	CM	PCC	Satisfactory	Saves time
12	HMHL CM**	PCC	Satisfactory	Remove all steel, leave 3 to 5 inches for welding steel for permanent patch.
13	HMCL CM**	PCC	Satisfactory	Asphaltic material is used as temporary patch, restores service quickly, delays progressive failure. Notice that pavement tends to fail adjacent to patch. Had experience with fast setting concrete such as "Fast Fix," "Speed Crete," "Duracal," etc., these tend to last about 1 year then break up.
15				
17	HMCL	PCC	Satisfactory	Used as temporary repair, provide service till permanent repair can be performed.
18	HMHL-AC* HMCL	PCC	Satisfactory	Asphaltic material used for temporary repair only.
20	CM	PCC	Good	Experience with quick setting concretes, only last a few years at the best.
25	HMCL	HMCL	Poor	Keep adding mix as patch consolidates and shoves, this is also the pavement repair.

*when available

**with cement added

1 inch = 25.4 mm

TABLE A1.7. REPAIR OF PUNCHOUT RELATED DISTRESS - QUESTION B7; ASPHALTIC REPAIR MATERIAL USED FOR PERMANENT REPAIR

District	Material	Specifications	Compaction	Cure	Performance Evaluation	Comments
1	HMHL-AC* HMCL	Type Class A, 18:1 pcc added	Mechanical Tamp	<1 hr	Very Good	Have not had to re-repair any of these type repairs. Some patches have side drains installed.
2						
3	HMHL-AC	Type D	Mechanical Tamp	<1 hr	Poor	No longer considered permanent, completely remove steel, cooled with water before opened to traffic.
4	--	--	--	--	--	--
9	HMHL-AC	Types B and D	Hand tamp	<1 hr	Satisfactory	
10	CM	Type AA, mod. rock asphalt	Mechanical tamp	<1 hr	Good	
12						
13	HMHL-AC* CM	Type A, rock asphalt	Hand tamp	<1 hr	Satisfactory, small patches only	Temporary patches which last a long time, used with and without steel removed.
15	HMHL-AC* HMCL	Type D, water and primer left out	Vibratory Hand tamp	<1 hr	Good	Vibratory compaction below the steel, hard to get density, use coarser graded material in lower portions.
17	--	--	--	--	--	--
18	--	--	--	--	--	--
20	--	--	--	--	--	--
25	HMCL	Type D	Hand tamp	<1 hr	Poor	Temporary patches which become permanent.

*When available

TABLE A1.8. REPAIR OF PUNCHOUT RELATED DISTRESS - QUESTION B.8: PORTLAND CEMENT CONCRETE USED AS A PERMANENT REPAIR MATERIAL

District	Type of Portland Cement	Specifications	Compaction	Type of Curing	Curing Time	Performance Evaluation	Comments
1	I III	Low slump	Hand tamp	No special curing proced.	12 to 24 hr	Very poor	Use existing steel; problems with patch moving then breaking up.
2	I III*	∿4-in. slump, $f_r > 500$ psi	Internal vibration	Water proof membrane curing pound	72 hours	Very good	Existing concrete faces are coated with epoxy sometimes.
3	I*	$f_r > 500$ psi	Hand tamp	Curing compound	72 hours	Very good	
4	I	$f_c > 6{,}000$ psi	External vibration	Curing compound	1 hour	Good	Use "quick-set" concrete on expansive material; make sure side cuts are vertical.
9	I*	Class C	Internal vibration		1 to 6 hr	Very good	Cal-Set added for quick set.
10	I	Class A, 5 sk/cy	Hand tamp	Wet coverings	72 to 96 hr	Good	
12	I III*	3-in. slump, class C, 6 sk/cy	Internal vibration	Wet coverings, curing compound	1 to 6 hours	Very good	Use steel plate to cover small patches while curing.
13	I II	3-4 in. slump, classes A & C, 6sk/cy, $f_r > 500$ psi	Internal vibration	Wet coverings	48 to 72 hrs	Good	
15							
17	I	2-2 1/2 in. slump, class A	Internal vibration, hand tamp	Curing compound	72 hours	Good	Should use 6-7 sk/cy concrete and low water admixture; consideration should be given to providing drainage under trench type CRCP.
18	III	2-3 in. slump, class A	Internal vibration	Curing compound	48 to 72 hr	Very good	When repairs are performed pumping problems should be corrected.
20	III	2-2 1/2 in. slump, class F, 7 sk/cy	Internal vibration	Curing compound	1 to 6 hr	Good	Make mix as dry as possible.
25	I		Hand tamp	Curing compound	1 week	Poor	Do not use PCC patching any more.

* Air entraining admixture

1 psi = 6.89 KPa

1 inch = 2.54 mm

1 sk/cy = 1.31 sk/m^3

TABLE A1.9. REPAIR OF SPALLING - Question C.1-5.

District	Type Spalling Required	Material Used	Performance Evaluation	Repair Method
1	NA			
2	NA			
3	Severe	HMCL type FF	Satisfactory	Cut vertical edges for bonding
4	Severe	PCC if large enough HMCL	Very poor	Cut at least 2 to 4 inches deep.
9	Minor and severe	Epoxy	Satisfactory	Clean crack, apply sand and epoxy.
10	Minor	HMCL	Good	Hand tamp material in spall.
12	Severe	HMCL	Poor	
13	Severe	PCC (fast set)	Good	
15	Minor severe	PCC (fast set) epoxy	Good	Clean spall and place material.
17	Severe	HMCL	Good	Clean spall, tack sides, place material, and hand tamp.
18	Severe	HMCL	Very good	Place material, compact
20				
25	Severe	PCC set)	Poor	

NA - No problems with spalling or not repaired.

1 inch = 25.4 mm

TABLE A1.10. CORRECTION OF PUMPING - Question D.1-5.

District	Type of Pumping Corrected	Correction Method	Performance Evaluation	Experience with Longitudinal Drains
1	Severe	Joint sealer longitudinal drains	Poor, very good	Yes
2	Severe (associated with failures)	Mudjacking, under drains, joint sealer	Mixed, very good	Yes
3	Severe	Under drains, joint sealer	Very good	Yes
4	Severe	Corrected with repair of punchout		No
9	Severe	Joint sealer	Poor	No
10	Severe	Mudjacking, under drains	Satisfactory	No
12	Severe	Mudjacking	Good	No
13	Severe	Mudjacking, under drains, joint sealer	Satisfactory, satisfactory, good	Yes
15	Severe	Corrected with repair of punchout		No
17	NA			
18	Severe	Underdrains, joint sealer	Very poor	Yes
20	NA			No
25	NA			No

NA - No problems with pumping or not corrected.

TABLE A1.11. MISCELLANEOUS PROBLEMS WITH CRCP

District	Miscellaneous Problems with CRCP
1	Bad concrete, honey combing under steel, asphaltic concrete level up to correct distortion.
2	Problems on approach slabs to bridges.
3	Faulting between slab and shoulder.
4	Distortion (corrected with level up), problems with approach slabs to bridge.
9	Distortion (corrected with level up).
10	Distortion (corrected with level up).
12	Same blow-ups on approach slabs.
13	Buckling, construction joint failures.
15	Distortion (corrected with level up).
17	-
18	Low friction on curves.
20	Blow-up of a patch poured earlier in the day.
25	-

APPENDIX 1b.

QUESTIONNAIRE ON REHABILITATION OF CRCP

APPENDIX 1b. QUESTIONNAIRE ON REHABILITATION OF CRCP

The following is a copy of the questionnaire used in the maintenance survey. A summary of results of this survey is shown in Appendix 1a. The questions are designated by topic letter heading and alphanumerical subheadings.

QUESTIONNAIRE ON REHABILITATION OF CRCP*

A. Warrant for repair of punchout related distress.

1. What level of distress in the development of a punch out do you repair?

Minor Punch Out	Severe Punch Out		
	Initial Stage	Moving Block	Separated Block

2. What other criteria are used to determine if a repair of this type of distress is warranted?

B. Repair of punch out related distresses

1. What method, if any, is used to determine the limits of sound and unsound concrete?

2. How much excavation of existing concrete is performed?

To the limits of unsound concrete ☐

Past the limits of unsound concrete ☐

How much past the limit of unsound concrete?

0 - 2 in. ☐

2 - 6 in. ☐

6 - 12 in. ☐

> 12 in. ☐

Please comment on amount of excavation.

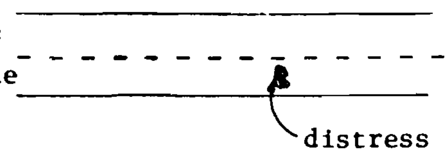
*Use reverse side for additional space in answering questions

3. If excavation is performed

a. What shape would it have? Please Sketch

Inside Lane

Outside Lane



b. What tools are used to remove existing concrete?

Jack Hammer ☐

Diamond Saw ☐

Dropping Weight ☐

Other (Please specify):

4. How is the base coarse prepared?

Left in existing condition			
Excavated to firm material			
Dry Material			
Tamp existing material		Mechanical tamper	Hand tamp

Backfill with		
Base Material		
Asphalt Material		Type:
Portland Cement Concrete		

other (specify):

Please comment on base coarse preparation:

5. What is done with the reinforcing steel?

Use existing steel	☐
Replace steel	☐

splice length ________ in.

weld length ________ in.

other type of connection (specify):

6. If asphaltic material is used as a temporary patch:

a. What type is used?

Hot-mixed Hot-laid Asphaltic Concrete	
Hot-mixed Cold-laid mix	
Cold Mix	

other (specify)

b. What material will be used as a permanent repair replacing this material?

Hot-Mixed Hot-Laid Asphaltic Concrete	
Portland Cement Concrete	

other (specify)

c. What are the differences between this procedure and a permanent repair procedure in terms of strength specifications, compaction, and curing?

d. How well do you think this method performs?

Very good	
Good	
Satisfactory	
Poor	
Very Poor	

e. Please comment on what you feel are some significant aspects of this procedure?

6. f. Do you have any ideas or suggestions on changes in current techniques or other possible methods of temporary repair?

7. If asphaltic repair material is used

a. What type is used?

Hot-Mixed Hot-Laid Asphaltic Concrete	
Hot-Mixed Cold-Laid Mix	
Cold Mix	

Other (specify):

b. What class of material is used? (Specify test and level)

c. How is the material compacted?

Vibratory	
Hand tamp	
Mechanical tamper	
Steel roller	
Pneumatic roller	
Not compacted	

Other (specify):

d. How long between repair to the application of traffic?

< 1 hr	☐
1 - 2 hr	☐
2 - 6 hr	☐
6 - 12 hr	☐
> 12 hr. (specify)	

e. How well do you think this method performs?

Very good	☐
Good	☐
Satisfactory	☐
Poor	☐
Very poor	☐

f. Do you have any ideas or suggestions on changes in current methods or other methods of repair using asphaltic materials?

8. If portland cement concrete is used

a. What type of portland cement is used?

	Air Entrained	Non-Air Entrained	Air Entraining Admixture
Type I			
Type II			
Type III			
Type IV			
Type V			

other (specify, include admixtures):

b. Slump Specification, if used ________in.

c. What class of material is used? (Specify test and level)

d. How is this material compacted?

- Internal vibration ☐
- External vibration ☐
- Hand tamp ☐
- Not compacted ☐
- other (specify)

e. What type of curing is used?

- Ponding ☐
- Wet coverings ☐
- Water proof membranes ☐
- Curing Compounds ☐ (Specify under other)
- other (specify)

f. How long this the concrete cured before application of traffic?

- < 1 hr. ☐
- 1 - 6 hrs. ☐
- 6 - 12 hrs. ☐
- 12 - 24 hrs. ☐
- 24 - 48 hrs. ☐
- 48 - 72 hrs. ☐
- > 72 hrs. (specify)

g. How well do you feel this method performs?

- Very good ☐
- Good ☐
- Satisfactory ☐
- Poor ☐
- Very poor ☐

h. Do you have any ideas or suggestions on changes in current methods or other repair methods using portland cement concrete? Use reverse side, if necessary.

9. What are your ideas on other repair methods not covered above?

C. If spalling is repaired

1. What type of spalling is repaired?

Minor	
Severe	

other (specify):

2. What type of material is used to repair spalling?

Portland Cement Concrete		
Asphaltic Materials		type
Epoxy		

other (specify)

3. How is the repair performed?

4. How well do you feel this method performs?

Very good	
Good	
Satisfactory	
Poor	
Very poor	

5. Do you have any ideas or suggestions on this repair procedure or other possible procedures?

D. If pumping is corrected

1. What type is corrected?

Minor	☐
Severe	☐
other (specify)	

2. What method is used to correct pumping?

Mudjacking	☐
Underdrains	☐
Longitudinal joint sealer	☐
other (specify)	

3. How well do you think this method of repair performs?

Very good	☐
Good	☐
Satisfactory	☐
Poor	☐
Very poor	☐

4. If you have had experience with longitudinal drains, what observations have you made on their construction and performance?

5. What are your ideas or suggestions on this repair procedure or other possible procedures?

E. If other types of distress are repaired

1. What type of other distresses are repaired?

2. What criteria do you use to determine the warrant for repair?

3. How are these distresses repaired?

APPENDIX 2

ANALYSIS OF WHEEL LOAD STRESSES ON STEEL CONNECTIONS

APPENDIX 2. ANALYSIS OF WHEEL LOAD STRESS ON STEEL CONNECTIONS

Introduction

To investigate possible destructive stress concentrations resulting from the placement of a precast slab in a continuously reinforced concrete pavement, an analysis was performed using the computer program SLAB49. The factors which are thought to influence the distribution of stresses that were investigated are

(1) increased stiffness at the locations where the precast slab is connected to the surrounding pavement,

(2) a decrease in bending stiffness of connection locations accompanied with a constant mesh twisting stiffness, and

(3) length of precast slab.

Table A2.1 presents the factorial breakdown of the factors mentioned above and indicates the combination of factors which were investigated. Further explanations of the values selected in Table A2.1 will be discussed in subsequent sections of this report.

The following assumed material properties were used throughout this analysis:

E_{conc} = Modulus of elasticity for concrete = 4.0×10^6 psi (2.76×10^7 kPa)

M = Poisson's ratio for concrete = 0.20

K_{sub} = Modulus of subgrade reaction = 150 pci (4.15×10^6 kg/m^3)

These material properties were selected because they are felt to be reasonably representative of concrete used in pavement construction and subgrade support. These properties will remain fixed throughout this analysis. The magnitude of stresses and deflections will vary with these properties. However, the indication of destructive stress concentrations will appear as the magnitude of change in stresses resulting from the inclusion of the factors

TABLE A2.1. FACTORIAL OF COMBINATIONS OF VARIABLES INVESTIGATED

Modulus Of Elasticity At Splice Locations, psi	Bending Stiffness At Splice Locations, percent	Slab Length, feet: 2	4	6	10
4×10^6	100%			X	
	60% Reduction			X	
	90% Reduction			X	
10×10^6	100%	X	X	X	X
	24% Reduction			X	
	60% Reduction			X	
	90% Reduction			X	
17×10^6	100%			X	
	60% Reduction				
	90% Reduction				

being investigated. Thus, the selection of these basic properties is not expected to exert significant influence on the results of this investigation.

SLAB49 Model

The SLAB49 program is a discrete-element method based upon representation of a slab by bars, springs, and torsion bars, grouped into a system of orthogonal beams for solution of discontinuous plates and slabs. This program was developed under Research Project 3-5-63-56 at the Center for Highway Research at The University of Texas; a brief description of these reports can be found in Ref 34.

The following geometric properties were input into the SLAB49 program to model a precast repair patch in a CRCP

Total length of pavement = 42 feet (12.8 m),
Width of pavement = 24 feet (7.32 m),
Width of precast slab = 12 feet (3.66 m),
Thickness of pavement and precast slab = 8 inches (203 mm), and
Increment length in X and Y directions = 1 foot (.3048 m).

These properties are meant to represent a two-lane section of CRCP with a precast patch occupying the total width of one lane. The length of the section was selected to approximate the continuity effect of CRCP.

The coordinate system used in this analysis has the X-coordinate in the transverse direction and the Y-coordinate in the longitudinal direction of the pavement section.

For most pavement applications of the SLAB49 program, three externally computed quantities are generally required. These quantities are the slab bending stiffness D_x or D_y, mesh twisting stiffness C^t, and foundation support S. The formulas for these quantities can be found in Ref 35.

The calculated inputs mentioned above must also be distributed over the coordinate system in a manner which approximates the system being modeled. For pavement problems, it is convenient to distribute the bending stiffness and foundation support over the entire slab in four passes, using quarter values of the calculated total magnitude of these quantities. This procedure facilitates 1/2 and 3/4 reduction of the stiffness and support for edge and corner conditions, respectively. Discontinuities in bending stiffness or support are then produced by specifying positive or negative quantities at specific locations. The property of a particular node becomes the summation of quantities specified for that node.

The mesh twisting stiffness has a different geometric interpretation than bending stiffness or foundation support. Bending stiffness and foundation support are specified at nodal points and are interpreted as representing the properties of the area centered about that nodal point. The mesh twisting stiffness is interpreted as representing the area encompassed between four adjacent nodal points. Thus, no reductions are made for edge and corner effects. Therefore, the mesh twisting stiffness is distributed over the slab in one pass at the total calculated value. Discontinuities are then modeled by specifying additional quantities at specific nodes such that the summation of these quantities results in the value of stiffness which is desired to represent the discontinuity being modeled.

The use of reinforcing steel in CRCP to anchor a precast repair patch to the surrounding pavement will require the use of a connection mechanism, such as a welded joint or clamp, which will produce a discontinuity in this location. The term "splice location" will be used hereafter to indicate the section of pavement where the connecting mechanism attaches the precast slab to the surrounding pavement. This connection is expected to increase the stiffness of the splice location.

To model the bending stiffness at the splice locations, additional stiffness was input at these sections. The value of bending stiffness at these locations was selected on the basis of an assumed elastic modulus.

An example of the first two pages of output from SLAB49 is presented in Figs A2.1 and A2.2. These pages summarize the parameters input into SLAB49 to model the system under consideration. The output shown is the model of a precast slab in a section of CRCP with a bending stiffness at the splice location corresponding to a modulus of elasticity of 10×10^6 psi (6.89×10^7 kPa). Table 1 in Fig A2.1 is control data which instructs the program how to process the data.

PROGRAM SLAB 49 -DEVELOPMENT DECK- MATLOCK,PANAK, ENDRES REV DATE 13 JUL 71

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* * * * * * * * * * * * * * * * * * * * * * * * * * * * *
*                                                       *
*   THIS PROGRAM IS BEING USED AT YOUR OWN RISK.        *
*   CHANGES MAY OCCUR AFTER THE ABOVE REVISION DATE.    *
*   PLEASE REPORT DIFFICULTIES TO THE ABOVE PEOPLE      *
*   AT THE CENTER FOR HIGHWAY RESEARCH, UT AT AUSTIN.   *
*                                                       *
* * * * * * * * * * * * * * * * * * * * * * * * * * * * *
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PRE-CAST SLAB REPAIR METHOD - INVESTIGATION OF STRESS CONCENTRATIONS-11/12/76
12+6FT PRECAST SLAB IN 24+42FT PAVEMENT-E(SPLICE)=10E+06SL(SPLICE)=1FT ELKINS

PROB
177.0 2-9KIP LOADS 6FT APART AT SPLICE,1FT FROM EDGE

TABLE 1. CONTROL DATA

	TABLE NUMBER 2	3	4	5	6	7	8	9
KEEP FROM PRECEDING PROBLEM (1=YES)	-0	-0	-0	-0	-0	-0	-0	-0
NUM CARDS INPUT THIS PROBLEM	1	10	0	5	0	2	5	1
MULTIPLE LOAD OPTION	1							
STATICS CHECK OPTION	0							
PRIN STRESS OPTION	1							
PROFILE PLOT OPTION	0							
3-D PLOT OPTION	0							

TABLE 2. CONSTANTS

NUMBER OF INCREMENTS IN X DIRECTION	24
NUMBER OF INCREMENTS IN Y DIRECTION	42
INCREMENT LENGTH IN X DIRECTION	1.200E+01
INCREMENT LENGTH IN Y DIRECTION	1.200E+01
POISSONS RATIO	2.000E-01
SLAB THICKNESS	8.000E+00

TABLE 3. JOINT STIFFNESS AND LOAD DATA

FROM JOINT		THRU JOINT		DX	DY	FX	FY	Q	S
0	0	24	42	4.445E+07	4.445E+07	-0	-0	-0	5.400E+03
0	1	24	41	4.445E+07	4.445E+07	-0	-0	-0	5.400E+03
1	0	23	42	4.445E+07	4.445E+07	-0	-0	-0	5.400E+03
1	1	23	41	4.445E+07	4.445E+07	-0	-0	-0	5.400E+03
12	18	24	18	-0	1.333E+08	-0	-0	-0	-0
13	18	23	18	-0	1.333E+08	-0	-0	-0	-0
12	24	24	24	-0	1.333E+08	-0	-0	-0	-0
13	24	23	24	-0	1.333E+08	-0	-0	-0	-0
12	18	12	24	-8.001E+07	-0	-0	-0	-0	-0
12	19	12	23	-8.001E+07	-0	-0	-0	-0	-0

Fig A2.1. SLAB49 modeling technique for precast repair slab.

TABLE 4. JOINT STIFFNESS AND LOAD DATA CONTD

FROM JOINT	THRU JOINT	RX	RY	TX	TY

NONE

TABLE 5. MESH STIFFNESS DATA

FROM MESH		THRU MESH		C
1	1	24	42	1.422E+08
13	18	24	18	1.067E+08
13	19	24	19	1.067E+08
13	24	24	24	1.067E+08
13	25	24	25	1.067E+08

TABLE 6. BAR STIFFNESS DATA

FROM BAR	THRU BAR	PX	PY	PBX	PBY

NONE

TABLE 7. MULTIPLE LOAD DATA

FROM JOINT		THRU JOINT		QM
23	18	23	18	-9.000E+03
17	18	17	18	-9.000E+03

TABLE 8. PROFILE OUTPUT AREAS

FROM JOINT		THRU JOINT		DEFL (1=YES)	X MOMENT (1=SLAB,2=BEAM)	Y MOMENT	PRIN MOM OR STRESS (1=YES)
13	12	13	30	1	1	1	1
17	12	17	30	1	1	1	1
23	12	23	30	1	1	1	1
5	18	24	18	1	1	1	1
5	19	24	19	1	1	1	1

TABLE 9. PRINTED OUTPUT LIMITS

FROM Y STA	THRU Y STA
10	30

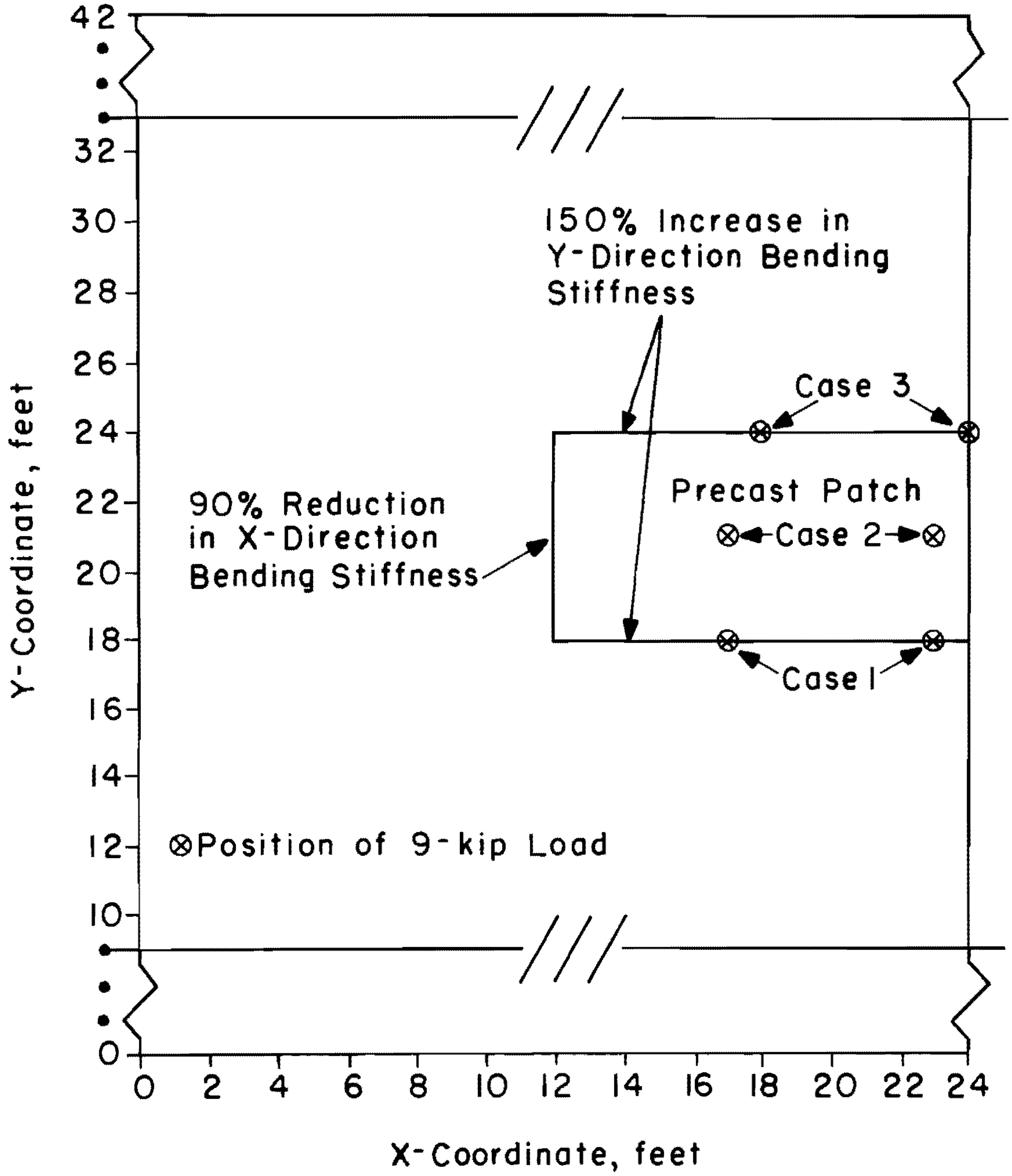
Fig A2.2. SLAB49 modeling technique for precast repair slab.

Table 2 is a printout of constants used in modeling. Table 3 in Fig A2.1. is a listing of joint stiffness and spring support of the modeled slab. The first four lines of Table 3 distribute the bending stiffness and spring support over the slab. The next four lines distribute the increased bending stiffness in the longitudinal direction at the two splice locations. The last two lines in Table 3 decrease the bending stiffness in the transverse direction along the inside longitudinal edge of the precast slab. Table 4 in Fig A2.2 is a continuation of joint stiffness and load data. Quantities in this table are not required for modeling a pavement slab. Table 5 is a listing of the mesh twisting stiffness distribution. The first line in Table 5 distributes the mesh twisting stiffness on an 8-inch (203-mm) thick concrete slab with no discontinuities and a modulus of elasticity of 4×10^6 psi (2.76×10^7 kPa) over the area of the pavement. The increased stiffness at splice locations is distributed at the splice locations by the next four lines in Table 5. Table 6 is not used in modeling a pavement slab. Table 7 in Fig A2.2 is the positioning of the two 9,000-pound (40-kN) loads which represent an 18-kip (80-kN) axle located at one splice location, one foot from the edge of the pavement. A schematic illustration of the system modeled by Figs A2.1 and A2.2 is shown in Fig A2.3.

Critical Position of Load

Three load positions were selected to study the effect of load position on the modeled precast slab. These three positions are shown in Fig A2.3 as Cases 1, 2 and 3. These positions are located near the edge of the slab where deflections and stresses are expected to be the greatest because of the reduced support. Since stress concentrations are the subject of this inquiry, the critical position of load is defined here as that position which produces the largest stress in the modeled slab.

The slab model utilized in the determination of the critical position of load consists of a 6-foot-long precast slab with bending stiffness at the splice locations corresponding to a modulus of 10×10^6 psi (6.89×10^7 kPa). Two 9-kip (40-kN) loads transversely postiioned 6 feet (1.83M) apart were chosen to approximate the wheel loads of an 18-kip (80-kN) single axle truck.



1 ft = .3048 m

Fig A2.3. Loading positions and model example.

Graphs are subsequently presented showing plots of moments in the X and Y directions, principal stress, and deflections against slab position for the three cases shown in Fig A2.3.

A plot of moments in the Y-direction (longitudinal) versus transverse position along the line of load for the three cases is shown in Fig A2.4. The shapes of these curves are similar. The maximum moment occurs under the outside load for Case 1 and is -3700 in.-lb. (-418 N-m).

Figure A2.5 shows moments in the X-direction (transverse) versus transverse position along the line of load. The largest moment occurs for Case 2 under the inside load position. This moment is -1950 inch-pound (-220 N-m), which is approximately half as large as the maximum moment in the Y-direction.

The SLAB49 program performs a Mohr's circle analysis on bending and twisting moments to calculate the largest principal moment.

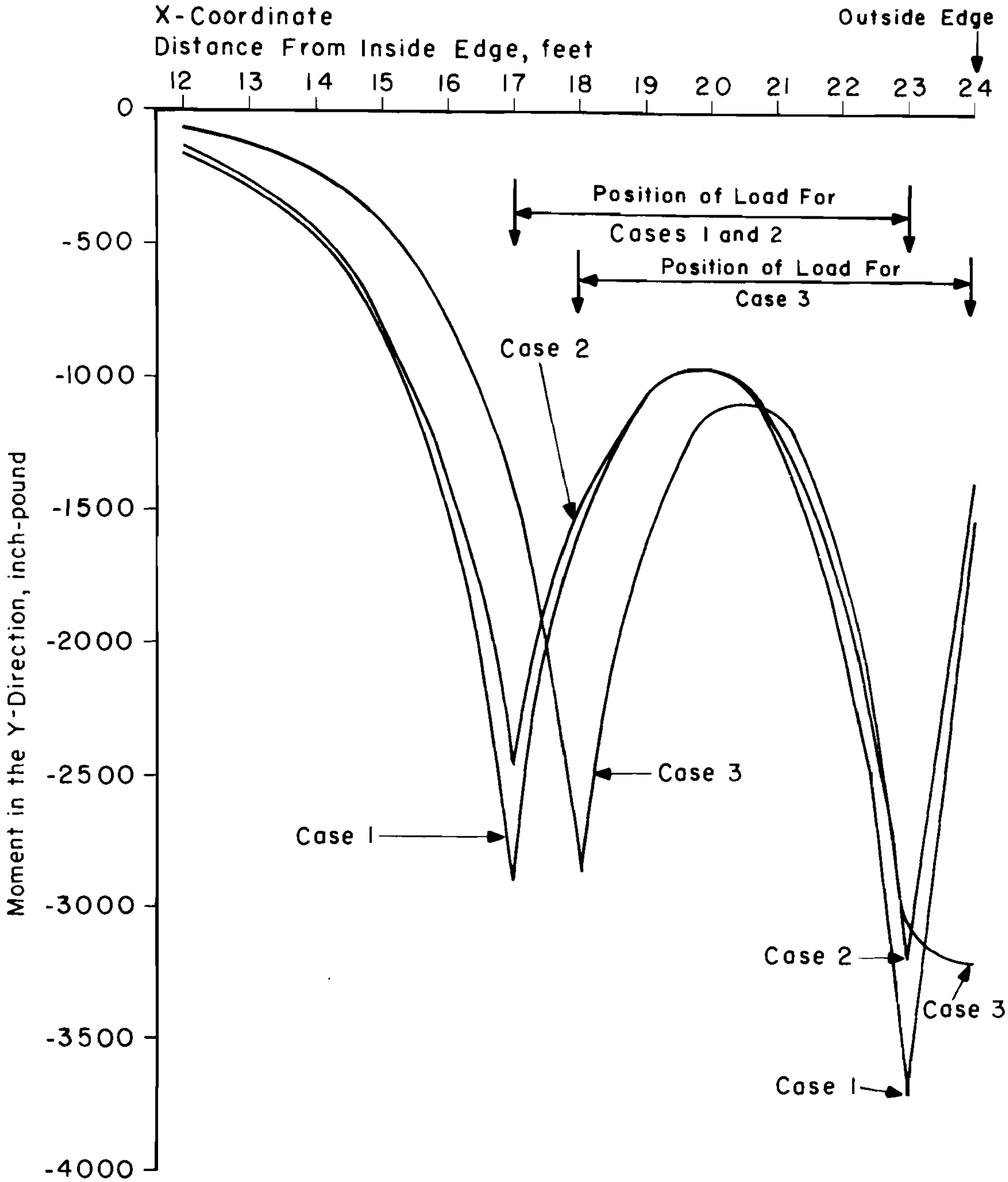
The stress in the extreme fibers due to the largest principal moment is calculated, then reported. A plot of the largest principal stress versus transverse position along the line of load is shown in Fig A2.6.

The largest stress occurred for Case 1. This stress was approximately 350 psi (2413 kPa). Case 1 was chosen as the critical load position for the slab model being analyzed, using stress as the criterion.

The load position which induces the largest stress does not also induce the greatest deflection. This can be seen in Fig A2.7, where deflection versus transverse position along the line of load is plotted. The maximum deflection occurred at the pavement's edge for Case 3. The least deflection at the edge of the pavement occurred for Case 1. If pumping were used as a criterion, Case 3 would be considered the critical load position.

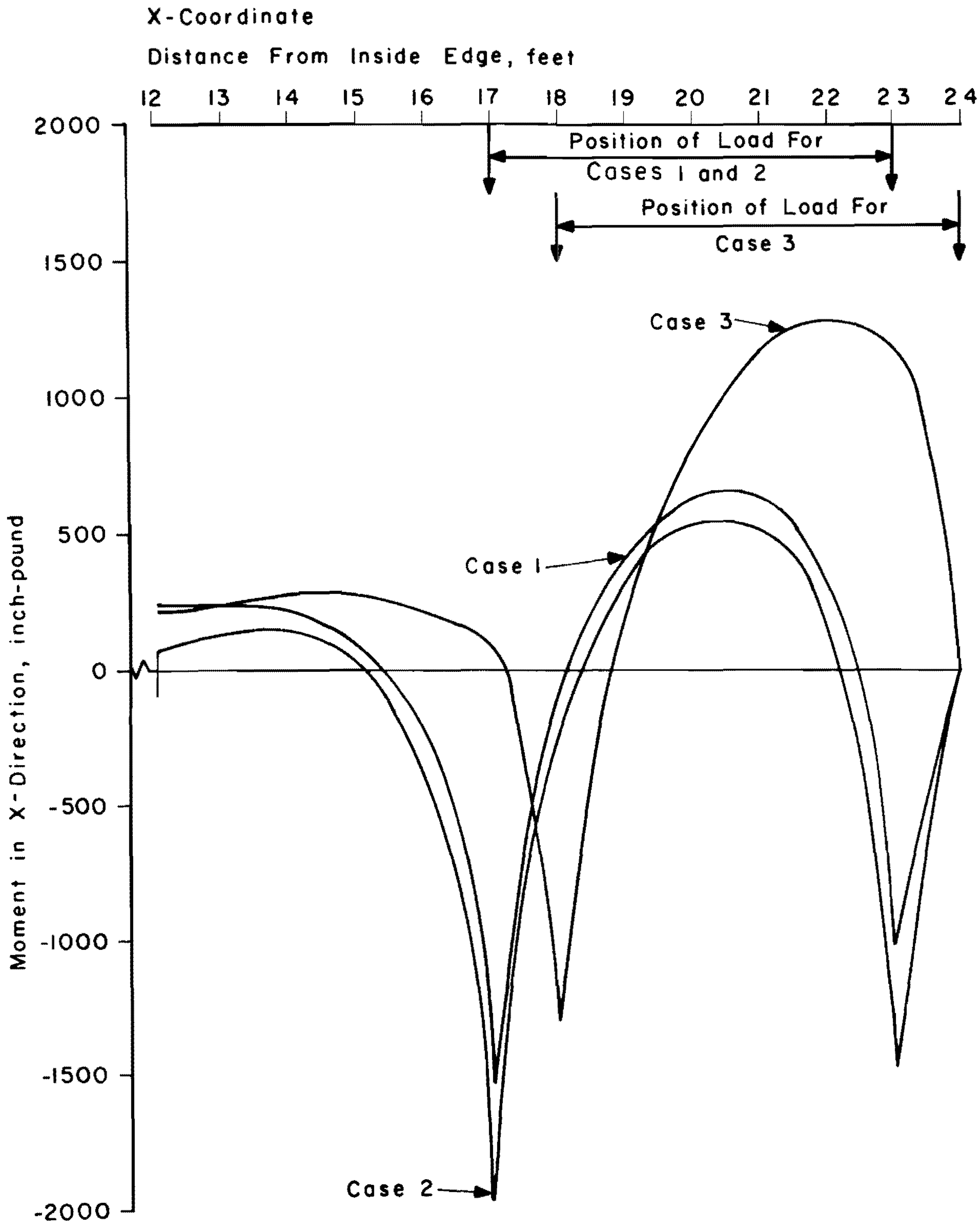
Increased Bending Stiffness

It is expected that the stiffness of the splice locations will be greater than that of the surrounding pavement. This increased stiffness is attributed to the additional steel at these locations required for splicing. The stiffness values for these locations were selected using the modulus



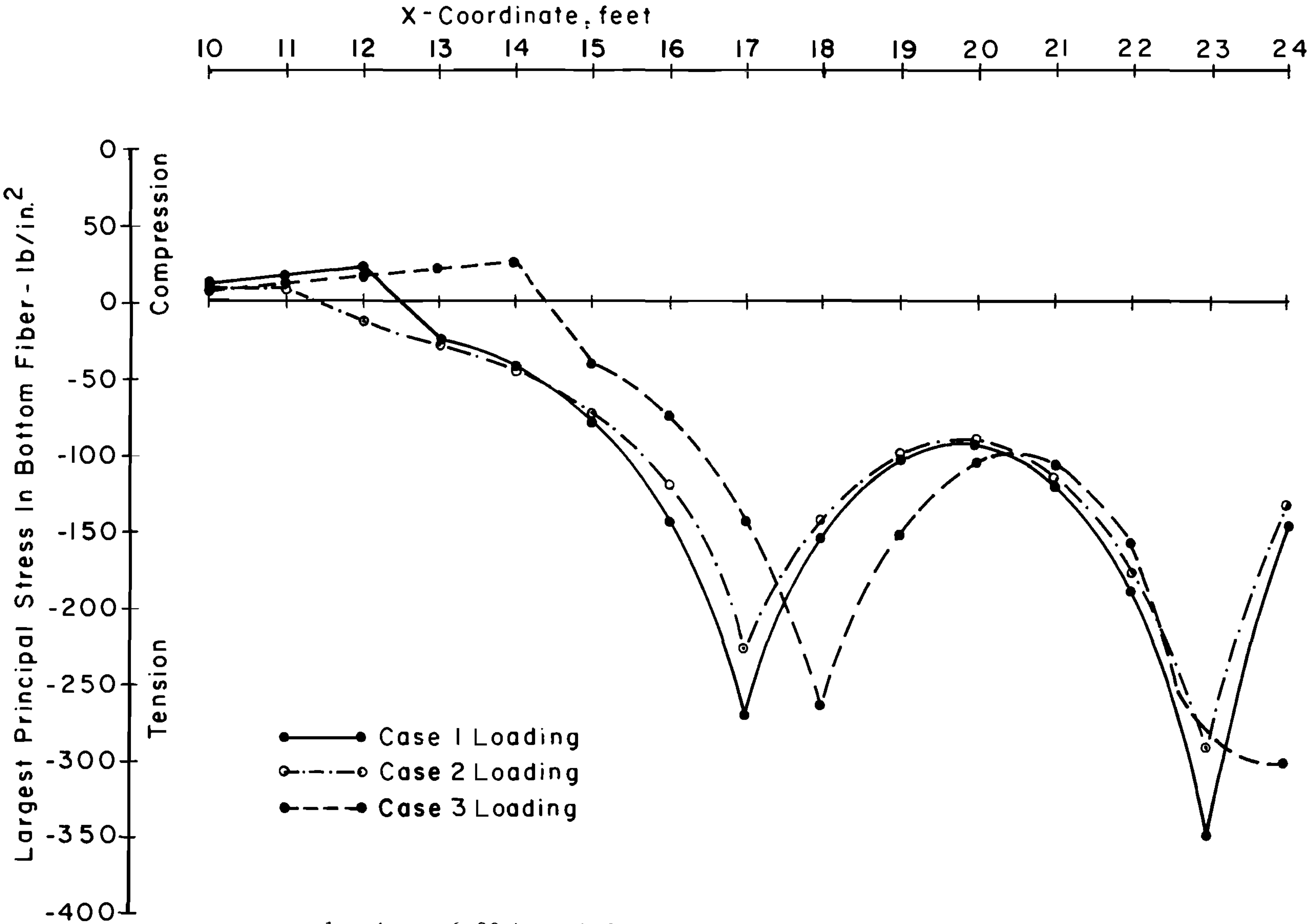
1 in.-lb = .113 N-m, 1 ft = .3048 m

Fig A2.4. Critical position of load-moments in the Y-direction.



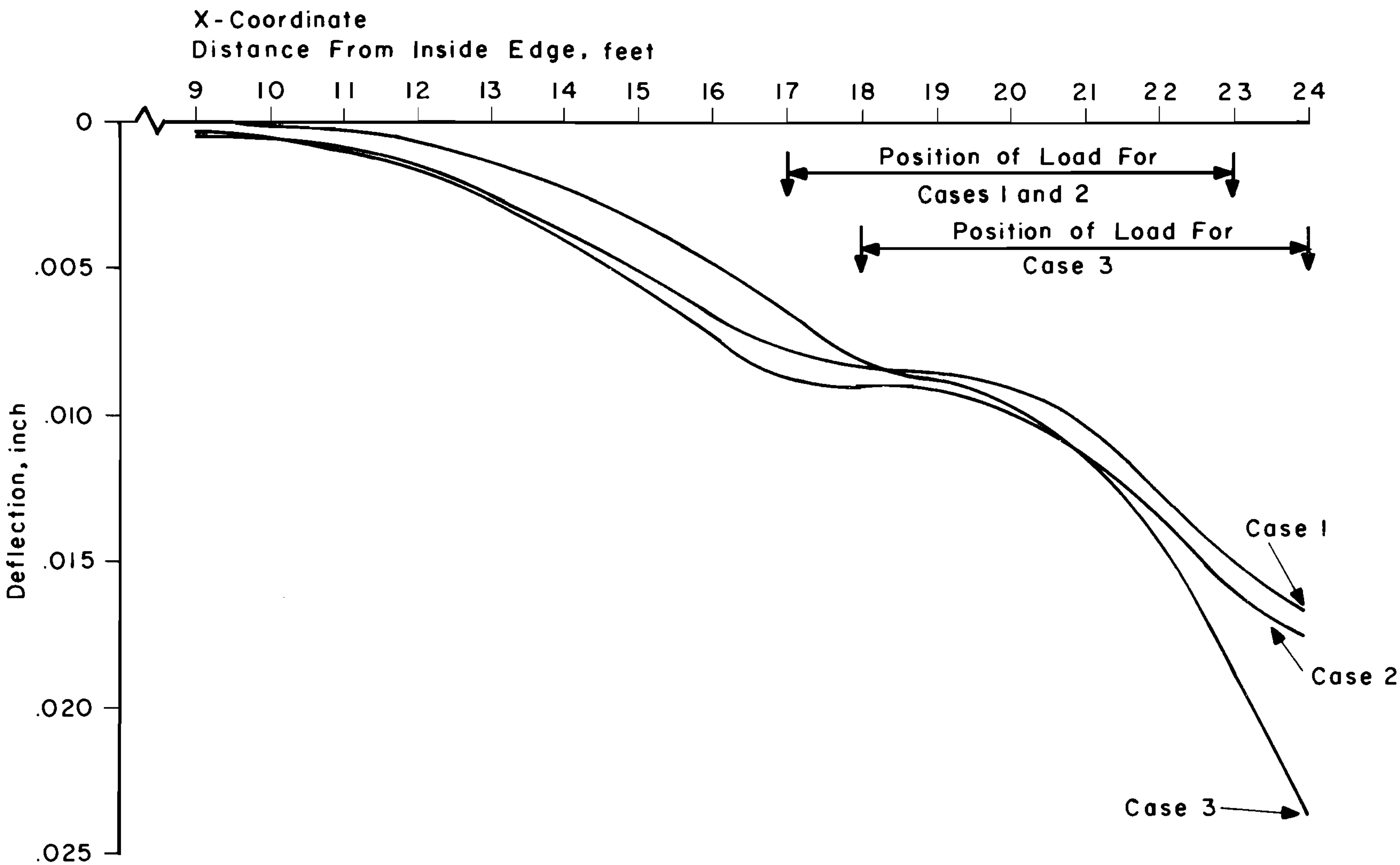
1 in.-lb = .113 N-m, 1 ft = .3048 m

Fig A2.5. Critical position of load-moments in X-direction.



1 psi = 6.89 kPa, 1 ft = .3048 m

Fig A2.6. Largest principal stress in bottom fiber-critical position of load.



1 in. = 25.4 mm, 1 ft. = .3048 m

Fig A2.7. Critical position of load-deflection.

values shown in Table A2.1 and the following equation for the bending stiffness of a slab:

$$D_i = \frac{E\ t^3}{12(1-M^2)}$$

where

D_i = bending stiffness in direction i,

E = modulus of elasticity,

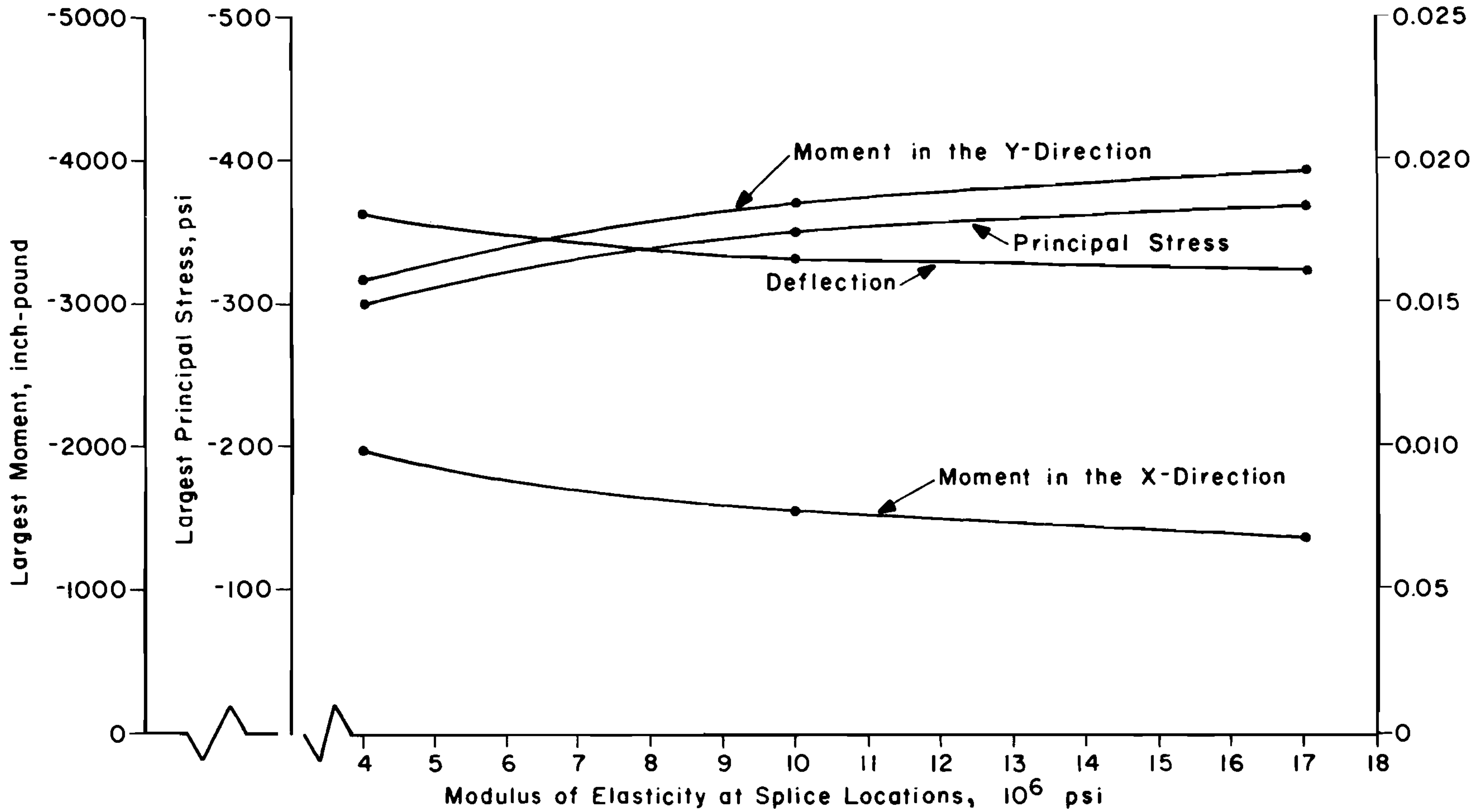
t = thickness,

M = Poisson's ratio.

As shown in Table A2.1, three levels of modulus were studied. A modulus of 4×10^6 psi (2.76×10^7 kPa) was selected to represent the modulus of concrete. The stiffness corresponding to this modulus is the control case to which the other stiffnesses are compared. The increased stiffnesses were selected corresponding to splice moduli of 10×10^6 psi (6.89×10^7 kPa) and 17×10^6 psi (1.17×10^8 kPa). These modulus values are approximately one-fourth and one-half the difference between the assumed modulus for concrete (4×10^6 psi) (2.76×10^7 kPa) and the assumed modulus for steel (29×10^6 psi) (2.0×10^8 kPa) added to the concrete modulus. A modulus of 17×10^6 psi (1.17×10^8 kPa) is higher than that expected in the field. More importantly, the stiffness corresponding to this modulus, assuming constant thickness and Poisson's ratio, is felt to be larger than that expected in the field. The moduli were selected in this way so that the levels of bending and twisting stiffness could be determined in a cognitive manner.

The modeled slab was loaded in the position corresponding to Case 1 in Fig A2.3. Each slab with different stiffness at the splice location was loaded in the same position.

A plot of largest moment, principal stress, and deflection versus the modulus of the splice location is shown in Fig A2.8. Since stiffness is directly proportioned to the modulus, the abscissa in Fig A2.8 is also a representation of the stiffness at the splice location. It is seen in



1 in.-lb = .113 N-m, 1 psi = 6.89 kPa, 1 in. = 25.4 mm

Fig A2.8. Influence of modulus of elasticity at splice locations on largest principal stress, moments, and deflection.

Fig A2.8 that, as the modulus of the splice section increases, the induced stress increases. The largest principal stress increased 23 percent for a 325 percent increase in the modulus, and stiffness, at the splice location.

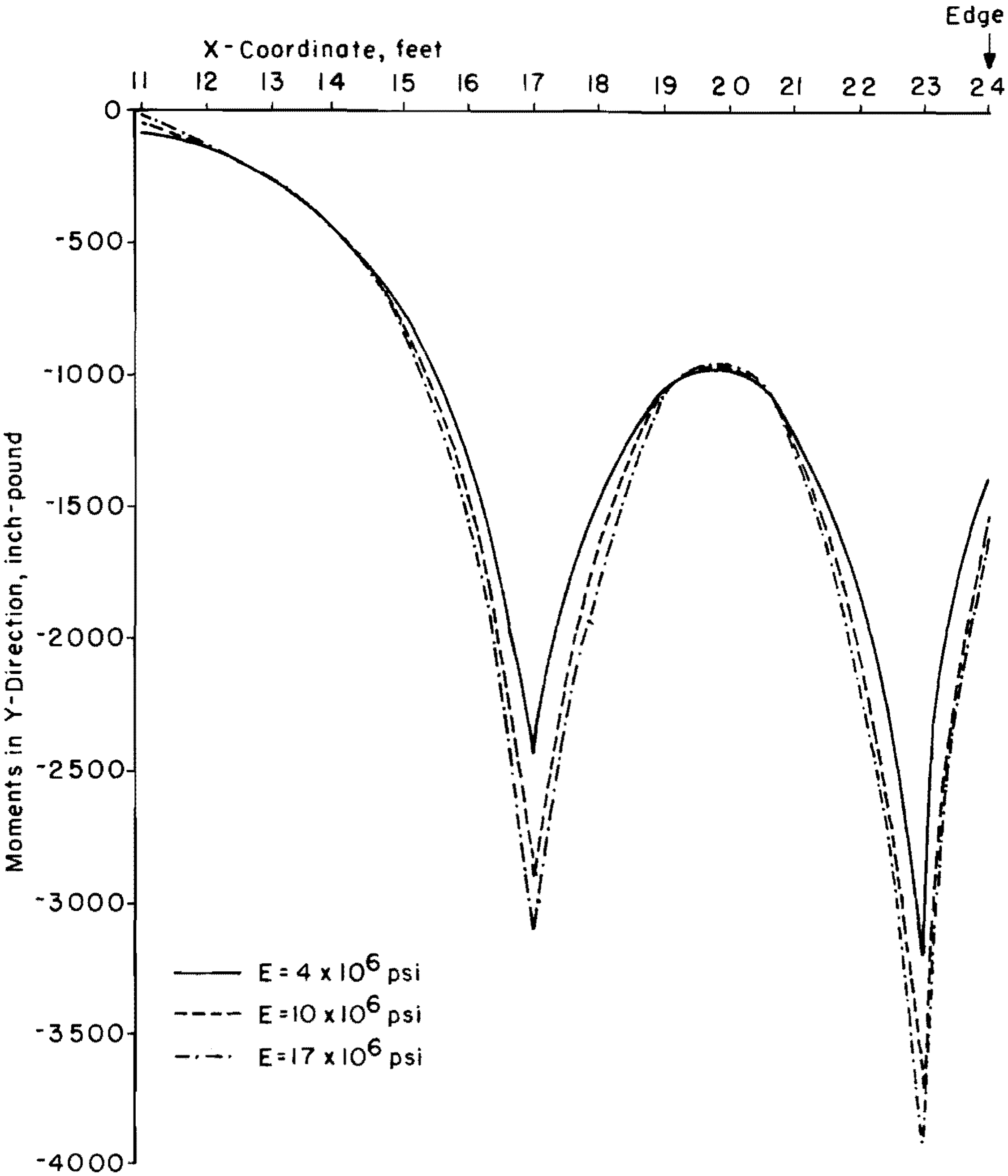
As previously noted, the stiffness at the splice location was increased only in the longitudinal (Y) direction. The increase in moment in the Y-direction was observed for this increase. However, deflection and moment in the X-direction both decrease for this increase. This suggests that, as the stiffness of the splice connection increases, more resistance is provided against flexure in the longitudinal direction within the slab. Thus, deflection is reduced, reducing the transverse curvature of the slab. In this way, moments in the X-direction are decreased.

Plots of the distribution of moments in the Y-direction, moments in the X-direction, principal stress, and deflection across the slab along the line of loading are shown in Figs A2.9, A2.10, A2.11, and A2.12. The curves for the splice stiffnesses have similar profiles. The maximum moment in the Y-direction and largest principal stress occurred beneath the outside load. The largest deflection occurred at the edge of the pavement near the position of the load.

A plot of principal stress showing the longitudinal distribution along a line passing through the outside load position is presented in Fig A2.13. It can be seen that the stress distribution exhibits a continuous profile with the maximum stress concentration occurring beneath the load as seen in the transverse profile. The distribution of moments and deflection in the longitudinal direction, not shown here, also exhibits curves with shapes similar to that in Fig A2.13.

From these findings, it is concluded that a stress concentration due to the increased stiffness of the splice connections occurs. However, the magnitude of such concentration for a higher value of stiffness than that expected is not great enough to suggest this as a probable destructive element within a pavement structure.

A splice stiffness corresponding to a modulus of 10×10^6 psi (6.89×10^7 kPa) was used in the determination of the critical position of load. The figures presented here show that the stiffness does not greatly affect the shape or magnitude of the induced stress curves. The determination of the critical position of load is, therefore, not adversely affected



1 in.-lb = .113 N-m, 1 ft = .3048 m

Fig A2.9. Splice stiffness - moments in the Y-direction.

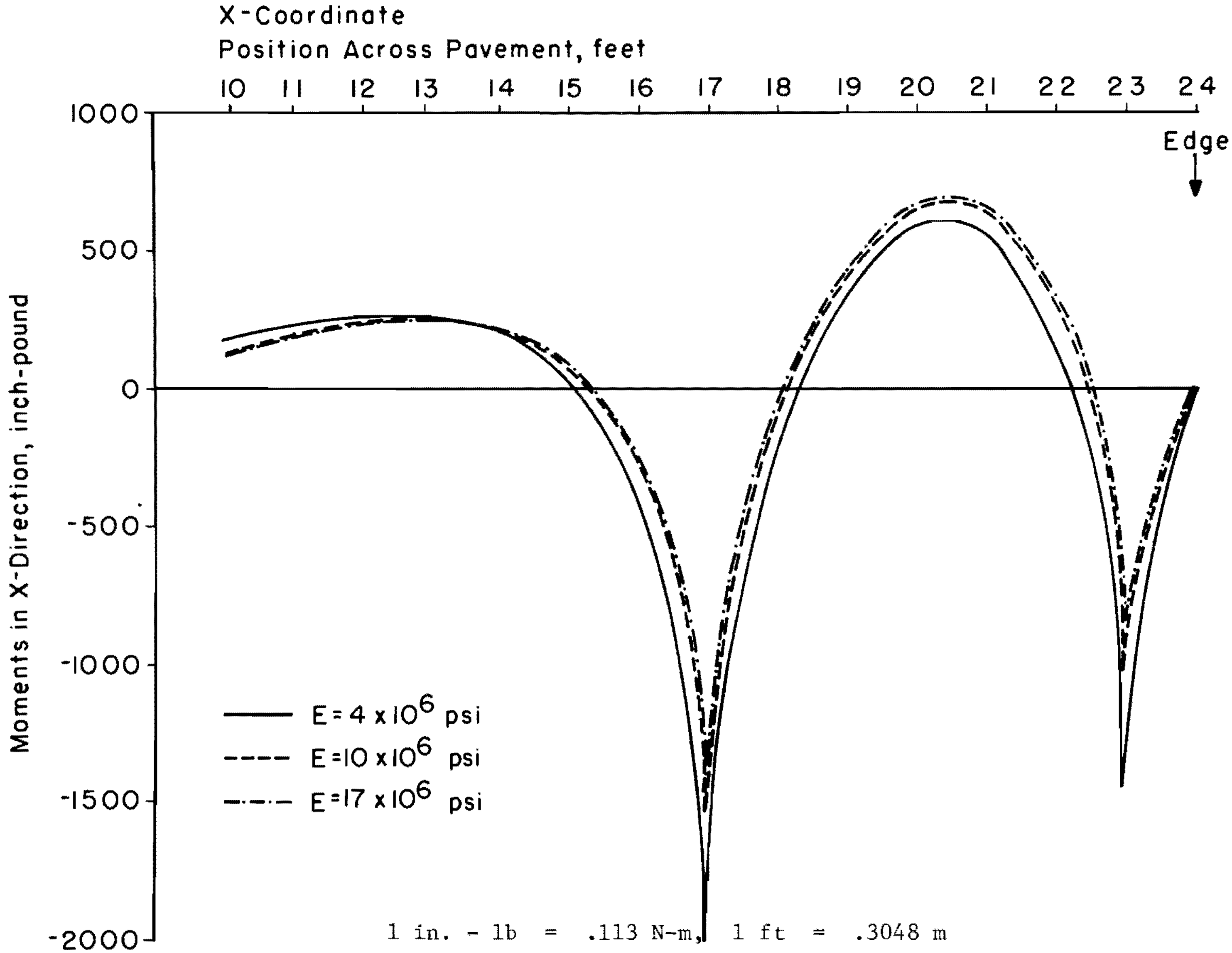
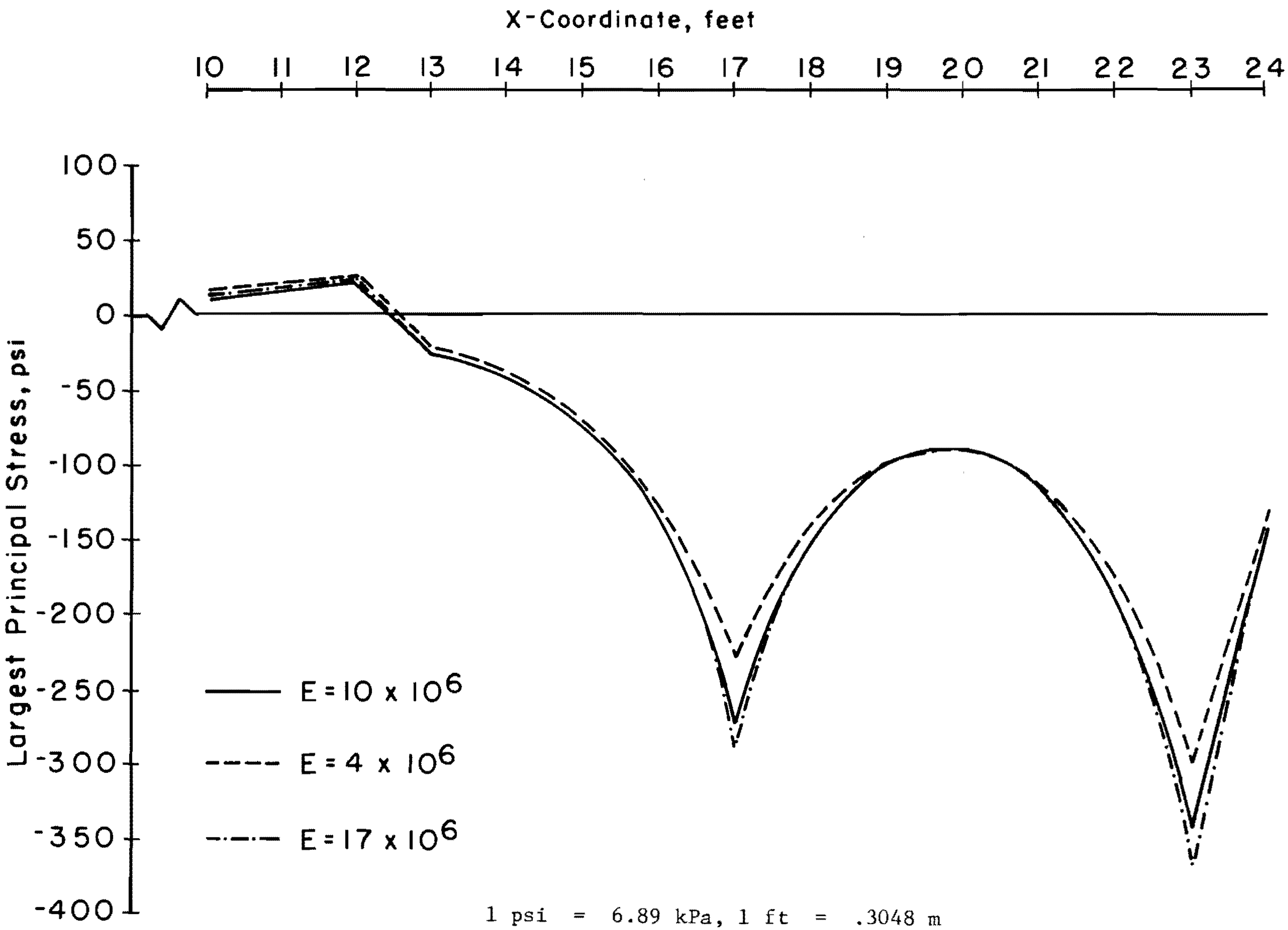


Fig A2.10. Moments in the X-direction due to two 9-kip loads at 17 and 23. Slab with varying modulus of elasticity at splice locations.



1 psi = 6.89 kPa, 1 ft = .3048 m

Fig A2.11. Distribution of principal stress across slab with varying splice modulus.

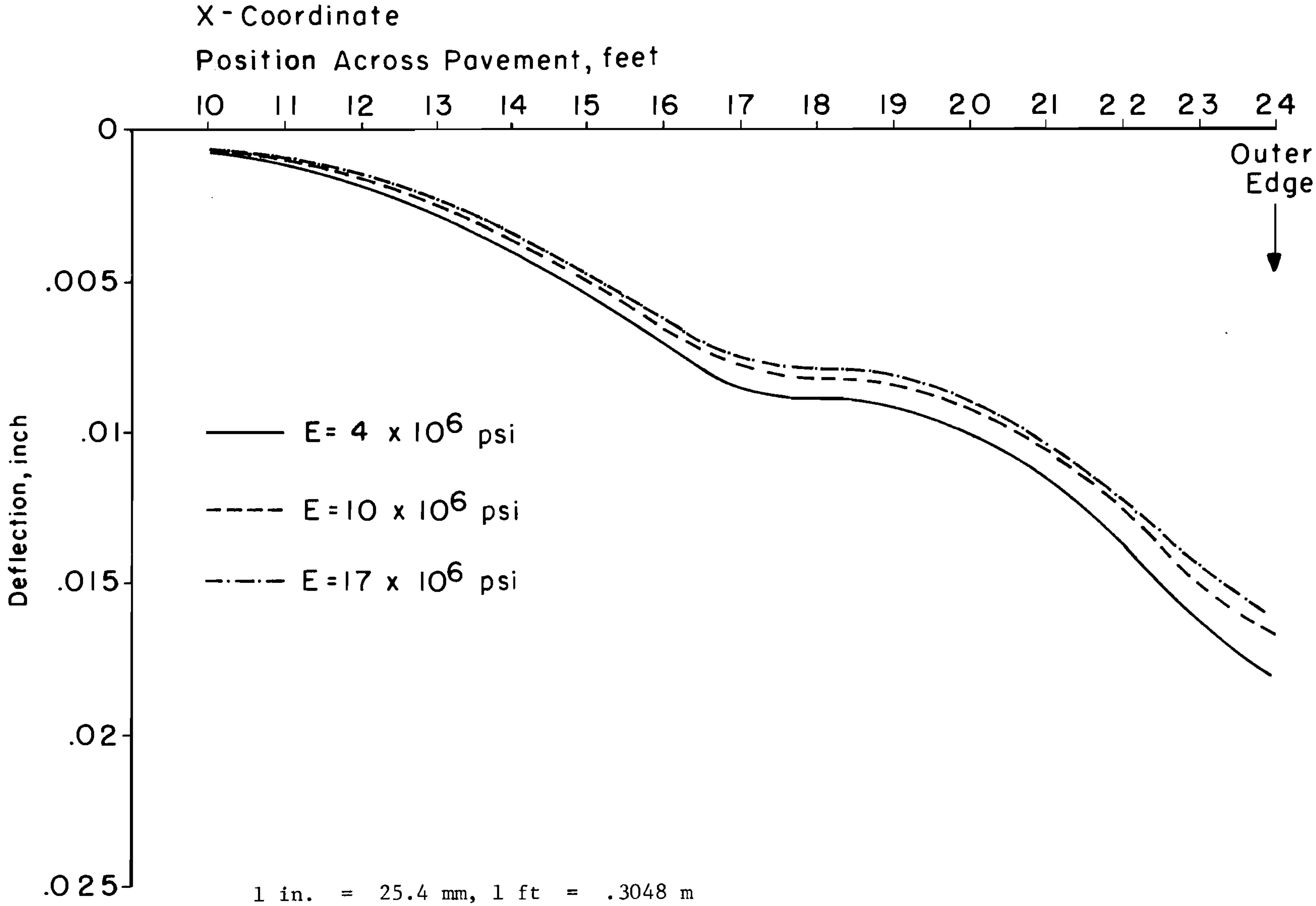
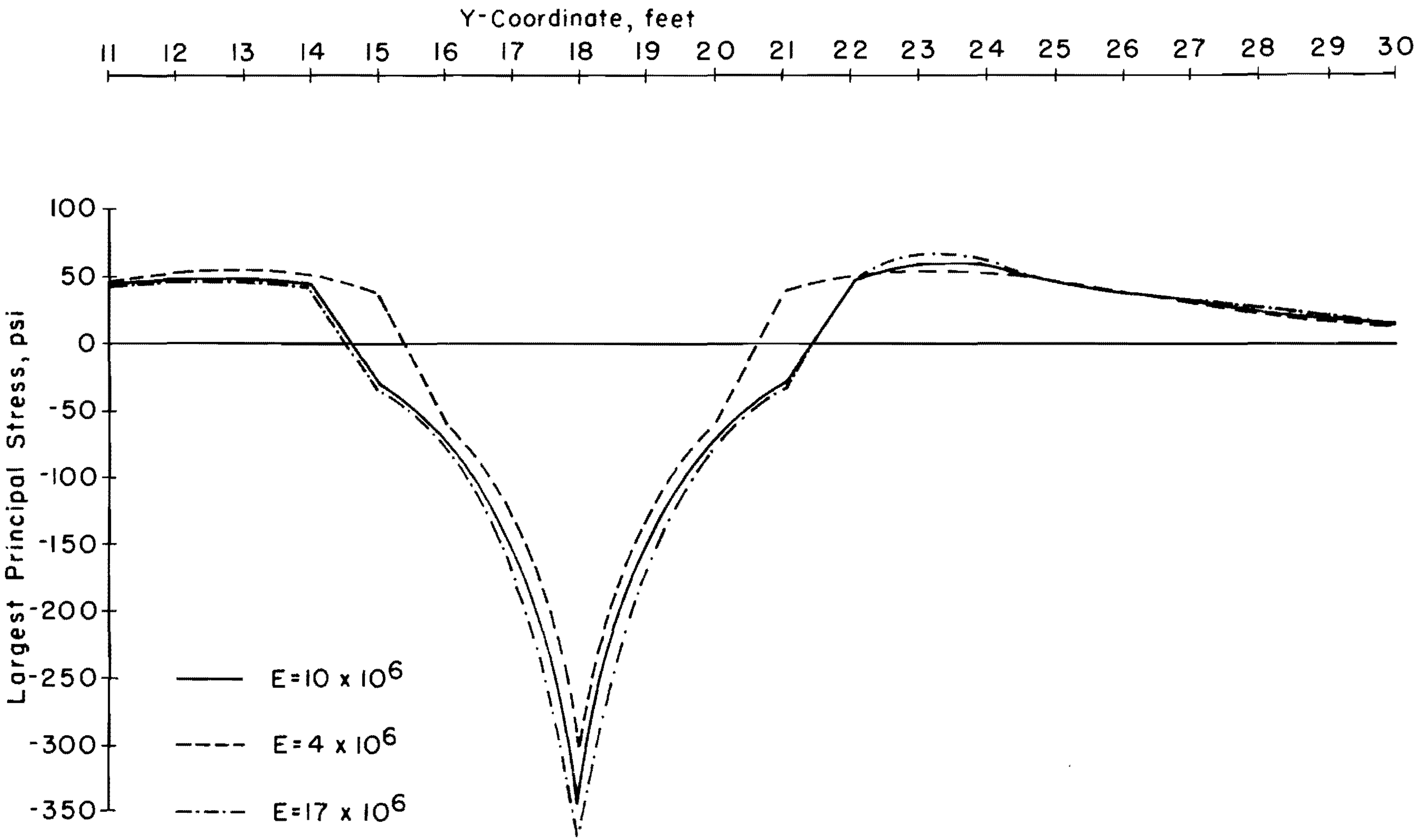


Fig A2.12. Deflection due to 9-kip loads at 17 and 23. Varying modulus of elasticity at splice locations.



1 psi = 6.89 kPa, 1 ft = .3048 m

Fig A2.13. Distribution of principal stress along slab with varying splice modulus.

by the use of this stiffness. This stiffness for the splice area was also used to investigate the effect of the length of precast slab.

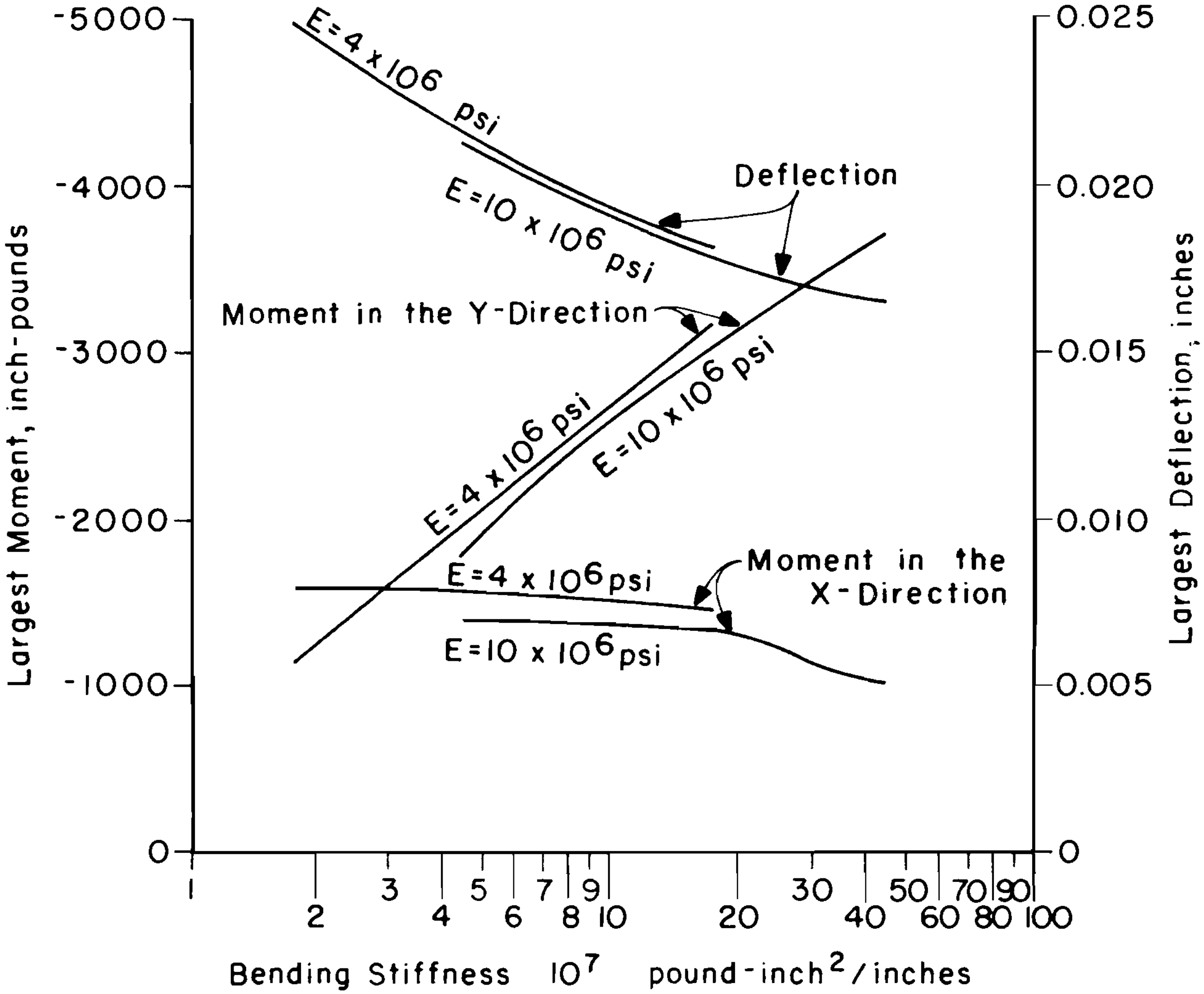
Decreased Bending Stiffness

At the splice location of a precast patch, thermal cracks or other discontinuities may reduce the longitudinal bending stiffness of this cross-section. To investigate this condition, the longitudinal bending stiffness at the splice location was reduced while the transverse bending stiffness and mesh twisting stiffness were held constant. Two levels of stiffness, corresponding to modulus values of 4×10^6 psi (2.76×10^7 kPa) and 10×10^6 psi (6.89×10^7 kPa), were compared against percentage reductions in stiffness ranging from 24 percent to 90 percent. The mesh twisting stiffness is assumed to be activated across these discontinuities by the longitudinal reinforcing bars. At the splice location, this reinforcement is roughly twice that of the existing pavement due to the lapping of the bars. Thus, the mesh twisting stiffness is assumed not to be affected by the discontinuities. The transverse bending stiffness, orthogonal to the discontinuities, is also assumed not to be affected. The loading position shown as Case 1 in Fig A2.3 was used in this investigation.

A summary of the result of lowering the Y-direction bending stiffness is shown in Fig A2.14. The moment in the Y-direction decreased with the decreased bending stiffness. The moment in the X-direction slightly increased for the decrease in stiffness. The deflection also increased for the decrease in bending stiffness. Thus, as the bending stiffness is reduced, the amount of support provided by flexure of the slab is also reduced and more support is taken up in the subgrade reaction.

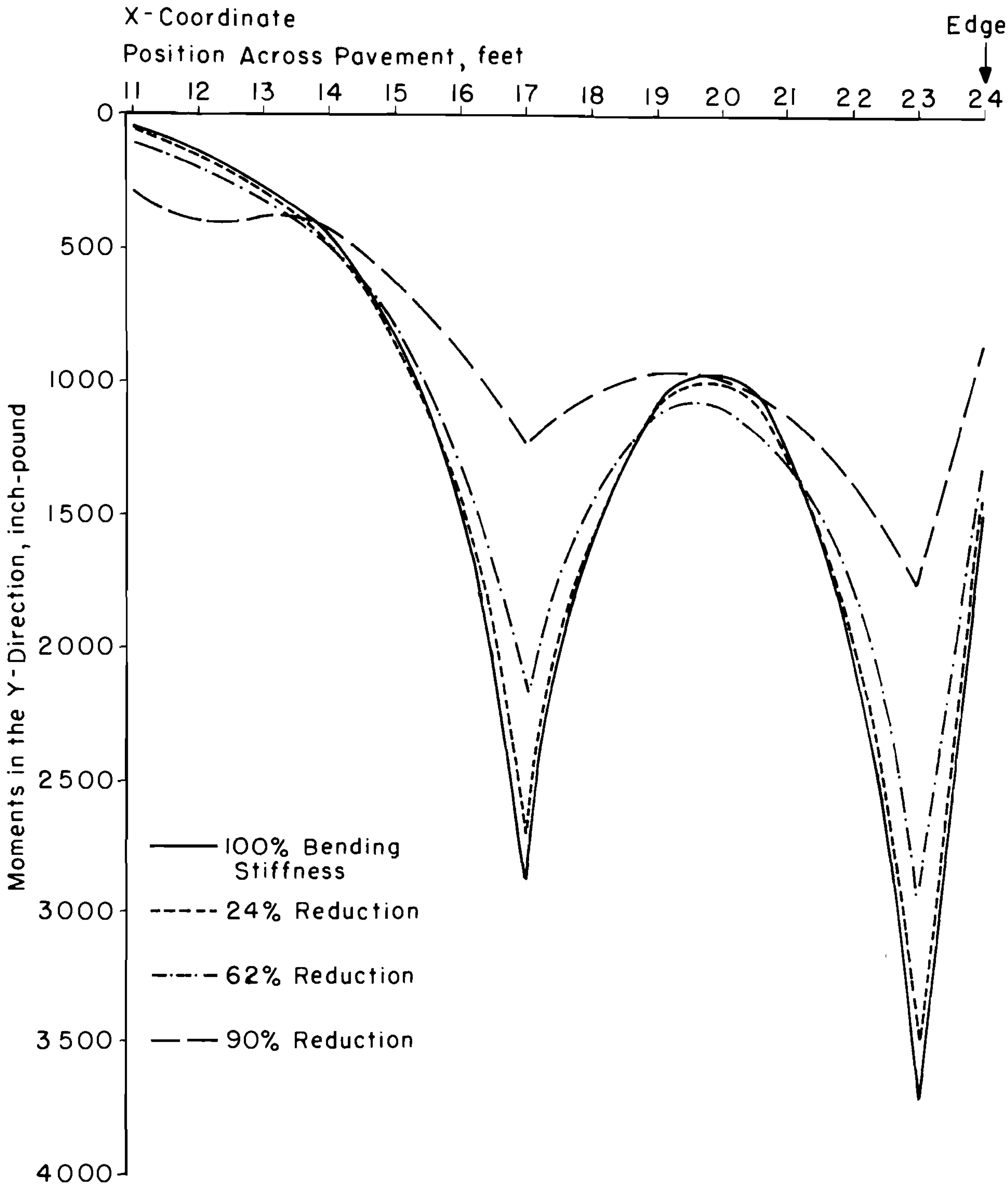
Examples of the typical distribution of moments, principal stresses, and deflection due to the reduction in the Y-bending stiffness are shown in Figs A2.15, A2.16, A2.17, A2.18 and A2.19. These stiffnesses correspond to a splice modulus of 10×10^6 psi (6.89×10^7 kPa). Similar distributions were observed for the case of a 4×10^6 psi (2.76×10^7 kPa) splice modulus.

The effects shown here suggest that possible destructive stresses may be induced into the supporting medium directly beneath the pavement if the stiffness of the splice location is low. This may result in pumping or faulting. This indicates that the stiffness of the splice should be maintained.



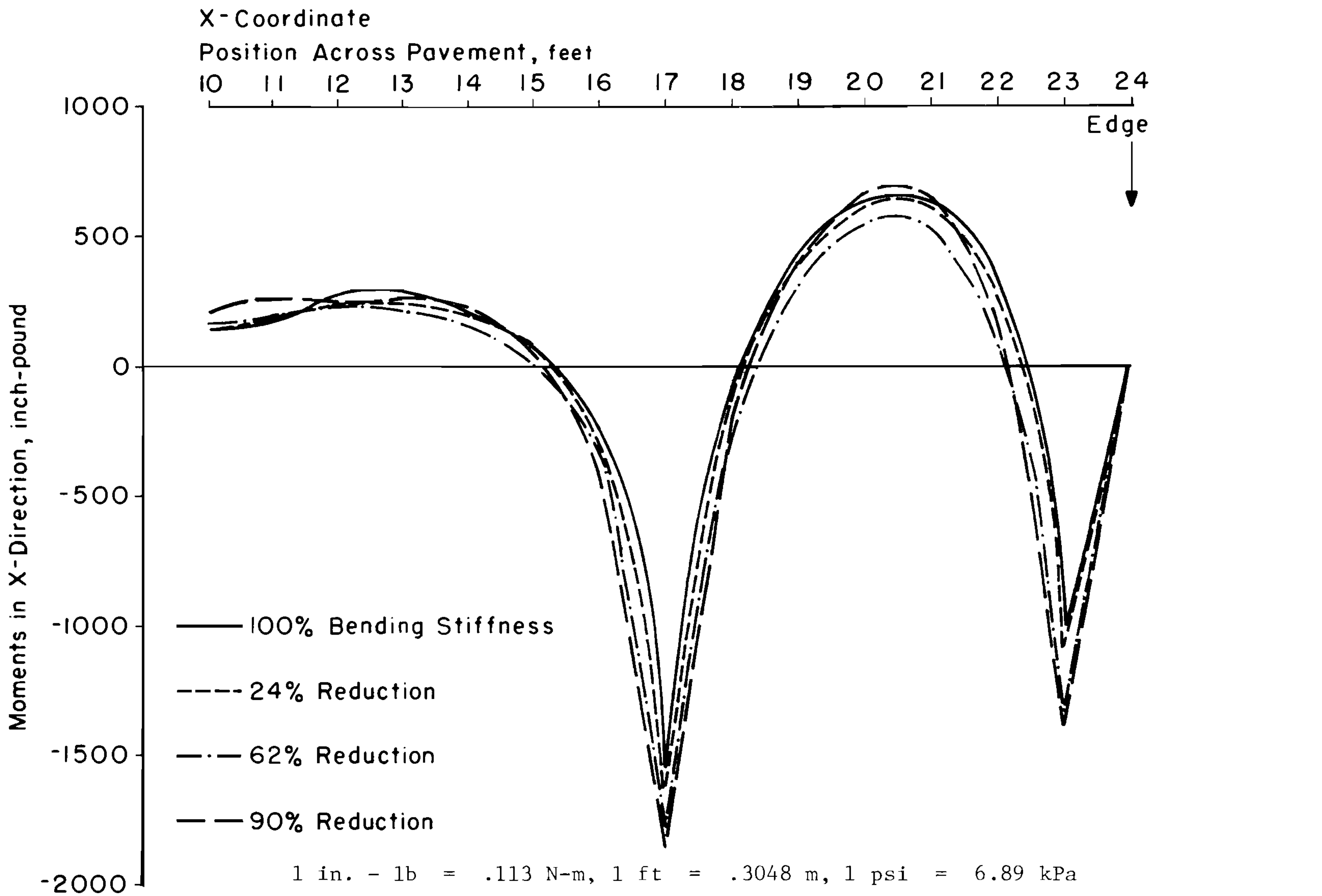
1 in. - lb = .113 N-m, 1 in. = 25.4 mm, 1 lb - in. ^{2}/in. = .113 N - m^2/m

Fig A2.14. Influence of reduced bending stiffness at splice locations holding elastic modulus constant on moment and deflection.



1 in.-lb = .113 N-m, 1 ft = .3048 m, 1 psi = 6.89 kPa

Fig A2.15. Moments in Y-direction due to 9-kip loads at 17 and 23. Modulus of elasticity at splice locations equal to 10 x 10^6 psi -- varying bending stiffness.



1 in. - lb = .113 N-m, 1 ft = .3048 m, 1 psi = 6.89 kPa

Fig A2.16. Moments in the X-direction due to 9-kip loa s at 17 and 23. Modulus of elasticity at splice locations equal to 10×10^6 psi, varying bending stiffness.

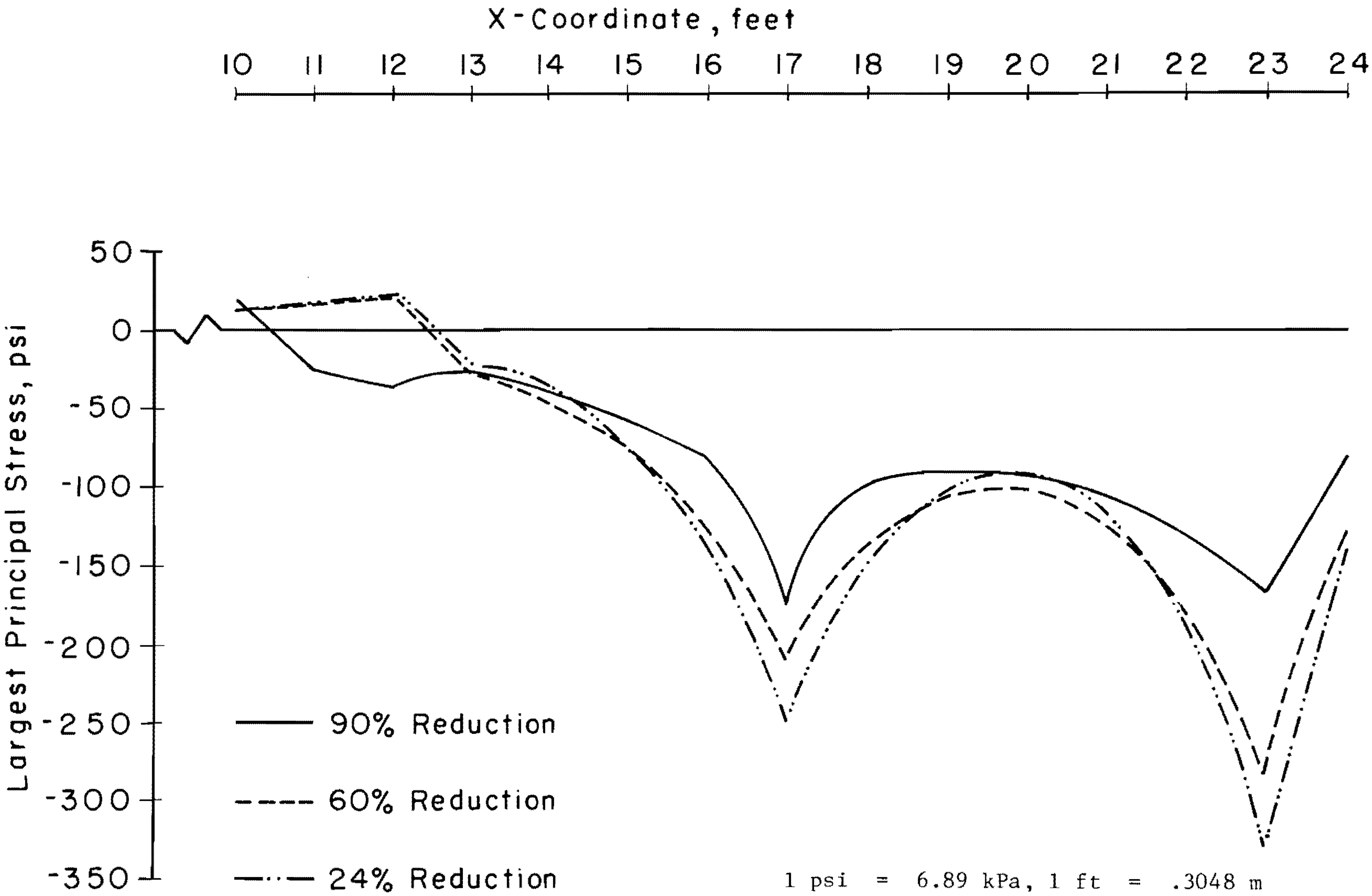


Fig A2.17. Distribution of principal stress across slab with reduced splice bending stiffness $E_{SPL} = 10 \times 10^6$ psi.

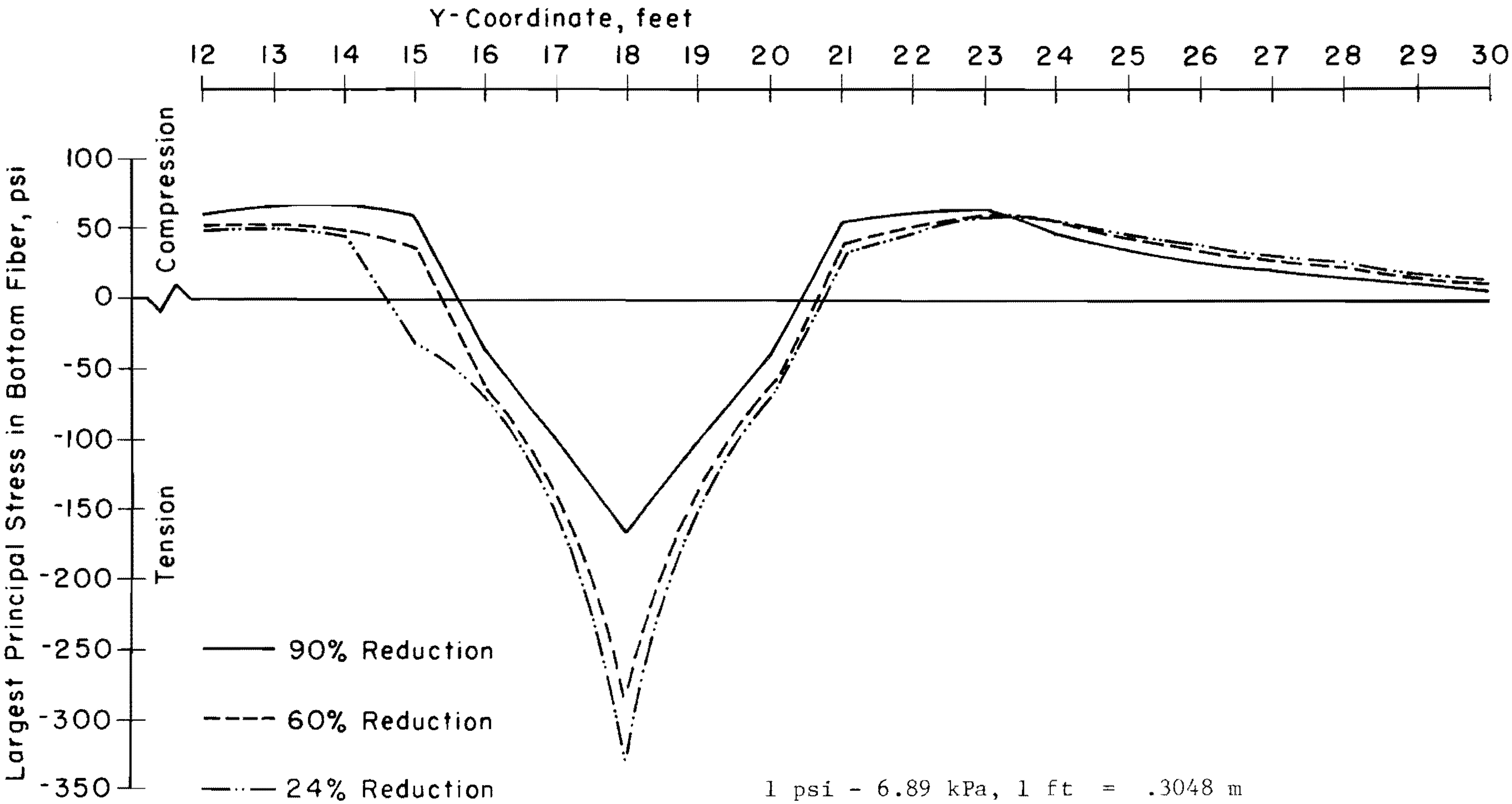


Fig A2.18. Distribution of principal stress along slab with reduced splice bending stiffness. $E_{SPL} = 10 \times 10^6$ PSI

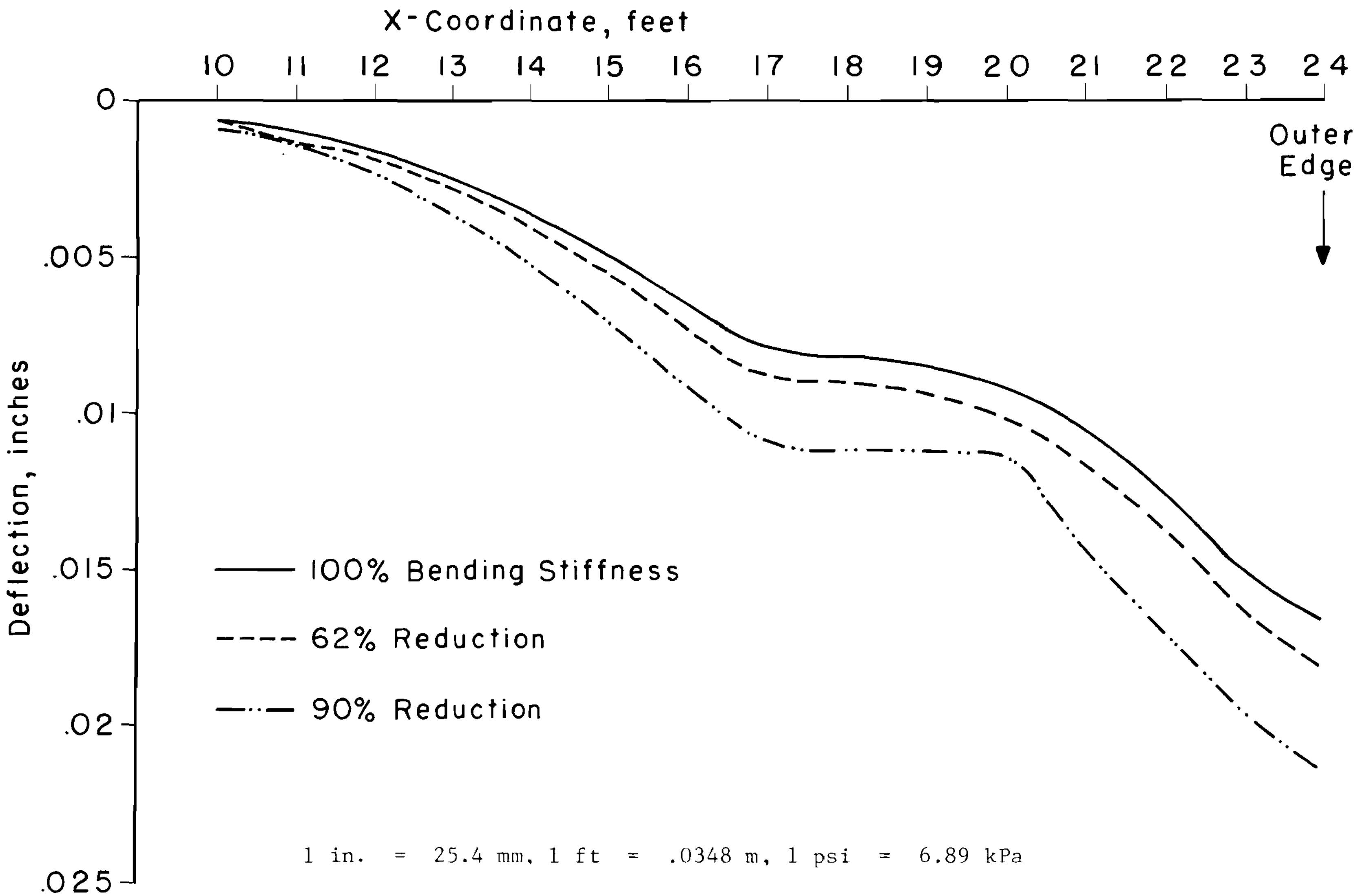


Fig A2.19. Deflection due to 9-kip loads at 17 and 23. Modulus of elasticity at splice locations equal to 10 x 10^6 PSI varying bending stiffness.

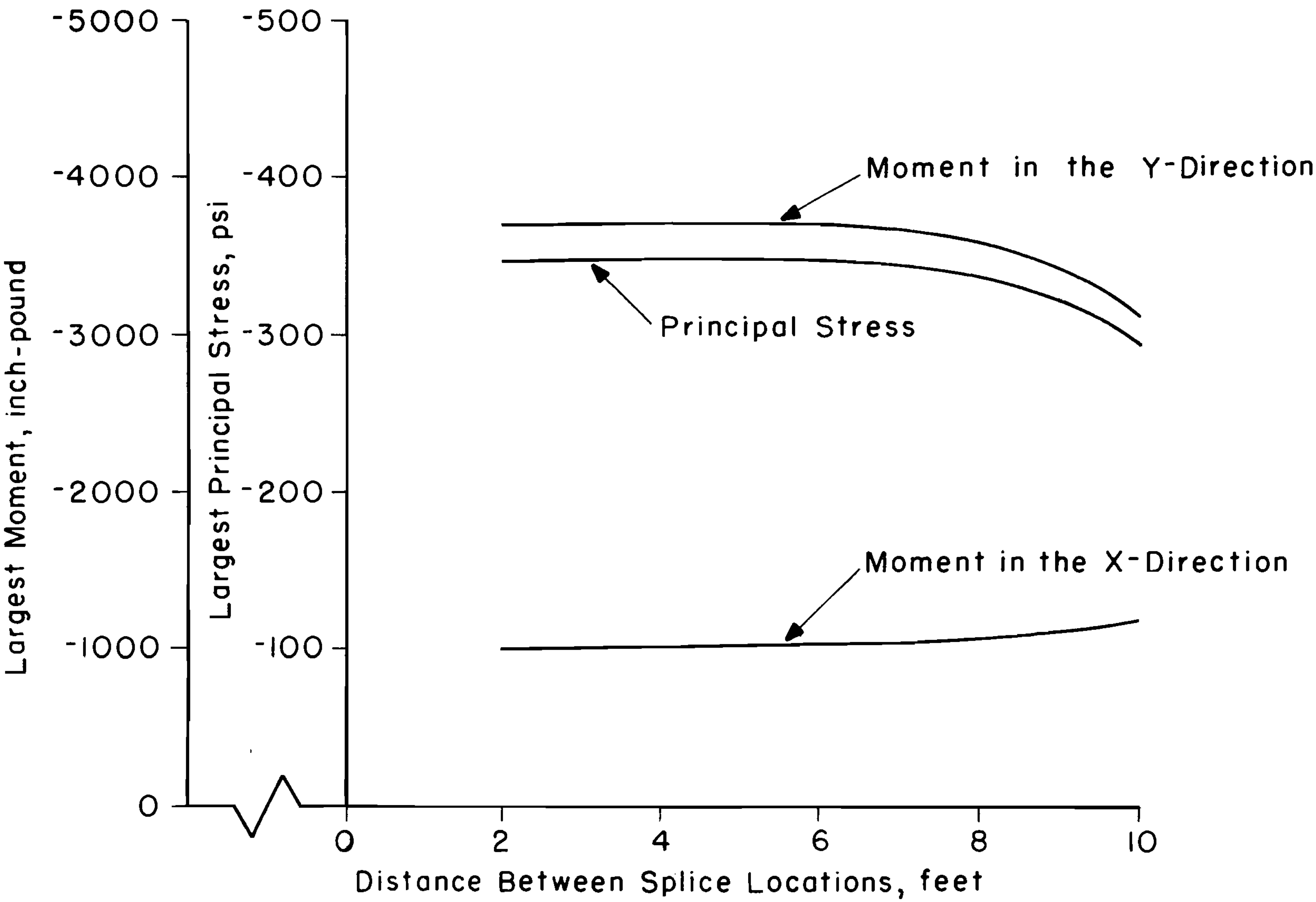
Precast Slab Length

The effect of the precast slab length on the pavement response was investigated using a length between splice location ranging from 2 to 10 feet (.61 m to 3.05 m) with a Y-bending stiffness corresponding to a splice modulus of 10×10^6 psi (6.89×10^7 kPa). A loading position corresponding to Case 1 in Fig A2.3 was used here.

The results of this investigation show that the induced stresses and moments decrease as the length of the precast slab increases. The additional stress resulting from shortening the slab from 6 to 2 feet (1.83 to .61 m) is small, as shown in Fig A2.20. The difference in stress between the 10 and 2-foot (3.05 and .61 m) long slabs is more significant. In terms of fatigue potential, the stresses calculated for the 2-foot (.16-m) long slab results in 53 percent less potential application than the 10-foot-long slab. This is assuming equal concrete strengths and that fatigue life is proportionate to the fourth power of the strength-stress ratio (Refs 40 and 41). The increase in fatigue potential between a 6-foot (1.83 m) and an 8-foot (2.44 m) long slab is 13 percent. The 12 percent decrease in stress between the 8 and 10-foot (2.44 and 3.05-m) slabs increases the fatigue potential 68 percent. Thus, as the slab length increases past about 8 feet (2.44 M), the decrease in stress becomes more and more significant in terms of fatigue.

The plots of the distribution of moments, stresses, and deflections in Figs A2.21, A2.22, A2.23, A2.24 and A2.25 demonstrate the relatively small effect of slab length on the other pavement response parameters. In Fig A2.24, it can be seen that the deflection for the 10-foot (3.05-m) long slab was greater than for the shorter lengths.

On the basis of this analysis, the lengths of precast slabs are not expected to appreciably affect pavement response. This analysis does indicate that shorter length slabs, less than 6 feet (1.83 m) have less fatigue potential than slabs longer than this. The lesser fatigue potential, however, is not detrimentally high. Thus, because of the higher stress levels in the shorter length slabs, the design and construction are expected to be more critical to their performance.



1 in. - lb = .113 N-m, 1 psi = 6.89 kPa, 1 ft = .3048 m

Fig A2.20. Influence of distance between splice locations on largest principal stress and moment.

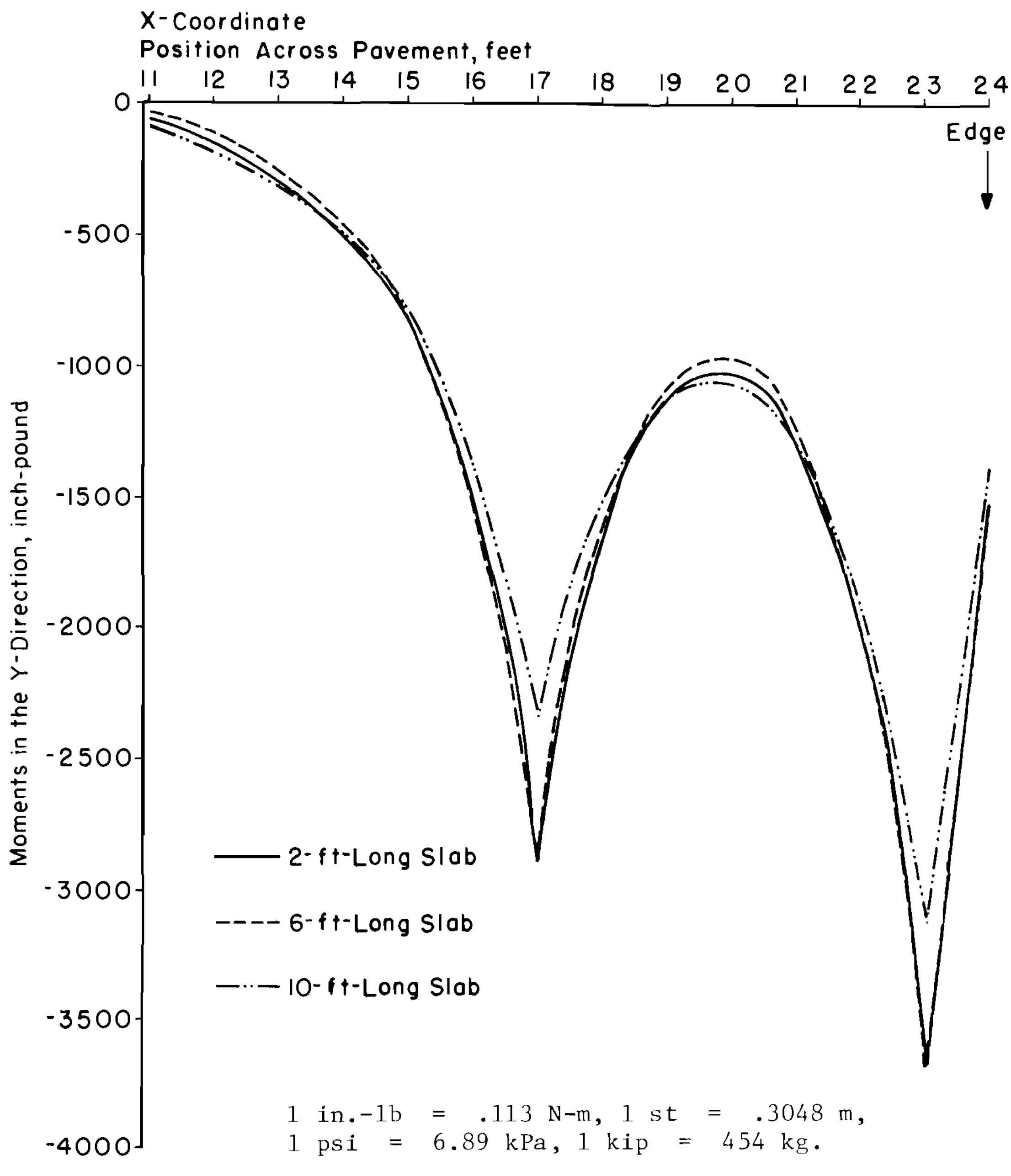


Fig A2.21. Moments in the Y-direction due to 9-kip loads at 17 and 23 ft. Modulus of elasticity at splice locations equals 10.0 x 10^6 psi. Varying slab length.

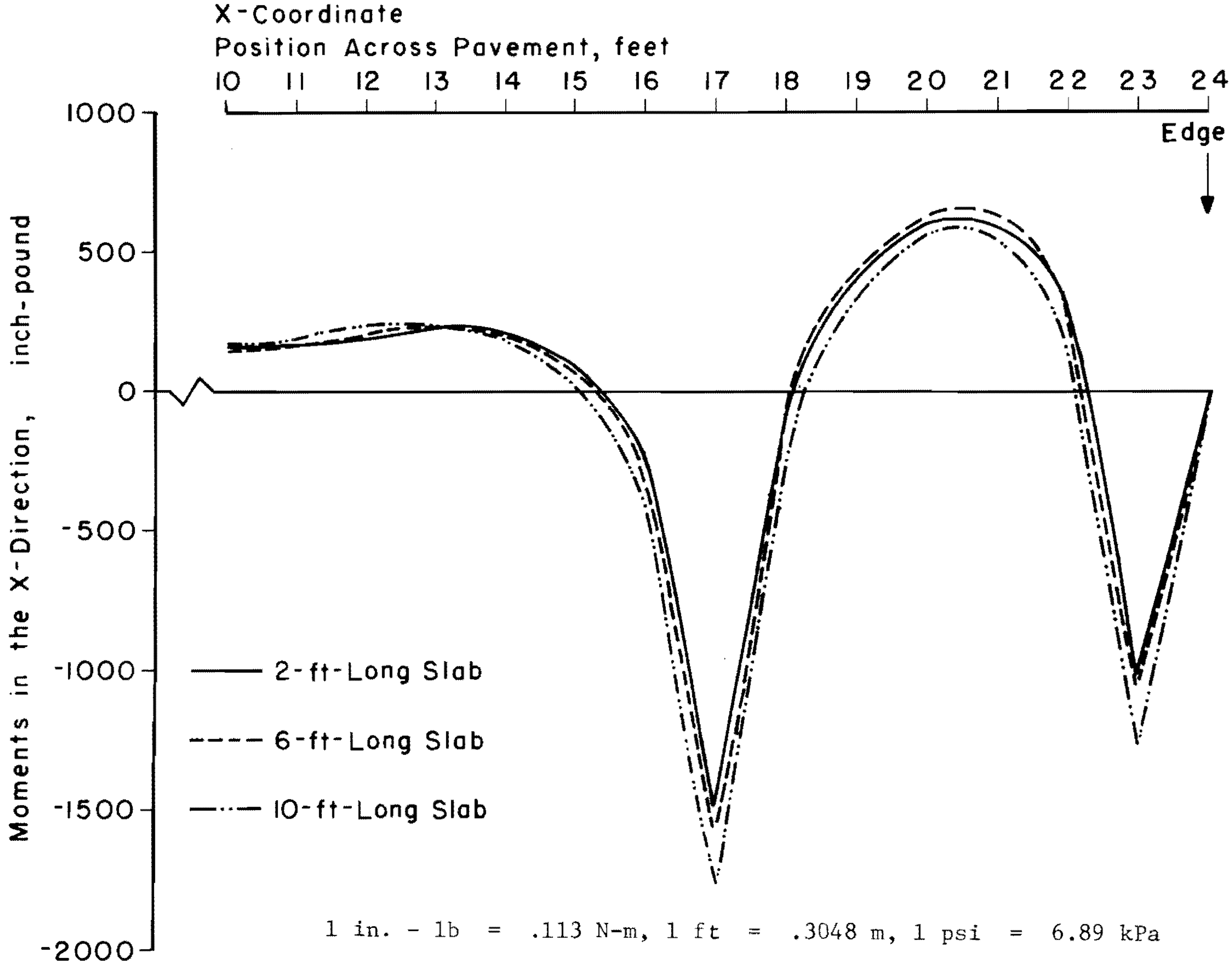


Fig A2.22. Moments in the X-direction due to 9-kip loads at 17 and 23. Modulus of elasticity at splice locations equal to 10.0 x 10^6 psi, varying slab lengths.

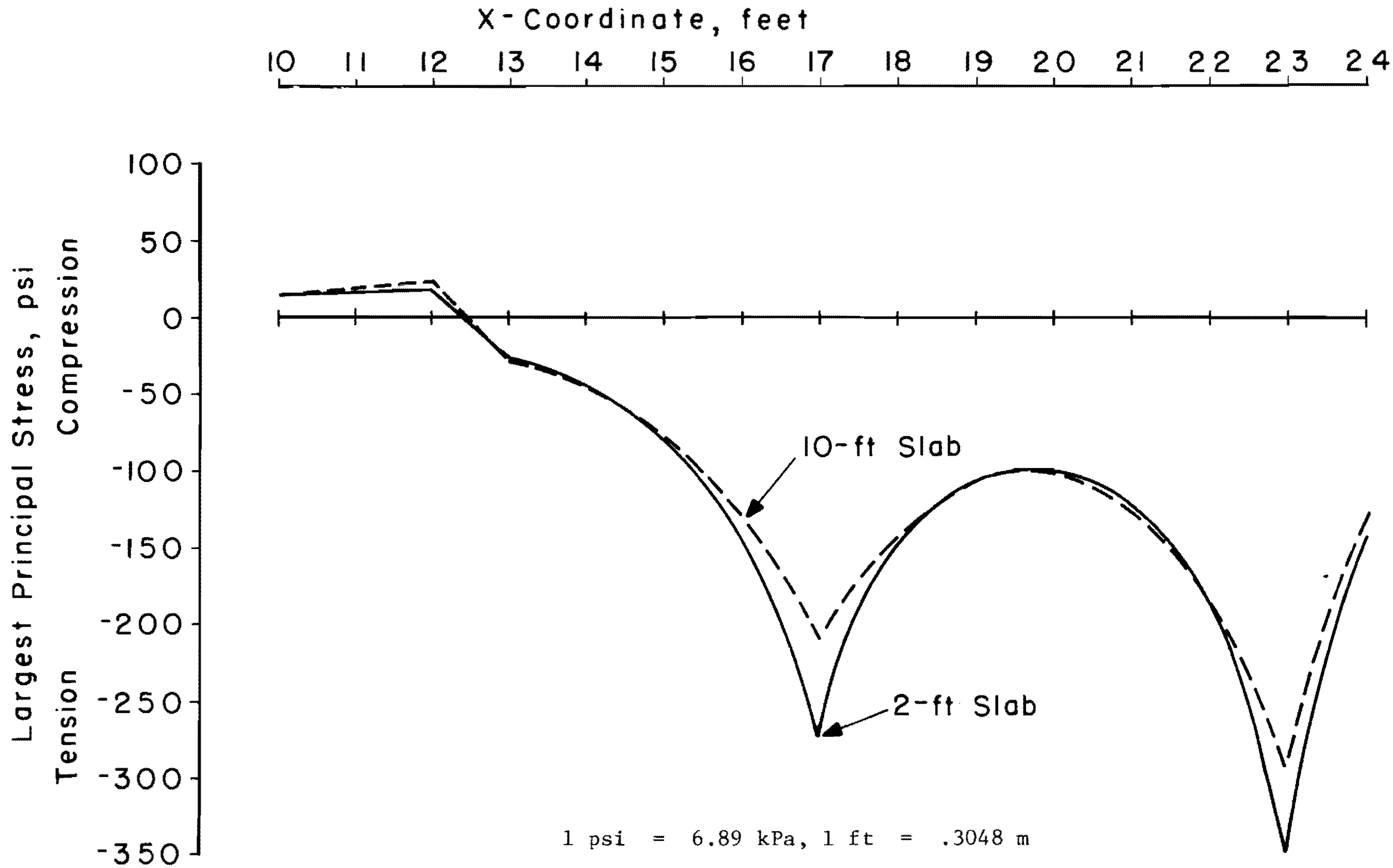
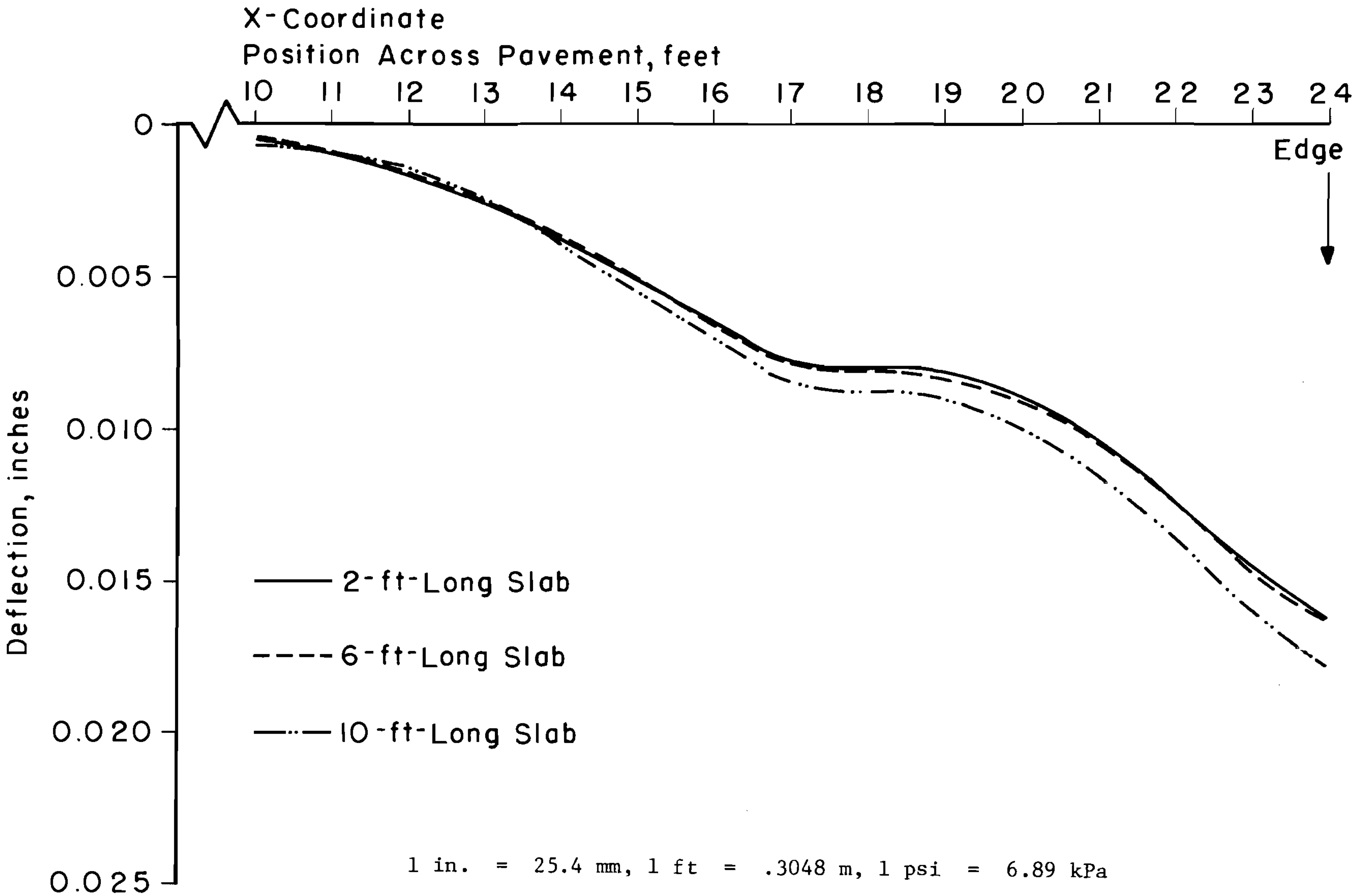


Fig A2.23. Largest principal stress in bottom of slab versus X-coordinate for two and ten-foot long slabs.



1 in. = 25.4 mm, 1 ft = .3048 m, 1 psi = 6.89 kPa

Fig A2.24. Deflection due to 9-kip loads at 17 and 23. Modulus of elasticity at splice locations equals 10.0 x 10^6 PSI varying slab length.

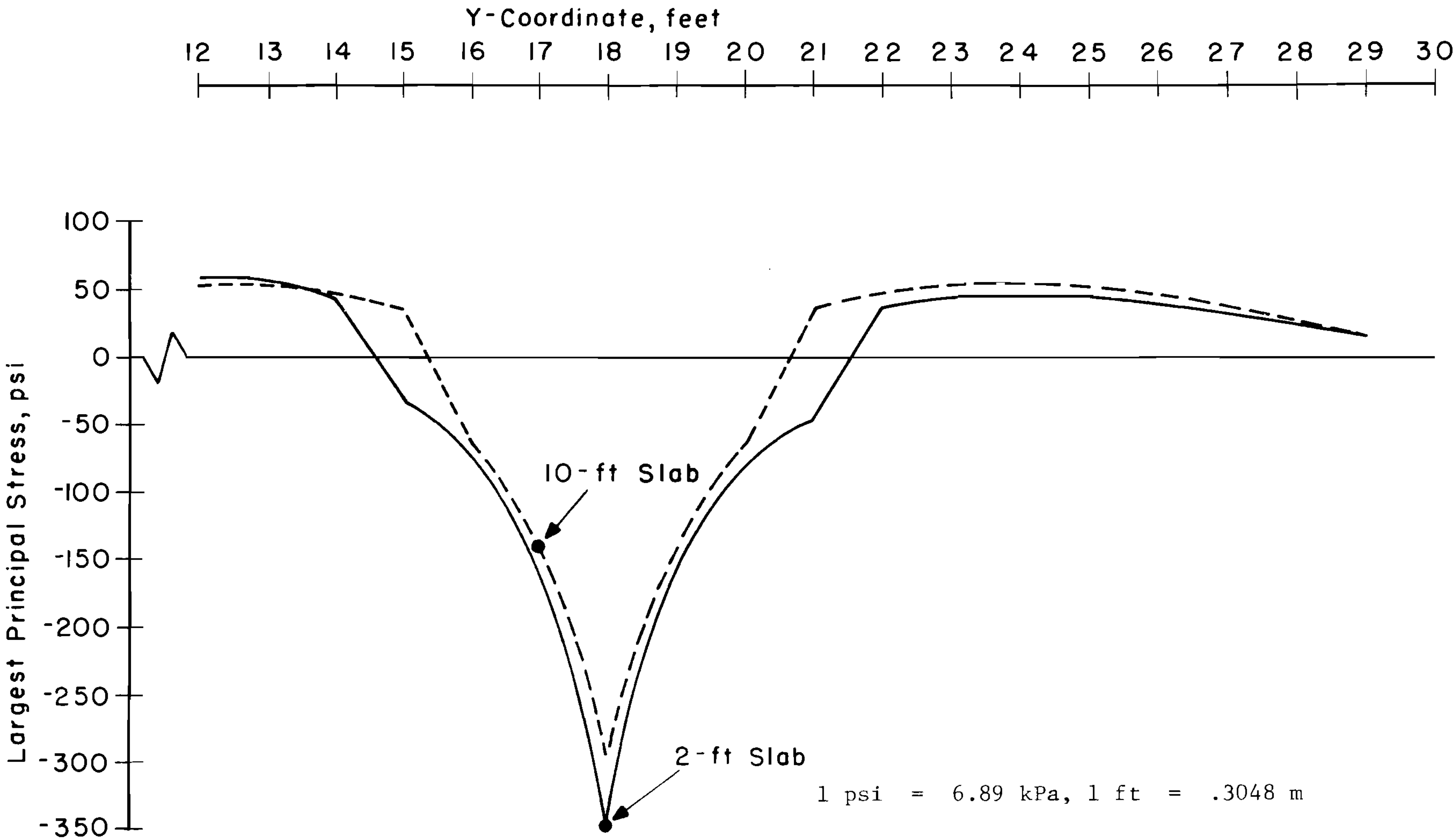


Fig A2.25. Largest principal stress versus Y-coordinate with two and 10-foot long slabs. X = 23.

Conclusion

The analysis performed here is intended to reveal possible problem areas in the use of a precast slab method of repair. Many simplifications are required to model a section of pavement. Particularly ignored here is the possible variability in the engineering properties of the pavement material and structure. The wheel loading was with two 9-kip (40-kN) point loads. These are static loads, whereas a pavement is subject to dynamic loading greater than these values. However, in light of these facts, the results of this analysis appear reasonable and satisfy its intentions.

The following conclusions can be drawn from this analysis:

(1) A high bending stiffness at the splice locations due to steel connections does not lead to excessive stress concentrations,

(2) a low bending stiffness should be avoided through a satisfactory connection mechanism,

(3) the length of the precast slab does not significantly affect wheel load stresses, although slabs shorter than approximately 6 feet (1.83 m) are more sensitive to design and construction errors due to their higher stress levels.

APPENDIX 3

LIFTING ANALYSIS FOR PRECAST CONCRETE SLABS

APPENDIX 3. LIFTING ANALYSIS FOR PRECAST REPAIR SLABS

Precast repair slabs must be lifted to be transported and installed. A simple lifting mechanism will result in a more economical repair than a more complex mechanism with extraneous hardware. The lifting mechanism studied here consists of four threaded inserts with attachable lifting plates. This mechanism is considered a simple method compared to a combination of steel I-beams to add stiffness to the slab during lifting.

The SLAB49 computer program was utilized to study the stresses and deflections induced into a concrete slab during lifting. The following properties were used in the concrete slab's modeling:

Modulus of elasticity of concrete	=	4.0×10^6 psi ($2.76 \times 10^{10} N/m^2$)
Poisson's ratio	=	0.2
Unit weight of concrete	=	150 lb/ft^3 (2403 kg/m^3)
Thickness	=	8 inches (203.2 mm)
Width	=	12 feet (3.66m)
Length	=	6 to 20 feet (1.83 to 6.10 m)

These properties were used as inputs into the SLAB49 program. An example listing of the inputs and modeling methodology used by SLAB49 for a 12 x 6 foot $(3.66 \times 1.83m)^2$ slab is shown in Fig A3.1.

The loading of the slab was accomplished by distributing the slab weight evenly over the slab. The lifting points were modeled by specifying subgrade support only at these points. The subgrade support reaction was set to an arbitrarily high value of 1×10^{15} in.-lb (1.75×10^{17} N-m). This resulted in a negligible deflection of 1.8×10^{-12} inches (4.572×10^{-11}mm) at these locations. Thus, a slab supported by cables or chains was simulated as a slab supported on incompressible stilts. The load pattern and support conditions are equivalent.

```
PROGRAM SLAB 49  -DEVELOPMENT  DECK-  MATLOCK,PANAK, ENDRES  REV DATE 13 JUL 71

          * * * * * * * * * * * * * * * * * * * * * * * * * * *
          *                                                   *
          *   THIS PROGRAM IS BEING USED AT YOUR OWN RISK.    *
          *   CHANGES MAY OCCUR AFTER THE ABOVE REVISION DATE. *
          *   PLEASE REPORT DIFFICULTIES TO THE ABOVE PEOPLE  *
          *   AT THE CENTER FOR HIGHWAY RESEARCH, UT AT AUSTIN. *
          *                                                   *
          * * * * * * * * * * * * * * * * * * * * * * * * * * *

PRECAST REPAIR SLAB DESIGN-MOMENTS,STRESSES,AND DEFLECTIONS INDUCED BY LIFTING
SLAB PROPERTIES- E=4.0E+06,THICKNESS=8IN.,UNIT WEIGHT=150LB/FT**3.-1/77-ELKINS


PROB
77.20      6FT LONG BY 12FT WIDE SLAB - LIFTING INSERTS AT 1/5 POINTS


TABLE 1.  CONTROL DATA
                                                            TABLE NUMBER
                                           2     3     4     5     6     7     8     9

     KEEP FROM PRECEDING PROBLEM (1=YES)  -0    -0    -0    -0    -0    -0    -0    -0
     NUM CARDS INPUT THIS PROBLEM          1     8     0     1     0     0     4     0

     MULTIPLE LOAD OPTION                 -0
     STATICS CHECK OPTION                 -0
     PRIN STRESS OPTION                   -0
     PROFILE PLOT OPTION                  -0
     3-D PLOT OPTION                      -0


TABLE 2.  CONSTANTS

     NUMBER OF INCREMENTS IN X DIRECTION                                           24
     NUMBER OF INCREMENTS IN Y DIRECTION                                           48
     INCREMENT LENGTH IN X DIRECTION                                        3.000E+00
     INCREMENT LENGTH IN Y DIRECTION                                        3.000E+00
     POISSONS RATIO                                                         2.000E-01
     SLAB THICKNESS                                                         8.000E+00


TABLE 3.  JOINT STIFFNESS AND LOAD DATA

   FROM    THRU        DX          DY          FX          FY          Q           S
   JOINT   JOINT

   0   0 24  48  4.445E+07  4.445E+07         -0          -0  1.563E+00          -0
   0   1 24  47  4.445E+07  4.445E+07         -0          -0  1.563E+00          -0
   1   0 23  48  4.445E+07  4.445E+07         -0          -0  1.563E+00          -0
   1   1 23  47  4.445E+07  4.445E+07         -0          -0  1.563E+00          -0
   5  10  5  10         -0         -0         -0          -0         -0   1.000E+15
  19  10 19  10         -0         -0         -0          -0         -0   1.000E+15
   5  38  5  38         -0         -0         -0          -0         -0   1.000E+15
  19  38 19  38         -0         -0         -0          -0         -0   1.000E+15
```

(Continued)

Fig A3.1. Example listing of SLAB 49 input variables.

(Continued)

TABLE 4. JOINT STIFFNESS AND LOAD DATA CONTD

FROM JOINT	THRU JOINT	RX	RY	TX	TY

NONE

TABLE 5. MESH STIFFNESS DATA

FROM MESH	THRU MESH	C
1 1	24 48	1.422E+08

TABLE 6. BAR STIFFNESS DATA

FROM BAR	THRU BAR	PX	PY	PBX	PBY

NONE

TABLE 7. MULTIPLE LOAD DATA

FROM JOINT	THRU JOINT	QM

NONE

TABLE 8. PROFILE OUTPUT AREAS

FROM JOINT	THRU JOINT	DEFL (1=YES)	X MOMENT (1=SLAB,2=BEAM)	Y MOMENT	PRIN MOM OR STRESS (1=YES)
5 0	5 48	1	1	1	1
19 0	19 48	1	1	1	1
0 10	24 10	1	1	1	1
0 38	24 38	1	1	1	1

TABLE 9. PRINTED OUTPUT LIMITS

FROM Y STA	THRU Y STA

NONE

Fig A3.1. Example listing SLAB 49 input variables. (Continued)

To select starting points for the positions of the lifting points, an analysis of a simple supported beam with a uniform distributed load was performed (Fig A3.2). The distance from the end of the beam to the support b was varied between 0 and $\frac{L}{2}$. Moments at the support and the center of the span were computed with L = 1.0 and q = 100. A plot of these moments versus $\frac{b}{L}$ is shown in Fig A3.2.

Two points can be selected from this simplified analysis as starting points for the slab analysis. The support at approximately $\frac{1}{5}$ the span length resulted in equal and opposite moments at the support and midspan. The other point is $\frac{1}{4}$ the span length. Here, the moment at midspan is zero and the sum of squares of the two moments plotted in Fig A3.2 is a minimum.

The slab analysis began with a 6 x 12 foot (1.83 x 3.66 m) slab. For this slab size, it was convenient to use 3-inch (76.2 mm) increments so that the $\frac{1}{5}$ and $\frac{1}{4}$ span lifting poings would coincide, or nearly coincide, with nodal points in the slab model. The $\frac{1}{6}$ span length also coincided with nodal points and was included in the analysis of this size slab.

Three runs of SLAB49 were made using the above three lifting positions. The output of these runs was used to plot the moment profiles in Figs A3.3 A3.4, and A3.5. The moments plotted are the largest principal moments from a Mohr's circle analysis. Stresses in the extreme fibers of the slab can be computed by the common expression

$$\sigma = \frac{Mc}{I}$$

where

σ = stress in extreme fiber of slab,

M = moment acting on particular section,

c = distance to extreme fiber $(\frac{h}{2})$,

I = moment of inertia per unit width of slab $(\frac{h^3}{12})$, and

h = slab thickness.

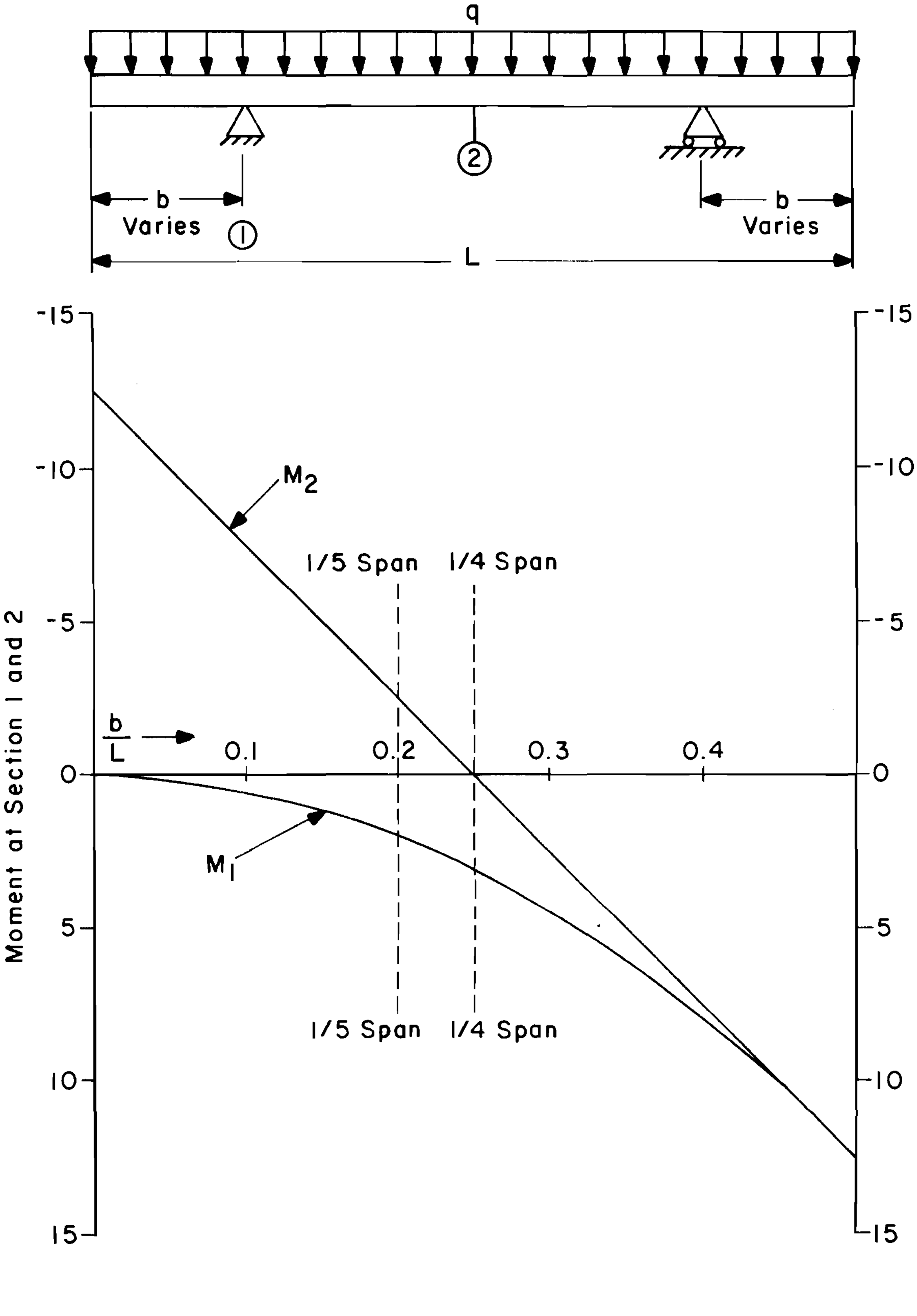
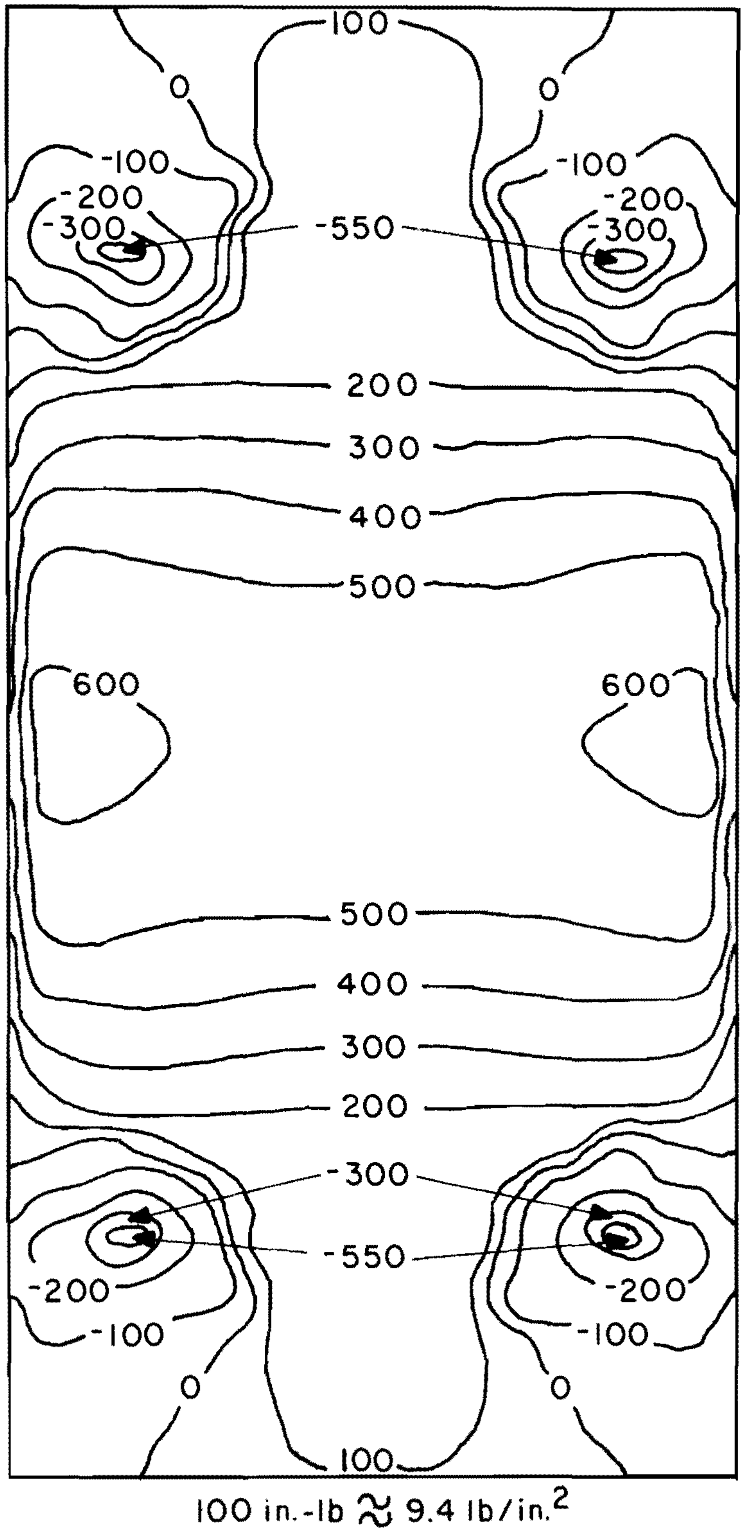
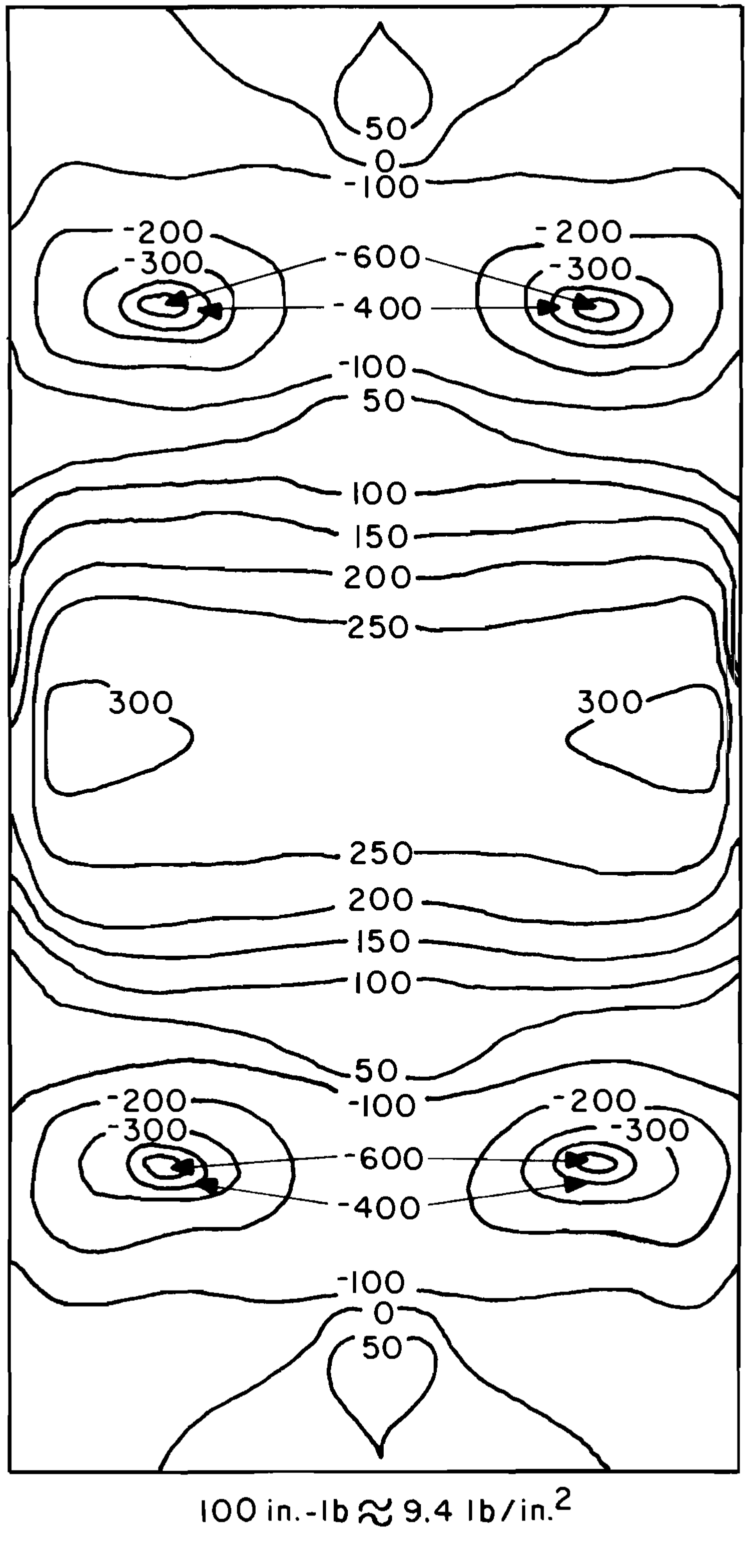


Fig A3.2. Moment analysis of simply supported beam with uniformily distributed load.



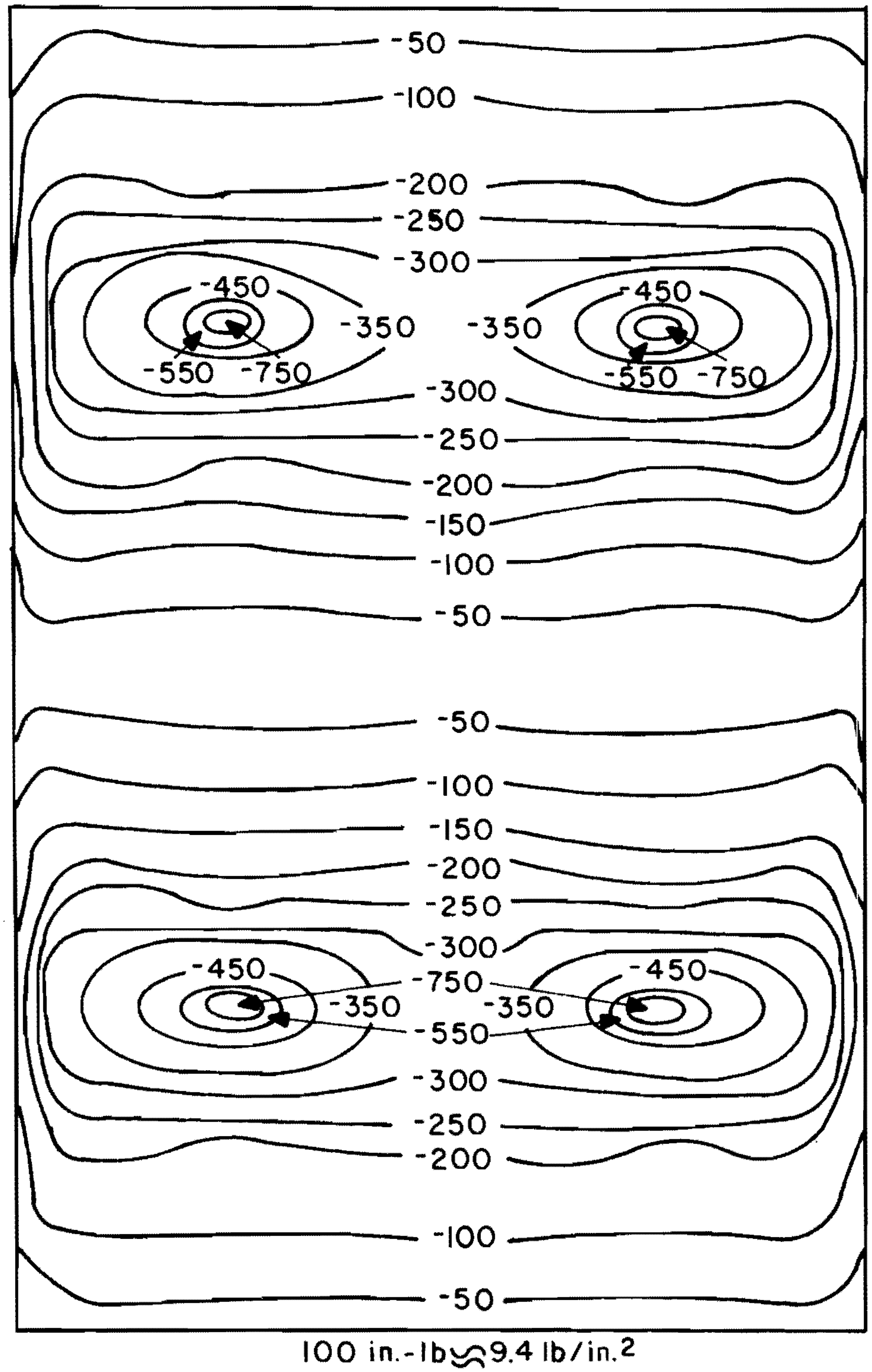
1 in.-lb = .11 N-m 1 psi = 6.89 kPa 1 ft = .30 m

Fig A3.3. Principal moments in a 6x12-foot slab with lift point at 1/6 span.



1 in.-lb = .11 N-m 1 psi = 6.89 kPa 1 ft = .30 m

Fig A3.4. Principal moments in a 6 X 12-foot slab with lift point at 1/5 span.



1 in.-lb = .11 N-m   1 psi = 6.89 kPa   1 ft = .30 m

Fig A3.5 Principal moments in a 6 X 12-foot slab with lift points at 1/4 span.

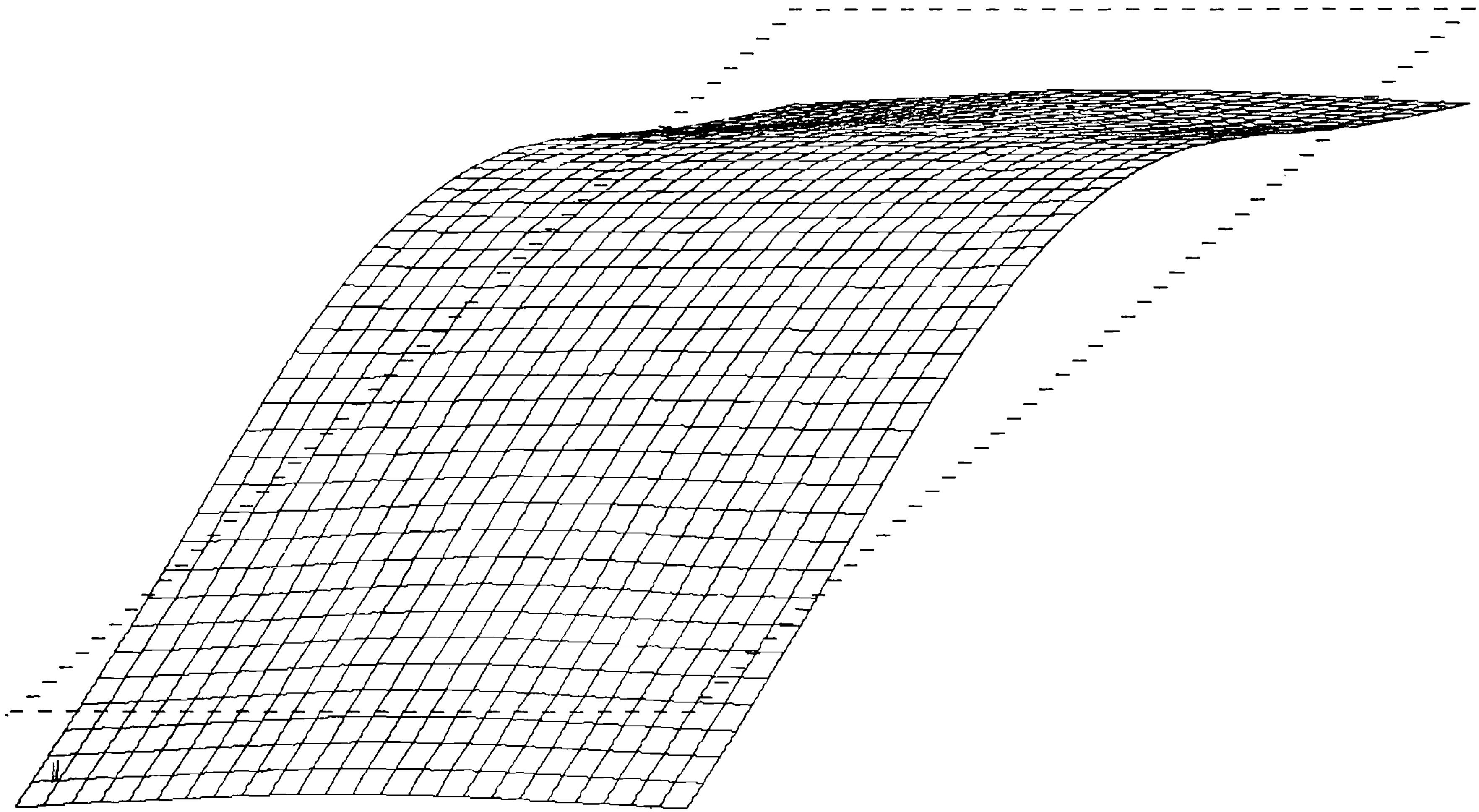


Fig A3.6. Distorted 3-D deflection profile for 6 x 12-foot (1.83 x3.66 m) concrete slab with lifting points at 1/6 span.

Substituting for c and I as indicated above reduces the equation to

$$\sigma = \frac{6M}{h^2}$$

Thus, for a constant slab thickness, principal stress is directly proportional to principal moment. The distribution of principal moment also represents the distribution of principal stress. For the slabs analyzed here, h = 8 inches (203.2 mm), and the principal stress is 0.094 times the principal moment. A positive moment results in a tensile stress in the lower fiber.

This distribution and magnitude of moments, and stress, was found to be quite sensitive to the location of lifting points. Comparison of Figs A3.3, A3.4, and A3.5 shows a decrease in principal moment at the center of the slab of 300 inch-pounds (33.9 N-m) for a change from $\frac{1}{6}$ to $\frac{1}{5}$ and $\frac{1}{5}$ to $\frac{1}{4}$ span. These shifts in the position of the lifting point amount to only a 5 and 8-inch (127 and 203-mm) movement, respectively.

Figures A3.6, A3.7, and A3.8 are pseudo three-dimensional deflection profiles. These plots readily exhibit curvature and relative deflection of the slabs for the lifting options. The magnitudes of each deflection plot have been scaled to fit into a standard plot dimension. A distorted image of the actual profile is, therefore, produced for macroscopic inspection. The cross marks which form a rectangle on the plots represent the reference plane indicating zero deflection.

The deflection plot for lifting points at $\frac{1}{6}$ span, shown in Fig A3.6, indicates a downward deflection in the center of the slab, with the ends deflected upward at a fairly straight slope. The profile in Fig A3.7 for the $\frac{1}{5}$ span option indicates downward deflection in the center, with the ends deflected above the reference plane and nearly parallel to it. The deflection profile for the $\frac{1}{4}$ span option in Fig A3.8 shows an upward deflection in the center of the slab, with a slight curvature to the ends of the slabs which are deflected below the reference plane.

Ideally, the lifting mechanism should produce minimum stresses throughout the slab. In any case, the expected stresses should not exceed the

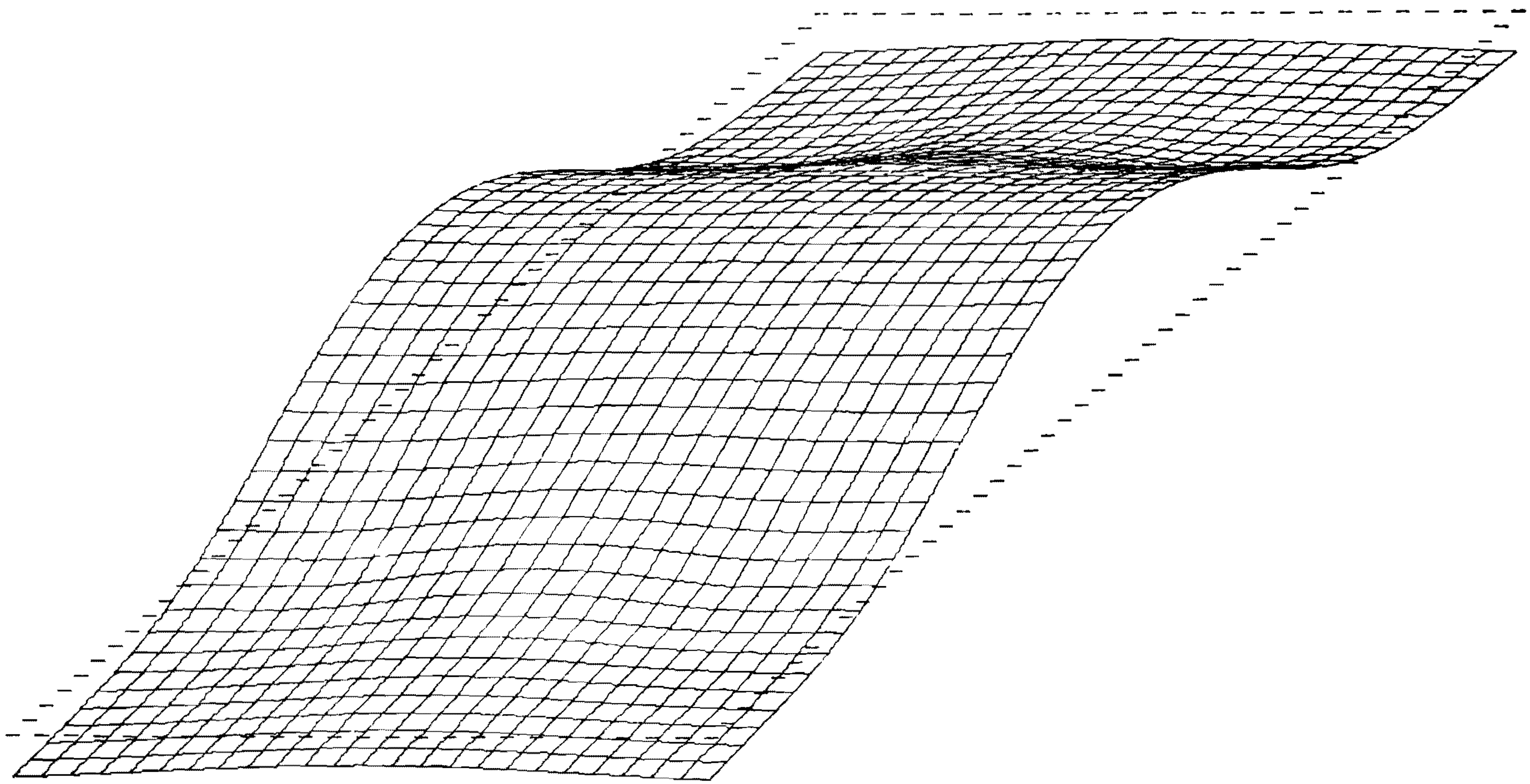


Fig A3.7. Distorted 3-D deflection profile for 6 x 12-loot (1.83 x 3.66 m) concrete slab with lifting points at 1/5 span.

77.20

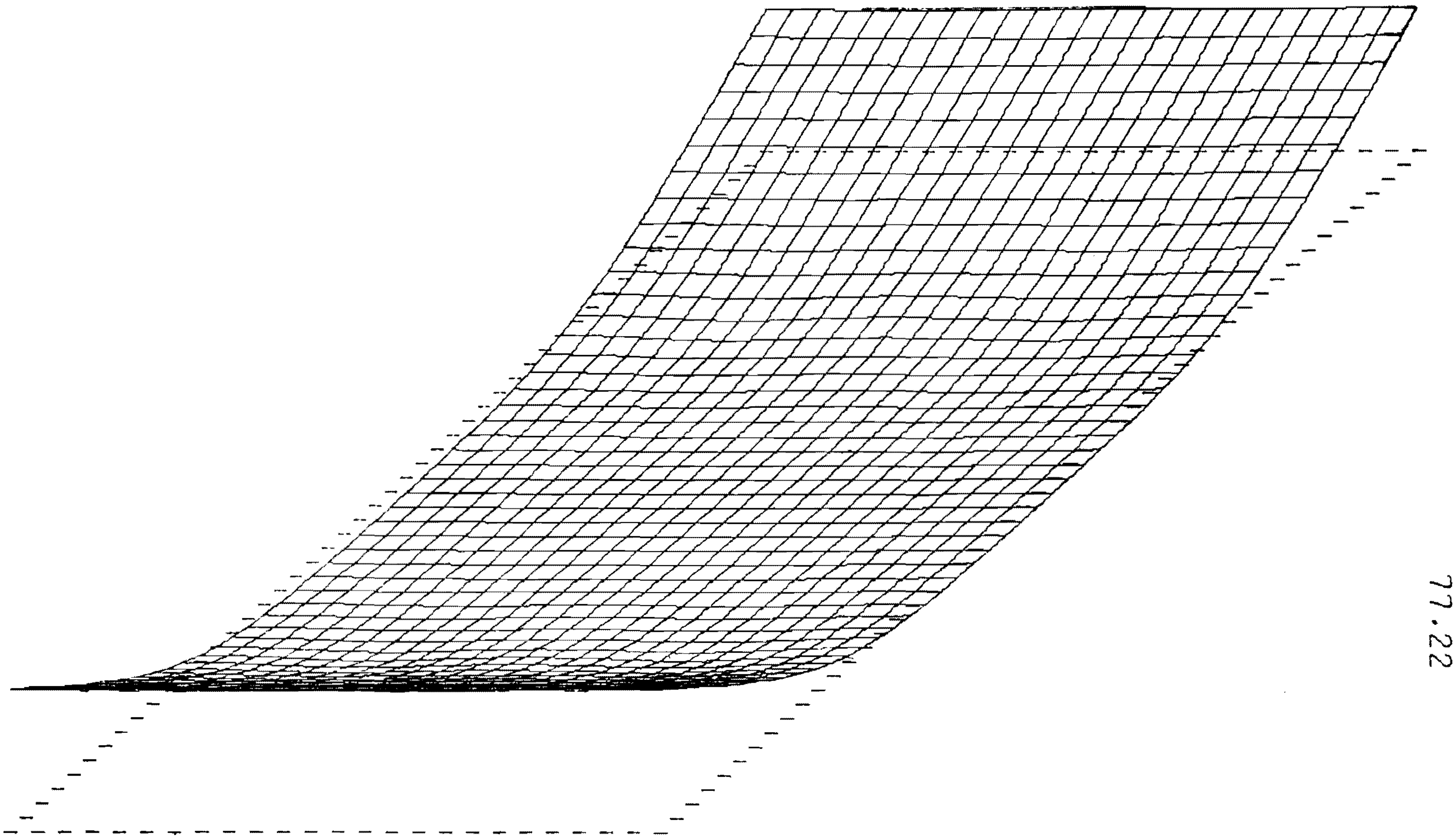


Fig A3.8. Distorted 3-D deflection profile for 6 x 12 foot (1.83 x 3.66 m) concrete slab with lifting points at 1/4 span.

allowable tensile strength of the concrete. Other considerations are the position of the largest stresses and possible reinforcement requirements.

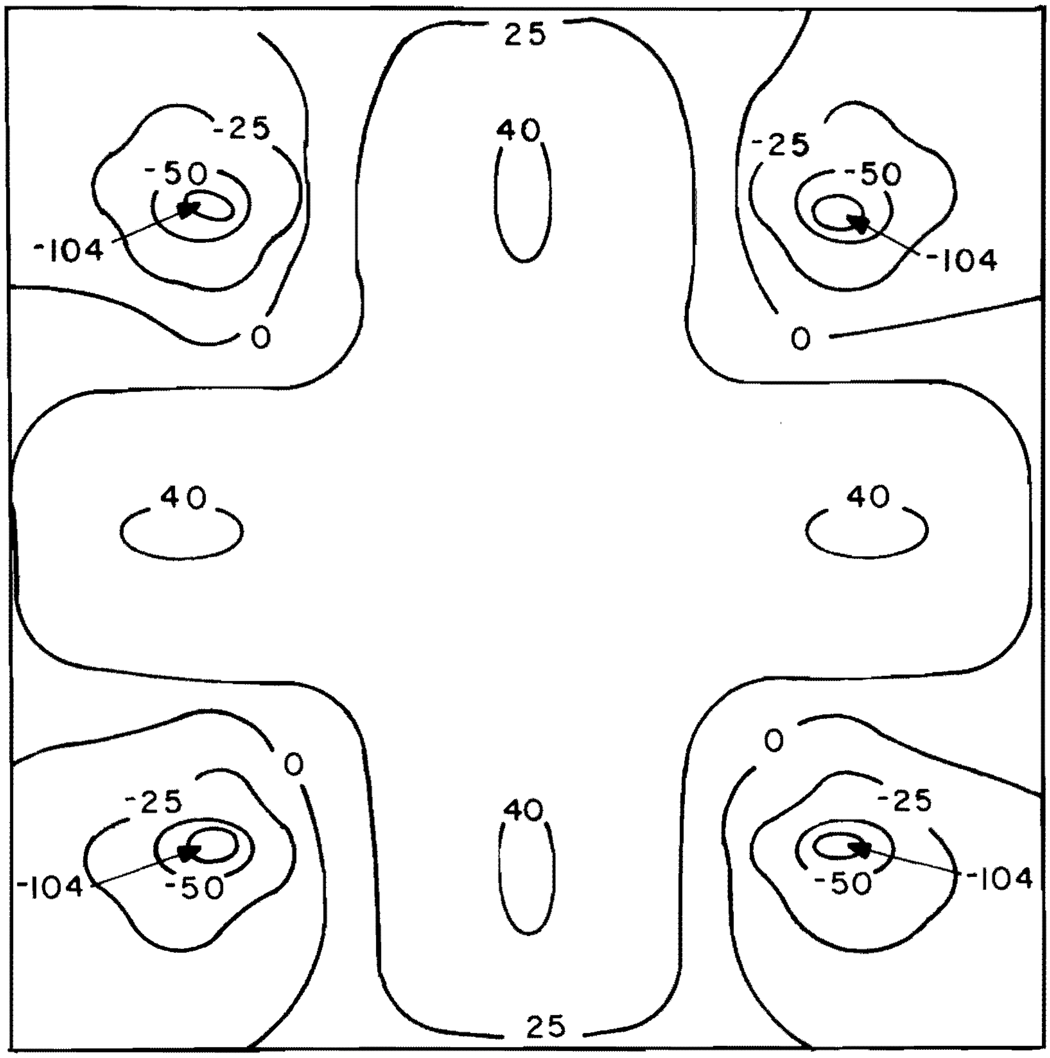
The stresses in the slab with lifting points at $\frac{1}{6}$ span were larger in the central interior portion of the slab and smaller at the supports than for the other positions analyzed. The stresses in the center of the slab for the $\frac{1}{6}$ span option were approximately twice those of the $\frac{1}{5}$ span option. The stresses at the support were about 8 percent less for the $\frac{1}{6}$ span option.

The $\frac{1}{5}$ span lifting option resulted in stresses at the center and supports that were greater and less for the same stresses in the $\frac{1}{4}$ span option, respectively. Stresses in the extreme fibers at the center of the slab were approximately 28 psi (193 kPa) for the $\frac{1}{5}$ span option and one psi (6.89 kPa) for the $\frac{1}{4}$ span option. Thus, stresses in the center of the slab for the $\frac{1}{4}$ span option were only 4 percent of those of the $\frac{1}{5}$ span placement. At the supports, the stresses for the $\frac{1}{4}$ span option were 25 percent greater.

At this point, the $\frac{1}{6}$ span placement of the lifting points was removed from further consideration. The stresses in the center of the slab were much greater than the other options and only a slight reduction in stress at the supports was observed.

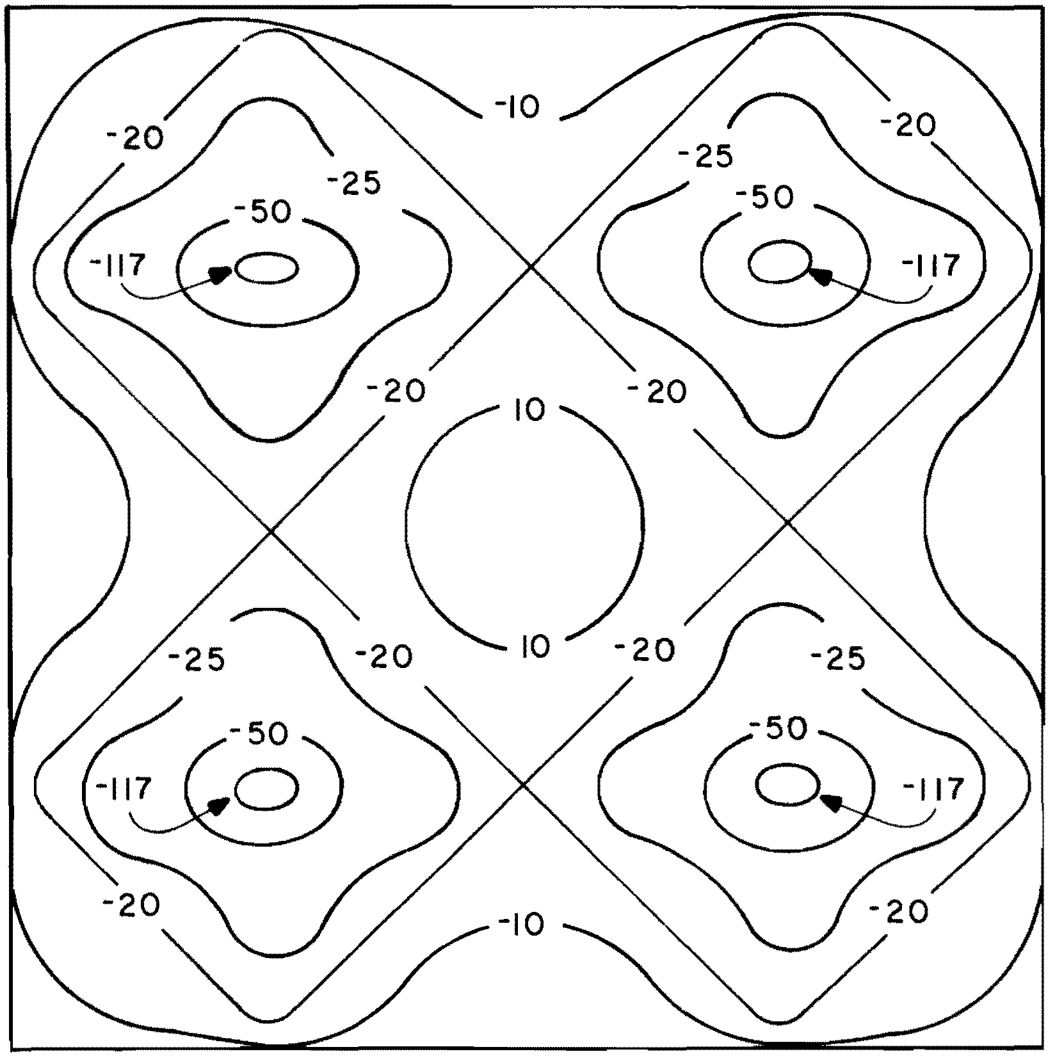
To further investigate the $\frac{1}{5}$ and $\frac{1}{4}$ span placement of the lifting points, 12 X 12-foot (3.66 X 3.66-m) and 12 X 14-foot (3.66 X 4.27-m) slabs were modeled and analyzed. The procedure previously described was also employed here. The moment-stress contours and three-dimensional deflection profiles are presented in Figs A3.9, A3.10, A3.11, A3.12, A3.13, A3.14, A3.15, and A3.16.

The same general distribution of stresses and deflections as seen in the 6 X 12-foot (1.83 X 3.66-m) slab was observed for these slabs. Stresses in the center region of the slab supported at the $\frac{1}{4}$ span positions were roughly twice those of the $\frac{1}{5}$ position. The largest principal stress occurred at support for the $\frac{1}{4}$ span position. For the 12 X 14-foot (3.66 X 4.27-m) slab, this stress was 153 psi. This is approximately 20 percent greater than the same stress for the $\frac{1}{5}$ position.



Principal Stress, psi (1 psi = 6.89 kPa)

Fig A3.9. Stresses in a 12 X 12-foot (3.66 X 3.66-m) slab with lift points at 1/5 span.



Principal Stress, lb/in.^{2} = 6.89 kPa

Fig A3.10. Stresses in a 12 X 12-foot (3.66 X 3.66-m) slab with lift points at 1/4 span.

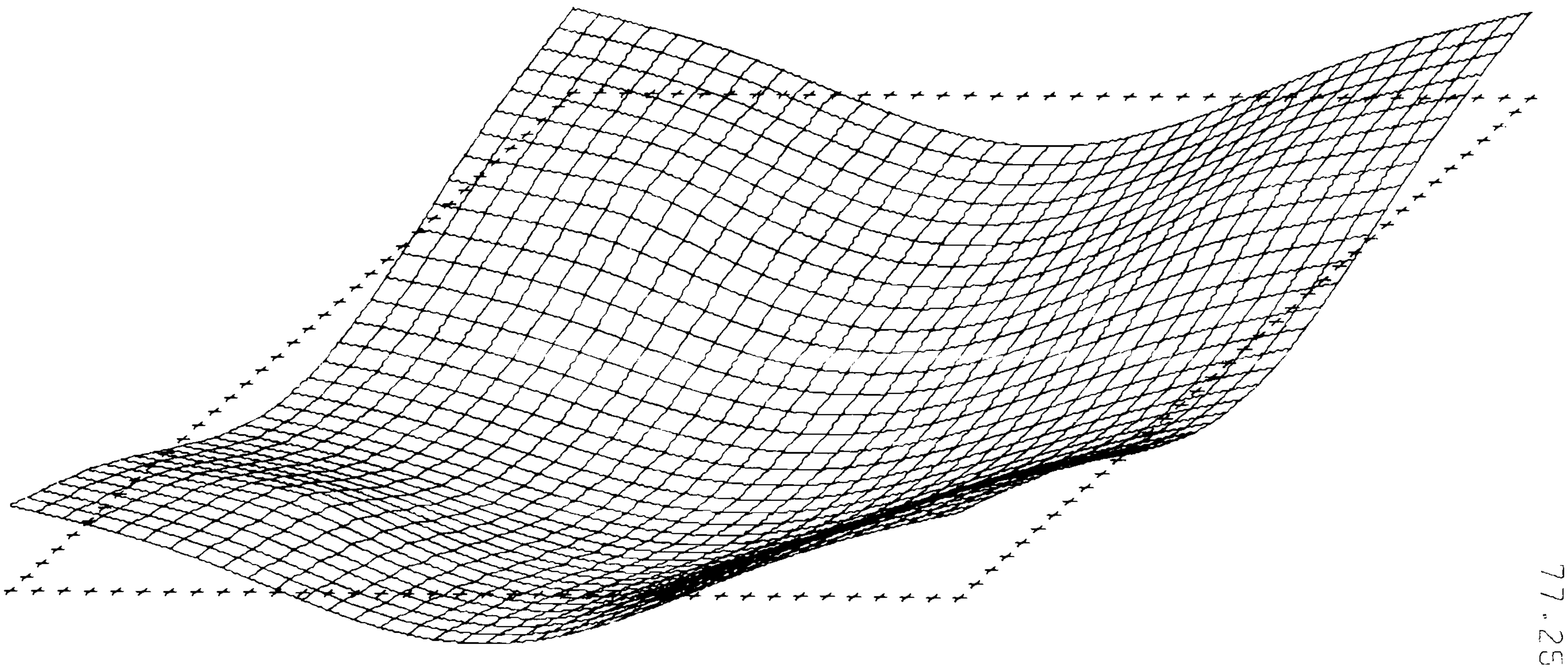


Fig A3.11. Distorted 3-D deflection profile for 12 X 12-foot (3.66 X 3.66-m) slab with lift points at 1/5 span.

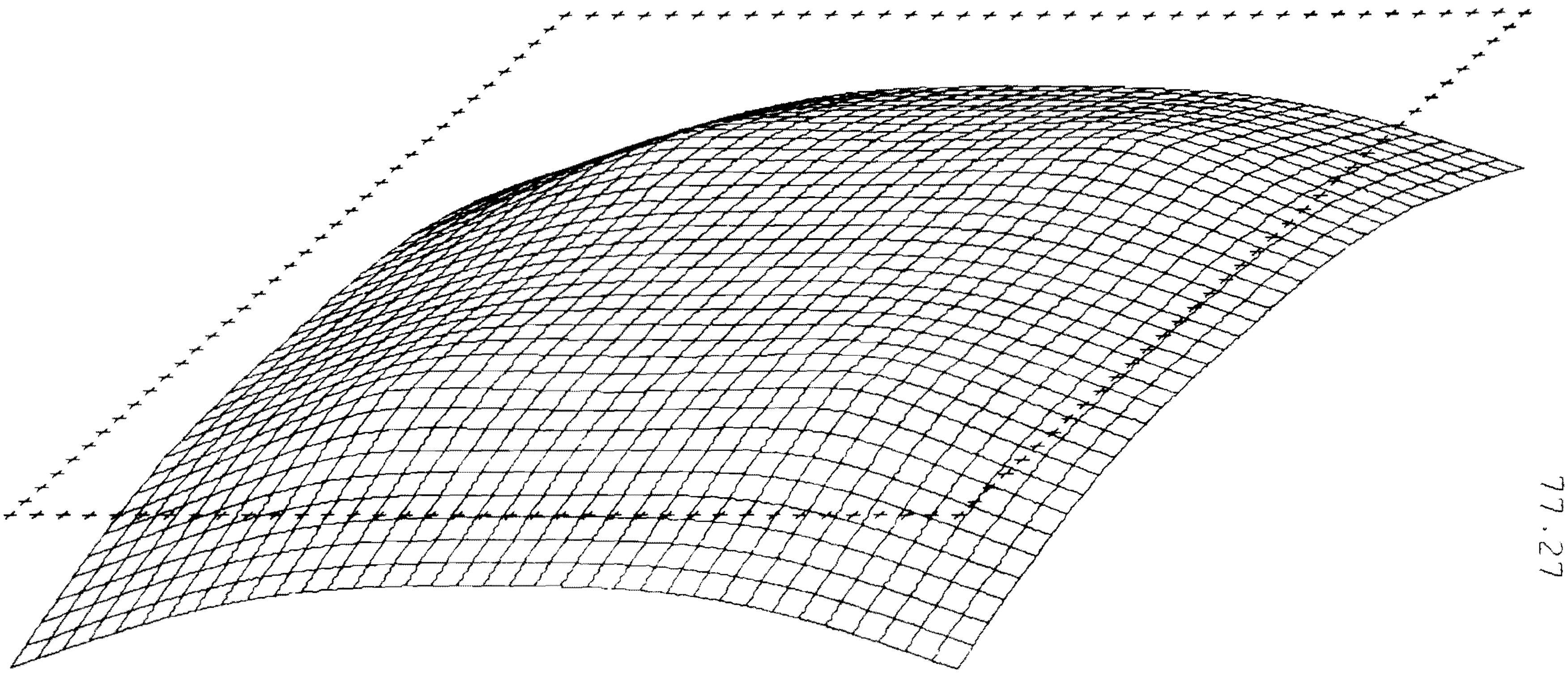
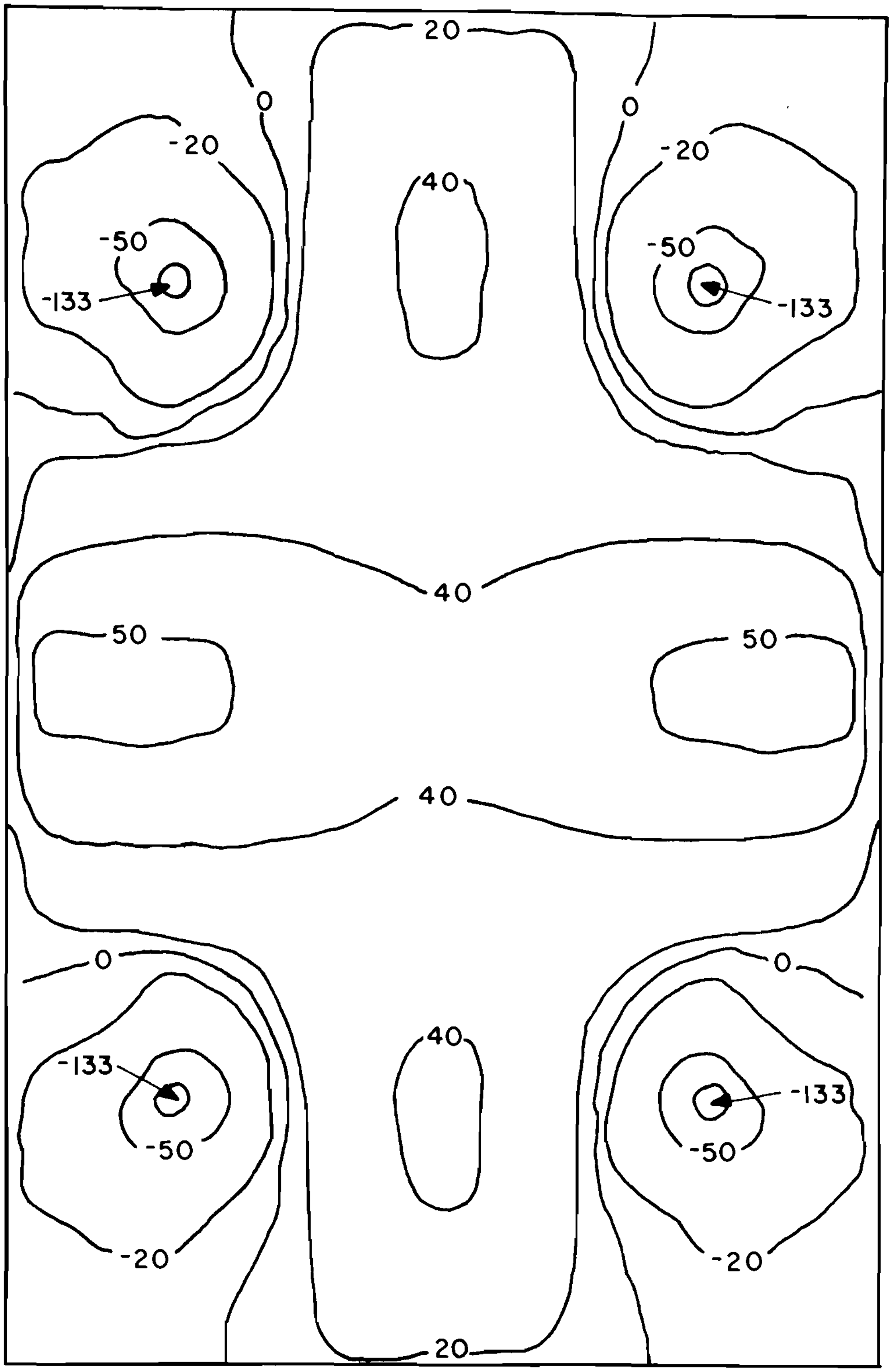
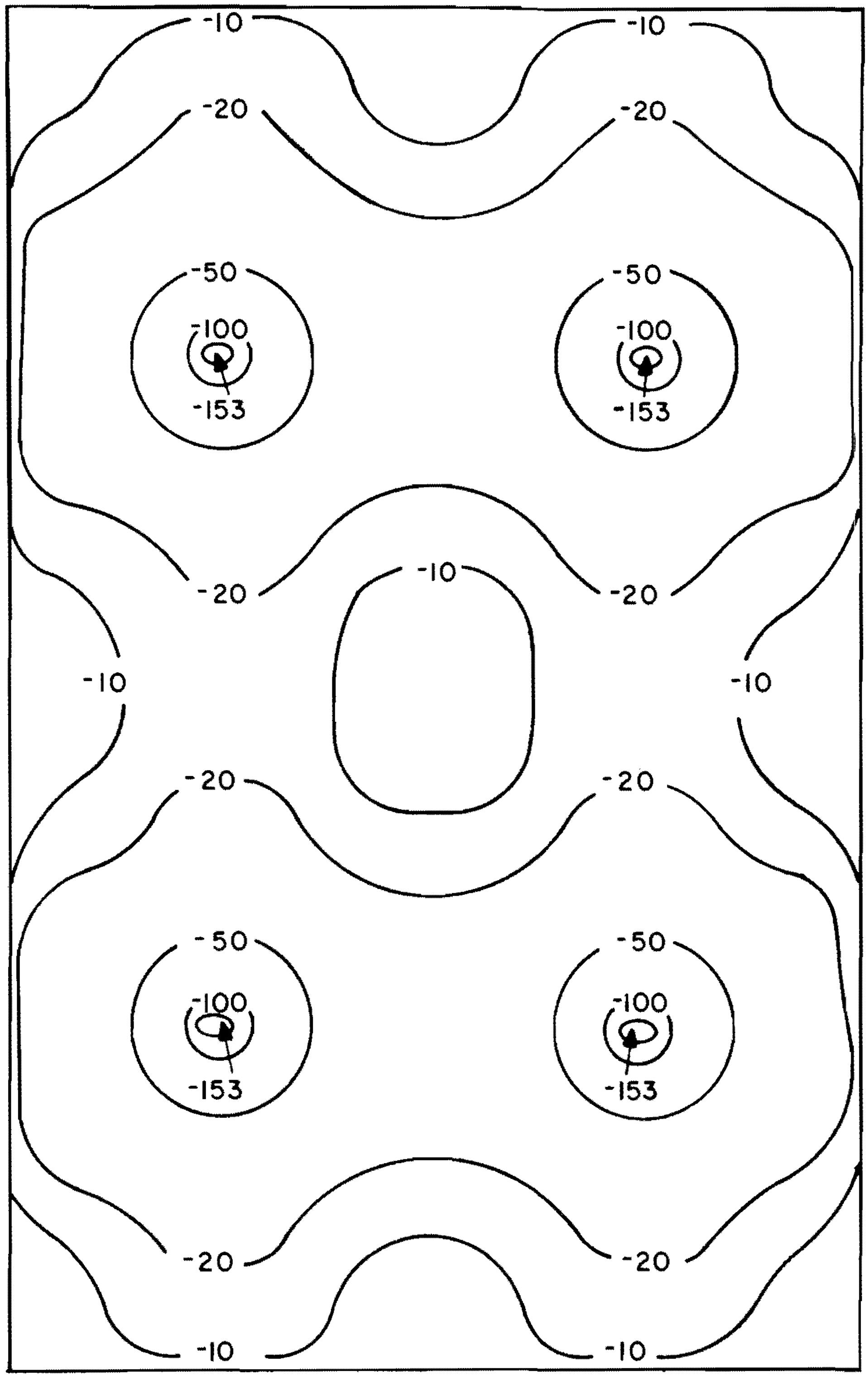


Fig A3.12. Distorted 3-D deflection profile for 12 X 12 foot (3.66 X 3.66-m) slab with lift points at 1/4 span.



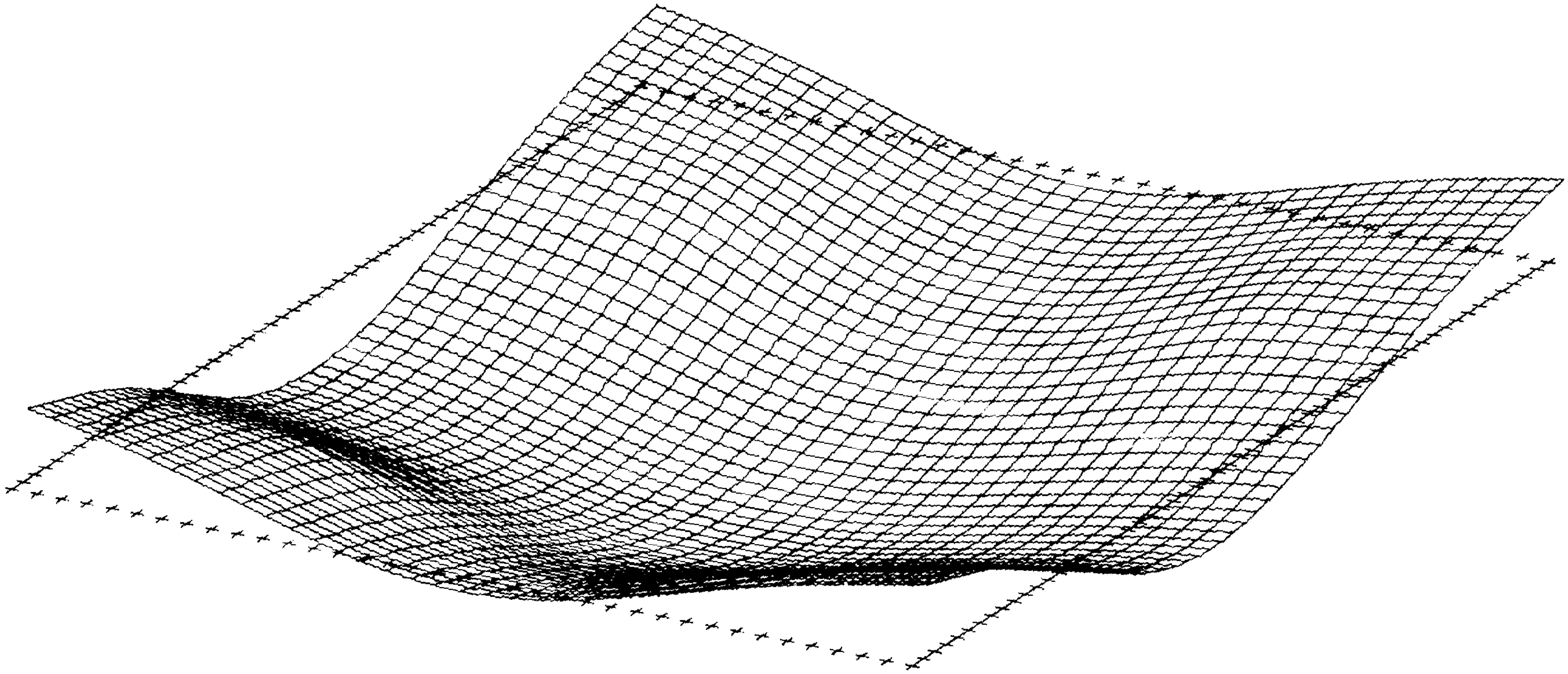
Principal Stresses, psi (1 psi = 6.89 kPa)

Fig A3.13. Stresses in a 12 x 14-ft. (3.66 x 4.27-m) slab with lift points at 1/5 span.



Principal Stresses, psi (1 psi = 6.89 kPa)

Fig A3.14. Stresses in a 12 x 14-ft. (3.66 x 4.27-m) slab with lift points at 1/4 span.



77.28

Fig A3.15. Distorted 3-D deflection profile for 12 X 14-foot (3.66 X 4.27-m) slab with lift points at 1/5 span.

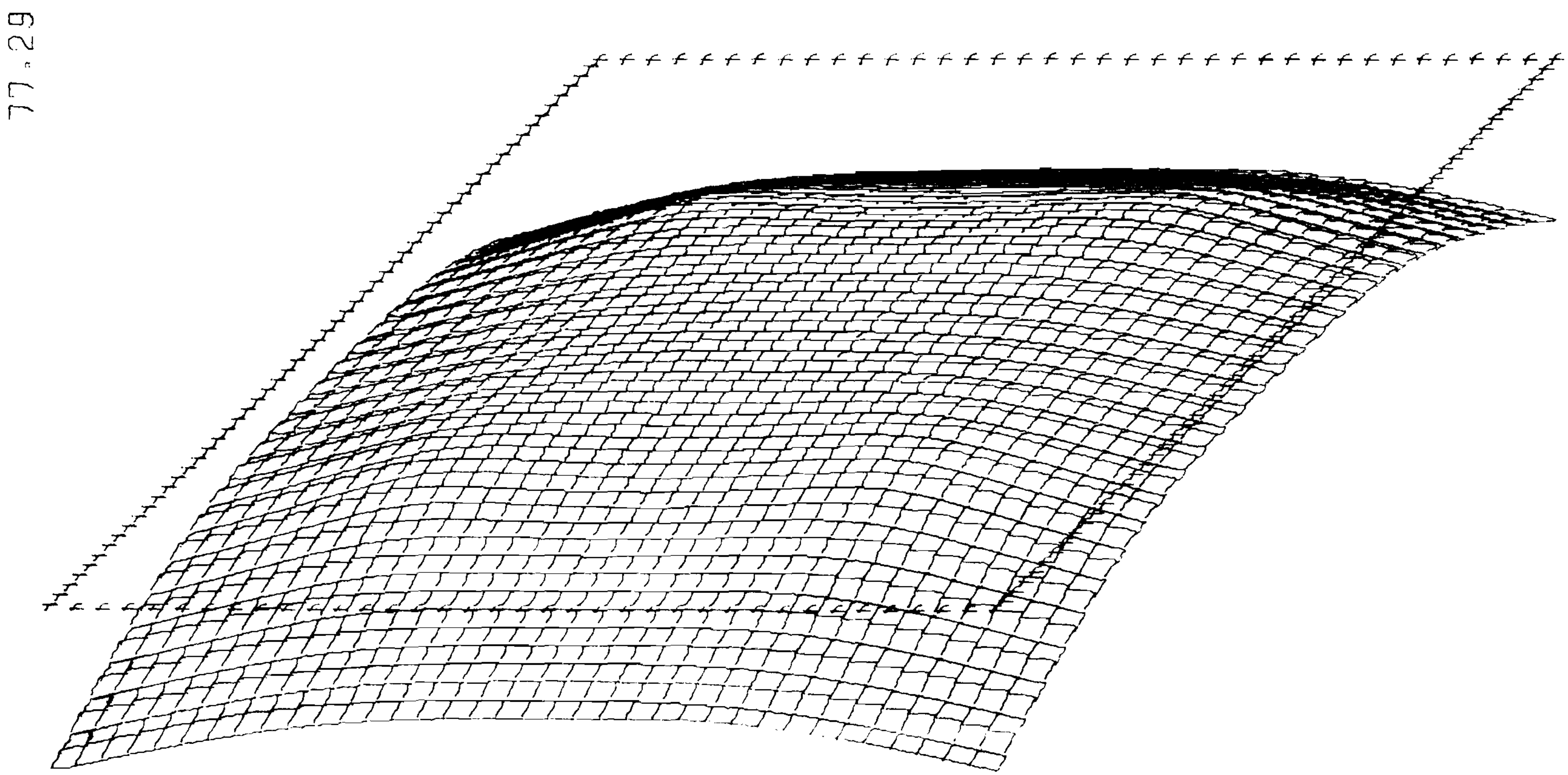


Fig A3.16. Distorted 3-D deflection profile for 12 X 14-foot (3.66 X 4.27-m) slab with lift points at 1/4 span.

To guard against cracking, required concrete strengths can be derived based on the calculated stresses, taking dynamic effects and material variability into account. The methodology for arriving at the required concrete strength is illustrated in Fig A3.17. The design stress, which accounts for dynamic effects, is calculated at twice the stress calculated by SLAB49. Given the standard deviation of concrete strength, and the number of samples used to evaluate the strength, the required mean concrete strength is found by adding the product of the t-statistic and the concrete strength's standard deviation to the design stress. This is expressed in equation form as

$$\bar{f} = \sigma_{design} + t(n-1,\alpha)S_{\bar{f}} = 2\sigma_{static} + t(n-1,\alpha)S_{\bar{f}}$$

where

$\bar{f}$ = required mean concrete strength,

σ_{design} = design stress = $2\sigma_{static}$,

σ_{static} = calculated stress by SLAB-49,

$t(n-1,\alpha)$ = t-statistic corresponding to $n-1$ degrees of freedom, at the α percentage level,

n = sample size,

α = percentage of concrete strengths less than design stress, and

$S_{\bar{f}}$ = standard deviation of concrete strength.

The required mean concrete strengths for a range of slab sizes, range of standard deviations of concrete strengths, and the $\frac{1}{5}$ and $\frac{1}{4}$ lift options are provided in Tables A3.1 and A3.2. The strengths indicated in these tables are based on a sample size of 10 at a 5 percent probability level. Table A3.1 lists the required flexural strengths measured by third point loading. Table A3.2 lists the corresponding compressive strengths based on the correlation

$$f_R = 7.5\sqrt{f'_c} \qquad \text{(Ref 43)}$$

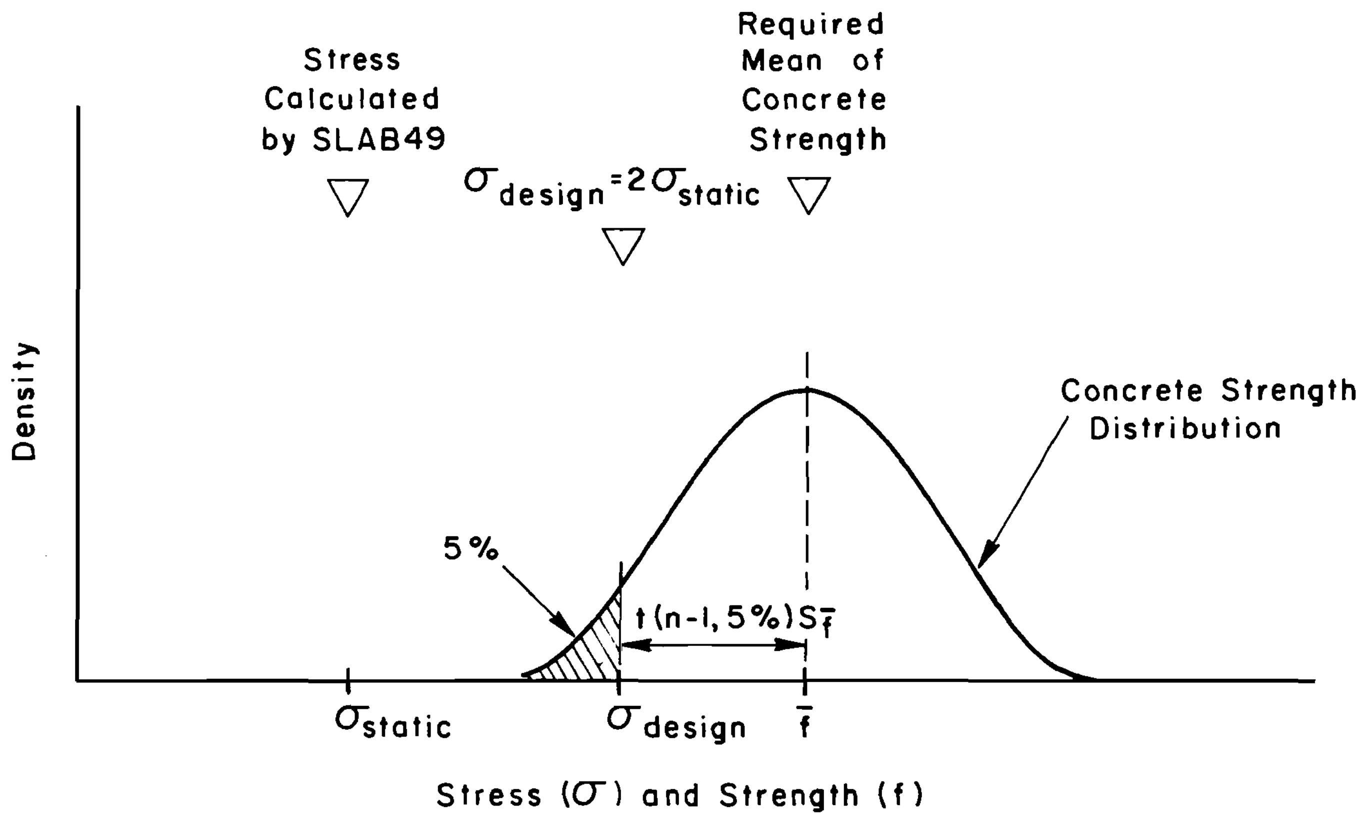


Fig A3.17. Selection of required concrete strength for lifting concrete slabs.

TABLE A3.1. REQUIRED MEAN FLEXURAL STRENGTHS FOR LIFTING SLABS (psi), (THIRD POINT LOADING) BASED ON SAMPLE SIZE OF 10.

Slab Size, ft	Standard Deviation of Conc. Strength, psi	Lift Points	
		$\frac{1}{5}$	$\frac{1}{4}$
12 X 6	20	150	178
	40	186	214
	60	223	250
12 X 12	20	245	270
	40	281	307
	60	318	344
12 X 14	20	303	343
	40	339	379
	60	376	416
12 X 20	20	407	502
	40	443	539
	60	480	576

1 ft = .305 m 1 psi = 6.89 kPa

TABLE A3.2. REQUIRED MEAN COMPRESSIVE STRENGTHS FOR LIFTING SLABS (psi), BASED ON SAMPLE SIZE OF 10.

Slab Size, ft	Standard Deviation of Conc. Strength, psi	Lift Points	
		1/5	1/4
12 X 6	20	400	560
	40	615	815
	60	885	1100
12 X 12	20	1100	1300
	40	1400	1700
	60	1800	2100
12 X 14	20	1650	2100
	40	2050	2550
	60	2500	3100
12 X 20	20	2900	4500
	40	3500	5200
	60	4100	5900

1 ft = .305 m   1 psi = 6.89 kPa

where

f_R = flexural strength, psi, and

f'_c = compressive strength, psi.

The strengths listed in these tables are applicable to the first lifting of a concrete slab. When the concrete gains the strengths indicated here, the slab can be lifted with a high degree of assurance that it will not crack. If smaller sample sizes are used to determine the appropriate time the concrete has reached sufficient strength, then a new t-statistic needs to be calculated. A listing of the stresses calculated by SLAB49 is presented in Table A3.3. The minimum desired sample size is six specimens.

It has been shown in this analysis that the magnitude and distribution of stresses induced into a slab by lifting are **very** sensitive to the location of the supports. For the simple lifting scheme investigated here, tables of required concrete strengths have been derived. These strengths are within the range of those easily obtainable with portland cement concrete. A more elaborate lifting mechanism does not appear to be warranted.

TABLE A3.3. CALCULATED STATIC TENSILE STRESSES FOR LIFTING CONCRETE SLABS (psi).

Slab Size feet	Lift Points	
	$\frac{1}{5}$	$\frac{1}{4}$
12 X 6	56.4	70.5
12 X 12	104	117
12 X 14	133	153
12 X 20	185	233

APPENDIX 4

INVESTIGATION OF VOIDS BENEATH PRECAST REPAIR SLABS

APPENDIX 4. INVESTIGATION OF VOIDS BENEATH PRECAST REPAIR SLABS

Some distress in CRCP has been attributed to the presence of voids beneath the pavement. This occurrence is sometimes associated with pumping of the subbase layer immediately below the slab. In any instance, the loss of subgrade support beneath a slab results in higher stress levels, accompanied with a shorter fatigue life.

The response of a precast patch to the presence of voids was investigated with the SLAB49 computer program. The precast slab modeling methodology used here is similar to that used in the stress concentration analysis. The assumed physical properties of the precast slab were

length of precast slab	=	6 ft (1.83 m),
width of precast slab	=	12 ft (3.66 m),
total width of pavement section	=	24 ft (7.32 m),
thickness of precast slab and CRCP	=	8 in. (203.2 mm),
modulus of elasticity of concrete	=	4×10^6 psi (6.41×10^7 kPa),
modulus of elasticity of steel connection	=	10×10^6 psi (1.60×10^8 kPa),
Poisson's ratio	=	0.2, and
K value of subgrade support	=	150 pci (4.15×10^6 kg/m^3).

Loading used in this investigation consisted of two 9-kip point (40-kN) loads positioned 6 ft (1.83 m) apart. Several void configurations and load placements, as shown in Fig A4.1, were investigated.

The void configurations in Fig A4.1 were chosen to relate to conditions which may occur beneath a precast patch. Case 1 is shown as a triangular void occurring beneath the steel connections. The actual modeling of this void is accomplished by using support values at each node which are reduced

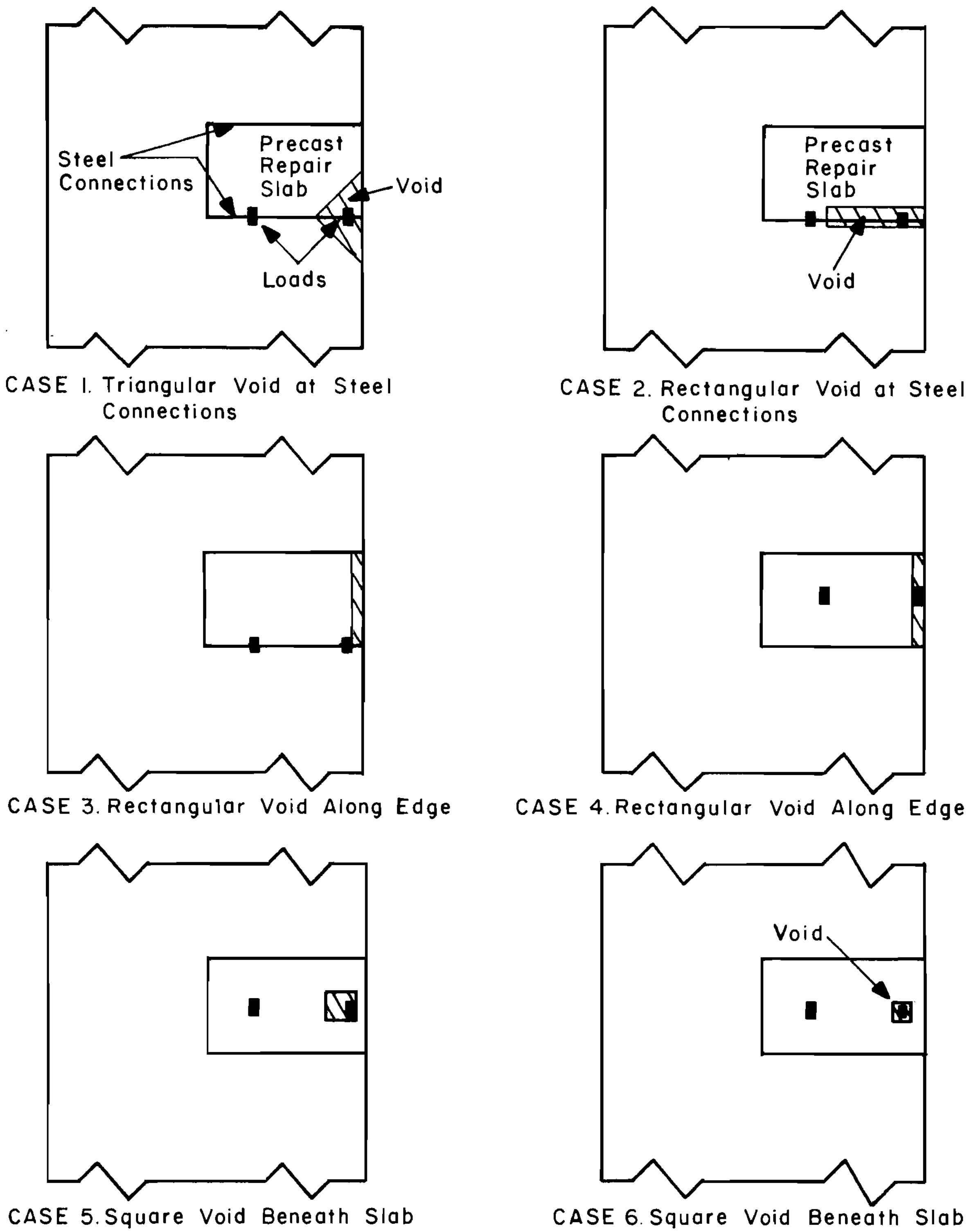


Fig A4.1. Void and load configurations investigated.

proportionally to the area of void which falls within that node. For example, if the void covers 25 percent of a node, then the support value for that node is reduced by 25 percent. Thus, a triangular representation of the void is not precisely correct; however, it is indicative of the distribution of support. This case can be physically interpreted as a void produced by pumping, with the outside edge larger than the inside edge.

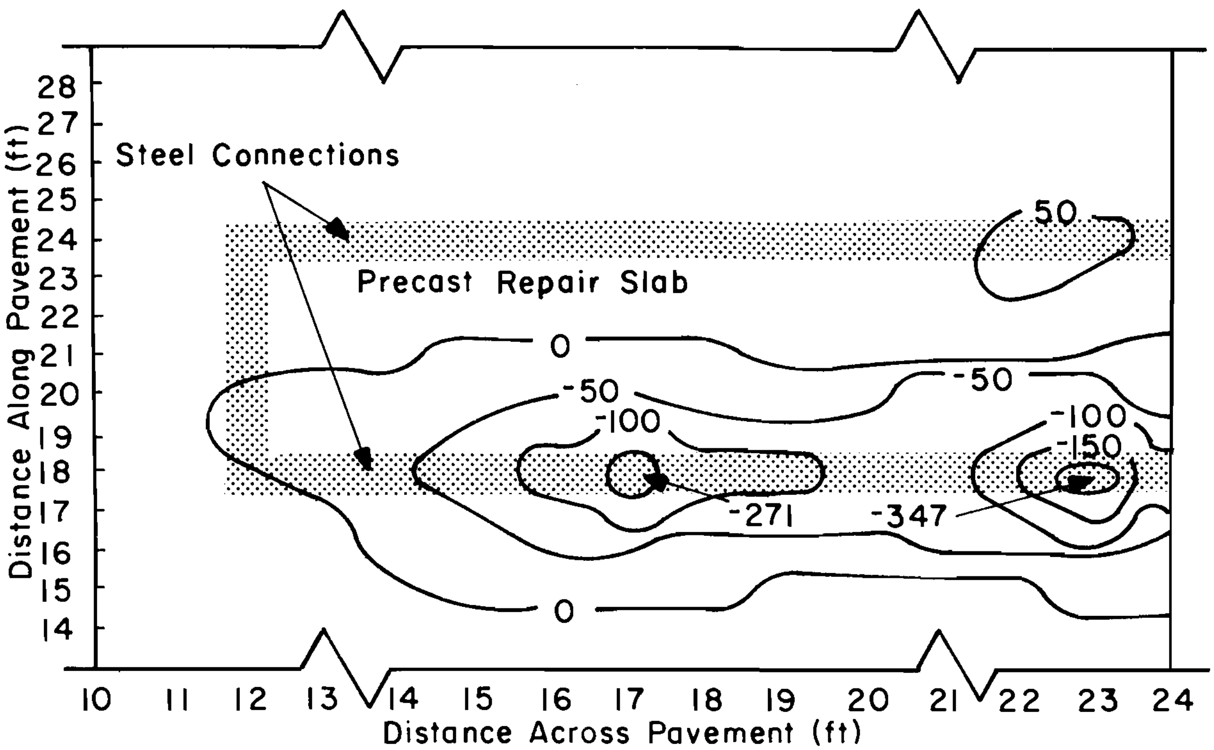
Case 2 was constructed to investigate the effect of no support at the steel connections. Cases 3 and 4 are voids along the edge of the precast slab which resemble pumping patterns of CRCP observed in the field. Two load positions were used with this void size for comparative purposes. Case 5 simulates a void produced during the construction process from uneven seating of the slab. Case 6 is similar to Case 4 but is smaller in size.

Profiles of largest principal stress and deflection have been prepared from the SLAB49 program's output. These profiles are contained within Figs A4.2 through A4.8. The profiles show contours of equal stress and deflection. The magnitudes of the largest principal stress beneath the load positions are indicated. Numbers along the side and bottom of each plot refer to nodal points used in modeling.

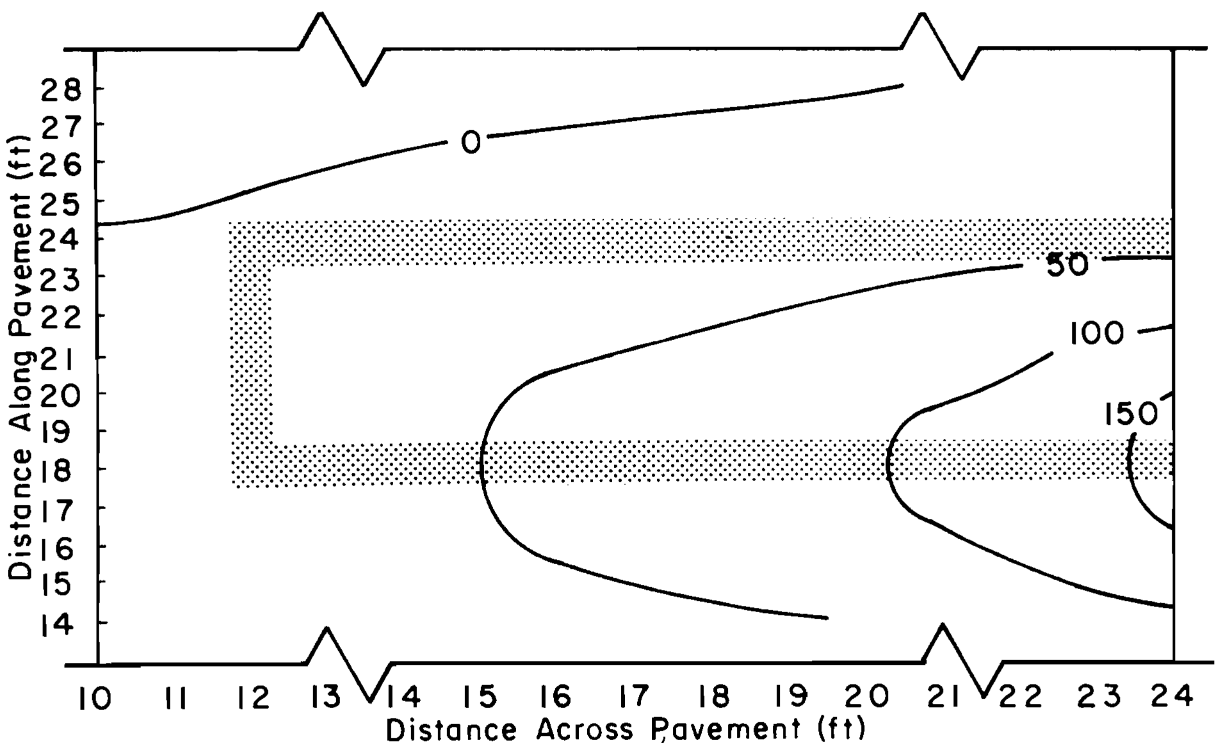
Figure A4.2 was prepared from SLAB-49 runs used in the stress analysis of splice connection for precast repair patch and represents the full support condition.

The contour labeled 0 on the principal stress plots should not be interpreted as a line of zero stress. This line represents the limits of the region within which the principal tensile stress at the bottom of the slab is greater than the principal compressive stress. This situation arises from the orthogonal state of bending within a slab and Mohr's circle analysis performed by the program.

The greatest stress occurred with the void and load located at the steel connections. This stress occurred, for Case 1, beneath the outside wheel load and was 392 psi (2703 kPa). This is shown in Fig A4.3. Case 2, shown in Fig A4.4, produced the next largest stress. This stress was 355 psi (2448 kPa). Comparison of Figs A4.2, A4.3, and A4.4 shows the void in Case 1 increased the stress over the full support condition by 13 percent. The increase in Case 2 was 9 percent.



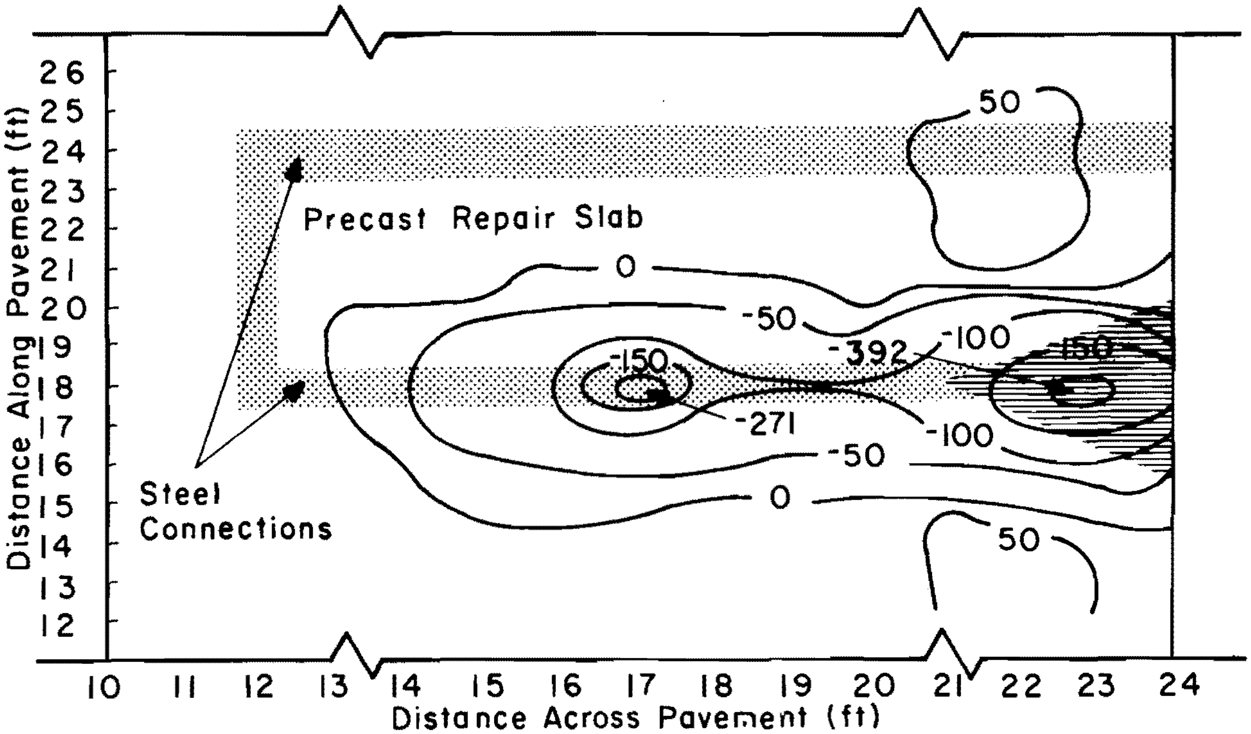
a. Principal stress contours (psi).



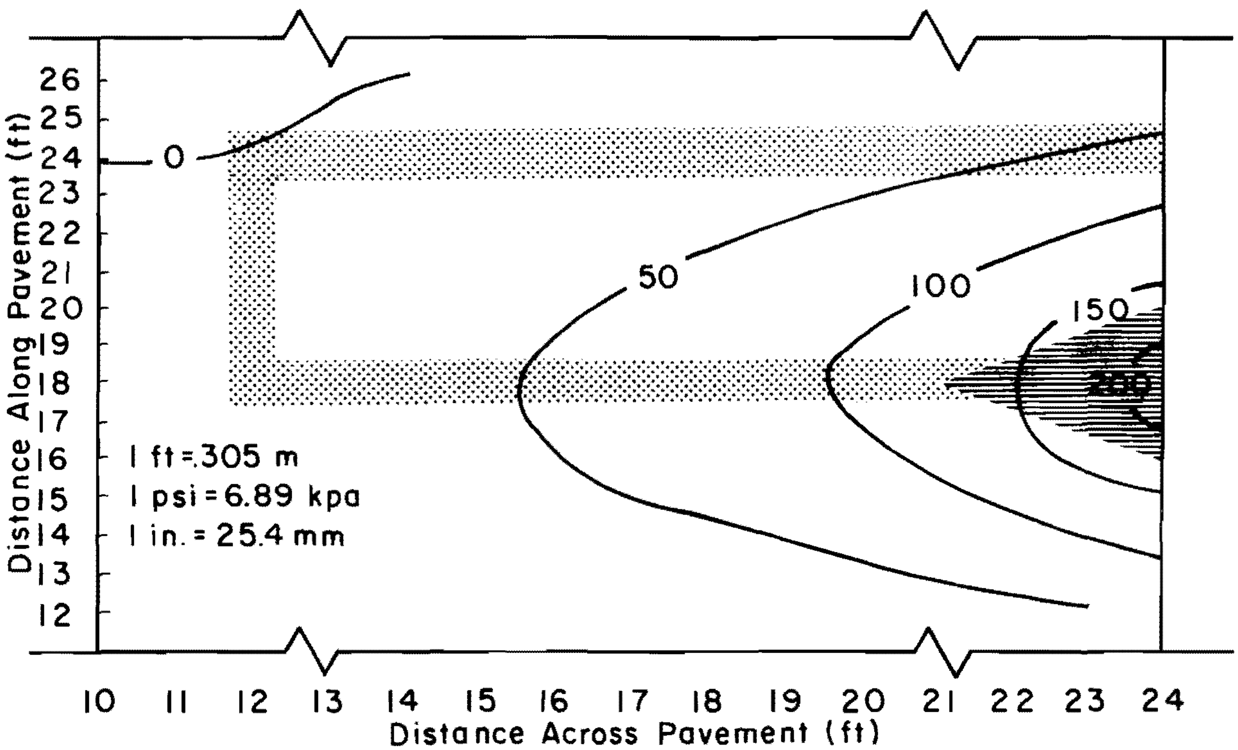
b. Deflection contours (in. x 10^{-4}).

1 ft=.305 m, 1 psi=6.89 kPa, 1 in.=25.4 mm

Fig A4.2. Principal stress and deflection contours for full support condition.

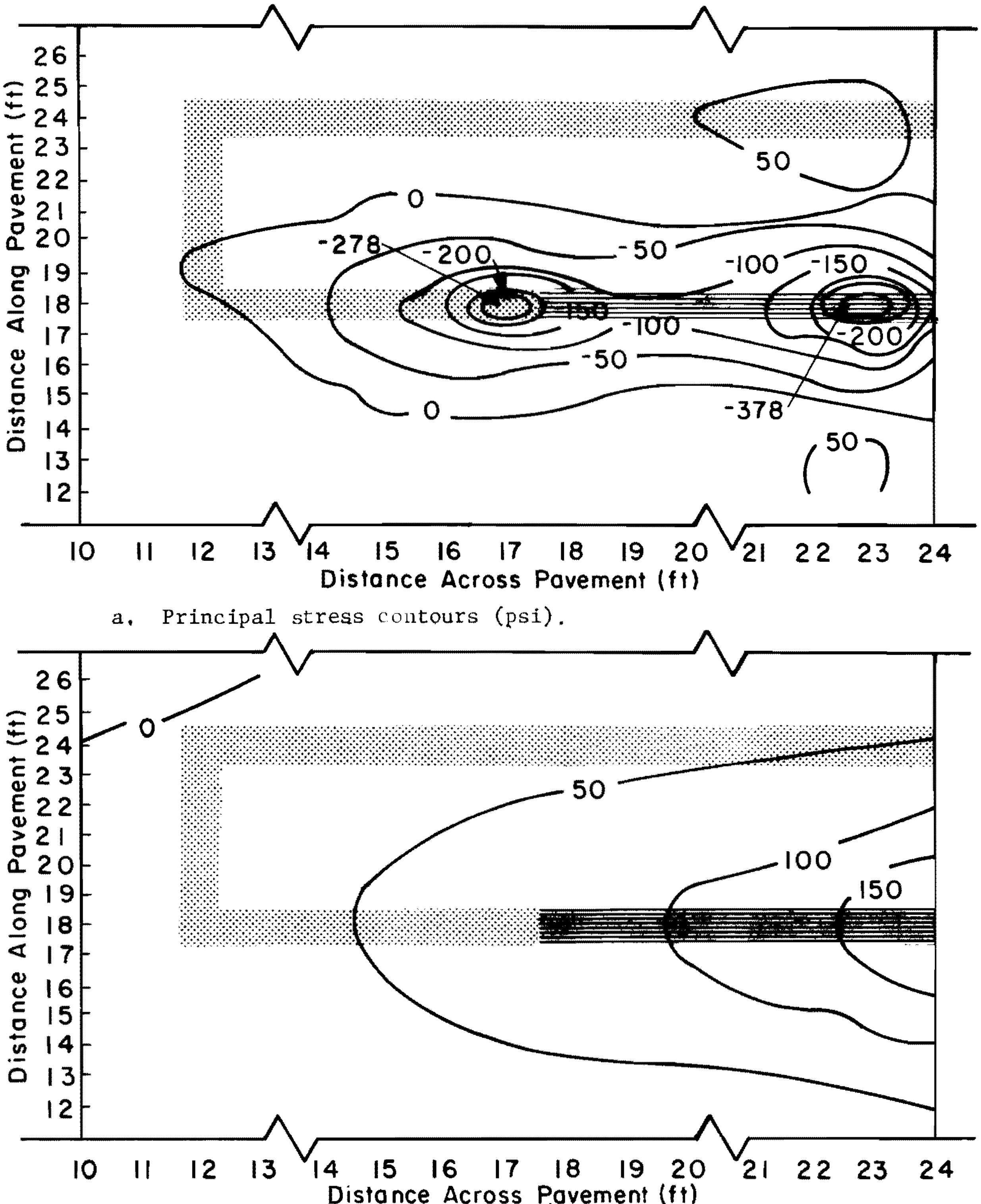


a. Principal stress contours (psi).



b. Deflection contours (in. x 10^{-4}).

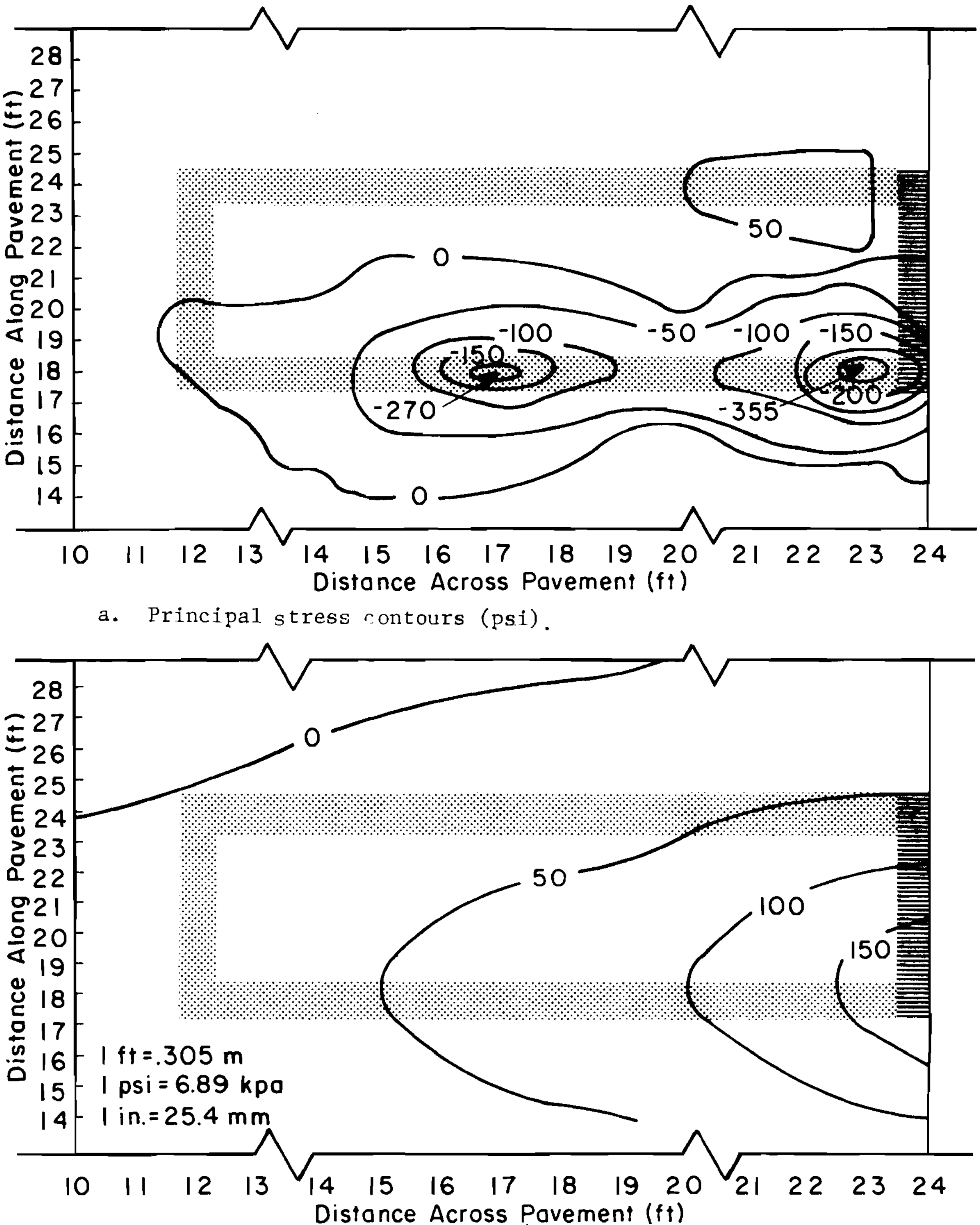
Fig A4.3. Principal stress and deflection contours for case 1.



b. Deflection contours (in. x 10^{-4}).

1 ft = .305 m, 1 psi = 6.89 kPa, 1 in. = 25.4 mm

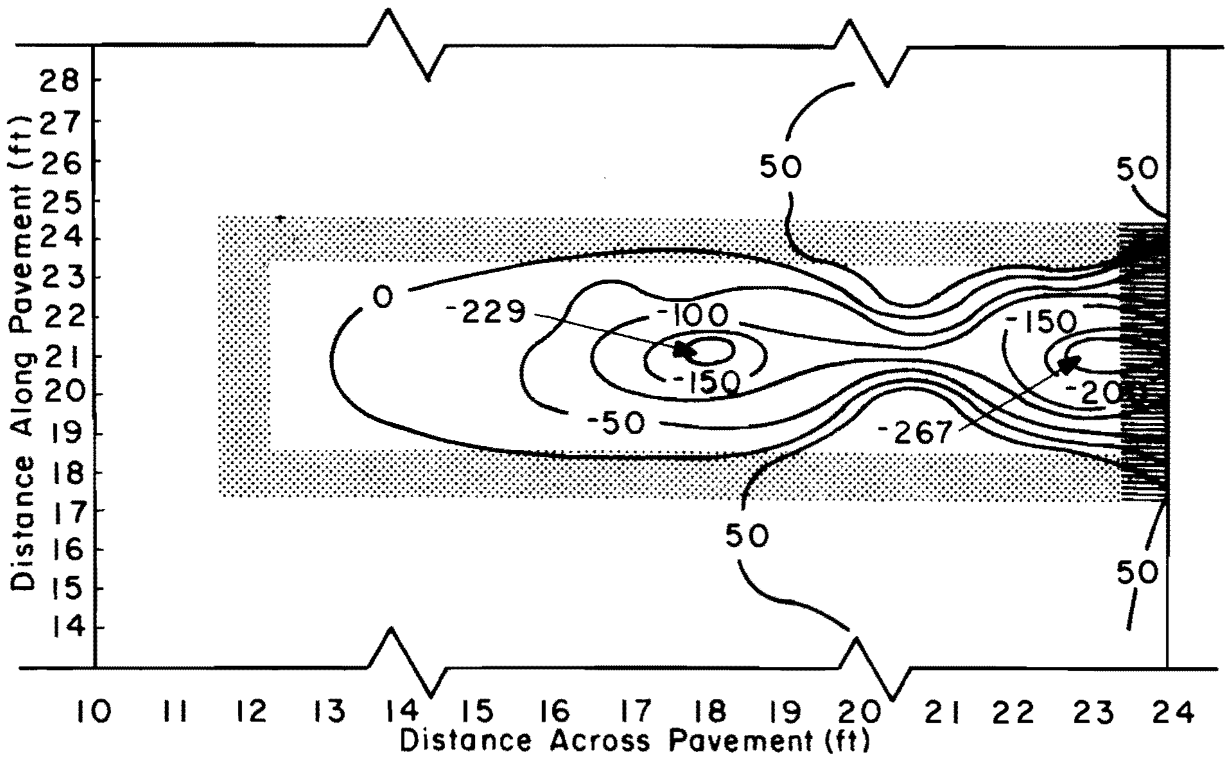
Fig A4.4. Principal stress and deflection contours for case 2.



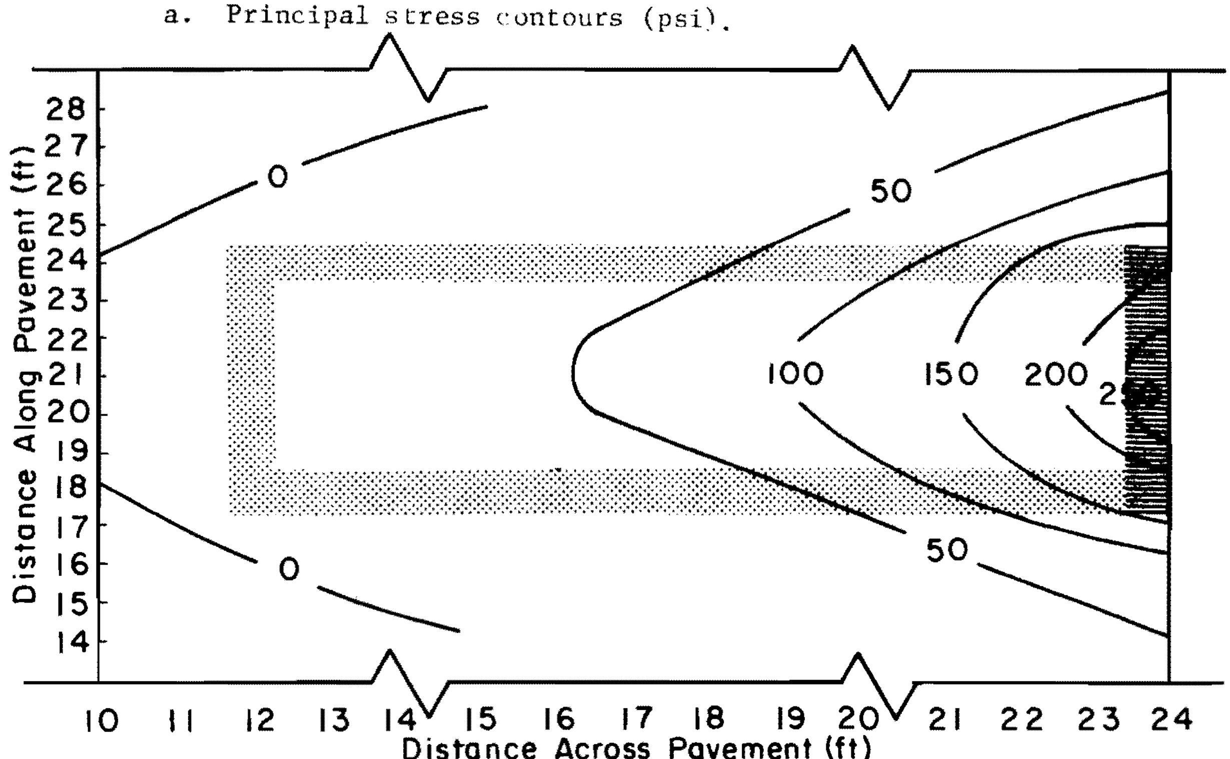
a. Principal stress contours (psi).

b. Deflection contours (in. x 10^{-4}).

Fig A4.5 Principal stress and deflection contours for case 3.



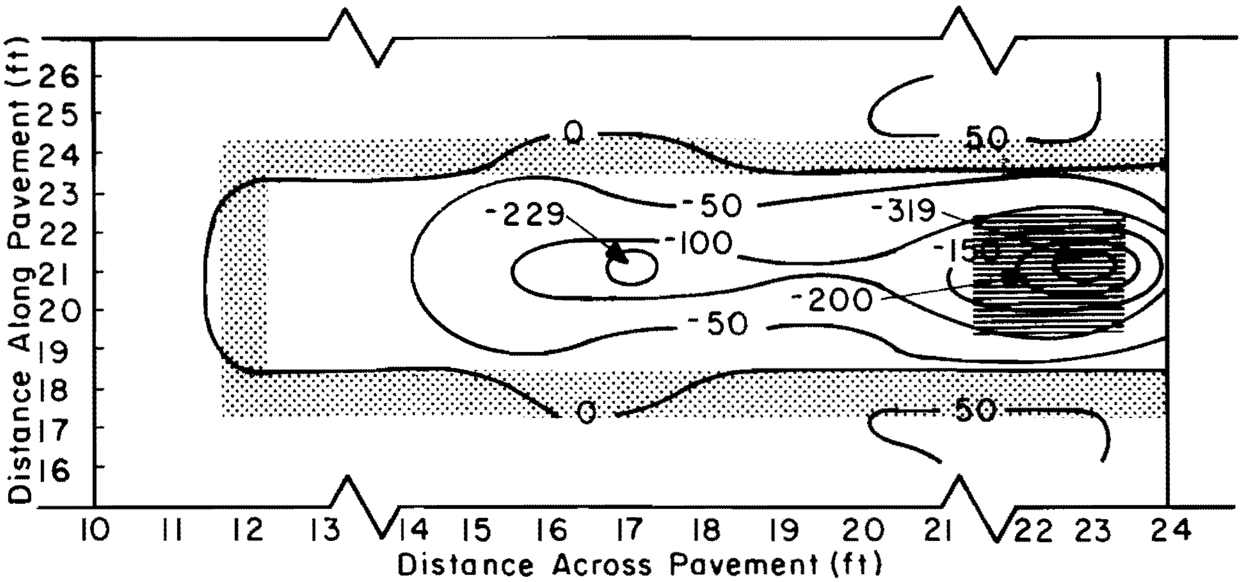
a. Principal stress contours (psi).



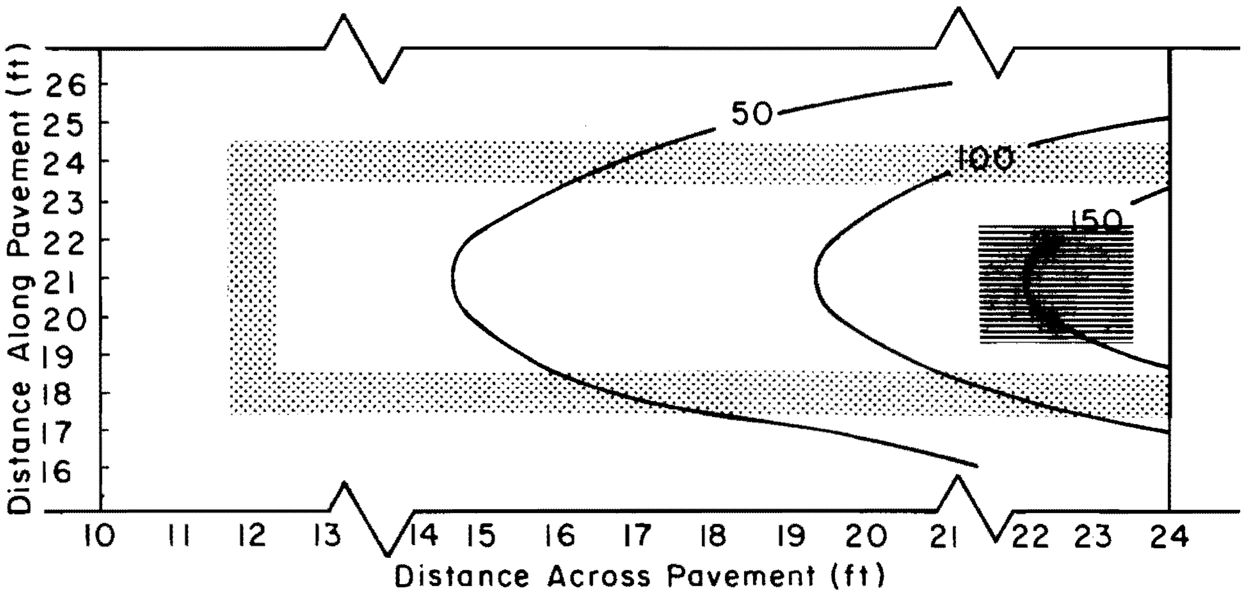
b. Deflection contours (in. x 10^{-4}).

1 ft=.305 m, 1 psi=68.9 kpa, 1 in.=25.4 mm

Fig A4.6. Principal stress and deflection contours for case 4.



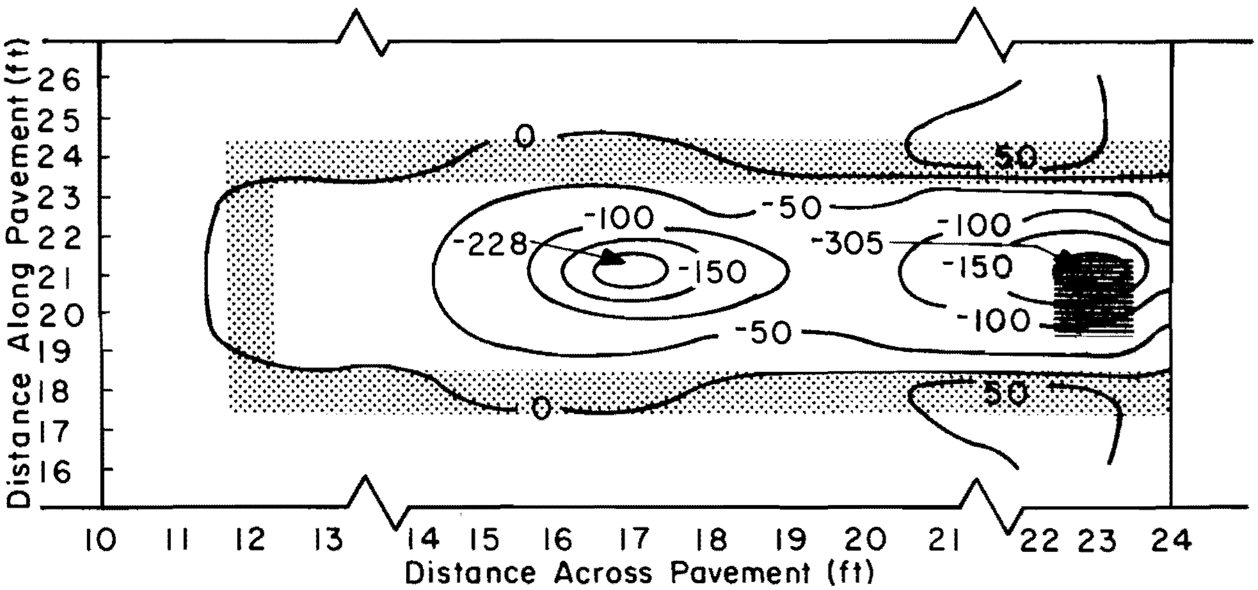
a. Principal stress contours (psi).



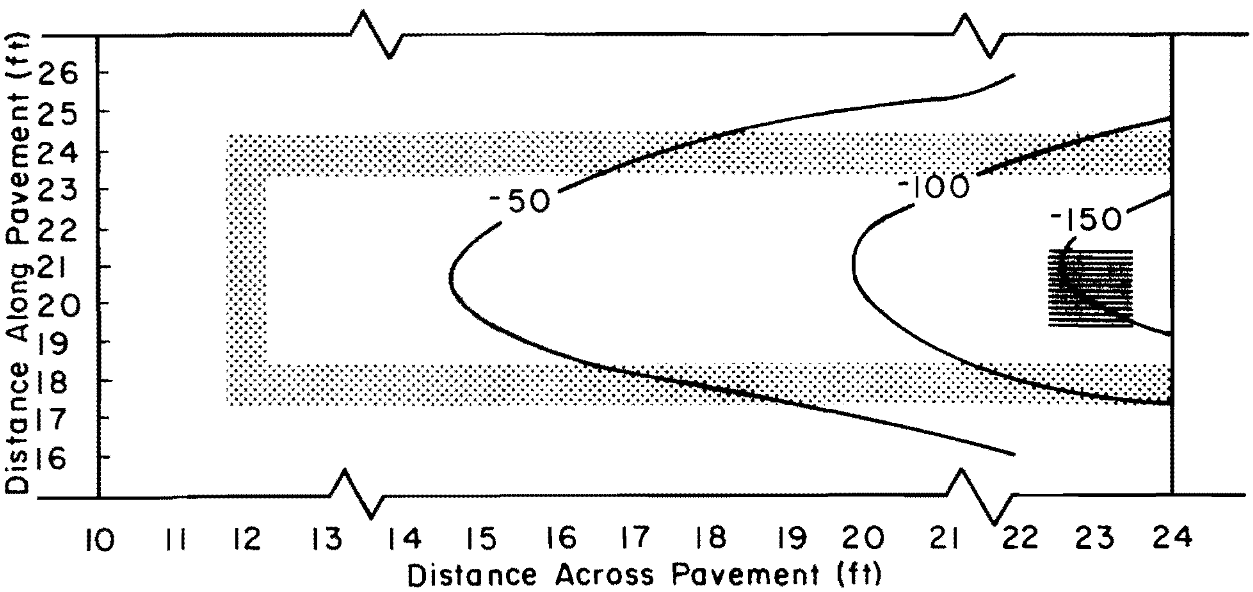
b. Deflection contours (in. x 10^{-4}).

1 ft=.305 m, 1 psi = 6.89 kpa, 1 in.= 25.4 mm

Fig A4.7. Principal stress and deflection contours for case 5.



a. Principal stress contours (psi).



b. Deflection contours (in. x 10^{-4}).

1 ft = .305 m, 1 psi = 6.89 kpa, 1 in. = 25.4 mm

Fig A4.8. Principal stress and deflection contours for case 6.

The deflection contours in Figs A4.2, A4.3, and A4.4 exhibit similar shapes. Case 1, which yielded the largest stress, exhibited a larger deflection with greater curvature than Case 2 or the full support condition. Deflection in Case 2 was a bit larger than the full support condition, otherwise the contours are nearly identical.

The void in Case 3 exhibited little effect on the stress and deflection contours. This is seen in a comparison of the full support condition in Figs A4.2 and A4.5. The stress and deflection beneath the outside wheel load in Case 3 are slightly larger than for the full support condition.

The void in Case 4 is identical to the void in Case 3. However, the load in Case 4 was positioned in the middle of the precast slab, with the outside load placed on the edge node. Deflections were greater but the stress level was considerably less. This result conforms to those found in the stress analyses of splice connections for precast repair patches.

The void in Case 5 was four times greater than in Case 6. The stresses and deflections for Case 5 were slightly greater than those for Case 6. The distribution of the stresses and deflections was very similar.

This analysis indicates that a void beneath the steel connections is critical. This is due to both the stress concentrations caused by the increased stiffness of these locations and the effect of the void.

The magnitude and, to some extent, the distribution of stress and deflection are affected by the material properties and values used in modeling. The values used here are thought to be reasonably representative of the situation being modeled. The qualitative results reported here are not expected to change within the range of these variables.

In general, the findings of this analysis are in line with those reported in "Laboratory Study of the Effect of Nonuniform Support on CRCP" (Ref 40).

APPENDIX 5

VOLUME CHANGE ANALYSIS

APPENDIX 5. VOLUME CHANGE ANALYSIS

As portland cement concrete cures, it shrinks due to evaporation of water. Concrete also changes volume with temperature. When concrete is restrained from making these volume changes, such as in CRC pavements, stresses are produced. These shrinkage and thermal stresses can become great enough to fracture concrete and cause reinforcing steel to yield. Concrete restraint in CRCP is provided by the steel reinforcement and friction along the bottom of the slab. The steel restraint is provided by the concrete slab. Shrinkage and thermal stresses in precast repair slabs are primarily related to the combination of steel reinforcement and concrete mass and secondarily to friction.

The analysis of volume change stresses in precast repair slabs is similar to that for CRC pavements. Differences in the two analyses are in the combination of material properties. The analysis of CRCP begins with the placing of the fresh concrete. At this time, the concrete is weak and will fracture at low levels of stress in both the concrete and the steel. Progressively shorter slabs are formed each time the concrete fractures. By the time the concrete has gained sufficient strength to induce harmful stresses into the steel reinforcement, a short slab has formed, with associated low stresses. A precast repair slab is placed after the concrete has cured and attained a high strength. Greater stresses are developed. This condition may lead to excessively large steel stresses prior to cracking of the concrete.

Solution of volume change stresses in restrained reinforced concrete slabs is inherently nonlinear and requires numerical techniques. Prior research at the Center for Highway Research has produced mathematical models and a computer program for volume change analysis of CRCP (Ref 1). This program analyzes CRCP as a function of time and does not allow for initial setting of some variables and easy manipulation of parameters. To overcome these difficulties, a new program, based essentially upon the same mathematical models, was written. It and a companion program can be used to analyze shrinkage and thermal stresses in reinforced concrete slabs.

These new programs have been called VCARCS and VCACS, which stand for volume change analysis of restrained concrete slabs and volume change analysis of concrete slabs, respectively. VCARCS is used for stresses in slabs whose steel reinforcement is restrained at the ends, such as in CRCP. VCACS analyzes stresses in unrestrained reinforced concrete slabs, such as precast slabs before installation and jointed reinforced concrete pavements. These programs were written as interactive programs and indicate the required units of measurements of input data.

INITIAL TRIAL INVESTIGATION

The first step in the investigation of volume change stresses was the selection of two CRCP designs. These designs were obtained from data on CRCP in Texas, collected and reported in NCHRP Report 1-15 (Ref 1). The principal difference between the two designs is the steel reinforcement. A 0.5 percent, by area, steel design was selected consisting of 5.74 in.^{2} (37.0 cm^2) of No. 6 grade 50 steel rebar. The other was a 0.6 percent design consisting of 6.75 in.^{2} (43.4 cm^2) of No. 5 grade 60 steel rebars. The most common CRCP design in Texas consists of 0.5 percent steel reinforcing using No. 5 grade 60 rebar. The designs chosen in this trial investigation represent a practical range of CRCP designs used in Texas.

The material properties and dimensions for this analysis were selected to approximate those expected for a precast CRCP repair slab. A one-lane CRC pavement, 8 inches (203 mm) thick and 12 feet (3.66 m) wide was modeled for both steel designs. The following material properties were assumed:

concrete compressive strength	=	4,000 psi (2.76×10^4 kPa),
concrete modulus of elasticity	=	4.0×10^6 psi (2.76×10^7 kPa),
concrete thermal coefficient	=	5.0×10^{-6} in./in./^{o}F (9.0×10^{-6} cm/cm/^{o}C),
steel modulus of elasticity	=	29×10^6 psi (1.99×10^8 kPa),
steel thermal coefficient	=	5.0×10^{-6} in./in./^{o}F (9.0×10^{-6} cm/cm/^{o}C).

TABLE A5.1. DESIGN FACTORIAL AND RESULTS FOR INITIAL INVESTIGATION

%Steel Reinforced Design	Friction	Concrete Shrinkage	Slab Length 3		6		9		15	
.5	Plastic Soil	.00005	-218	170	240	233	573	279	1070	348
			33.7	.00480	46.7	.00902	56.1	.0129	70.1	.0199
		.0002	-4270	211	-3690	291	-3260	350	-2610	439
			37.8	.00741	54.3	.0141	66.4	.0203	84.6	.0317
		.0004	-9470	256	-9020	355			-7680	541
			41.4	.0109	61.7	.0209			99.6	.0481
	None	.00005	-219	170	237	232	567	278	1060	346
			33.7	.00480	46.7	.00902	56.1	.0129	70.1	.0199
		.0002	-4270	211	-3690	291	-3270	349	-2630	437
			37.8	.0074	54.3	.0141	66.4	.0203	84.5	.0318
		.0004	-9740	256	-9030	354			-7700	537
			41.4	.0109	61.7	.0294			99.5	.0481
.6	Plastic Soil	.00005	235	232	832	315	1250	373	1867	457
			39.6	.00454	54.2	.00831	64.4	.0116	79.2	.0174
		.0002	-3690	290	-2930	396	-2370	473	-1560	585
			45.5	.00708	64.2	.0132	77.6	.0187	97.3	.0285
		.0004	-9030	354	-8070	486			-6330	727
			51.0	.0105	74.3	.0198			116.4	.0440
	None	.00005	234	232	829	314	1250	372	1850	455
			39.7	.00454	54.2	.00831	64.4	.0116	79.2	.0175
		.0002	-3700	290	-2930	396	-2380	471	-1570	583
			45.5	.00708	64.2	.0132	77.6	.0187	97.3	.0286
		.0004	-9030	354	-8080	486			-6350	723
			51.0	.0105	74.3	.0198			116.4	.0440

Key

A	B
C	D

A. Steel stress in center of slab, psi (- is compression).

B. Concrete stress in center of slab, psi.

C. Steel stress at end of slab, ksi.

D. Concrete movement at end of slab, in.

1 ksi = 6.89 mPa, 1 psi = 6.89 kPa, 1 ft = .305 m, 1 in. = 25.4

A constant temperature drop of 50°F (27.8°C) was used throughout this analysis. Although this drop is not an extreme, it is representative of a likely condition in the southern United States. The concrete shrinkage, slab length, and friction were allowed to vary from low to high to demonstrate their effects on stresses. The factorial breakdown of the combinations of the above variable quantities is shown in Table A5.1.

The concrete shrinkage was varied from 5.0×10^{-5} to 4.0×10^{-4} in./in. (mm/mm). A concrete shrinkage of 4.0×10^{-4} in./in/ (mm/mm) is a common expectation for the final shrinkage of ordinary portland cement concrete. Some of the concrete shrinkage in a precast repair slab occurs during curing while the steel is unrestrained. As much as half the total concrete shrinkage can occur during the first 28 days. Thus, the median value shown in Table A5.1, 2.9×10^{-4} in./in. (mm/mm), is a reasonable limit on the maximum of shrinkage loading expected for an installed precast repair slab. Shrinkage stresses in a precast slab are properly evaluated by considering both stresses produced during the initial curing period, when the steel reinforcement is unrestrained, and those developed after installation, when the steel is restrained. In this initial investigation, only those stresses produced after slab placement were investigated.

The lengths of the precast slabs were varied between 3 and 15 feet (0.914 and 4.57 m). Three feet (0.914 m) is a practical lower limit on precast slab lengths. The analysis demonstrated that 15-foot (4.57-m) long slabs were large enough to fulfill the purpose of this investigation.

A highly frictional subbase was used to compare against the frictionless case. The force-displacement curve for an 8-inch-thick concrete slab upon a plastic nontreated subbase was taken from data reported in Ref 44. This friction curve was the greatest frictional resistance for this size slab reported in this reference. This curve is shown below.

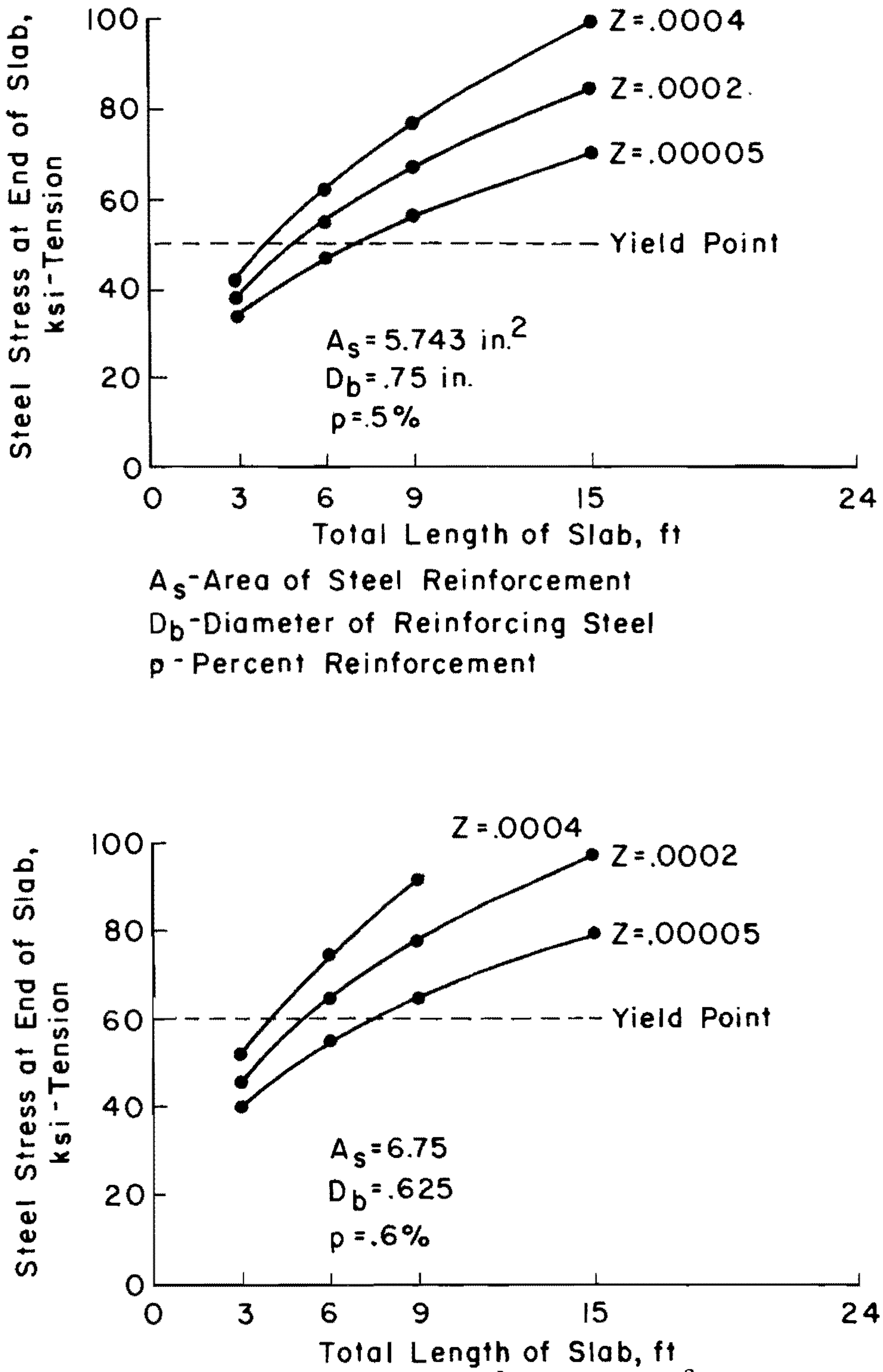
Displacement, in. (mm)	0.012 (.305)	0.025 (.635)	0.05 (.127)	0.10 (2.54)	0.15 (3.81)	0.25 (6.35)	0.4 (10.2)
Friction Stress, psi (kPa)	0.242 (1.67)	0.333 (2.30)	0.455 (3.14)	0.593 (4.09)	0.651 (4.49)	0.694 (4.78)	0.735 (5.07)

The results of the analysis using VCARCS are shown in Table A5.1. Steel stress at the end of the slab, steel stress in the center of the slab, concrete stress in the center of the slab, and concrete movement at the end of the slab are indicated for the complete factorial of variables. The significant results of this analysis are plotted in Figs A5.1 and A5.2.

In Fig A5.1, steel stress at the end of the slab is plotted against slab length for different shrinkage levels. A graph is provided for each reinforcement design. The plot for the 0.5 percent steel design shows stresses in excess of the yield point for slabs longer than 6 feet (1.83 m) at the lowest shrinkage level and for slabs longer than 4 feet (1.22 m) at the median shrinkage level. Similar behavior is seen for the 0.6 percent steel design.

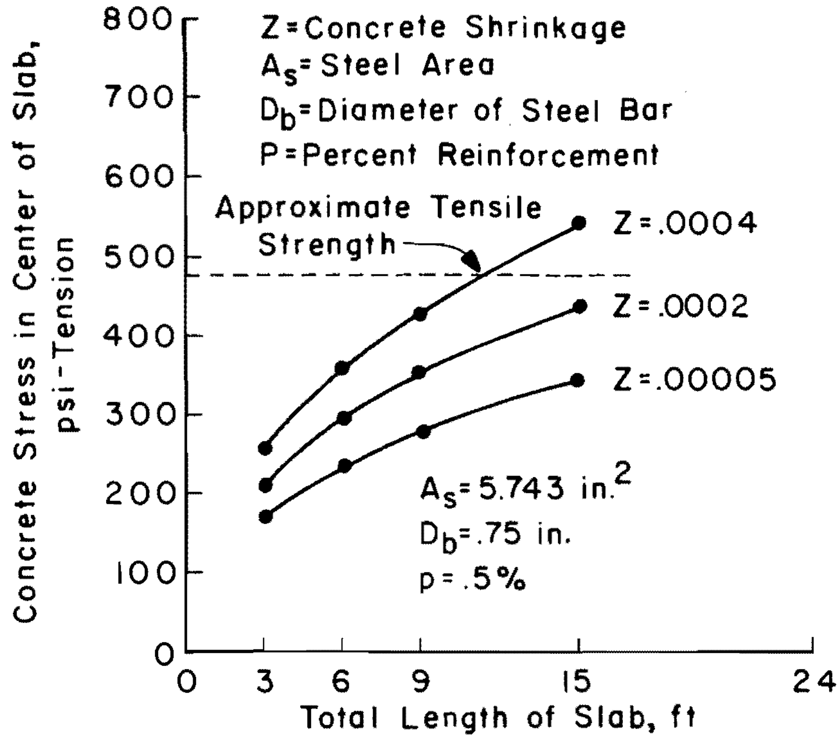
A close comparison of the graphs in Fig A5.1 reveals higher calculated stresses for the 0.6 percent steel design. This result may appear contrary to the reasoning that inclusion of more steel into a concrete slab should result in lower stresses. The situation shown here arises from the difference in rebar sizes between the designs. Rebar sizes influence the steel's development length, which is reflected directly by the calculated stresses. The smaller bars in the 0.6 percent design have a shorter development distance, which results in higher stresses. This effect is shown in subsequent analysis.

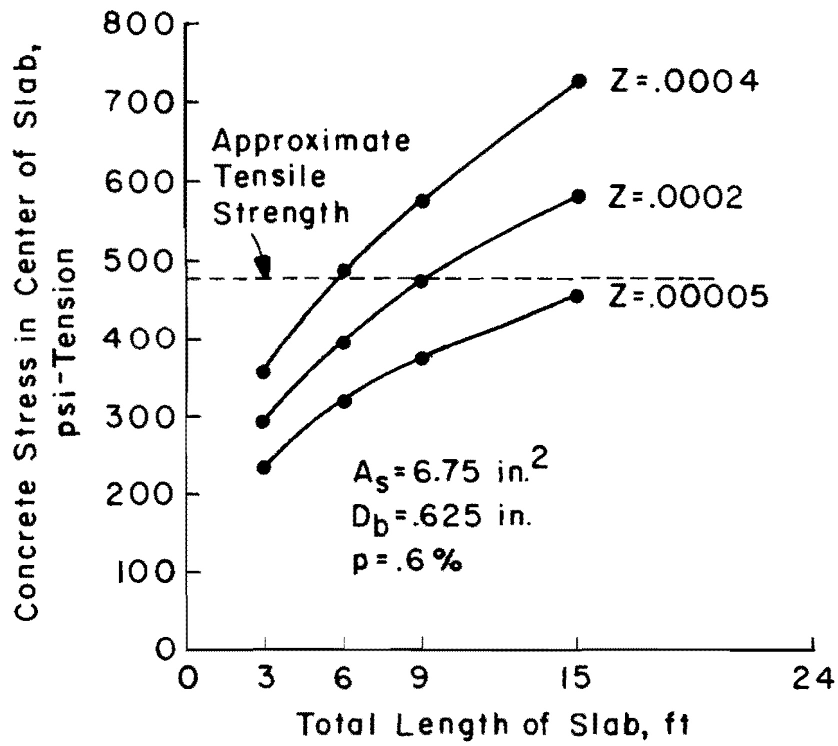
In Fig A5.2, concrete stress in the center of the slab for varying slab lengths and shrinkage levels is plotted for both steel designs. An approximate tensile strength of the concrete is indicated. For concrete shrinkage less than 2.0×10^{-4} in./in. (mm/mm), concrete stresses for the 0.5 percent design were below the estimated tensile strength for slabs up to 15 feet (4.57 m) long. At the same shrinkage level, stresses for the 0.6 percent design were in excess of the tensile strength for slabs longer than 9 feet (2.74 m). At the low level of concrete shrinkage, concrete stresses for both designs were below the estimated tensile strength for the range of lengths investigated. At the 4.0×10^{-4} in./in. shrinkage level, included for comparative purposes, slabs longer than 9 feet (2.74 m) for the 0.5 percent design and longer than 6 feet (1.38 m) for the 0.6 percent design showed concrete stresses in excess of the estimated strength.



1 ksi = 6.89 mPa, 1 in^2 = 6.45 cm^2, 1 ft = .305 m
1 in. = 25.4 mm

Fig A5.1. Steel stress at end of slab for initial investigation





1 psi = 6.89 kPa, 1 in.^{2} = 6.45 cm^2, 1 ft = .305 m, 1 in. = 25.4 mm

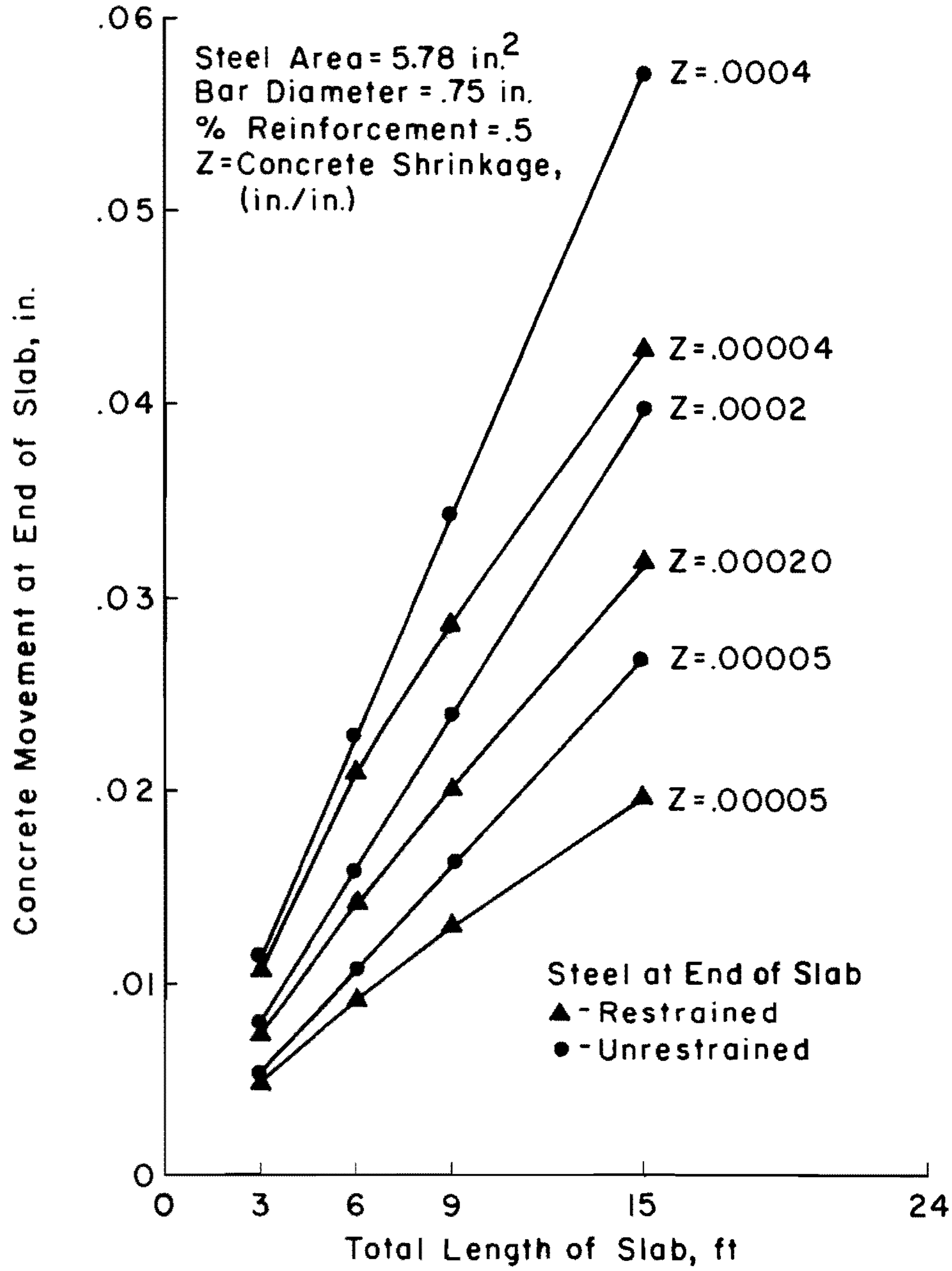
Fig A5.2. Concrete stress in the center of the slab for initial investigation.

In this analysis addition of friction resistance exhibited little effect. Friction resistance exhibited the greatest effects on the longer length slabs. The 15-foot (4.57-m) long slabs showed an increase in concrete stress of about 0.1 percent, an increase in steel stress at the end of the slab of 0.03 percent, a decrease in steel stress in the center of the slab of approximately 0.3 percent, and a decrease in concrete movement at the end of the slab of 0.1 percent for friction calculations relative to the frictionless case. On the basis of this comparison, use of the frictionless model for volume change calculations results in negligible errors considering the uncertainty associated with the material properties.

Figures A5.1 and A5.2 indicate the possible development of excessive steel stresses in a precast repair slab for CRCP. Concrete stresses in a 9-foot (2.74-m) long slab with 0.5 percent steel reinforcement and 0.0002 in./in. of concrete shrinkage, shown in Fig A5.2, are well below the estimated tensile strength. The corresponding steel stress at the end of the slab, shown in Fig 5.1, is greatly in excess of the yield point. A similar situation occurs for some of the 0.6 percent steel reinforcement designed slabs.

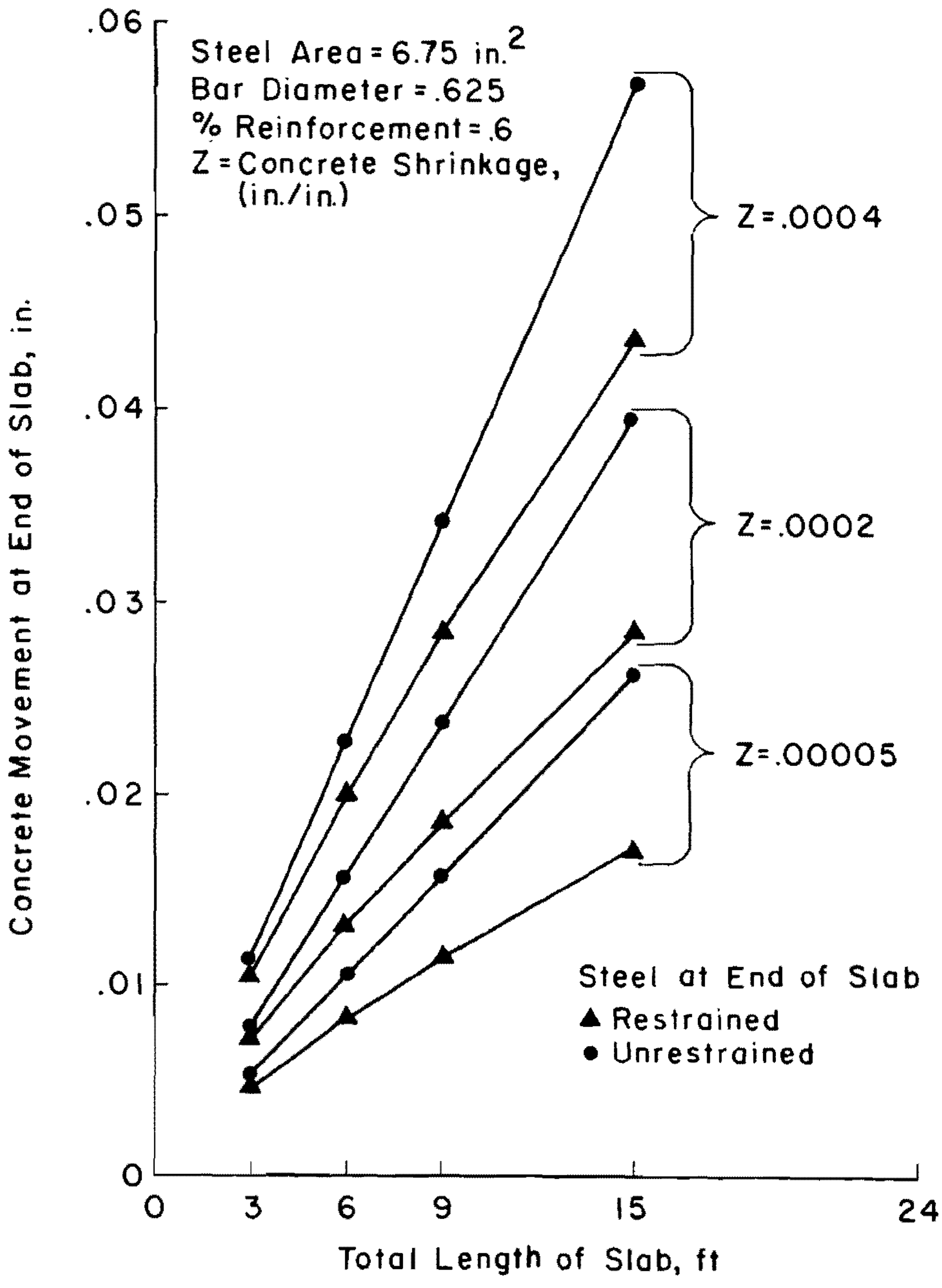
A comparison of concrete movements at the end of the slab for slabs with and without the steel at the end of the slab restrained is shown in Figs A5.3 and A5.4. The VCACS program was used to analyze the unrestrained slabs for comparison against the restrained slabs analyzed with the VCARCS program. As might be expected, movements for the slabs with the steel restrained were less than those without restraint. Slabs with the 0.6 percent steel design had movements slightly less than those with the 0.5 percent design.

If the crack spacing in the pavement immediately adjacent to the ends of the repair slab was equal to the length of the repair slab, the concrete movement at the end of the slab, shown in Figs A5.3 and A5.4, would be expected to account for half of the crack widths at these locations. However, the concrete movement of the precast repair slab is expected to account for more than half of the crack widths at the ends of the slab. This is a consequence of the steel connection area and probable discontinuities in the surrounding concrete. A limit on crack widths of 0.023 inch (0.584 mm) in CRCP has been suggested to guard against water infiltration and provide load transfer (Ref 1). A conservative limit of 0.011 inch (0.279 mm), approximately half of



1 in. = 25.4 mm, 1 ft = .305 m, 1 in.^{2} = 6.45 cm^2

Fig A5.3. Comparison of concrete movement at end of slab for restrained and unrestrained slabs.



1 in. = 25.4 mm, 1 ft = .305 m, 1 in^2 = 6.45 cm^2

Fig A5.4. Comparison of concrete movement at end of slab for restrained and unrestrained slabs.

the above crack width limit, can be placed on concrete movements of the ends of precast repair slabs. For each steel design, restrained slabs less than 9 feet long (2.74 m) at 0.00005 in./in. (mm/mm) concrete shrinkage exhibited movements less than the above limit. At the same shrinkage, unrestrained slabs less than 6 feet long (1.83m) had movements less than the limit. At the 0.0002 in./in. (MM/MM) shrinkage level, restrained slabs less than 5 feet long (1.5 m); unstrained slabs less than 4 feet long (1.22m) had acceptable movements.

The above calculations indicate one of the reasons for providing steel anchorage at the end of a precast repair slab. Restraining the steel reinforcement helps maintain narrow crack widths. Narrow crack widths aid the performance of the repair through exclusion of incompressible material and resistance to water percolation.

The calculations presented for the trial steel designs indicated that excessive steel stresses can be developed at low concrete stresses. This was particularly true for slabs longer than about 6 feet (1.83 m). It would be preferable for the concrete to crack in the center of the slab and create two shorter length slabs rather than for the steel at the end of the slab to yield. One method of increasing concrete stress in the center of the slab is to reduce the cross-sectional area there with a weakened plane. This approach would allow the concrete to crack at acceptable steel stresses.

The factorial breakdown of variables presented in Table A5.2 was used to investigate the effects of a reduction in area of concrete in the center of a precast slab. Three levels of area reduction were investigated; 25, 50, and 75 percent of the cross-sectional area. Four levels of concrete shrinkage for four slab lengths are shown in Table A5.2. Not all of the combinations were examined in this investigation.

Steel stress at the end of the slab and concrete stress in the center of the slab are plotted with respect to percent of concrete area reduction for 6-foot (1.83-m) and 9-foot (2.74-m) slabs for each steel design in Figs A5.5, A5.6, A5.7, and A5.8. Stresses corresponding to the lowest three levels of concrete shrinkage shown in Table A5.2 are plotted in these figures.

The effect of reduction on concrete cross-sectional area can be thought of in terms of a safety valve. For example, the stress curves for a 9-foot (2.74-m) slab with the 0.6 percent steel design, shown in Fig A5.8, show steel stresses in excess of yield with corresponding concrete stress below

TABLE A5.2. DESIGN FACTORIAL FOR REDUCED CONCRETE AREA IN CENTER OF SLAB, INITIAL INVESTIGATION

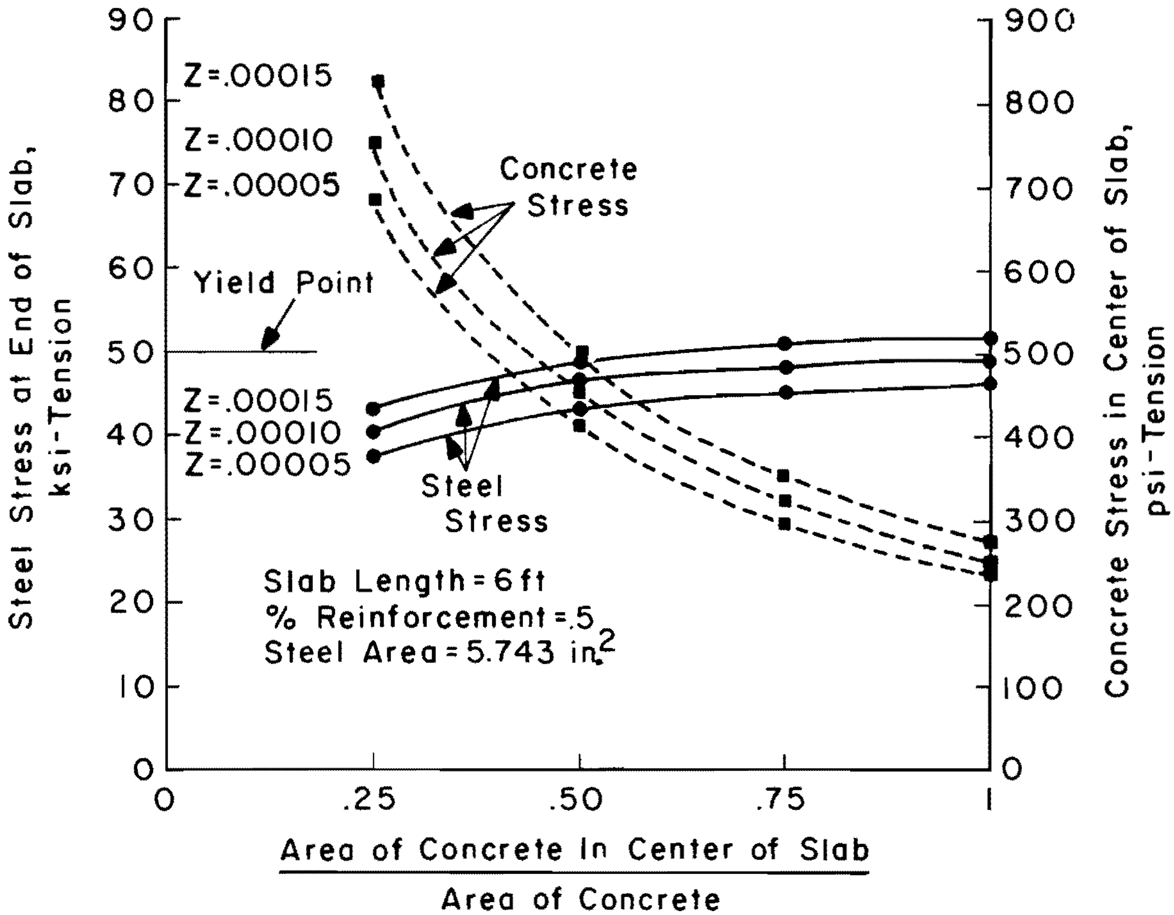
% Steel Reinforcement Design	Concrete Area in Center of Slab, in^2	Concrete Shrinkage	Slab Length 3	Slab Length 6	Slab Length 9	Slab Length 15
.5	3/4(1146.26) 859.7	.00005	150 221 33.2	721 299 45.5	1125 355 54.3	1711 436 67.0
		.00010	-1160 240 34.8	-530 327 48.4	-82.8 389 58.1	571 479 72.2
		.00015	-2480 258 36.2	-1790 352 50.9	-1310 420 61.5	-589 519 77.1
		.00020	-3800 275 37.4	-3070 376 53.3	-2540 449 64.7	-1770 556 81.5
	1/2(1146.26) 573.13	.00005	833 315 32.3	1580 418 43.3	2090 489 50.8	2800 586 61.3
		.00010	-407 344 33.9	428 459 46.2	100 538 54.7	1800 649 66.5
		.00015	-1660 371 35.4	-747 497 48.8	-115 584 58.2	776 707 71.3
		.00020	-2930 396 36.6	-1940 533 51.2		
	1/4(1146.26) 286.56	.00005	2480 543 29.6	3490 681 37.5	4090 765 42.2	4830 865 48.0
		.00010		2600 758 40.4	3300 855 46.0	4170 975 52.8
		.00015		1670 830 43.1	2470 941 49.4	3480 1079 57.3
.6	3/4(1145.25) 858.94	.00005	717 299 38.7	1440 399 52.2	1940 467 61.4	2630 562 74.2
		.00010	-534 326 41.0	268 437 55.9	823 514 66.2	
		.00015	-1800 352 43.0	-922 473 59.2	-312 557 70.6	
		.00020	-3080 376 44.7	-2130 507 62.3	-1460 598 74.6	
	1/2(1145.25) 572.62	.00005	1580 418 37.0	2480 541 48.4	3060 622 55.8	3820 727 65.5
		.00010	423 458 39.3	1440 598 52.2	2100 690 60.6	
		.00015	-752 496 41.3	368 651 55.6	1110 753 65.0	
		.00020	-1940 532 43.2	-721 700 58.7		
	1/4(1145.25) 286.31	.00005	3480 681 32.4	4510 822 39.4	5060 898 43.2	
		.00010		3790 923 42.9	4460 1020 47.5	
		.00015		3040 1020 46.2	3820 1130 51.6	

Key

A	C
B	

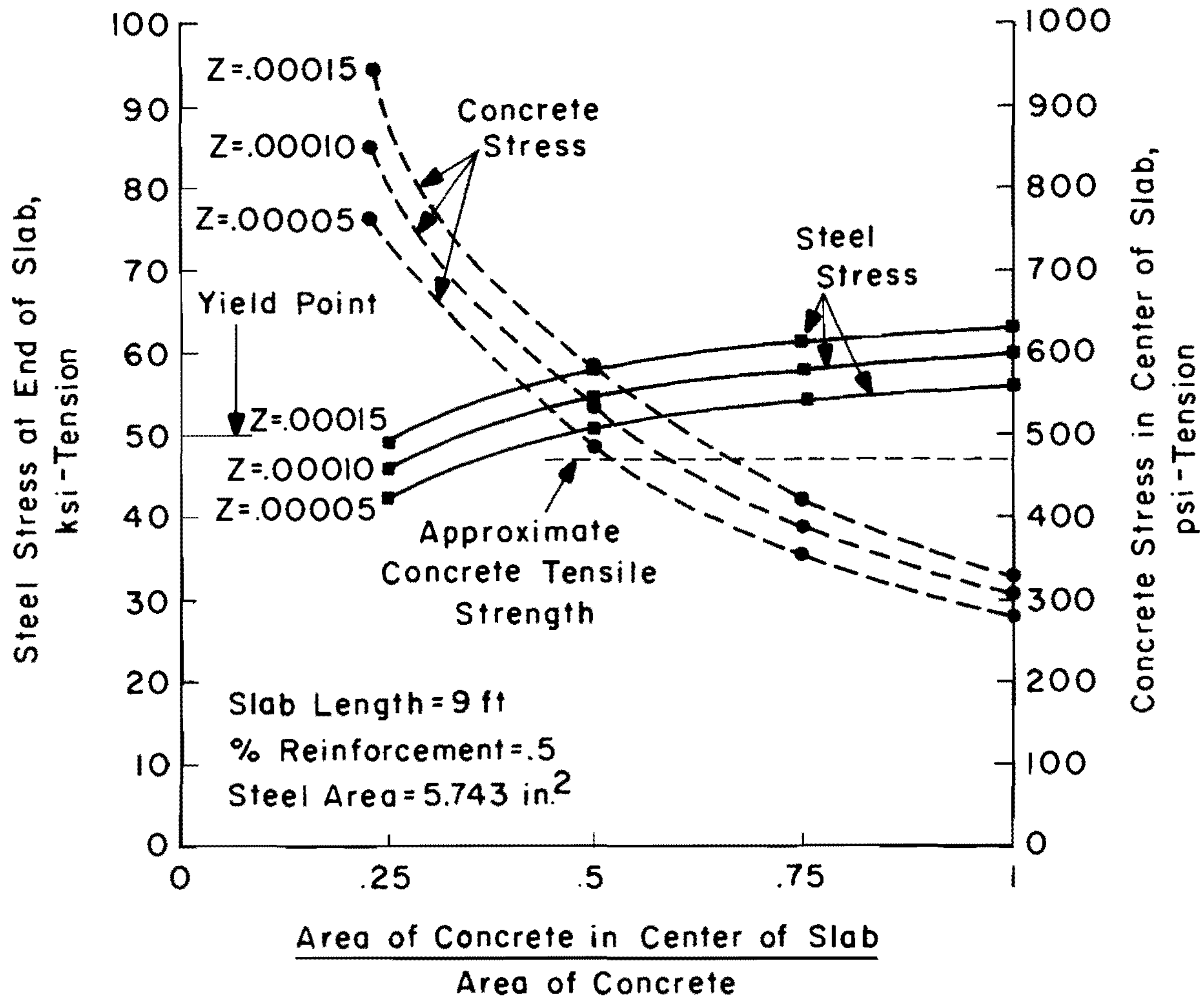
A. Steel stress in center of slab, psi (- is compression)
B. Steel stress at end of slab, ksi
C. Concrete stress in center of slab, psi

1 ksi = 6.89 mPa, 1 psi = 6.89 kPa, 1 ft = .305 m, 1 $in.^2$ = 6.45 cm^2



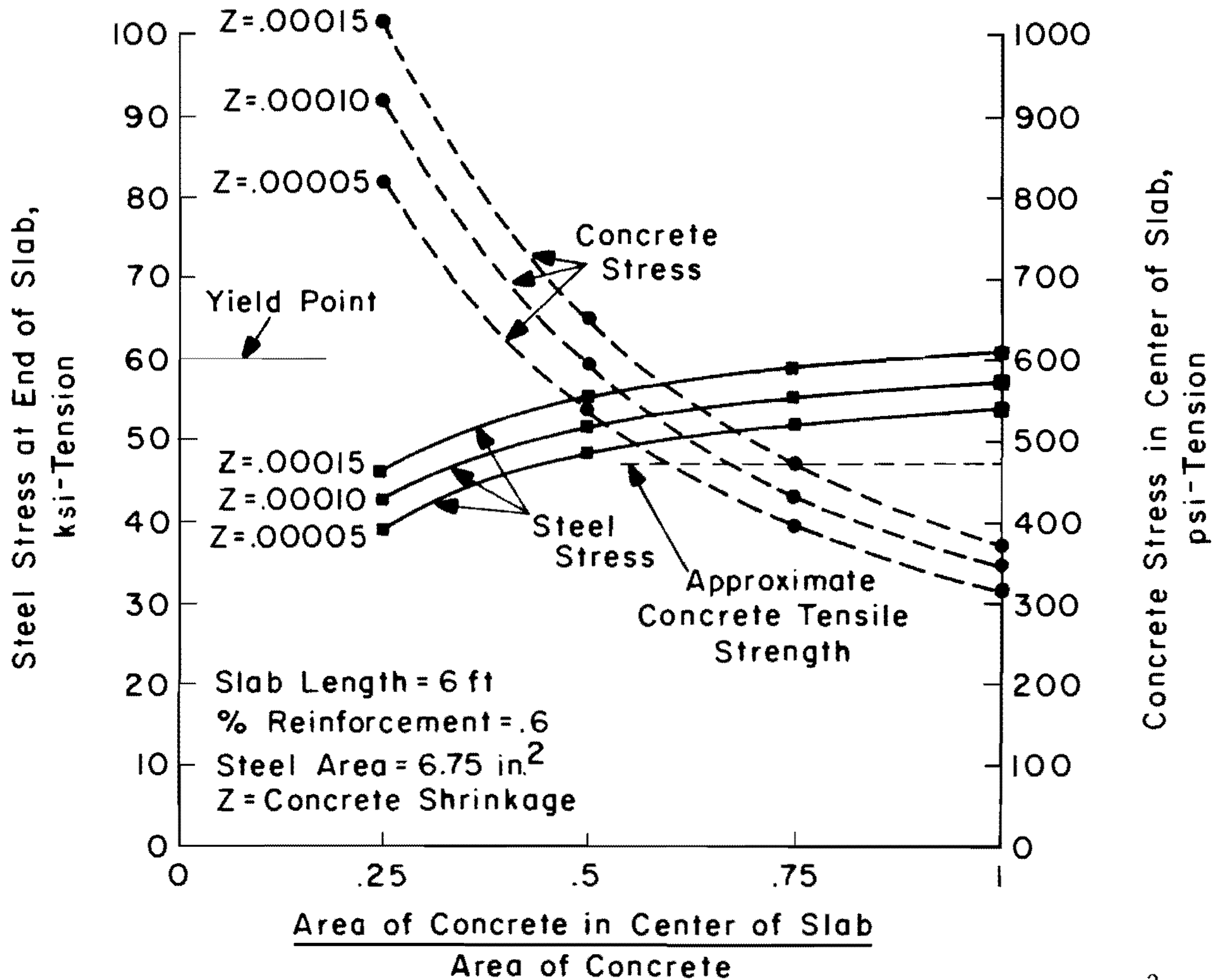
1 ksi = 6.89 mPa, 1 psi = 6.89 kPa,
1 ft = .305 m, 1 in.^{2} = 6.45 cm^2

Fig A5.5. Effect of reduced concrete area on steel and concrete stresses, initial investigation.



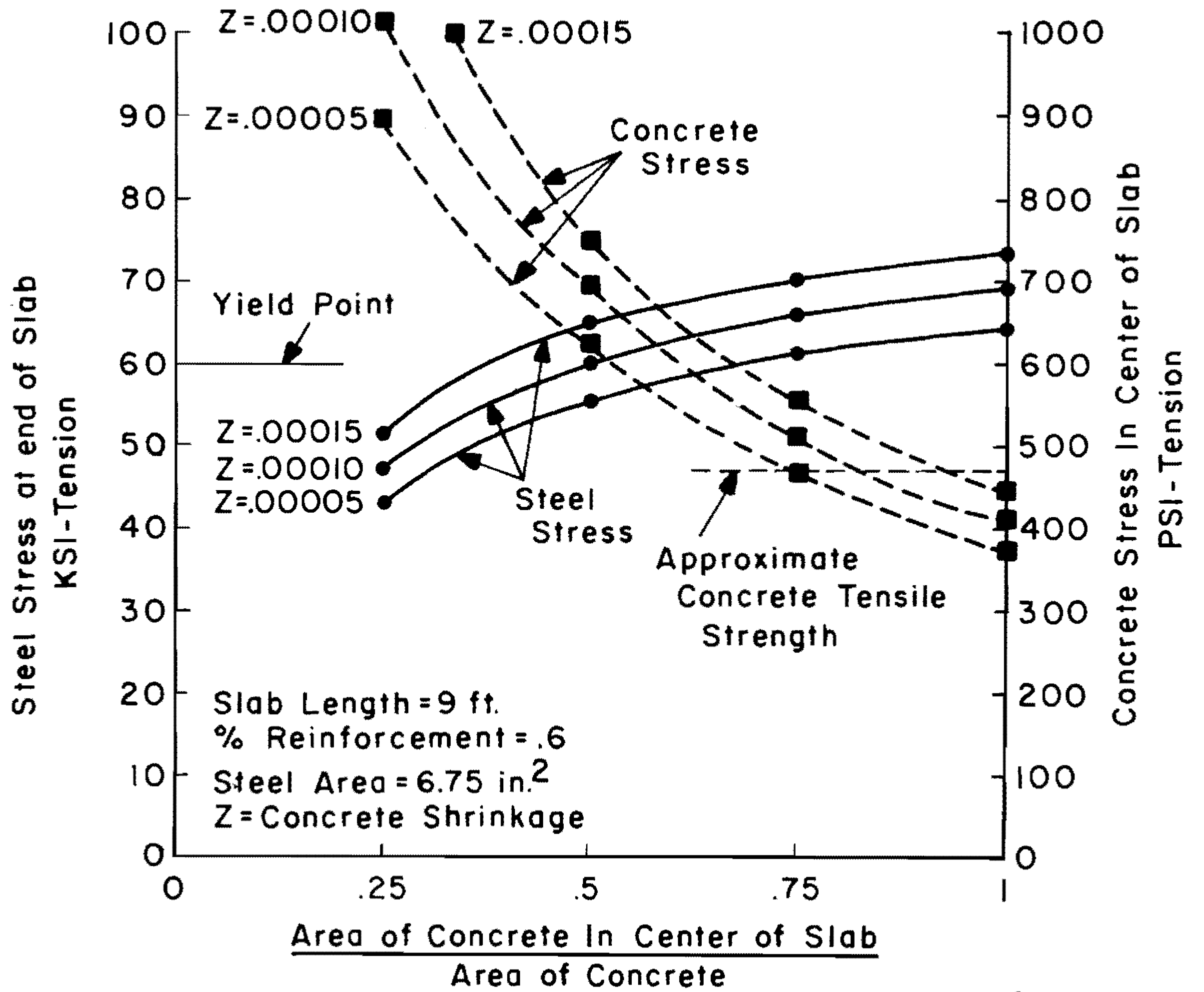
1 ksi = 6.89 mPa, 1 psi = 6.89 kPa, 1 ft = .305 m, 1 in.^{2} = 6.45 cm^2

Fig A5.6. Effect of reduced area of concrete in the center of the slab on steel and concrete stresses, initial investigation.



1 ksi = 6/89 mPa, 1 psi = 6.89 kPa, 1 ft = .305 m, 1 in.^{2} = 6.45 cm^2

Fig A5.7. Effect of reduced area of concrete in the center of the slab on steel and concrete stresses, initial investigation.



1 ksi = 6.89 mPa, 1 psi = 6.89 kPa, 1 ft = .305 m, 1 in.^{2} = 6.45 cm^2

Fig A5.8. Effect of reduced area of concrete in the center of the slab on steel and concrete stresses, initial investigation.

its tensile strength for no area reduction. A 50 percent area reduction increased the concrete stress in excess of the estimated tensile strength and reduced steel stresses. This indicates that for some previous combination of concrete shrinkage and temperature drop the concrete would crack at the weak plane without allowing steel stresses to become excessive. If the weak plane is located in the center of the slab, two 4.5-foot (1.37-m) long slabs would be produced. The stresses in a 4.5-foot (1.37-m) long slab can be estimated from Fig A5.1. For a concrete shrinkage of 0.0002 (in./in.), steel stress at the end of the slab would be near 55 ksi (3.79×10^5 kPa) and concrete stress in the center approximately 350 psi (2.41×10^3 kPa). Thus, the weak plane acts as a safety valve by allowing the concrete to crack and reduce stresses before they become excessive.

A simple force equilibrium calculation may be used to estimate the reduction of concrete area to guard against development of excessive steel stress. First, a limiting value of steel stress is selected. An often used limit on steel stress, sometimes referred to as the steel working stress (Ref 4), σ_s , is 0.75 times the yield point. The concrete stress with full concrete area, σ_c , which corresponds to the working steel stress is then calculated using the techniques presented in this appendix. A high level of concrete strength, f_α , is selected from the concrete strength distribution so that the actual concrete strength in the precast slab has a high probability of being less than f_α . The selection of a high concrete strength for this calculation is to assure that the concrete fractures. The reduced concrete area, A'_c , may then be calculated as follows:

$$A'_c f_\alpha \geq A_c \sigma_c \tag{A5.1}$$

$$A'_c \geq \frac{A_c \sigma_c}{f_\alpha} \tag{A5.2}$$

where

A'_c = reduced concrete area,

A_c = full concrete area,

σ_c = concrete stress on full concrete area corresponding to the working stress in the steel,

f_α = $\bar{f} + Z_\alpha S_f$ = concrete strength at α level,

$\bar{f}$ = mean concrete tensile strength,

S_f = standard deviation of concrete strength,

α = probability of concrete strength exceeding this value, and

Z_α = Z statistic at α level.

The percentage reduction of concrete area, P_R, is then

$$P_R = \frac{A_c - A'_c}{A_c} \times 100 \tag{A5.3}$$

GENERAL ANALYSIS

The trial investigation has shown the need to consider shrinkage and thermal volume change in the design of precast slabs for repair of CRCP. To further investigate the effects of this volume change on precast repair slabs and determine design implications, a more general analysis was conducted using the variable combination factorial in Table A5.3. The principal factors varied in this analysis were amount of steel reinforcement and reinforcing bar size. Three levels of reinforcing were selected, with exact areas varying due to reinforcing bar sizes. Two levels of concrete shrinkage and three slab lengths were used in the analysis. The factorial shown in Table A5.3 was repeated in Table A5.4 with a 50 percent reduction in concrete cross sectional area to investigate the weak plane-safety valve concept. The factorial shown in Table A5.5 is an expansion of the 0.6 percent reinforcement design shown in Table A5.3 to bring concrete compressive strength and modulus of elasticity into the analysis.

TABLE A5.3. GENERAL ANALYSIS FACTORIAL

% Steel Reinforcement	Steel Area, in^2, and Bar Diameter, in.	Concrete Shrinkage	Slab Length 3		Slab Length 6		Slab Length 12	
.51	5.89 in^2	.00005	39.0	205	576	279	1260	374
			40.0	.00464	54.9	.0089	74.1	.0154
	.625 in	.0002	-3950	256	-3260	351	-2360	474
			45.8	.00721	65.0	.0135	89.9	.0247
	5.72	.00005	-223	169	231	232	825	314
			33.7	.00481	46.7	.00903	63.7	.0165
	.75	.0002	-4280	210	-3700	290	-2940	395
			37.8	.00741	54.3	.0141	76.2	.0262
.62	7.13	.00005	319	244	938	329	1710	436
			39.5	.00450	53.8	.00819	71.7	.0144
	.625	.0002	-3590	305	-2790	415	-1770	557
			45.4	.00703	63.9	.0130	87.6	.0234
	7.04	.00005	35.0	205	571	279	1260	373
			33.3	.00468	45.9	.00866	62.0	.0155
	.75	.0002	-3950	255	-3260	350	-2370	473
			37.5	.00726	53.6	.0136	74.6	.0249
.72	8.37	.00005	588	281	1280	376	2120	493
			39.0	.00435	52.7	.00781	69.5	.0134
	.625	.0002	-3240	353	-2340	477	-1210	634
			45.0	.00686	62.8	.0125	85.4	.0221
	8.36	.00005	285	239	893	323	1660	429
			33.0	.00455	45.1	.00831	60.3	.0146
	.75	.0002	-3630	299	-2850	407	-1840	547
			37.3	.00711	52.9	.0132	72.9	.0237

Key

A	B
C	D

A. Steel stress in center of slab, psi (- is compressive stress)

B. Concrete stress in center of slab, psi

C. Steel stress at end of slab, ksi

D. Concrete movement at end of slab, in.

1 ksi = 6.89 mPa, 1 psi = 6.89 kPa, 1 ft = .305 m, 1 in. = 25.4 mm, 1 $in.^2$ = 6.45 cm^2

TABLE A5.4. FACTORIAL COMBINATIONS OF TABLE A5.3 WITH ONE HALF CONCRETE AREA IN CENTER OF SLAB

Concrete Area, in.^{2}, Steel Area, in.^{2}, Rebar Size	Concrete Shrinkage	Slab Length					
		3		6		12	
573.05	.00005	1260	374	2110	491	3080	625
5.89		37.7	.00141	49.8	.00394	63.9	.0105
#5	.00020	-2360	474	-1230	630	122	817
		43.8	.00169	60.1	.00489	79.6	.0134
573.14	.00005	825	314	1570	417	2470	541
5.72		32.3	.00112	43.4	.00324	56.7	.00888
#6	.00020	-2940	395	-1950	531	-730	699
		36.6	.00132	51.2	.00395	69.3	.0111
572.44	.00005	1710	436	2630	563	3650	703
7.13		36.7	.00165	47.8	.00455	60.1	.0118
#5	.00020	-1760	557	-507	730	948	931
		42.9	.001999	58.1	.00570	75.6	.0153
572.48	.00005	1260	373	2100	489	3070	624
7.04		31.6	.00135	41.9	.00383	53.8	.0103
#6	.00020	-2370	473	-1240	628	109	815
		36.1	.00160	49.9	.00472	66.4	.0131
571.82	.00005	2130	493	3100	627	4130	770
8.37		35.8	.00188	46.0	.00510	56.7	.00130
#5	.00020	-1210	634	151	821	1670	1030
		42.1	.00229	56.2	.00645	72.0	.0171
571.82	.00005	1660	429	2570	555	3580	694
8.36		31.0	.00156	40.5	.00437	51.1	.0115
#6	.00020	-1840	547	-593	718	851	917
		35.5	.00186	48.5	.00543	63.6	.0148

Key

A	B
C	D

A. Steel stress in center of slab, psi
(- is compressive stress)

B. Concrete stress in center of slab, psi

C. Steel stress at end of slab, ksi

D. Concrete movement at end of slab, in.

ksi = 6.89 mPa, psi = 6.89 kPa, ft = .305 m, in. = 25.4 mm, in.^{2} = 6.45 cm^2

A temperature drop of $50^{o}F$ ($28.7^{o}C$) was used in this analysis. Granted, conditions exist where greater temperature variations are expected; however, this drop in temperature is thought to be adequate to serve the purposes of this investigation. The design implications with respect to the safety valve concept will apply to greater temperature drops due to stress control by the weak plane.

The following material properties and dimensions were assigned to the modeled slab:

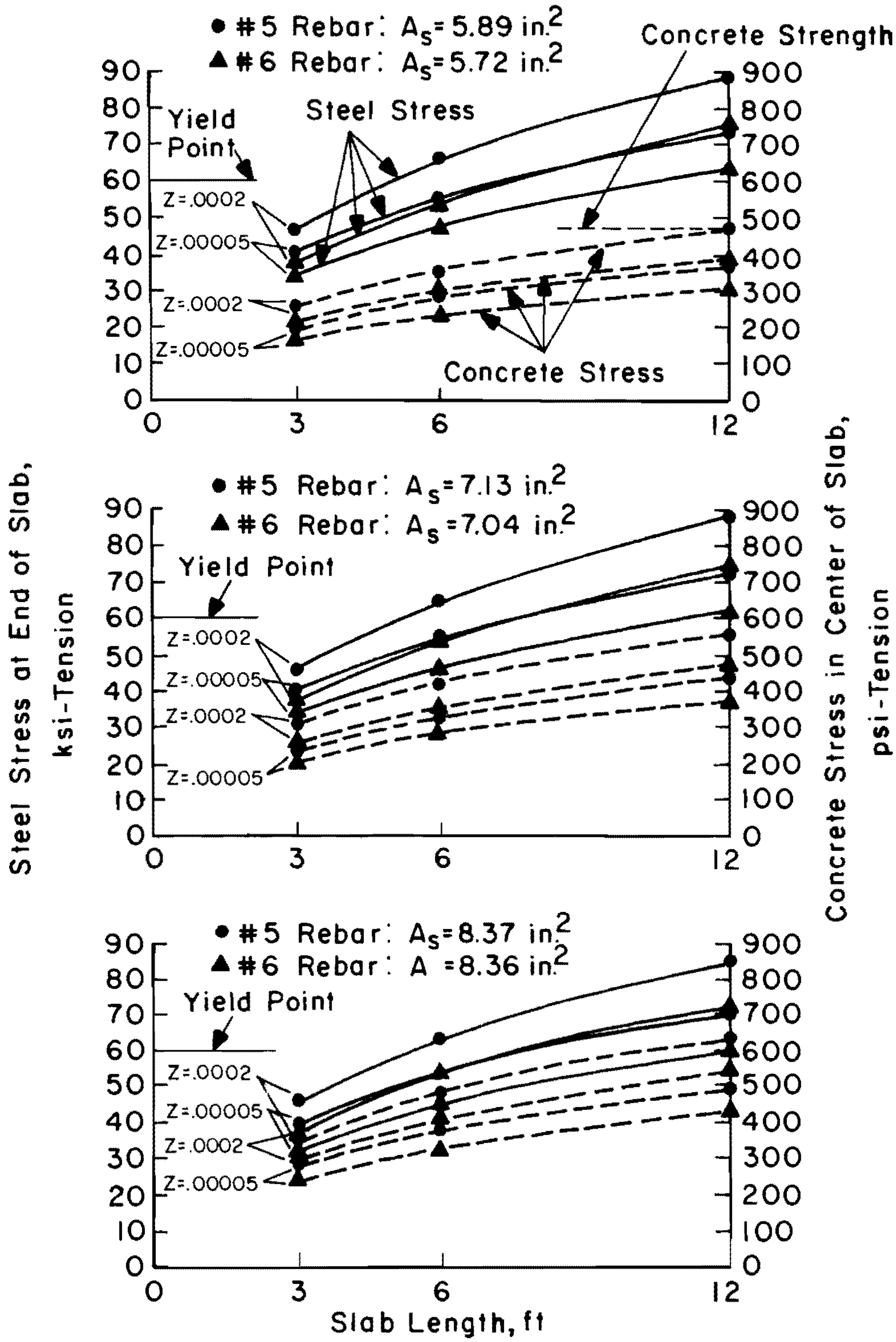
concrete modulus of elasticity = 4.0×10^{6} psi (2.76×10^{7} kPa),
concrete thermal coefficient = 5.0×10^{-6} in./in/^{o}F (9.0×10^{-6} cm/cm/^{o}C),
concrete compressive strength = 4,000 psi (2.76×10^{4} kPa),
steel modulus of elasticity = 29.0×10^{6} psi (1.99×10^{8} kPa),
steel thermal coefficient = 5.0×10^{-6} in./in./^{o}F (9.0×10^{-6} cm/cm/^{o}C),
width = 12 feet (3.65m), and
thickness = 8 inches (203mm).

These values remained fixed throughout the analyses shown in Table A5.3 and A5.4. These values are felt to be representative of expected properties for precast pavement repair slabs.

Calculations for these analyses were based upon the frictionless model in the VCARCS program. Use of the friction model for volume change analysis is more conservative in that higher stresses are calcualted. However as previously shown, the frictionless model yields results comparable to those from the friction model. The frictionless model also requires less computational time.

The calculated stresses for concrete in the center of the slab and steel at the end, shown in Table A5.3, are plotted in Fig A5.9 against slab length. This same information is plotted in Figs A5.10 and A5.11 against the area of steel reinforcement.

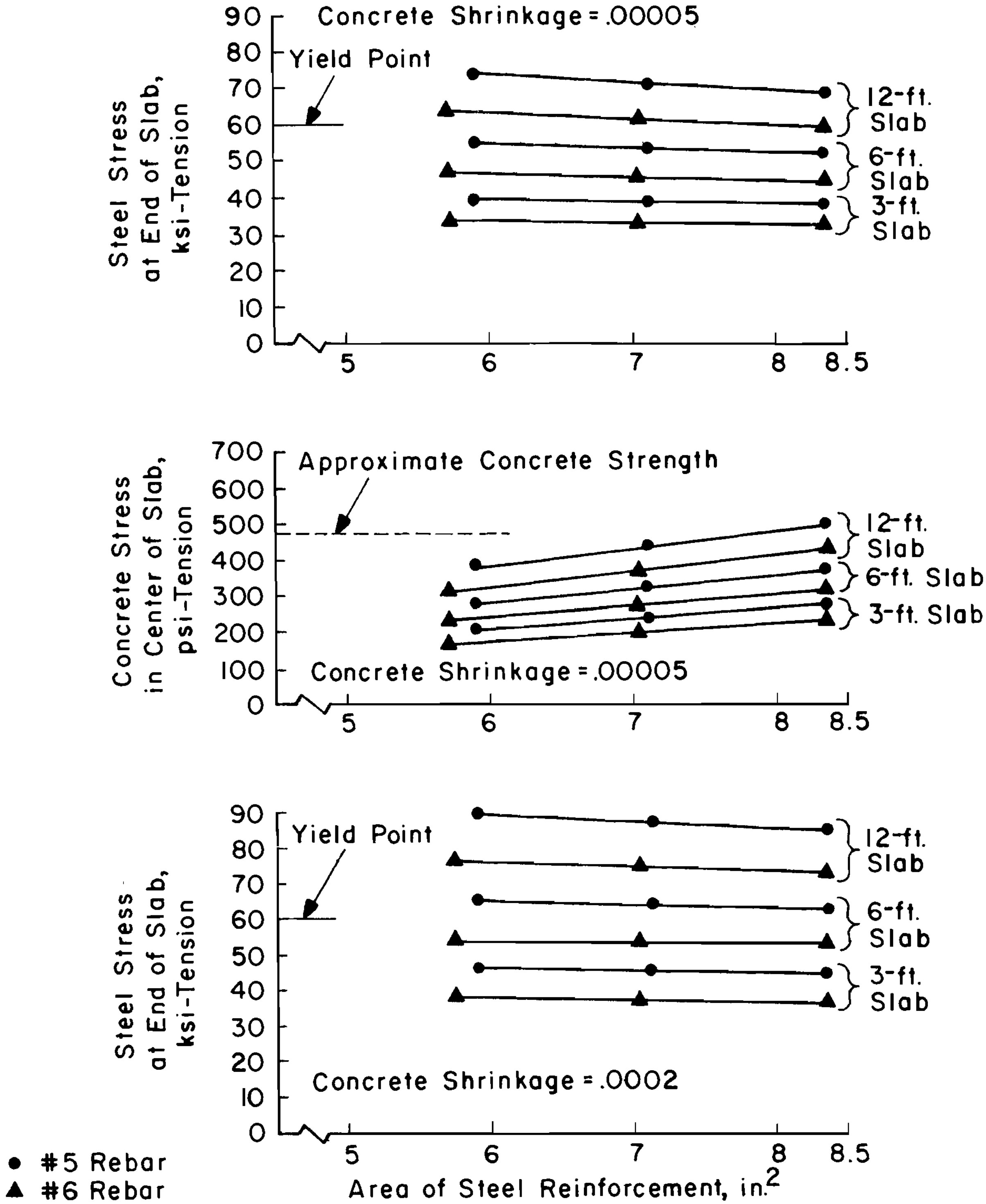
The occurrence of excessive steel stresses, similar to those exhibited in the previous analysis, is seen for the longer slabs. In Fig A5.10, steel stress for a 6-foot (1.83-m) long slab with No. 5 rebars and a concrete shrinkage of 0.0002 is above the yield point for all levels of reinforcement. The corresponding concrete stresses in Fig A5.11 are below the approximate tensile strength.



1 in.^{2} = 6.45 cm^2, 1 ksi = 6.89 mPa,

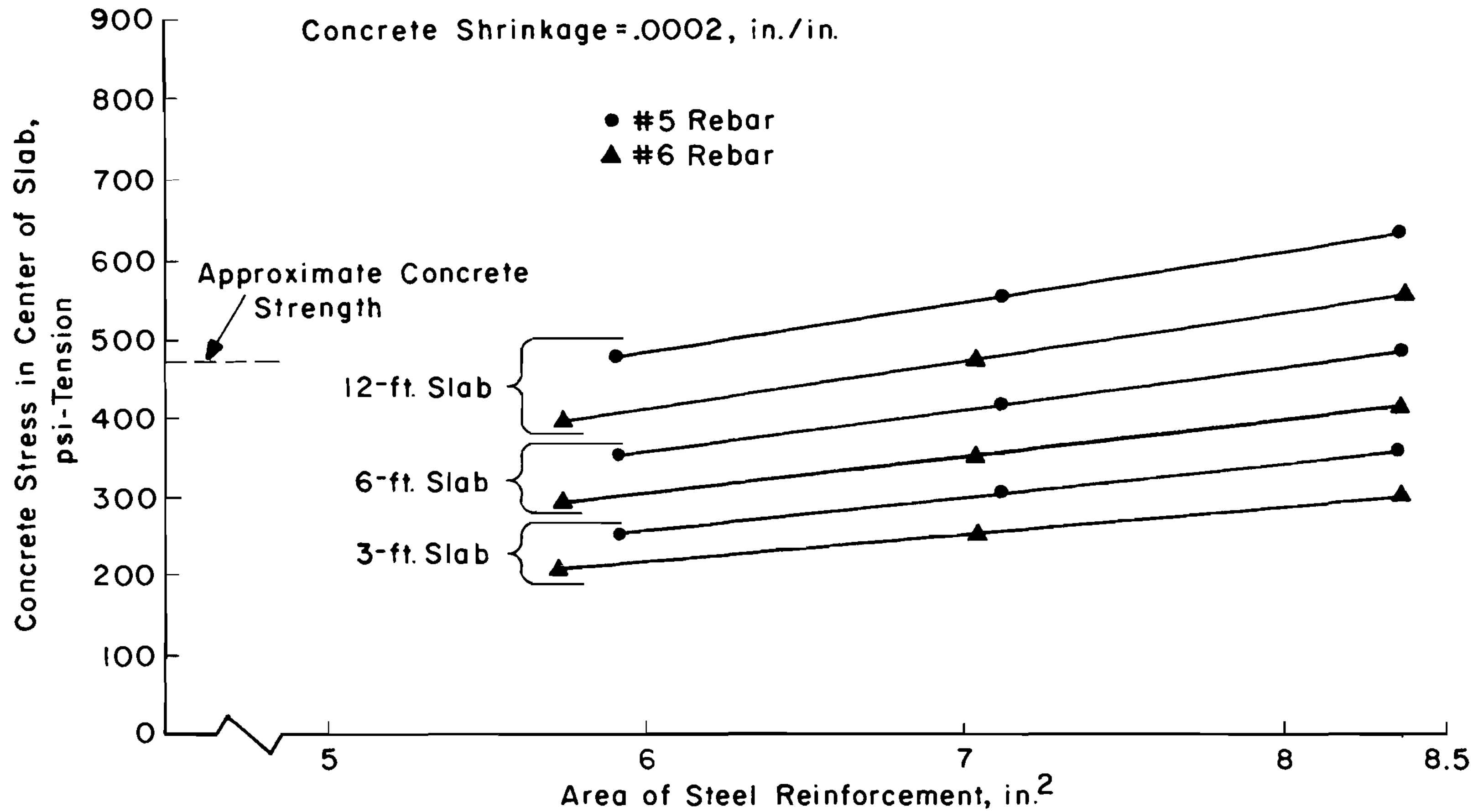
1 psi = 6.89 kPa, 1 ft = .305 m

Fig A5.9. Steel and concrete stresses plotted against slab length, general analysis.



1 in,^{2} = 6.45 cm^2, 1 ksi = 6.89 mPa, 1 psi = 6.89 kPa

Fig A5.10. Steel and concrete stresses plotted against steel area, general analysis.



1 in.^{2} = 6.45 cm^2, 1 ft = .305 m, 1 psi = 6.89 kPa

Fig A5.11. Concrete stress versus area of steel reinforcement, general analysis.

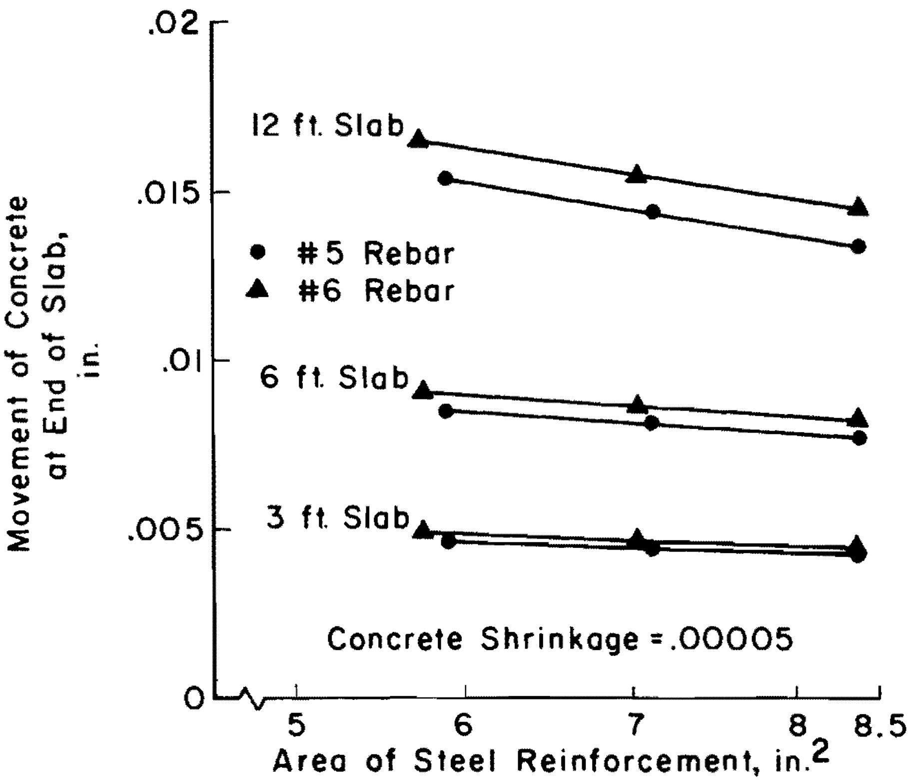
It can be seen in Figs A5.9, A5.10, and A5.11 that the stresses calculated for the No. 5 rebar steel designs were greater than those for the No. 6 rebars for equivalent areas of steel reinforcement. This reinforces the contention in the earlier analysis that lower stresses for the greater steel area were a result of differences in reinforcing bar sizes. The trend in Figs A5.10 and A5.11 shows a decrease in steel stress and an increase in concrete stress for an increase in steel area.

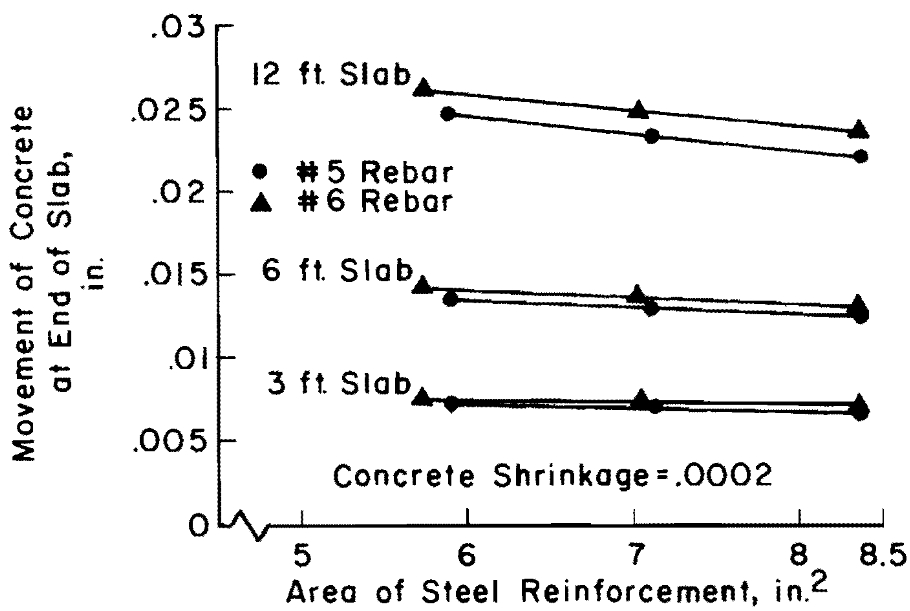
Concrete movement at the end of the slab is plotted against the area of steel reinforcement for both levels of shrinkage in Fig A5.12. The movements increased with slab length and decreased with greater areas of steel. Movements for slabs with the No. 5 rebars were less than those with the No. 6 rebars.

The results of the concrete area reduction tabulated in Table A5.4 are plotted in Figs A5.13 through A5.15. Steel stress at the end of the slab and concrete stress in the center of the slab are plotted against percent of area reduction for each level of steel reinforcement and concrete shrinkage. The steel yield point and approximate concrete tensile strengths are indicated on the appropriate graphs.

In Fig A5.13a, steel stress at the end of the slab is plotted for the 0.51 percent steel design and concrete shrinkage of 0.00005 (in./in.). Stresses for the 3-foot (0.914-m) and 6-foot (1.83-m) long slabs remained below the yield point. Stresses for the 12-foot (3.66-m) long slabs with full concrete area were in excess of yield. Reduction of the concrete area by half produced stresses for the No. 6 rebar design below the yield point. Stresses for the No. 5 rebar design remained above the yield point. Concrete stresses in the center for the slab corresponding to the stresses in Fig A5.13a are shown in Fig A5.13b.

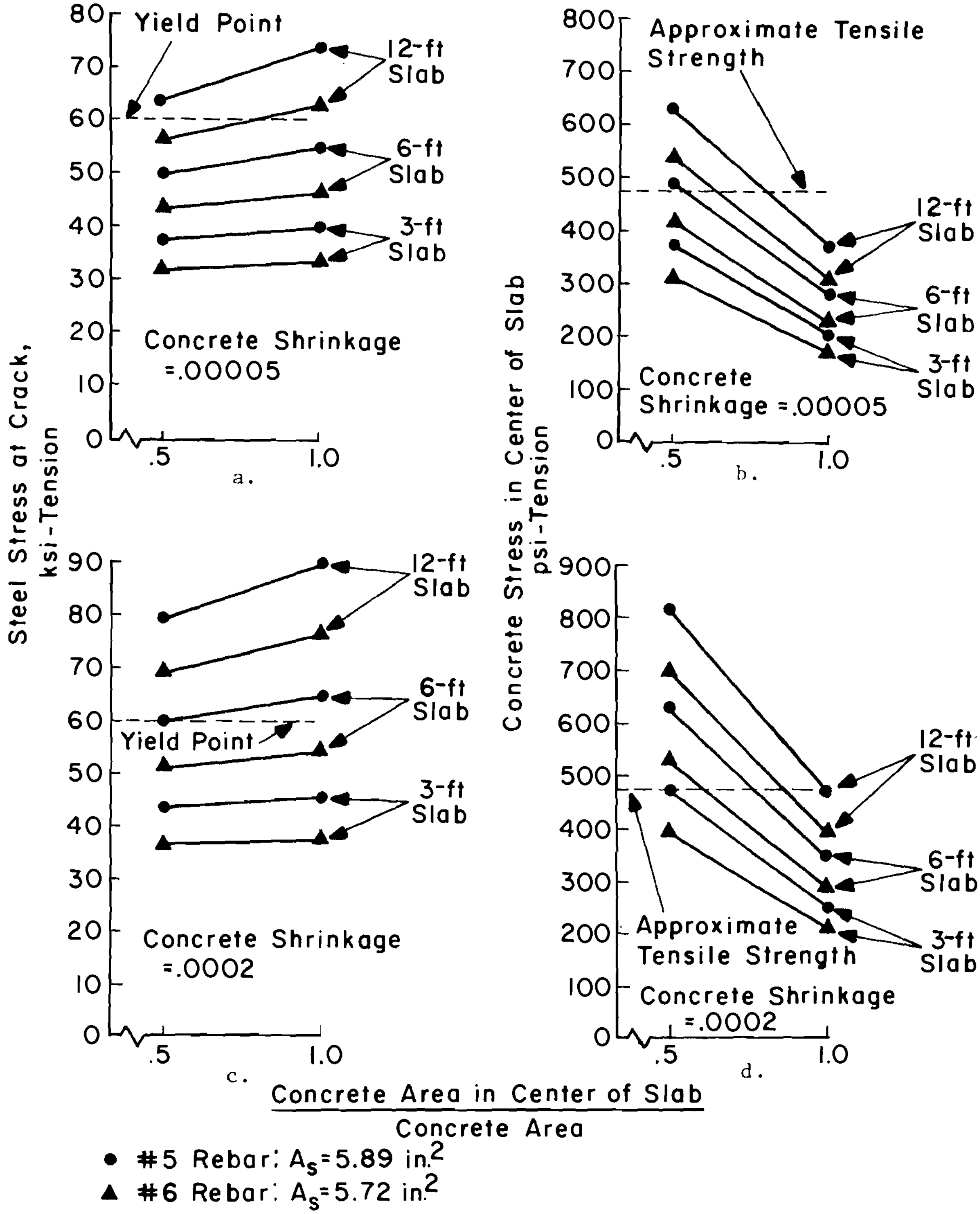
The concrete stresses in Fig A5.13b for full concrete area were below the approximate tensile strength. A one-half reduction in concrete area greatly increased these stresses. Stresses for the 12-foot (3.66-m) long slabs were well above the tensile strength, indicating probable cracking. Stresses for the 6-foot (1.83-m) long slab with the No. 5 rebar exceeded the indicated tensile strength while those for the No. 6 size rebar remained below this level. In general, concrete stresses in the center of the slab increased approximately 70 percent for a 50 percent decrease in concrete area.





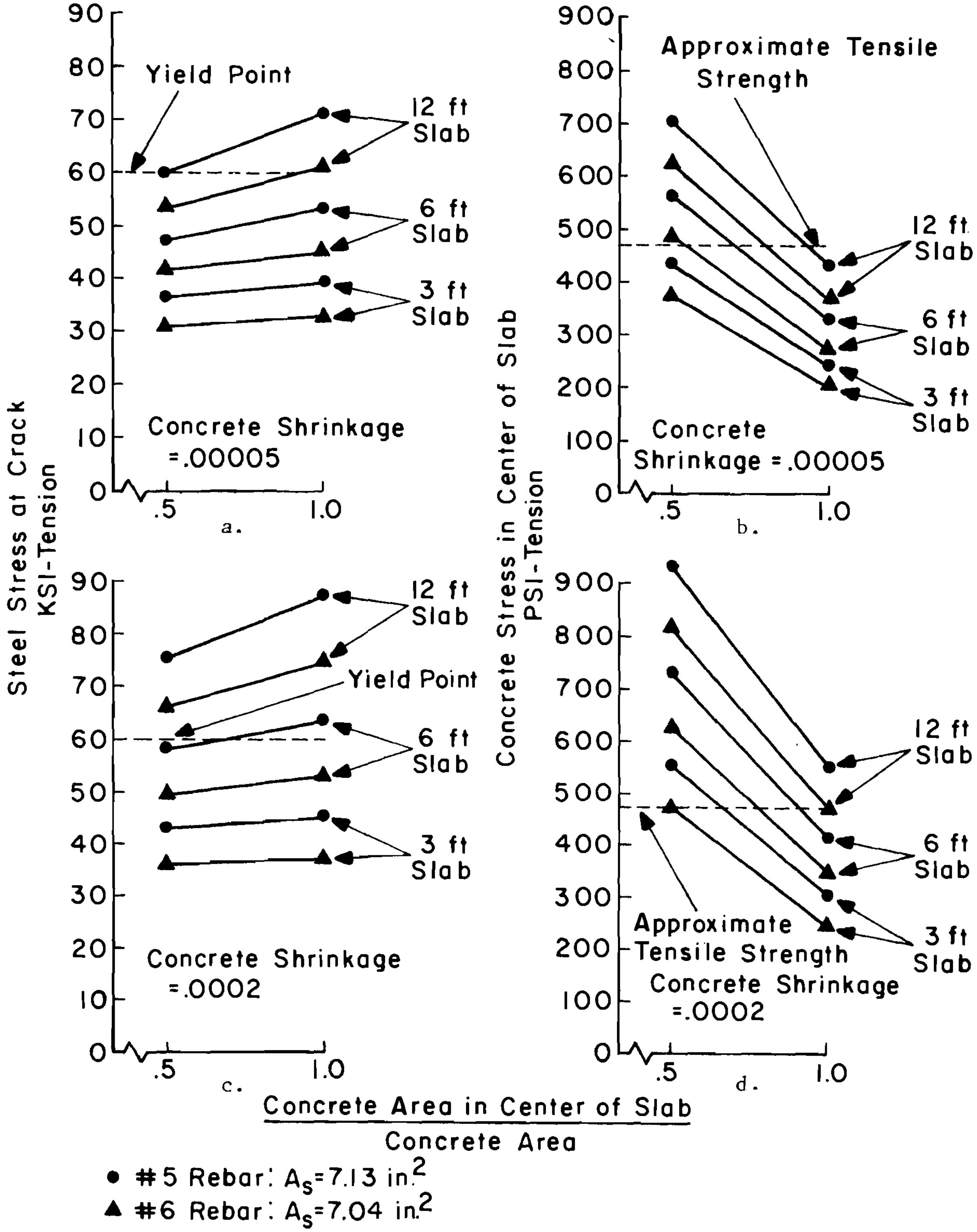
1 in. = 25.4 mm, 1 in^2 = 6.45 cm^2, 1 ft = .305 m

Fig A5.12. Concrete movement versus steel area, general analysis.



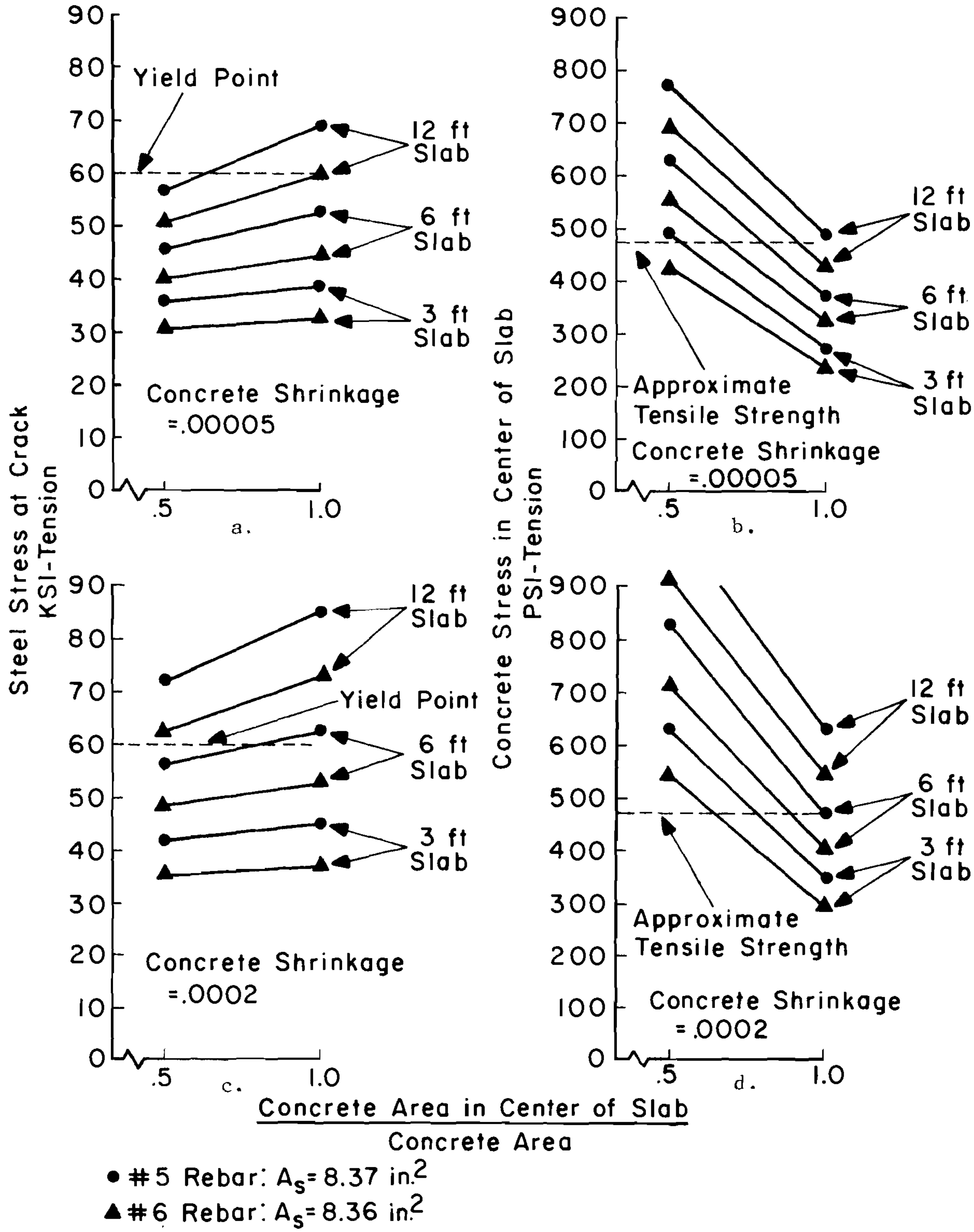
1 ksi = 6.89 MPa, 1 psi = 6.89 kPa, f ft = .305 m, 1 in^2 = 6.45 cm^2

Fig A5.13. Effect of reduced concrete area in center of slab on steel and concrete stresses, general analysis.



1 ksi = 6.89 MPa, 1 psi = 6.89 kPa, 1 ft = .305 m, 1 in^2 = 6.45 cm^2

Fig A5.14. Effect of reduced concrete area in center of slab on steel and concrete stresses, general analysis.



1 ksi = 6.89 MPa, 1 psi = 6.89 kPa, 1 ft = .305 m, 1 in.^{2} = 6.45 cm^2

Fig A5.15. Effect of reduced concrete area in center of slab on steel and concrete stresses, general analysis.

Concrete and steel stresses for the slab designs in Figs A5.13a and A5.13b are plotted in Figs A5.13c and A5.13d for a 0.0002 concrete shrinkage. The additional shrinkage increases stresses past those in fig A5.13a and A5.13b. Steel stresses for the 6-foot (1.83-m) slab with the No. 5 rebar are above the yield point. These stresses at 0.00005 -(in./in.)(mm/mm) concrete shrinkage were below the yield point. Corresponding concrete stresses in Fig A5.13d are below the tensile strength for no reduction in concrete area. Reduction of the concrete area increased the concrete stresses past the tensile strength for 12-foot (3.66-m) and 6-foot (1.83-m) long slabs for both sizes of rebars. Corresponding steel stresses dropped, however, for the 12-foot (3.66-m) slabs and the 6-foot (1.83-m) slabs. With the No. 5 rebar, these stresses were still in excess of yield.

The behavior of a precast repair slab can be inferred from Figs A5.13a, A5.13b, A5.13c, and A5.13d. Take, for example, a 6-foot (1.83-m) long slab with 5.89 square inches (38.0 cm^2) of No. 5 steel reinforcement with a one-half reduction in concrete cross-sectional area in the center of the slab. During the first months after installation, stresses are developed as the concrete shrinks and temperature varies. The concrete stress shown in Fig A5.13b is greater than the tensile strength. The corresponding steel stress is below the yield point. Thus, for some combination of shrinkage and temperature, the concrete can be expected to crack in the center of the slab without steel stress exceeding the yield point. When the concrete fractures, two slabs approximately 3 feet (0.914 m) long are formed. Expected stresses in these slabs are shown in Figs A5.13c and A5.13d for slabs with no area reduction. It is seen that both steel and concrete stresses are below the yield point and tensile strength, respectively.

The general characteristics of the influence of rebar sizes on stress can also be seen through continuation of the above example. In the same figures, stresses for the 6-foot (1.83-m) slab with the No. 6 rebar are less than those with the No. 5 rebar. For 0.00005 - in./in. (mm/mm) concrete shrinkage, concrete stresses have not exceeded the strength. In Fig A5.13d, concrete stresses for the No 6 size rebar are greater than the strength. This indicates that the concrete has probably fractured before reaching this shrinkage level. Through iteration on concrete shrinkage, it was found that for the concrete shrinkage which produced concrete stresses equal to the estimated tensile strength, steel stress at the end of the slab for the No. 6 size rebar was approximately equal to the steel stress at concrete fracture for the No. 5 rebar slabs, which occurred at a lower shrinkage level.

The effects of inclusion of more reinforcing steel seen in Figs A5.10 and A5.11 can also be observed in comparisons amongst Figs A5.13 through A5.15. The addition of greater amounts of reinforcement increases concrete stresses and reduces steel stresses. Thus, slabs with more steel will crack at lower levels of shrinkage, temperature, and steel stress.

The factorial in Table A5.5 was used to investigate the effects of concrete compressive strength. Empirical relationships have been derived which relate the concrete compressive strength to its elastic modulus and bond stress with steel reinforcement. The quantities are directly related and as the magnitude of strength increases, the modulus and bond stresses increase. The results of this investigation are shown in Table A5.5 and plotted in Figs A5.16 through A5.18.

Steel stress at the end of the slab, concrete stress in the center of the slab, and concrete movement at the end of the slab all increased for an increase in concrete compressive strength. These increases were due to the increase in modulus of elasticity and bond stress as they are related to the compressive strength. These results are for restrained concrete slabs with no area reduction in the center. Similar increases in stresses due to increased modulus and bond stresses for slabs with reduced concrete area in the center of the slab are expected.

As previously mentioned, volume change stresses are developed in a precast repair slab prior to restraining of the steel upon installation. These stresses are a result of concrete shrinkage, temperature, and restraint provided by the steel and friction. The factorial in Table A5.6 was used with the VCACS program to investigate these stresses. The material properties, slab dimensions, and temperature drop used in the previous analyses were used. The numerical results of this analysis are also indicated in Table A5.6.

Frictionless analysis of reinforced slabs with no restraint at the ends does not consider the slab's length. The friction-stress curve shown in Fig A5.19 was devised to demonstrate the effect of slab length. This friction curve is representative of a highly frictional surface. The coefficient of friction was varied from 1.4 to 2.1.

TABLE A5.5. FACTORIAL WITH VARYING CONCRETE STRENGTH FOR .62 PERCENT STEEL DESIGNS FROM TABLE A5.3

Concrete Compressive Strength and Elastic Modulus, psi	Steel Area, in^2 Rebar Diameter, in Concrete Area, in^2	Concrete Shrinkoge in./in.	Slab Length, Ft					
			3		6		12	
F_c = 2,000	A_s = 7.13	.00005	609	201	1310	269	2160	352
	D_b = .625		32.9	.00439	44.4	.00785	58.6	.0135
	A_c = 1144.87	.0002	-3220	252	-2310	341	-1160	452
			37.3	.00691	52.4	.0126	71.4	.0222
E_c = 2.83 x 10^6	A_s = 7.04	.00005	285	169	894	229	1660	303
	D_b = .75		27.8	.00460	38.1	.00839	51.0	.0148
	A_c = 1144.96	.0002	-3630	212	-2850	288	-1840	387
			30.8	.00718	44.0	.0133	61.0	.0240
F_c = 4,000	A_s = 7.13	.00005	319	244	938	329	1712	436
	D_b = .625		39.5	.00450	53.8	.00819	71.7	.0144
	A_c = 1144.87	.0002	-3590	305	-2790	415	-1760	557
			45.4	.00703	63.9	.0130	87.6	.0234
E_c = 4.0 x 10^6	A_s = 7.04	.00005	35.0	205	571	279	1260	373
	D_b = .75		33.3	.00468	45.9	.00866	62.0	.0155
	A_c = 1144.96	.0002	-3950	255	-3260	350	-2370	473
			37.6	.00726	53.6	.0136	74.6	.0249
F_c = 6,000	A_s = 7.13	.00005	166	273	741	370	1470	493
	D_b = .625		44.0	.00456	60.2	.00838	80.7	.0149
	A_c = 1144.87	.0002	-3780	341	-3040	465	-2090	627
			50.9	.00710	71.7	.0133	98.6	.0240
E_c = 4.90 x 10^6	A_s = 7.04	.00005	-96.0	229	399	312	1040	421
	D_b = .75		37.1	.00472	51.2	.00881	69.4	.0160
	A_c = 1144.96	.0002	-4120	285	-3480	391	-2660	531
			42.2	.00731	60.1	.0138	83.7	.0255
F_c = 8,000	A_s = 7.13	.00005	64.6	296	609	402	1300	538
	D_b = .625		47.5	.00460	65.1	.00851	87.6	.0152
	A_c = 1144.87	.0002	-3910	368	-3220	504	-2310	682
			55.2	.00715	77.8	.0134	107.1	.0245
E_c = 5.66 x 10^6	A_s = 7.04	.00005	-183	247	285	339	894	457
	D_b = .75		40.0	.00476	55.3	.00892	75.3	.0163
	A_c = 1144.96	.0002	-4230	307	-3630	423	-2850	576
			45.7	.00735	65.2	.0139	90.9	.0258

Key

A	B
C	D

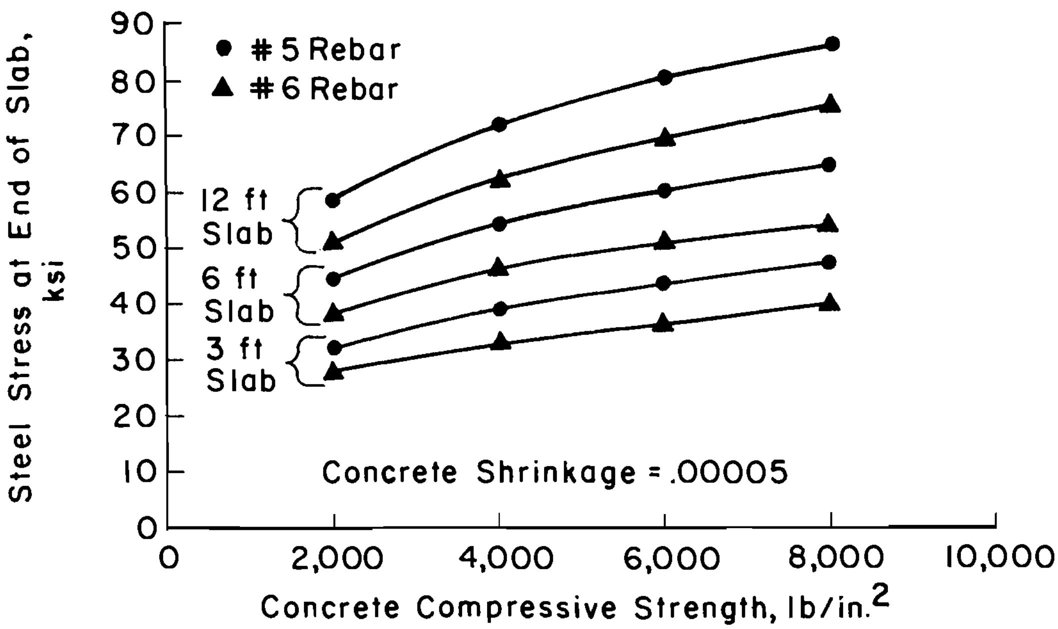
A. Steel stress in center of slab, psi (- is compressive stress)

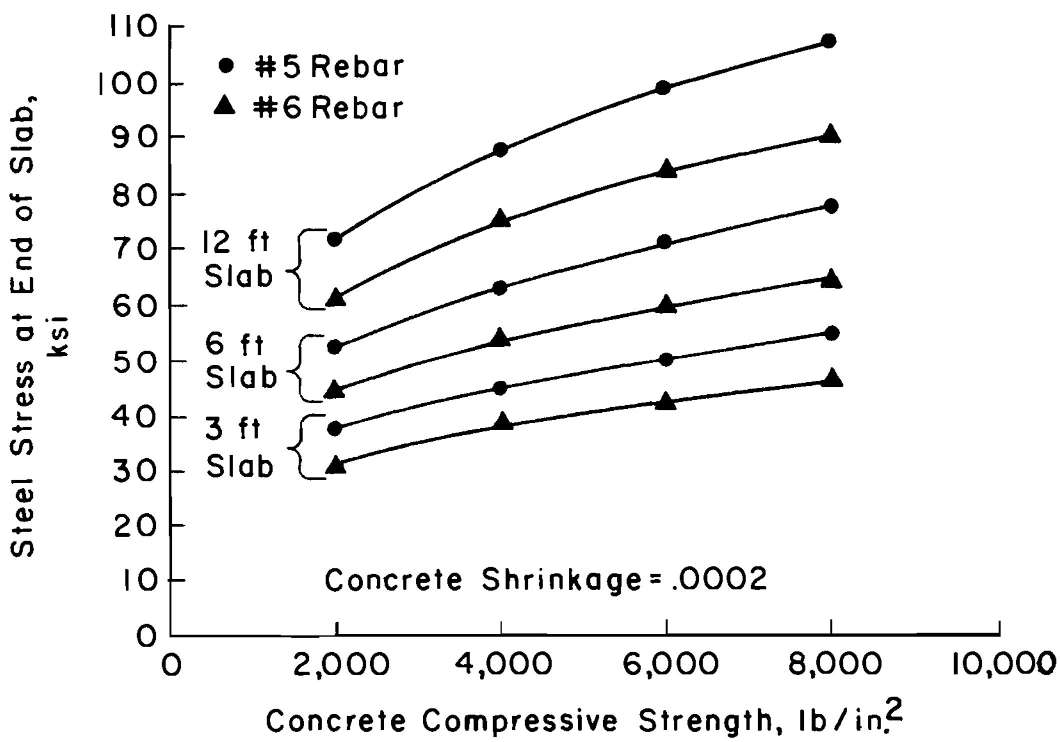
B. Concrete stress in center of the slab, psi

C. Steel stress at end of slab, ksi

D. Concrete movement at end of slab, in.

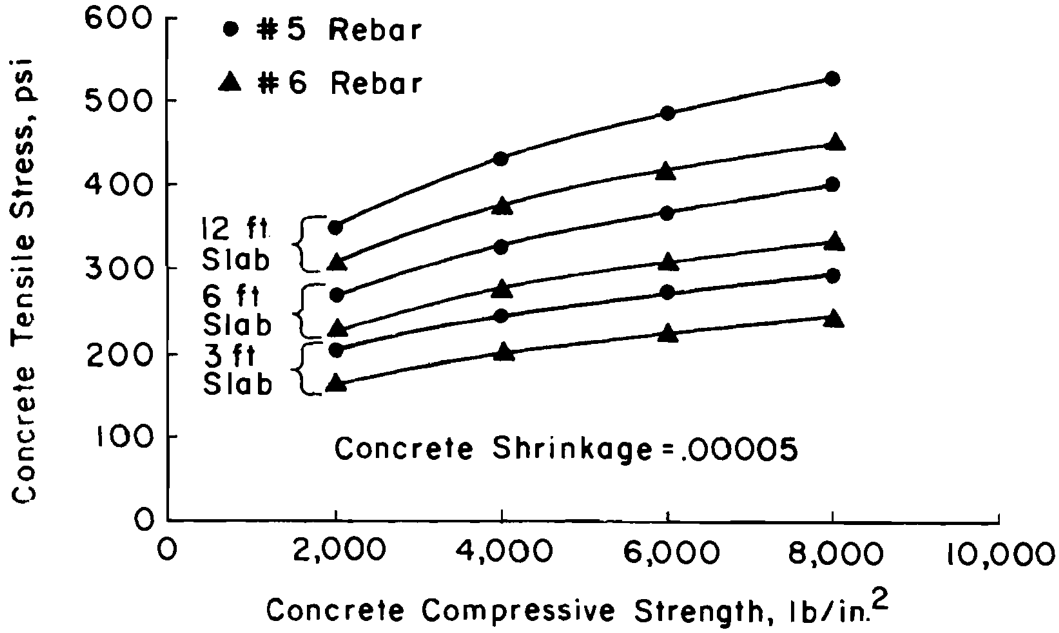
1ksi = 6.89 mPa, 1psi = 6.89 kPa, 1ft = .305 m, 1in. = 25.4 mm, 1in.^{2} = 6.45 cm^2

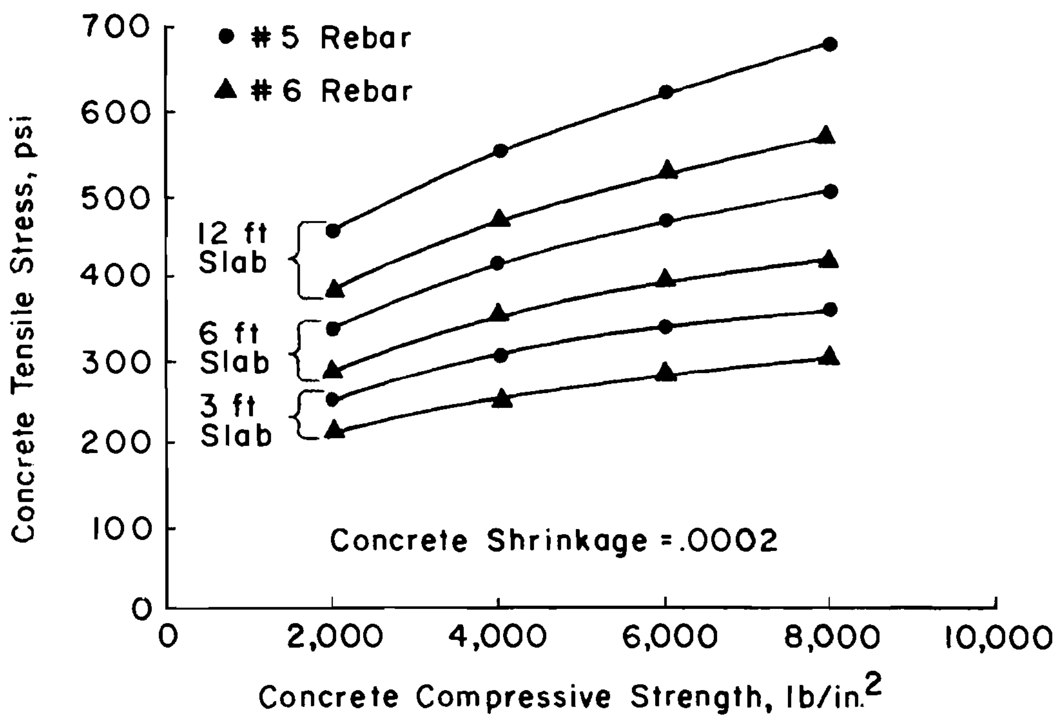




1 lb/in.^{2} = 6.89 kN/m^2, 1 ft = .305 m, 1 ksi = 6.89MPa

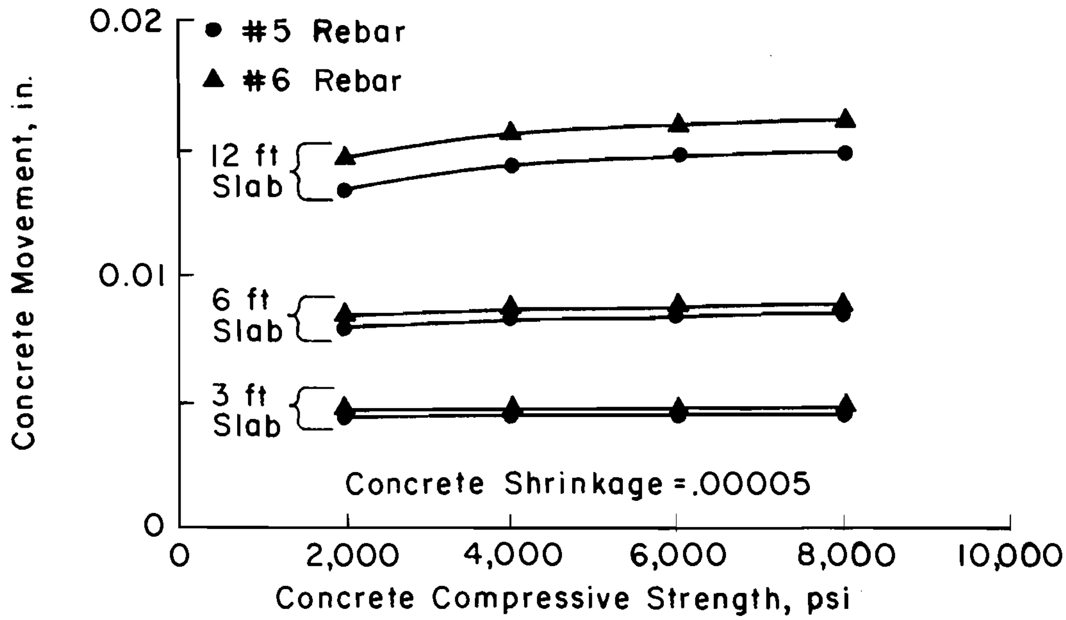
Fig A5.16. Steel stress versus concrete strength for factorial in Table A5.5.

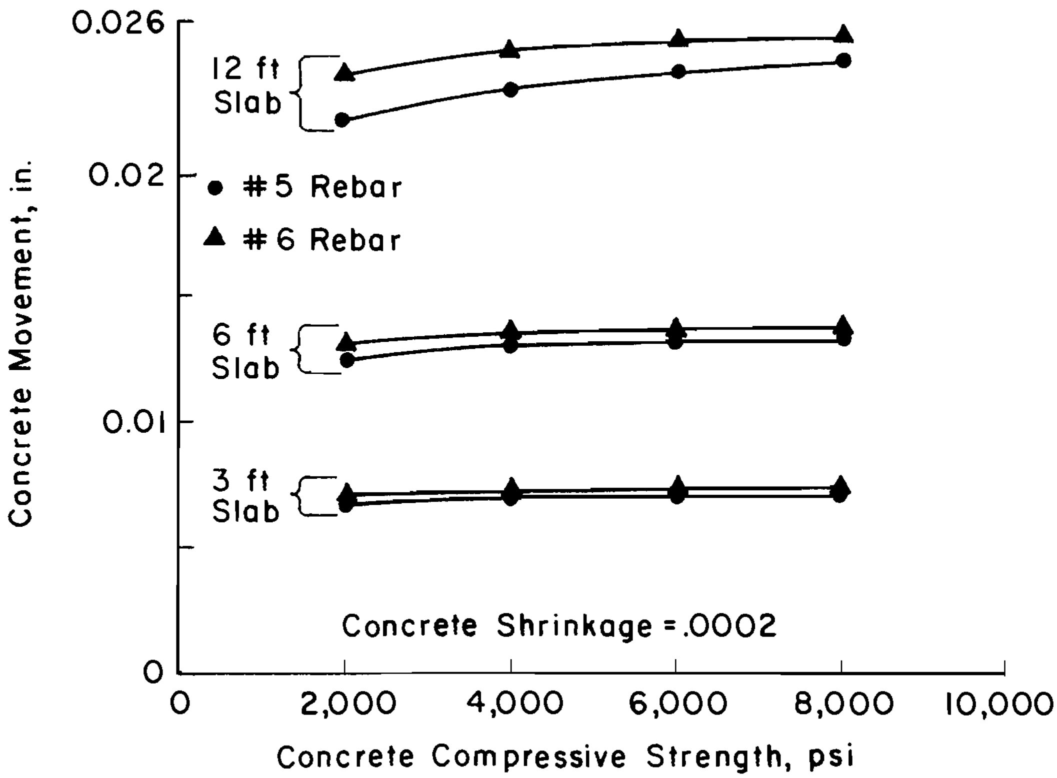




1 lb/in.^{2} = 6.89 kN/m^2, 1 ft = .305 m

Fig A5.17. Concrete stress in center of the slab versus concrete strength for factorial in Table A5.5.





1 lb/in.^{2} = 6.89 kN/m^2, 1 in. = 25.4 mm, 1 ft = .305

Fig 5.18. Movement of concrete at end of slab versus concrete strength for factorial in Table A5.5.

TABLE A5.6. FACTORIAL FOR VOLUME CHANGE ANALYSIS OF CONCRETE SLABS WITH NO STEEL RESTRAINT AT THE END

Percent Steel Reinforcement	Concrete Shrinkage	Slab Length, ft		
		3	6	12
.51	.00005	-1394 7.8 .00536	-1381 9.5 .0107	-1349 13.9 .0214
	.00020	-5585 29.6 .00796	-5570 31.7 .0159	-5538 36.1 .0318
.62	.00005	-1383 9.22 .00536	-1371 10.9 .0107	-1339 15.3 .0214
	.00020	-5443 35.4 .00794	-5528 37.5 .0159	-5496 41.9 .0317
.72	.00005	-1373 10.6 .00535	-1360 12.4 .0107	-1329 16.7 .0213
	.00020	-5502 41.1 .00791	-5487 43.2 .0158	-5455 47.6 .0316

1 psi = 6.89 kPa

1 ft = .305 m

1 in. = 25.4 mm

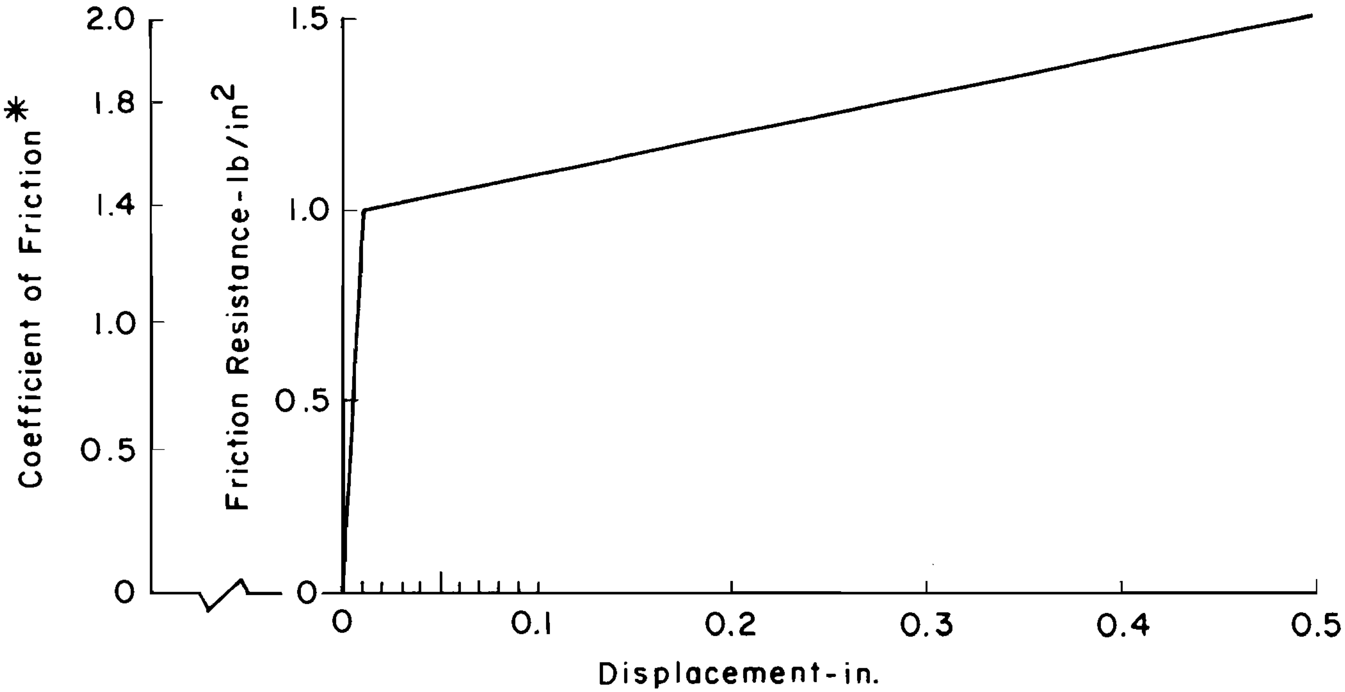
Key

A B C

A. Steel stress in center of slab, psi (- is compressive stress)

B. Concrete stress in center of slab, psi

C. Concrete movement of end of slab, in.



*Based Upon 8 in. Thick Slab With Unit Weight = 154.3 lb/ft^3

1 in. = 25.4 mm, 1 psi = 6.89 kPa, 1 lb/ft^3 = 16.0 kg/m^3

Fig A5.19. Stress-displacement curve used for friction calculations on slabs with no steel restraint at the end.

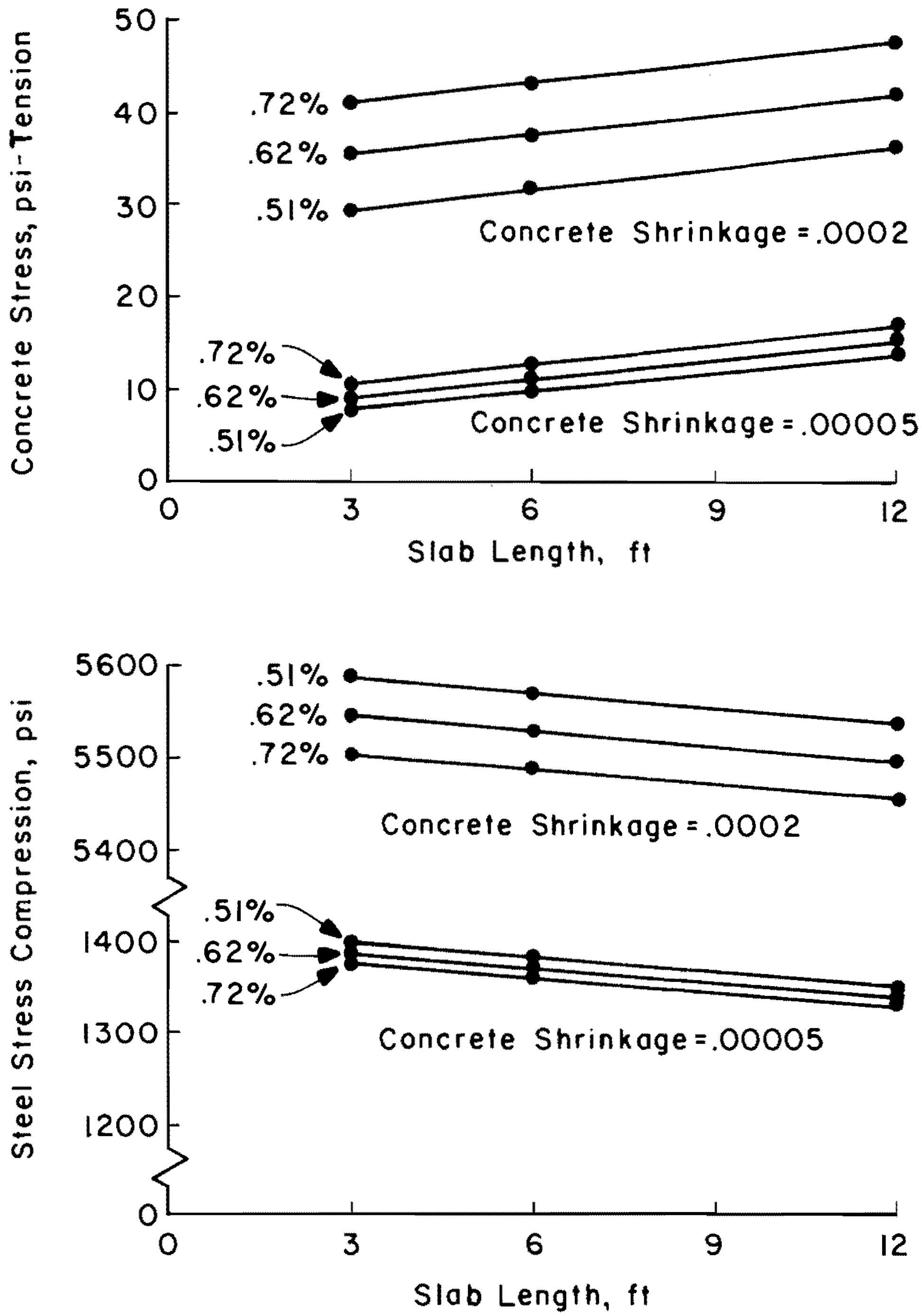
The concrete and steel stresses in the center of the slab, plotted in Fig A5.20, are for full concrete area. The effects of the variables shown in the factorial are exhibited in these graphs. Note the inverse relationship between steel and concrete stress with respect to both slab length and percent steel reinforcement.

Concrete stress is plotted for the 12-foot (3.66-m) slab against the concrete area in the center of the slab in Fig A5.21. This was the largest stress calculated in this analysis. The largest tensile stress calculated was less than 100 psi (689 kPa). This result indicates that inclusion of a weak plane in the center of the slab will probably not cause the slab to fracture before installation into a pavement.

CONCLUSIONS

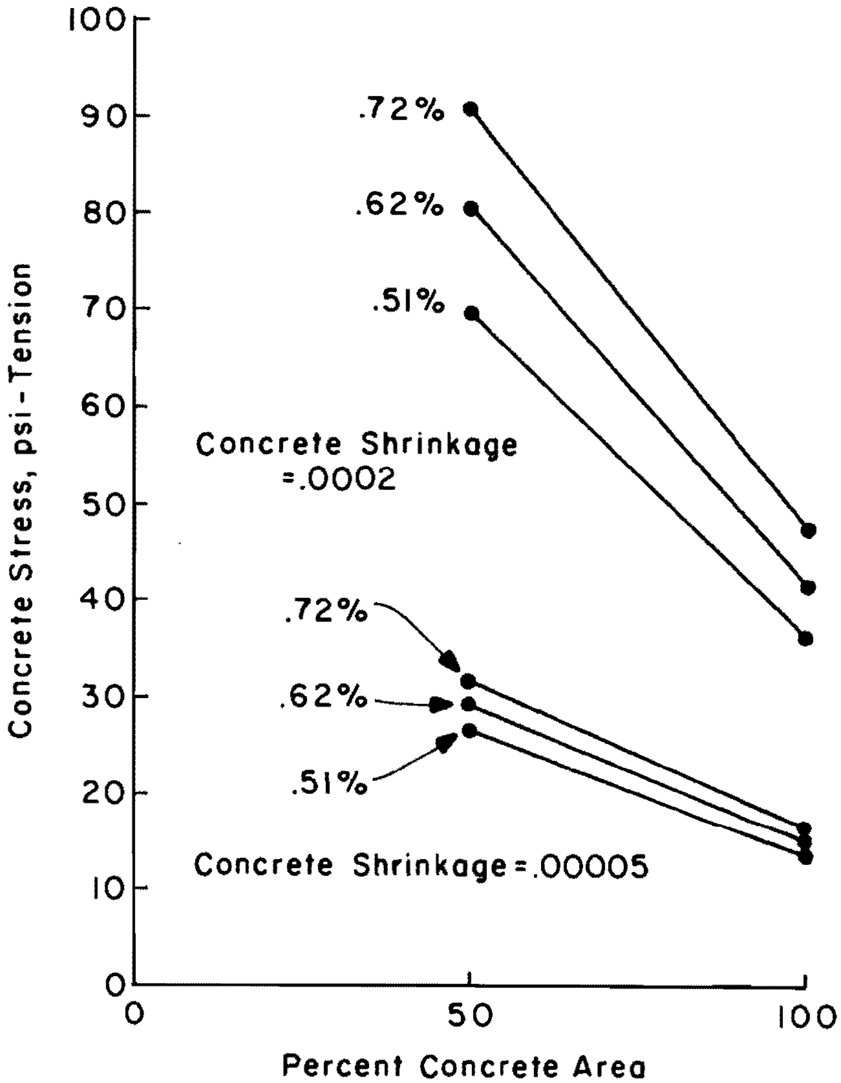
This analysis has shown the expected behavior of precast repair slabs for CRCP due to volume change. Design implications can be drawn from this behavior. Slabs longer than about 6 feet will have potential problems with excessive steel stresses at the end of the slab. On longer slabs it was shown that reduction of the concrete area in the center of the slab can act as a safety valve, allowing the concrete to fracture before steel stresses become excessive. It was also observed that inclusion of more steel into a slab increased concrete stresses while decreasing steel stresses, aiding the safety valve mechanism. Smaller reinforcing bars exhibited greater steel and concrete stresses and less concrete movement than the larger rebars. Thus, if a trial reinforcement design produces excessive steel stresses, increasing the amount of steel reinforcement and decreasing the rebar size can be tried on the next design iteration to limit excessive stresses. Higher strength concrete mixes were found to also increase both steel and concrete stresses.

The analysis presented here has shown only slight differences in results between the frictionless and friction models in the computer program. Larger differences can be expected for higher friction curves than those used in the frictional analysis presented here. It is suggested that initial slab designs be evaluated with the frictionless model. This model requires less time than the friction model. When a feasible design is achieved, it can be further evaluated with friction model calculations.



1 ft = .305 m, 1 psi = 6.89 kPa

Fig A5.20. Concrete and steel stress in the center of the slab, no steel restraint at end of slab.



1 psi = 6.89 kPa

Fig A5.21. Concrete stress in center of slab versus concrete area in center of slab, no steel restraint at end.

APPENDIX 6

TEST OF WELD AND CABLE CLAMP STEEL CONNECTIONS

APPENDIX 6. TEST OF WELD AND CABLE CLAMP STEEL CONNECTIONS

A series of simple tensile tests were conducted on Number 5 grade 60 deformed steel reinforcing bars joined together by welding and cable clamps. These tests were conducted to evaluate the applicability of using this type of connections for steel anchorage at the end of a precast repair slab.

TESTING OF WELDED REINFORCING BARS

Twelve number 5 deformed steel reinforcing bars were obtained from District 19 of the SDHPT (State Department of Highways and Public Transportation). These reinforcing bars were removed from a portion of continuously reinforced concrete pavement (CRCP) which was being repaired. Six of the reinforcing bars were lap welded in the field under similar conditions to those of new steel reinforcement being welded into the repair sites. These six welded bars were prepared to provide a comparison of the weld strength to the strength of the unwelded reinforcing bars.

Eleven of the reinforcing bars had similar deformations and grade marks. One of the unwelded reinforcing bars had different deformation pattern and grade marks than the other bars. The similar bars' grade marks indicated they were made from new billet steel with a yield stress of 60 ksi (414 MPa) and were all produced at the same manufacturing plant. The dissimilar bar's grade marks indicated that it was made with new billet steel and produced at a different plant than the other bars. The yield stress of this steel could not be interpreted from the grade marks. All of the reinforcing bars were approximately 32 inches (813 mm) long.

The welds all appeared to have similar construction and uniformity. The length of each weld was 7 inches (178 mm). The welds were performed by placing two bars side by side with a 7-inch lap (178 mm). One side of the bars was then welded along the lap.

Testing

A Bud Instruments 60k hydraulic testing machine equipped with tension jaws was used to test the reinforcing bars. A dial guage was installed on some of the specimens to measure deformation. Figure A6.1 shows a welded reinforcing bar in the testing machine with a dial guage attached.

The test procedure consisted of placing the bars in the test machine's grips, loading the bar to 3000 pounds (13.3 kN), placing a dial guage on the specimen (if used), and loading until failure. The dial guage was not used on all specimens.

This method of testing the welded reinforcing bars induces moments into the steel due to the eccentric loading. This loading is necessitated by the fact that the only available testing machines have loading heads positioned directly above each other. Under field loading, the welded bars are also subject to a similar type of eccentric loading, inducing moments into the bars. Thus, the induced moments are not a detrimental factor in this testing, considering field conditions.

Test Results

The results of the testing are given in Table A6.1. The yield and ultimate strengths are given in units of 1,000 pounds (4.45 kN). A brief description of the position of the fracture is also given in Table A6.1.

It is seen in Table A6.1 that all the welds developed the yield strength of the steel. In fact, most of the welds developed nearly the ultimate strength of the reinforcing steel. One of the welds developed the ultimate strength of the steel and fractured in the bar above the weld. The majority of the welded bars fractured at the lower tip of the weld. This is a zone of high stress due to bending of the bar. The average ultimate strength of the welded bars which fractured near the weld was 27,000 pounds (120 kN). The average yield strength of these bars was 18,000 pounds (80 kN).

The first five unwelded reinforcing bars shown in Table A6.1 are made with the same type steel as the welded bars. The yield strength of these bars was 18,000 pounds (80 kN). This corresponds to a stress of 60 ksi (414 MPa), which was expected for this steel. The average ultimate strength for these bars was 30,800 pounds (137 kN).

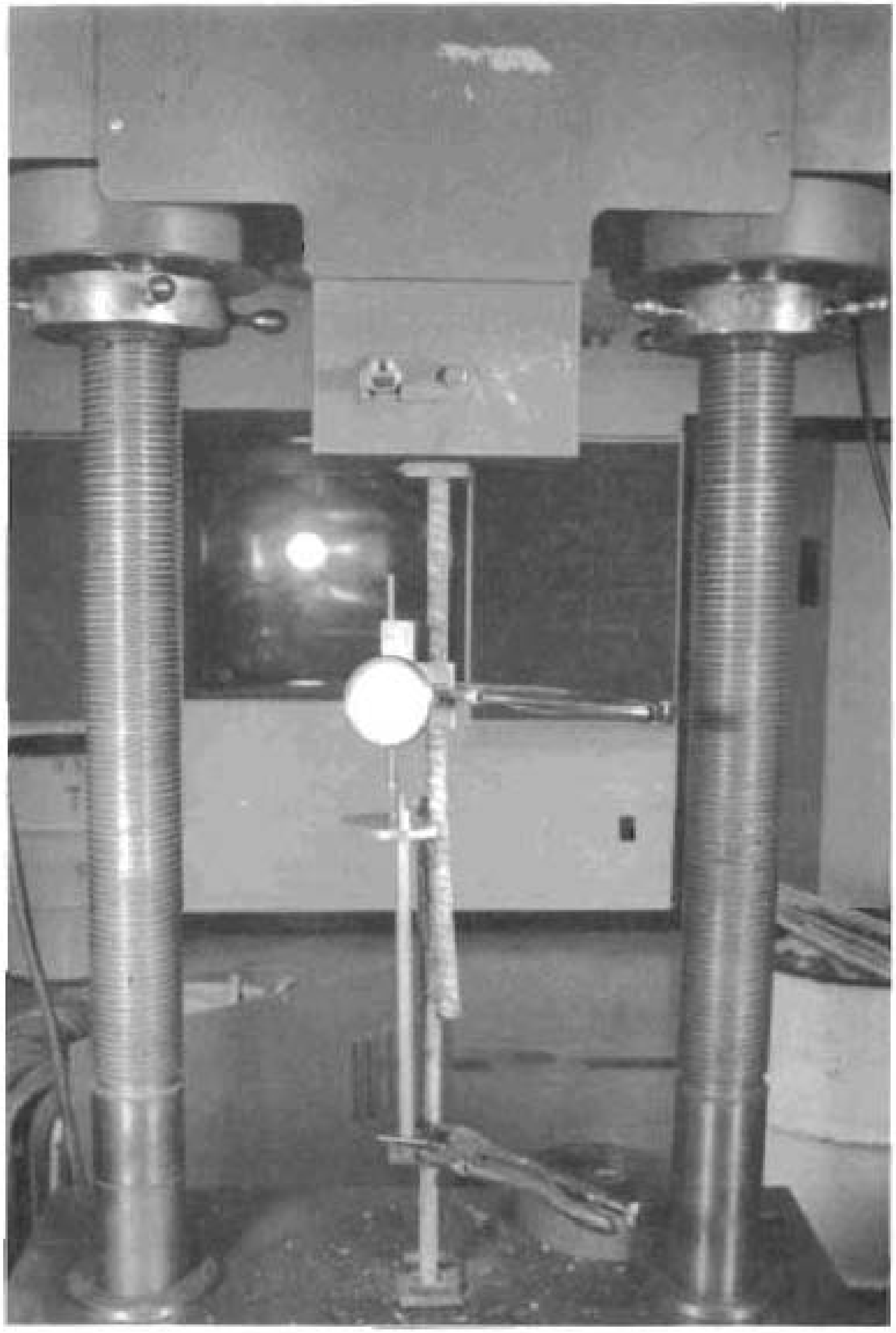


Fig A6.1. Welded reinforcing bar in test machine.
Note bending of bar at tips of the welds.

TABLE A6.1 TENSILE TEST RESULTS

WELDED REINFORCING BARS

Bar Number	Ultimate Strength, kips	Yield Strength, kips	Location of Fracture
1	27.1	18	Lower tip of weld
2	28.8	18	Lower tip of weld
3	24.2	18	Lower tip of weld
4	30.0	18	In bar above weld
5	26.7	18	Lower tip of weld
6	28.1	18	Lower tip of weld

UNWELDED REINFORCING BARS

Bar Number	Ultimate Strength, kips	Yield Strength, kips	Location of Fracture
1	31.0	18	Center of bar
2	28.0	18	At a discontinuity
3	32.0	18	Lower 1/3 point of bar
4	32.0	18	Center of bar
5	31.0	18	At a discontinuity
6	37.0	24	Center of bar

1 kip = 4.45 kN

Two of the five unwelded reinforcing bars fractured at identifiable surface defects. These defects were small and were no deeper than 0.1 inch (2.54 mm). The defects were smooth, worn places accompanied with some corrosion. These defects appear to have been caused by differential movements between concrete and steel while the bars were in the pavement. The defects may possibly have been caused either by placement of the steel or during removal of the steel. The bars which fractured at these defects had a slightly lower ultimate strengths than the other bars.

The sixth bar listed in Table A6.1 under the unwelded bars was the dissimilar bar previously mentioned. The yield of this reinforcing bar was 24,000 pounds (107 kN) which corresponds to 75 ksi (334 mPa). The ultimate strength of this bar was 37,000 pounds (165 kN). This steel must be grade 75 new billet steel.

The deformation readings collected for some of the specimens are not included in this report. The plot of deformation versus load revealed no significant information.

In Table A6.2, a statistical t-test is presented to test the hypothesis that the mean ultimate strength for the population of welded bars is equal to the mean ultimate strength of the population of unwelded bars. The mean ultimate strength of the sample of welded bars was calculated using all six results. The mean ultimate strength of the sample of unwelded bars was calculated excluding bar number 6, the grade 75 steel bar. The results indicate that at the 98 percent level a significant difference is observed and the hypothesis should be rejected.

In practical terms, the difference between the sample means is small. The important result of this testing is that the welds developed strengths well in excess of the yield point.

TESTING OF REBARS CONNECTED WITH CABLE CLAMPS

Cable clamp specimens consisted of two 18-inch (457.2-mm) lengths of number 5 grade 60 billet steel deformed reinforcing bars, lapped 7 inches and bolted together with two 5/8-inch (16mm) drop forged cable clamps placed 2.5 inches (63.5 mm) apart in the center of the lap. Each nut on the clamps

TABLE A6.2. STATISTICAL T-TEST ON ULTIMATE STRENGTH OF WELDED AND UNWELDED REINFORCING BARS

Hypothesis H_o

$$M_w = M_{uw}$$

where

M_w = mean ultimate strength of the population of welded reinforcing bars, and

M_{uw} = mean ultimate strength of the population of unwelded reinforcing bars.

Sample Parameters

U_w	= 27.5k	U_{uw}	= 30.8k
N_w	= 6	N_{uw}	= 5
S_w^2	= 4.0	S_w^2	= 2.7

where

U_w , U_{uw} = mean ultimate strength of the sample of welded and unwelded reinforcing bars,

N_w , N_{uw} = number of welded and unwelded samples, and

S_w^2 , S_{uw}^2 = variance of the sample of welded and unwelded bars.

t-test

$$t = \frac{U_{uw} - U_w}{\sqrt{(N_{uw} - 1)S_{uw}^2 + (N_w - 1)S_w^2}} \sqrt{\frac{N_{uw}N_w(N_{uw} + N_w - 2)}{N_{uw} + N_w}}$$

$$= \frac{30.8 - 27.5}{\sqrt{(5 - 1)2.7 + (6 - 1)4.0}} \sqrt{\frac{(5)(6)(5 + 6 - 2)}{5 + 6}} = 2.94$$

$t(9, .02) = 2.82 < 2.94$ reject at 98% level was tightened as tight as possible by hand. A 100-inch-pound torque wrench showed torques in excess of the wrench's capacity. Three specimens of this type were prepared.

Two other specimens were prepared for this test. These were 18-inch (457.2 mm) long rebars of the same steel. These two specimens were used as a control to determine the tensile characteristics of the steel.

Testing

The test procedure consisted of assembling the specimens, inserting them between the tension grips of the test machine, and then loading them until a maximum load was achieved. The total length between the loading heads at the start of each test was approximately the same. The maximum load was defined as the highest load that could be applied to a specimen without undue slippage. However, no measurements were made of the slippage or strain in the steel. Determination of the maximum load was based on observation of the load indicator dial and the opinion of the people performing the test. Pictures of a prepared specimen and the test setup are shown in Figs A6.2 and A6.3.

The yield point of the single bar specimens was also estimated by watching the load indicator dial. When the load was in the range of the yield, determined from the gradation of the steel and its cross-sectional area, the yield point was recorded as the point where the load remained nearly constant for continuing deformation of the steel. The ultimate strength of the steel was the greatest load applied before rupture.

Results

A summary of test results is presented in Table A6.3. The maximum strength of each of the bolted specimens, yield and ultimate strengths of the single bars, and notes on the test are included in Table A6.3.

Bolted specimen Number 3 was tested first. A protective cage was placed around this specimen to restrain flying pieces of steel in case the specimen shattered. This cage obstructed the view of the specimen during loading. After the load and protective cage had been removed, it was discovered that the steel bars had slipped and the clamps were now resting

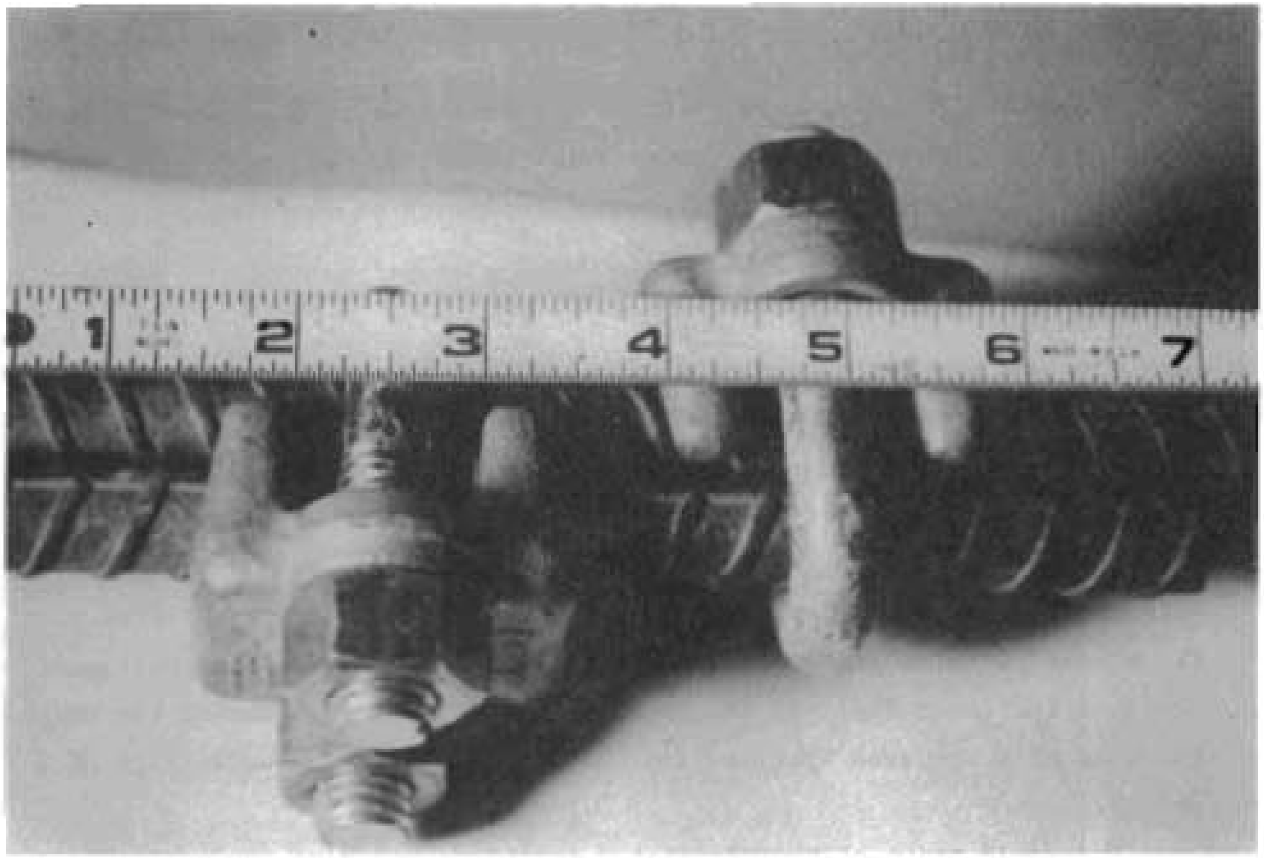


Fig A6.2. Close-up of prepared specimen.

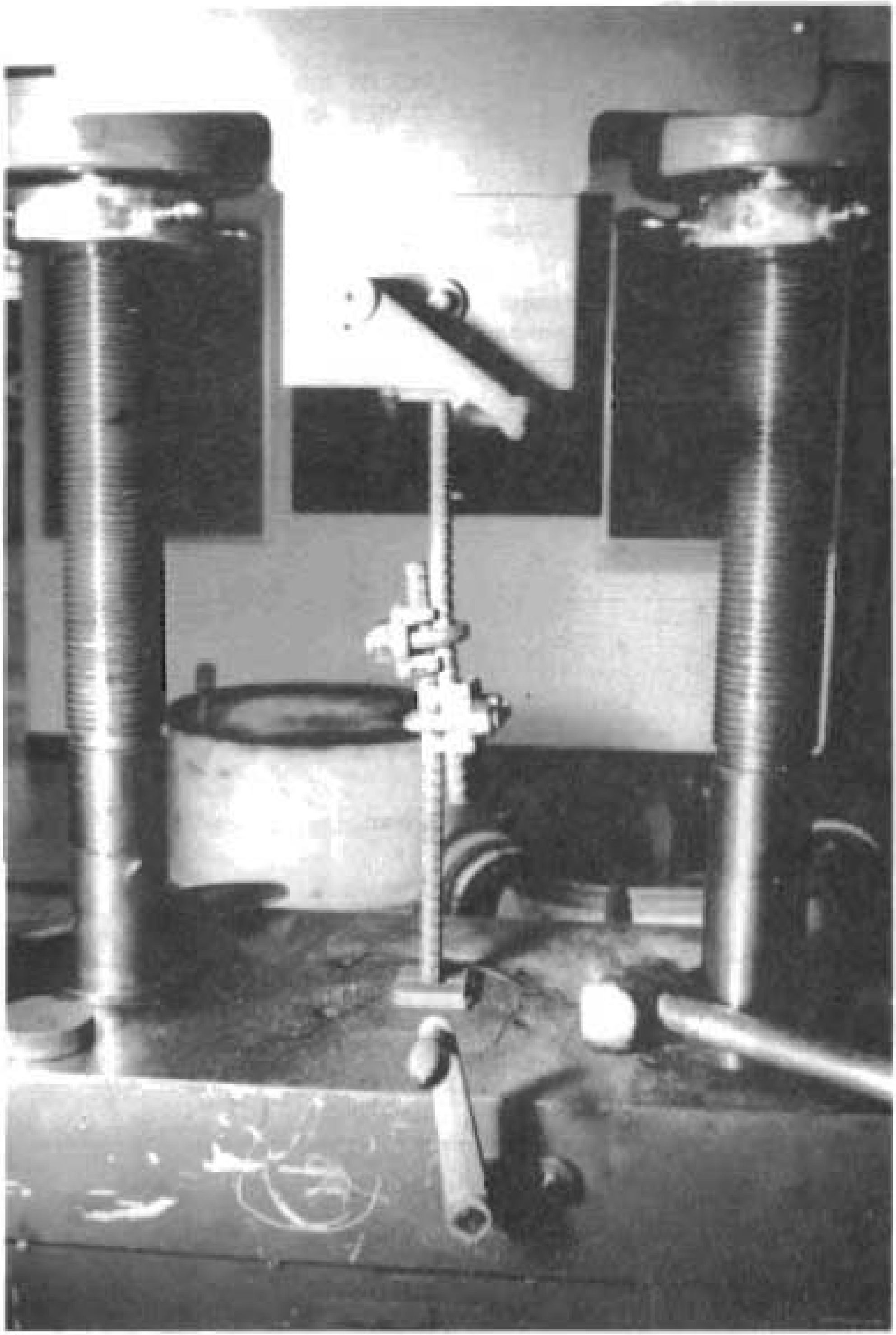


Fig A6.3. Specimen Number 2 in testing machine.

TABLE A6.3. RESULTS OF TENSILE TESTS ON NUMBER 5 GRADE 60 REBARS CONNECTED WITH CABLE CLAMPS AND SINGLE BAR SPECIMENS

Clamped Specimen

Specimen Number	Maximum Strength Developed, pounds	Note
3	23,000	Clamps slipped together
2	19,500	No apparent slippage of clamps
1	21,500	No apparent slippage of clamps

Single Bar

Specimen Number	Estimated Yield Point, pounds	Ultimate Strength pounds
1	18,500	30,500
2	18,500	30,500

1 pound = 4.45 kN

against each other in the center of the bars (Fig A6.4). The largest strength developed by the bolted specimens was recorded for this specimen. However, the previous definition of ultimate load was not strictly adhered to because of the rather large slippage. Photographs of specimens 2 and 1 after testing was completed are shown in Figs A6.5 and A6.6.

The other bolted specimens were tested without the protective cage. These specimens developed their maximum strengths with no noticeable slippage or change in length between the clamps. The average maximum load on these two specimens was approximately 20.5 kips (91.2 kN).

The two single bar specimens exhibited nearly identical behavior. The yield points, roughly estimated, occurred around 18.5 kips (82.3 kN). The ultimate strength of each bar was 30.5 kips (135.7 kN).

Using the estimated yield points and the ultimate strength of the straight bar specimens to represent the tensile strength of the material and the average ultimate load developed by the bolted specimens which did not slip drastically to represent the strength of the clamped specimens, the following statement can be made. The U-bolt cable clamps developed 110 percent of the yield strength and 67 percent of the ultimate strength of the steel.

CONCLUSIONS

The results of these tests indicate that welds and cable clamps may be applicable for use in making steel connections for precast repair slabs. In this situation, the connections will be encased in concrete, which will provide additional strength. Thus, the development of steel yield strengths in the reinforcing bars tested not encased in concrete is very significant. With a proper design, steel stress at these locations should not exceed the yield strength of the steel.

Fig A6.4. Close-up of specimen Number 3 after test.

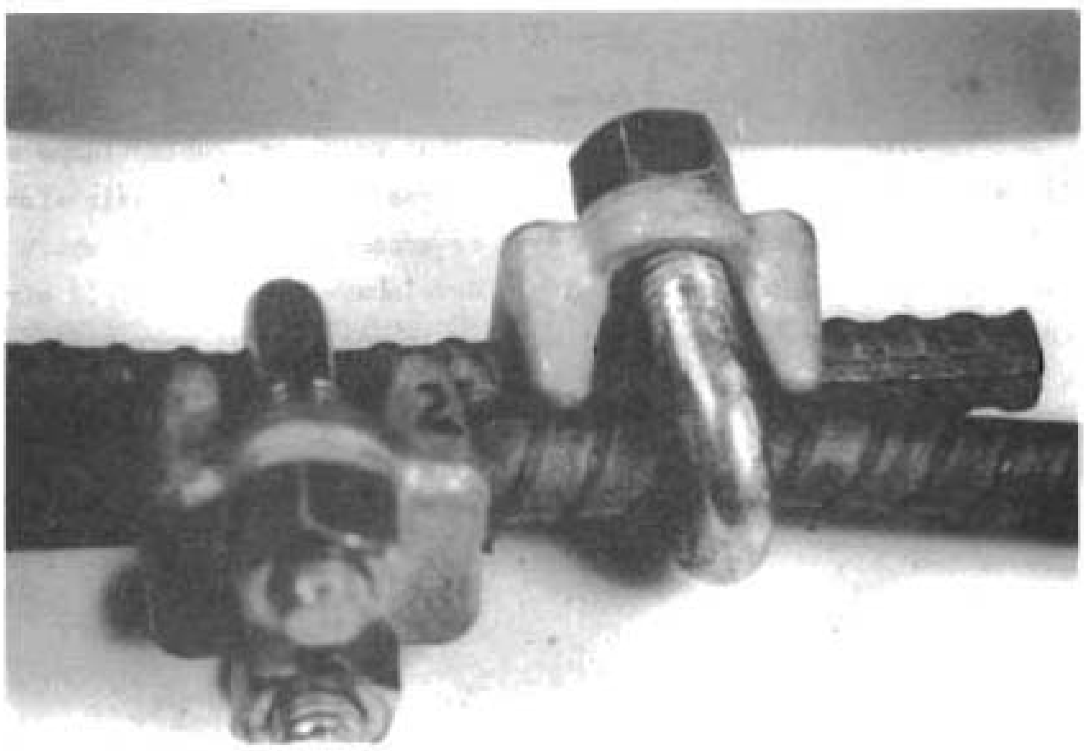


Fig A6.5. Close-up of specimen Number 2 after test.

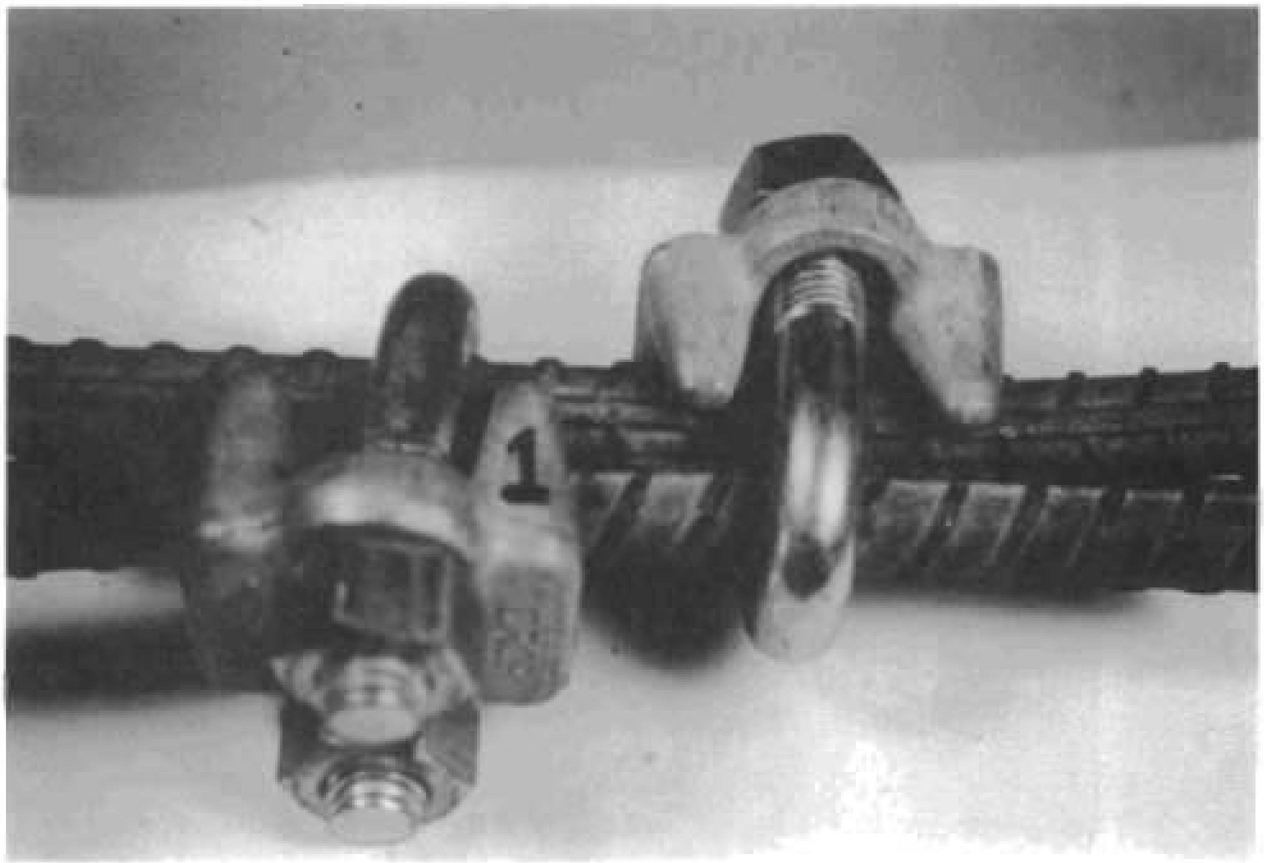


Fig A6.6. Close-up of specimen Number 1 after test.

THE AUTHORS

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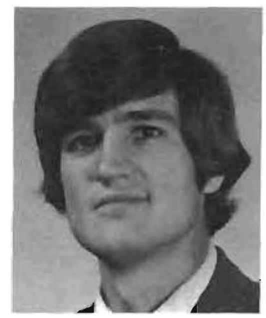

B. Frank McCullough is a Professor of Civil Engineering at The University at Austin. He has strong interests in pavements and pavement design and has developed design methods for continuously reinforced concrete pavements currently used by the State Department of Highways and Public Transportation, U. S. Steel Corporation, and many more. He has also developed overlay design methods now being used by the Federal Aviation Administration, U. S. Air Force, and the Federal Highway Administration. During the nine years with the State Department of Highways and Public Transportation, he was active in a variety of research and design developments. He worked for two years with Materials Research and Development, Inc., in Oakland California, and for the past nine years, for The University of Texas at Austin as a Professor of Civil Engineering. He participates in many national committees and is the author of over 100 publications that have been published nationally.

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