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# Void Development Under Bridge Approaches

**Study SD90-03**

**Final Report - Executive Summary**

Prepared by
Department of Civil Engineering
South Dakota State University
Brookings, South Dakota 57007-0495

November, 1992

## DISCLAIMER

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16. Abstract

The development of voids under approach slabs to bridges can lead to distress of the approach system resulting in decreased life expectancy, increased maintenance costs for repair and stabilization, and rider discomfort due to "bumps" resulting from grade differences in the approach system. The mechanisms involved in the development of the void generally are attributed to settlement problems of the foundation and embankment soils. A special backfill is often used under approach slabs to reduce such settlement problems. In recent years the South Dakota Department of Transportation (SDDOT) has observed increasing subsidence problems associated with the special backfill. The objectives of the research were to estimate the number of bridges affected by bridge end backfill settlement, to determine the cause and significance of bridge end settlement, to evaluate the cost-effectiveness of the current bridge end backfill design, and to develop recommendations for the maintenance of existing bridge end backfill systems and for design improvements for future systems.

The work included an extensive survey of some 140 bridges across South Dakota to determine and document the extent of the problem; a review of previous work related to bridge approach settlement; a survey of state transportation departments concerning approach system and bridge abutment problems and design methodology; and development of a field scale model of an integral abutment bridge/approach system to isolate and determine the mechanisms controlling backfill subsidence.

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# CHAPTER 1
# INTRODUCTION

## 1.1 Problem Statement

In South Dakota bridge approach design has evolved to the use of select granular backfill behind the abutment wall as a strong and free-draining support for reinforced concrete approach slabs at bridge ends. Since 1988 the select backfill has been encapsulated in geotextiles in an effort to prevent erosion of the backfill. Despite modifications to the design of the approach systems, problems continue to occur, appearing as voids under approach slabs, continued differential movements, and cracking and erosion of the approach embankment. The resulting maintenance necessary to correct the problems is expensive. In addition, the use of a select backfill and reinforced approach slabs adds considerably to the original cost of each bridge.

The primary problem which has developed is the appearance of voids under the reinforced concrete approach slabs in new and rehabilitated approach systems across the state. The void development has led to distress to concrete approach slabs in the form of transverse and longitudinal crack development resulting in a decrease of the life expectancy of the system and increased maintenance costs for repair and stabilization of the system. While no immediate threat to the structural integrity of the bridge structure or the approach slab exists, continued traffic loading and water infiltration may lead to sufficient deterioration to necessitate the eventual replacement of the approach system. Maintenance personnel have resorted to mudjacking, the placing of cement slurry in the void space, to stabilize the approach slab and prevent further distress from occurring.

The mechanisms involved in the development of this void are unknown. Maintenance costs to maintain the structural integrity of the approach system prompted this investigation into the mechanisms causing this subsidence in an effort to reduce or prevent such occurrences in the future.

## 1.2 Project Objectives and Tasks

To investigate problems with bridge end backfill and approaches, the South Dakota Department of Transportation developed study SD-90-03 "Bridge End Backfill Study" with the following research objectives:

1. To estimate the number of bridges affected by bridge end backfill settlement.

2. To determine the cause and significance of bridge end backfill settlement.

3. To evaluate the cost-effectiveness of the current bridge end backfill design.

4. To develop recommendations for maintenance of existing bridge end backfill designs.

5. To recommend changes to the current bridge end backfill design.

Consistent with these research objectives, the Department of Transportation identified the following research tasks as part of SD-90-03:

1. Inspect a representative number of structures constructed since 1980 in various parts of the state and in various soil types.

2. Review current design and performance of other states' bridge approach designs.

3. Assemble and review existing SDDOT construction and maintenance data relating to bridge end backfill.

4. Perform field tests and measurements to determine the cause of bridge approach subsidence.

5. Observe a representative sample of bridges being constructed with bridge end backfill.

6. Analyze the cost effectiveness of the current bridge end backfill design.

7. Develop recommendations for construction and maintenance of bridge end backfill systems.

8. Prepare a final report describing the research methodology, findings, conclusions and recommendations.

The overall objective of this study can be summarized as follows: To develop a more cost effective bridge end backfill design consistent with the goal of reducing subsidence problems at the approaches.

### 1.3 Project Work Phases

In the spring of 1990 a study of the problem was undertaken by faculty and graduate students from the Civil Engineering Department at South Dakota State University. To meet the research objectives and tasks, a series of field and laboratory investigations was conducted to determine the extent and severity of the problem, to identify potential mechanisms causing subsidence, and to aid in the development of possible solutions. These investigations are summarized below:

1. An extensive field survey of some 140 bridges across the state was conducted to determine and document the extent and severity of the approach system problem and to identify mechanisms in the field that may contribute to backfill subsidence.

2. Existing South Dakota Department of Transportation design, construction, and maintenance practices for bridge end backfill were reviewed, summarized, and constructively critiqued. A survey of selected state Departments of Transportation was made to ascertain the extent of the subsidence problem nationally and to determine the performance of other states' approach designs.

3. An extensive literature search was conducted to develop a background for the most recent developments and practices in this area.

4. An in-depth field investigation of selected bridges was conducted to attempt to isolate the mechanisms in the field that may contribute to backfill subsidence. Field measurements and laboratory testing of soil samples were made to assess the strengths and weaknesses of the current system in controlling subsidence in various soils, climatic conditions, and bridge functions. Level surveys of each site were made to distinguish vertical differences along the roadway-approach slab-bridge deck profile.

5. A model study of an integral abutment bridge having an approach system constructed using current South Dakota design practices was conducted to determine the role of thermal movements of the bridge in inducing backfill settlement. Extensive instrumentation was placed in the model to monitor movements and abutment pressures caused by the cyclic expansion and contraction of the abutment against the backfill. A model study was used

to reduce variations in soil and site conditions in attempting to isolate the mechanisms involved in backfill subsidence.

6. All the information collected in the investigations described above was thoroughly analyzed with the goal of identifying and/or isolating those causes which could be addressed economically by changes in current design or maintenance strategies and those which could not.

The following chapters contain a description of the research methodology, the findings, and conclusions and recommendations of this investigation. The chapters are organized according to major topics and thus the tasks identified above may be discussed in one or more chapters. Chapter 2 provides a review of the literature pertaining to approach systems and problems and solutions which have been used. Chapter 3 provides a detailed description of the field investigations conducted including the preliminary study of some 140 bridge sites and the detailed investigations conducted on 16 bridges. Chapter 4 summarizes past and present South Dakota Department of Transportation design practices and discusses the results of a survey conducted of selected state Departments of Transportation. Chapter 5 describes the development, testing, and results of the model study conducted. Chapter 6 presents the conclusions and recommendations of the study. Additional information about the field data collected, the survey information, and the data collected during the model study is presented in several appendices.

# CHAPTER 2
# REVIEW OF LITERATURE

## 2.1 Introduction

Numerous factors contribute to the differential movements between bridge abutments and the approach areas to the bridge. Although the problems are easily identified - they are readily seen and felt in the physical condition of the approach pavement - the basic cause or source of the problem usually lies elsewhere, often buried beneath the ground surface out of sight. Typical bridge approach problems include settlement at the pavement end of the approach slab, uplift of the approach slab due to swelling soils, settlement of the approach slab near the abutment, abutment settlement, and rotation or lateral movement of the abutment. In addition, voids often develop under approach slabs near the abutment, in spite of adequate slab support at the abutment. The problems are manifested in a differential movement between the bridge end and the approach area. The interaction of problems and settlements is often complex depending upon a number of factors. Below is presented a summary of the literature of the causes of bridge approach movements, the problems associated therewith, and solutions that have been used to mitigate the problems.

The cause of the differential settlements has been attributed, either singly or in combination, to: 1) pavement expansion, 2) vertical, lateral, or longitudinal movements of the embankment and/or foundation soils, 3) water action, 4) frost action, and 5) abutment settlement and rotations (Hopkins and Deen 1970, Timmerman 1976). The relative importance of each of these causes depends on abutment design, approach slab design, construction sequence and techniques, soil types, and embankment geometry characteristics. The second cause, or that of movements within the embankment and foundation soils, is generally regarded as the most common cause of settlement. Hopkins (1985) identified the following types of behavior of embankment/foundation soils as causing problems: primary consolidation of the embankment foundation soils; secondary compression and shear strain of the foundation and embankment soils; improper compaction of the approach embankment soils; loss of material from and around the abutment and approach pavements and soils; lateral and vertical deformations, or creep, of the bridge approach embankment; and lateral squeeze of the embankment foundation.

The factors causing bridge approach settlement are most easily discussed if categorized into factors relating to abutment type, approach backfill provisions, embankment/foundation soil conditions, approach pavement type, and construction techniques. These are discussed in more detail below. In addition, rating criteria and remediation techniques are discussed and previous research is summarized.

### 2.2 Abutment Type

The different types of bridge abutments are separated into two classifications: non-integral abutments and integral abutments. The difference between the two types is the joint at the bridge end. Integral abutments are rigidly connected to the bridge beams and deck with no expansion joint provision made for thermal movements of the bridge. The non-integral, or conventional, bridge abutment is separated from the bridge beams and deck by a mechanical joint that allows the bridge to undergo thermal movements without affecting the abutment. The design philosophy for these two abutment types is different and their effects on settlement warrants separation of their discussion.

#### 2.2.1 Non-Integral Abutments

The effect of type of abutment has been discussed in several investigations (Jones 1959, Hopkins and Deen 1970, NCHRP 1990, Timmerman 1976). The types of non-integral abutments used for bridges generally are closed abutments, stub or shelf abutments, and spill-through abutments. Compaction of backfill is difficult with closed and spill-through abutments and, as a consequence, their use has declined. Hopkins and Deen (1970) showed that considerably more approach settlement problems occurred with open-end (spill-through) abutments than with stub abutments.

The means for foundation support for the abutment is either through pile foundations or spread footings. In general spread footings are subject to settlement due to consolidation of underlying foundation and embankment soils and most agencies use pile foundations. Pile supported abutments generally provide better bridge support than do spread footings. Indeed, in studies of bridge approach settlement it is common to assume that settlement of bridge abutments on pile foundations is negligible (Hopkins 1985 and Tadros and Benak 1989). However, the relatively unyielding behavior of pile supported abutments is not necessarily a blessing. Hopkins and Deen (1970) and Timmerman (1976) report that the observed differential settlement between abutment and approach is generally higher for pile supported abutments than for those supported on spread footings. Shields *et al.* (1980) advocate founding bridge abutments on approach fills to reduce differential settlements between the abutment and approach.

### 2.2.2 Integral Abutment Bridges

While non-integral abutment bridges are still commonly used, the use of integral abutment bridges has increased dramatically. The 1969 NCHRP report "Bridge Approach Design and Construction" (NCHRP 1969) made no reference to integral abutment bridges. The 1990 updated report (NCHRP 1990) now includes integral abutment bridges in its discussion on design and construction. Integral abutment bridges provide a rigid connection to the bridge beams and deck which allows elimination of the joint at the bridge abutment/approach interface. This joint has been a continuous maintenance problem for bridge maintenance personnel, thus its removal reduces maintenance and construction costs associated with bridge joints. Integral abutment bridges have been in service for approximately thirty years, and as their use has increased so has their design length. Bridges are being built with lengths exceeding 400 feet, with some over 600 feet in length presently in service (NCHRP 1990).

The basic design of integral abutment bridges consists of a reinforced concrete stub abutment supported on a single row of steel "H"piles. The bridge beams are attached directly to the piles with reinforcing steel added to make the connection to the abutment rigid when the abutment is poured. The bridge deck is then poured forming a jointless connection to the abutment (SDDOT 1990). The piles are oriented with their weak axis parallel to the bridge beams which allows the piles to flex causing a plastic hinge effect in the first two feet below the abutment. The effects of thermal movements on pile performance has been studied by Jorgensen (1981), Girton *et al.* (1989), and Griemann *et al.* (1984 and 1987). The studies provide no reduction in pile capacity with lateral movements up to four inches in steel "H" piles and two inches in timber and concrete piles (Greimann *et al.* 1984).

Bridge thermal movements have been studied to develop better geometric design criteria. AASHTO provides cold climate temperature ranges of -30 to 120 degrees Fahrenheit for steel girder bridges and plus or minus 40 degrees Fahrenheit from the installation temperature for concrete structures. The coefficients of thermal expansion provided by AASHTO are 0.0000065 and 0.0000060 per degree Fahrenheit for steel and concrete structures respectively (Greimann *et al.* 1984). Length, skew, and curvature were the focus of a study conducted by Roeder and Moorty (1990). Their results showed good correlation to AASHTO thermal guidelines for steel structures, but the AASHTO thermal guidelines for concrete structures did not correlate well. Their study provided lower thermal coefficients and larger temperature ranges. The AASHTO guidelines were found to be conservative for the design of integral abutment bridges. Skewed and curved bridges were determined to require refined thermal calculations due to their geometry, thus requiring designs based on an individual evaluation of the site.

The effects of bridge thermal movements on bridge beams and the abutment were the focus of a 1973 study conducted at South Dakota State University for the South Dakota Department of Transportation reported by Lee and Sarsam (1973). A model bridge, with an integral abutment supported on piles, was constructed and instrumented to monitor bridge thermal induced stresses during and after construction. The model utilized a system of hydraulic jacks to simulate temperature expansion and contraction of the bridge. The jacks were mounted on a rigid abutment supported on piles opposite the integral abutment. Cycles were run to simulate construction stages and the different seasons of the year. The study showed that the current design by the SDDOT was adequate for the thermal stresses the bridge would undergo.

## 2.3 Select Backfill Provisions

To reduce settlement behind the abutment wall several states have adopted the use of select backfill, a special gradation of granular backfill material expected to have low volume change characteristics. Generally, select granular materials with less than five percent passing the #200 sieve are recommended. These soils can be compacted easily with small vibratory compactors to achieve desired field densities (NCHRP 1990). The success in using select backfill is not yet conclusive. Hopkins and Deen (1970) reported that Kentucky used special granular backfill extending 20 to 60 feet behind the abutments with mixed success. They noted the frequency of unsatisfactory performance of bridge approaches using the special granular fill was actually higher than similar bridge approaches which did not utilize the special fill. The use of special backfill was subsequently discontinued in Kentucky. It should be noted that Kentucky has experienced considerable problems with consolidation of foundation materials and the use of select backfill would probably not ameliorate consolidation induced settlements. In a recent survey of transportation departments, Tadros and Benak (1989) report that approximately 15 states utilize a select backfill gradation behind the abutment. The reported success is mixed, with most states noting some improvement, but unsure if it is due to the backfill or the use of reinforced concrete approach slabs. The survey responses and the National Cooperative Highway Research Program study "Design and Construction of Bridge Approaches" (NCHRP 1990) note that restricted work areas cause difficulty in achieving proper compaction next to bridge abutments. Practices that help to minimize either settlement or swell include the use of select materials, placement of relatively thin (6 to 8-inch) layers, strict control of moisture and density, and provisions for positive drainage (NCHRP 1990). Thermal movements of bridge abutments also result in settlement of backfill material behind integral abutment bridges. Although the select backfill possesses good elastic properties, thermal movements of the abutment cause material deformations that result in settlement of the backfill (NCHRP 1990).

Studies conducted by Broms and Ingelson (1971 and 1972) involved the investigation of earth pressures on a rigid frame bridge during compaction of the backfill and with subsequent thermal movements of the structure after completion. High earth pressures were recorded during the compaction of the backfill, and wide variations of earth pressure were recorded over the next two years. The highest earth pressures were recorded during the summer of 1970 during the second year of service. The earth pressures increased with each year of service indicating the material was settling and "self" compacting, resulting in larger earth pressures. This settling was attributed to the material falling into the void space during winter contraction cycles and compacting when the bridge expanded in the spring and summer. With subsequent years of service it was speculated that Rankine passive earth could be reached (Broms and Ingelson 1971). The 1972 study by Broms and Ingelson involved a 500 foot long structure with 14 foot high abutment. During their investigation Rankine passive earth pressures were recorded in the field during summer expansion cycles. A movement of H/600 was determined to be enough to develop full Rankine passive earth pressure. On both studies a uniform gravelly sand was used for backfill material.

## 2.4 Foundation/Embankment Soil Conditions

Numerous studies have shown that foundation and embankment soils play a major role in the amount of approach settlement. Bridges are often constructed in areas where poor soil conditions exist and consequently the approach embankments have to be constructed on soft, compressible cohesive soils. These foundation soils will compress and decrease in volume when loaded by an embankment. In the NCHRP 1990 report it is pointed out that post-construction consolidation of foundation soils is the main contributor to rough-riding bridge approaches. This conclusion has been born out by the work of many investigators (Hopkins and Scott 1970, Timmerman 1976, Hopkins 1985, Stewart 1985, Tadros and Benak 1989, and Kramer and Sajer 1991). Creep induced settlements caused by vertical and lateral movements in the foundation material can also result in rough-riding bridge approaches (Kramer and Sajer 1991). The NCHRP study (1990), Hopkins and Scott (1970), and Tadros and Benak (1989) point out the necessity for adequate site investigations and settlement studies to characterize the susceptibility of the foundation and embankment soils to settle. Settlement prediction techniques available provide the needed basis for design to minimize approach settlements due to poor foundation soil conditions. Methods by which foundation soil settlement can be controlled are the use of drains, surcharges and staged construction to allow consolidation to occur, the removal of unsatisfactory materials, dynamic compaction techniques, and the use of light weight embankment materials.

The conventional method of embankment construction is a rolled earth process. This consists of the use of standard roadway excavation or convenient borrow material. Since settlement of the embankment material is of major concern, some states specify select materials and/or increased density requirements. California specifies a fill with a maximum plasticity index (PI) of 15 and less than 40 percent fines within 150 feet of the abutment. The density requirement is increased from 90 percent to 95 percent of Standard Proctor (AASHTO T-99) (NCHRP 1990). South Dakota does not specify a select material, but increases the density requirement from 95 percent to 97 percent of Standard Proctor (AASHTO T-99) within 100 feet of the abutment (SDDOT 1990).

When the compaction of the embankment is controlled, it is assumed that subsequent settlements within the embankment are negligible (NCHRP 1990). Placing embankment soils in lifts of six to twelve inches (eight inches as a common thickness) is generally considered acceptable to obtain desired densities. Continuous visual inspection is required to ensure proper field compaction (NCHRP 1971). When a uniform compactive effort is provided on the embankment, no detrimental settlements are likely to occur as a result of climatic or traffic conditions (HRB 1952). A study by Nielson (1966) showed that granular embankment materials were susceptible to settlement under traffic loadings. The traffic loadings induced soil vibrations which resulted in further settlement of granular embankment materials. To reduce these settlements, Nielson recommended a granular gradation having 20 to 25 percent of the material passing the #200 sieve, with the PI of this material restricted to a narrow range between 3 and 6.

Local compression at the bridge/pavement interface may result in a rough transition. These localized effects may be the result of inadequate compaction, inadequate drainage, pavement rutting, or thermal bridge movements (Kramer and Sajer 1991). Inadequate compaction near the bridge abutment is common. Structural impediments limit the use of small vibratory compaction equipment near the abutment face and large equipment used near the abutment results in non-uniform compaction of the area. Inadequate drainage can create serious erosion and piping problems that can undermine the approach slab and abutment, causing large movements. Clogged joints on bridges and approach pavements can inhibit movements of the structure causing large thermal pressures to build up. These large pressures displace the pavement and the soil adjacent to the abutment, creating a gap when the bridge retracts. Subsequent traffic loading pushes the pavement and underlying soil into the gap, creating a localized settlement problem.

In an Ohio survey of bridge approach problems, Timmerman (1976) noted a general increase in differential settlements of the approach areas as embankment height increased. This tendency was also noted by Tadros and Benak (1989), who found that embankments with heights over 20 feet increased the need for special investigations to study the settlement potential. In a comprehensive study of case histories in Kentucky, Hopkins (1985) found that the embankment factor of safety correlated well with the tendency for approach settlements. He found that when embankments are designed with factors of safety against slope failure of 1.5 or greater, the settlements are on the order of one-half to one-third the settlements of approaches in which the embankments have factors of safety less than 1.5. This points out the need to consider the entire system in effecting reductions in approach settlements.

## 2.5 Approach Pavement Type

Differences in the behavior of flexible pavements and that of reinforced concrete pavements have been noted by several studies. In 1973 the State of Wisconsin (Tadros and Benak 1989) surveyed 200 approaches and found significant differences between the performance of flexible and reinforced concrete pavement approaches. Seventy six percent of flexible pavement approaches were rated as poor while 93 percent of the reinforced concrete approaches were rated as good. For non-reinforced approach slabs, 56% rated good, 12% fair, and 32% poor. A 1973 California survey (Stewart 1985) found that 74 percent of the asphalt approaches had been or needed to be repaired compared with 43 percent of reinforced concrete approaches. There is a general consensus that reinforced concrete approach slabs offer superior performance compared to asphalt concrete approach slabs (NCHRP 1990).

Reinforced concrete slabs help bridge problem areas, but each site should be evaluated separately to obtain the most feasible design (Kramer and Sajer 1991). To reduce approach settlements the use of reinforced concrete approach slabs has become common. In the survey of transportation departments conducted by Tadros and Benak (1989), it was found that more than 30 states presently use reinforced concrete approach slabs on some or all bridge approaches. The length of the slab varies from 10 to 100 feet, with slabs 20 to 30 feet in length being the most common. In general the states report that the slabs have been successful in reducing differential movements, although some report that it simply moves the bump to the slab-pavement interface.

Pavement expansion is also often a source of distress in the bridge approach area. Pavement contraction during periods of low temperature allows loose material to enter pavement joints from either above or below the pavement. During periods of high temperature when the pavement expands, the joints cannot close due to the presence of the loose material and pavement shoving occurs which can cause blowups or impact the bridge abutment. The NCHRP study "Design Construction and Maintenance of PCC Pavement Joints" (NCHRP 1973) describes various design practices used to eliminate or reduce the severity of shoving against the bridge.

### 2.6 Construction Sequence and Control

In areas where foundation conditions are such that soft soils will undergo consolidation settlements, construction sequence can have an impact on future settlements. The use of staged construction and/or cambered embankments can counterbalance some of the consolidation settlement effects. Construction deficiencies are often blamed for poor performance of approach slabs. Numerous studies (Peck and Ireland 1957, Jones 1959, Timmerman 1976, and Hopkins 1985) have shown that poor compaction behind abutment walls may be responsible for much of the "bump" in the approach. This has been discussed above in reference to select backfill behind the abutment walls. Benching of the embankment near the abutment when select backfill is used has been identified as a construction practice that can reduce differential settlements (NCHRP 1969).

The compaction control executed on the approach embankments can also have significant effects on the later performance of the embankment soils. The key to reducing settlements in the embankment is in obtaining proper compaction of the embankment materials in the field. Compaction control in the field is highly variable. Studies on compaction have been conducted by many investigators (D'Appolonia *et al.* 1969; Jorgensen 1969; Metcalf 1969; Morris *et al.* 1969; Noorany 1990; Selig *et al.* 1967; Wahls *et al.* 1966; Williamson 1969). The study conducted by Noorany (1990) showed widespread variability in measured field densities. In the study nine different geotechnical firms tested three different soil sites. Each site was to meet 90 percent relative compaction and the same compactive effort was used at each site. The testing provided good comparisons for silty clays and silty sands where the average relative density was 90% and 91% respectively. The third soil site consisted of granular soils. Sixteen tests at the site provided an average relative density of 88% with a 6.4% standard deviation. It was also noted that one geotechnical firm used the wrong size sand cone for testing the granular material. The study showed the need for improving placement techniques and the importance of following standard practice in field density testing. The specification and control of the compaction requires an understanding of the variability of the process. With a knowledge of this process it is possible to reach a standard that is acceptable to the contractor and the engineer (Metcalf 1969).

Although compacted embankment soils are generally considered to be less susceptible to settlement than natural foundation soils, it has been observed that settlement of these soils can contribute significantly to the settlement of the approach areas (Hopkins and Scott 1970). Volume changes in embankment soils generally result from changes in moisture content which either lead to shrinkage of the material (loss of moisture) or swell of the material (increase in moisture). Interactions between the compaction variables of water content, dry density, soil type, compaction method, and post-compaction stress and moisture regimes are complex and not yet completely understood. Recent work by Lawton *et al.* (1989) has shown that moderately plastic soils can sustain collapse of their soil structures under relatively unchanging vertical stresses. They found that the drier a soil was at compaction, the greater was both the maximum collapse and swell potential. The implications of this are that compacted embankment soils do have the potential for deformation and may be a mechanism for settlements of the approach slabs.

The influence of water on performance of approaches is often detrimental. The use of proper drainage of the backfill to remove water from under the approach slab will reduce problems due to seasonal frost action and dynamic loads in subbase materials. Proper surface drainage is necessary to convey water away from the approach area to reduce erosion of the approach embankment. The flow of water off the bridge should be channeled such that water does not simply run into the backfill.

### 2.7 Performance Rating Criteria

Criteria for the objective performance and potential need for maintenance of bridge approaches are not well defined among bridge and maintenance officials. The determination of whether an approach is performing satisfactory or unsatisfactory is very often simply subjective. In general the performance of the bridge approach system may be expressed in a quantitative way by measuring the settlement of the approach system, the settlement of the abutment, and determining the difference between the two. An Ohio study (Timmerman 1976) used differential settlement of 0.10 feet as a criterion for unsatisfactory performance requiring maintenance. A California study (Stewart 1985) developed a roughness rating using a special test vehicle equipped with a strip-chart recorder. The strip-chart instrument recorded vertical movements of the vehicle body relative to the rear axle housing, at a speed of 50 miles per hour. For their study a roughness rating of 17.0 (1.5 inches) was considered significantly rough to warrant maintenance repair. In Kentucky studies, Hopkins and Deen (1970) grouped behavior as follows: Group 1 - no maintenance necessary and no approach fault noticed; Group 2 - no maintenance performed, however, an approach fault was observed; and

Group 3 - maintenance had been performed on the approach. In a recent study by Tadros and Benak (1989) a subjective rating was assigned: Good - no visual settlement; Moderate - settlement causing a noticeable transition bump; and Severe - settlement causing a severe transition bump.

### 2.8 Remediation Methods

A discussion of remediation methods can be divided into two topics: 1) changes in design and construction techniques, and 2) repair of unsatisfactory approaches. Certainly one of the more obvious methods of reducing approach settlements is to preload foundation soils to induce consolidation settlement prior to construction of the bridge and approach. A number of design and construction techniques are available to speed up consolidation settlement in foundation soils such as pre-loading, stage construction, wick drains, replacement of bad soils, lightweight fills, and dynamic compaction. These are considered outside the scope of the present study and more information is available in NCHRP (1971) "Construction of Embankments". Many of the techniques discussed above have evolved as design changes to foster reductions in differential settlements. These include the use of select backfill, approach slabs, erosion control, benched slopes, and increased foundation investigation.

Once the differential settlement has occurred and progressed to the point that repairs are required, a number of repair methods can be used. One common practice is simply to correct the settlement or swelling using bituminous materials to overlay the approach slab and smooth the transition. Epoxy resin overlays have also been used to smooth the transition. For correcting settlements in reinforced concrete slabs slab-jacking or mud-jacking has been utilized to raise the slab back to grade or to fill voids beneath the slab for stabilization. Some agencies have anticipated the need for mud-jacking and have provided precast access holes through the approach slab for later use when required. Generally the sooner problems are identified and corrected, the lower will be the total cost compared to allowing more severe problems to develop to the point of slab removal and replacement. As several investigations have stated, the complete elimination of the bump at the end of the bridge is impossible and thus a comprehensive maintenance strategy to address approach problems can go a long way to mitigating overall costs.

### 2.9 Summary of Previous Research

The review of literature shows that settlements of bridge approaches have received considerable attention from transportation officials. While many causes have been identified, the interaction of the cause and effect are complex. In summary, the following conclusions can be stated on the basis of previous work:

1. Approach settlements are often associated with weak foundation soils and high embankments.
2. The use of approach slabs is common.
3. Reinforced concrete approaches generally perform better than asphalt concrete ones.
4. The use of select backfill materials is common, but their success is mixed.
5. Proper compaction of backfill and embankment materials is considered key to the performance of the approach.
6. Little quantitative criteria exist on the performance and maintenance rating of bridge approach systems.
7. Integral abutment bridge performance has been studied, but the effects of thermal-induced movements on approach embankments has received little attention.

The work reviewed provides a background for the bridge end backfill study. Although several causes of settlement have been identified above, the problem of void development under an approach slab as is occurring in South Dakota was not readily delineated in the literature review.

# CHAPTER 3

# FIELD STUDIES

## 3.1 Preliminary Field Investigations

### 3.1.1 Purpose of Field Investigations

The purpose of the field investigations was to gather information on bridge approach problems across the state of South Dakota. Task One of the study was to inspect a representative number of structures constructed since 1980 in various parts of the state and in various soil types. By conducting these inspections, the extent of void development beneath approach slabs and other related problems could be determined. Through inspection of different bridge sites across the state it was hoped that the mechanisms causing the problem could be isolated.

### 3.1.2 Site Selection

Initial field investigations to gather information on the extent of the problem and to gather data for development of a data base were performed over the summer and fall of 1990. Dan Strand, SDDOT Office of Research, contacted the regional bridge maintenance engineers to ask for their assistance in identifying problem bridges, bridges performing well, new construction activities, and construction and maintenance procedures. A copy of the memorandum sent to the four regions is presented in Appendix A. Contact was made with the regional engineers during the summer of 1990 to solicit their input on bridge selection and obtain background material. The regional bridge maintenance personnel helped select bridges for initial survey and generally accompanied us on initial investigations. Approximately 130 to 140 bridges were initially selected bridges for site visits. Table 3.1 shows the distribution of the selected bridges by region.

| Table 3-1. Distribution of Inspected Bridges by Region. | | | | |
|---|---|---|---|---|
| | Region | | | |
| | Aberdeen | Mitchell | Pierre | Rapid City |
| No. Bridges Inspected | 24 | 38 | 23 | 21 |

### 3.1.3 Evaluation Procedures

Over 130 bridges were visited during the initial phase and site investigations were performed on 111 of the selected bridges. A four page inspection form was filled out for each site; the form is shown in Appendix A. The information obtained and recorded included location, date(s) of visit, site characteristics and geology, bridge type, abutment type, foundation and embankment soils data, approach slab details, pavement and subgrade data, maintenance history and traffic density criteria. Information for completing the inspection form was obtained by site visits and by reviewing bridge files and plans at each of the regional offices. Not all of the information was available for each site. As the project involved bridges across the state, site characteristics could play an important role.

The sites were inspected for void development under the approach slab to identify bridges with or without backfill subsidence problems. This was accomplished by excavating the embankment adjacent to the approach slab and probing with a rod or hand to estimate void size. The bridge sites were studied to determine if problems were occurring in the approach and abutment embankments. This involved an inspection for cracking and settlement evidence.

Two evaluation procedures were used. One procedure was essentially a qualitative one in which problems, such as erosion and drainage conditions, were noted on the inspection forms and photographs taken. Such characteristics varied from site to site and must be evaluated in a non-quantitative manner based simply on observation. The second procedure consisted of developing a data base of the occurrence of specific characteristics. Some of the characteristics, such as void size, could be further quantitatively evaluated. For analysis purposes, the data collected was entered into a computer spreadsheet (Quattro Pro). Once in the data base, the data could be sorted by structure type, approach type, year of construction or other variable. 104 bridge sites were included in the data base. The location of the 104 sites are shown in Figure 3-1. The numbers shown in Figure 3-1 are referenced in the data base summary in Appendix A. In addition, four sites labeled A, B, C, and D, are shown in Figure 3-1. These sites were not included in the data base because they are scheduled for approach slab replacement. The sites labelled N in Figure 3-1 are locations where new approaches were constructed during the time period of this study.

### 3.1.4 Observations

#### 3.1.4.1 General

The initial inspections provided valuable information about the different types of bridge structures in the state. The state uses two basic styles of bridges with respect to abutments. The primary type is an integral abutment bridge with a stub abutment supported on a single row of piles. The other type of bridge is a non-integral abutment bridge with a stub abutment supported on piles. These two types account for most of the structures built in the state. The primary approach system is a reinforced concrete approach slab. The approach slab is tied to the abutment with dowel bars, making it an integral connection. There has been some use of asphalt approaches, but few were built during the 1980's to present time period.

Of the 104 bridges listed in the data base, 90 of the structures have been constructed with integral abutments, leaving 14 bridges built with non-integral abutments. The study was primarily concerned with 1980 and post-1980 bridges built using the 1980 standard specification for bridge end backfill. Of the 90 integral abutment bridges in the data base, 84 were built in or after 1980. There were 12 non-integral abutment bridges constructed in this same time period. These 96 bridges were analyzed to determine trends in void development and other problems.

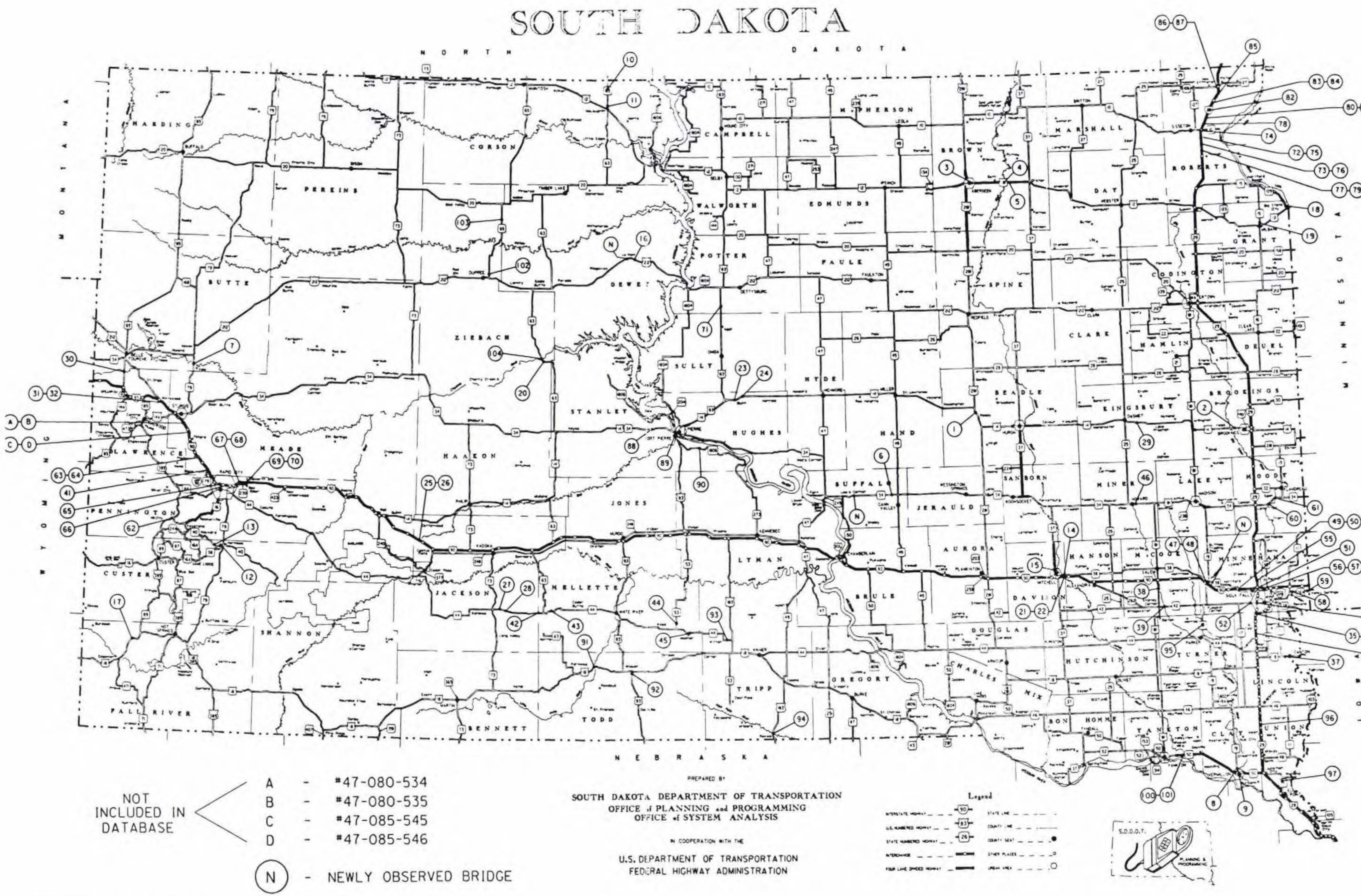



Figure 3-1. Location of bridges inspected/visited during preliminary field investigations.

The integral abutment bridges built without reinforced concrete approach slabs consisted of three with asphalt and two with gravel approaches. None of the five approaches were performing well. Two of the asphalt approaches had been mudjacked to stabilize the approach and curb system. All three of the approaches were rough and had required considerable maintenance. Structure #64-060-241 on SD 11 north of Elk Point is a bridge with asphalt approaches that had not been mudjacked, but the curbs are cracked and broken. The approach has also had asphalt wedges and overlays to smooth the approach. There were no bridges with asphalt approaches selected by the SDDOT that were performing well. The two structures with gravel approaches were in the Pierre region. One of the structures was severely eroded on the approach embankment and onto the road itself at the time of inspection. The other bridge showed signs of erosion, but had freshly-graded approaches before the inspection.

A summary of the observations at 104 bridges is presented in Table 3-2, which shows the number of bridge-approach systems which have developed voids under the abutment and the approach slab. Also shown in the table are the number of bridges that have been mudjacked, have developed cracks on the approach slab, berm or embankment, and/or have serious erosion problems.

**3.1.4.2 Void Development**

During the preliminary field investigations the size and extent of void development under the approach slab and the abutment were determined. A steel rod or pipe was generally used to estimate the extent of the void and tape measurements were made of the void height where accessible. As discussed later, a special probe was subsequently developed to better measure the void extent and height.

Of the 104 approach-bridge systems in the data base, 96 have been constructed since 1980, with 84 having integral abutments and 12 having non-integral abutments. The 84 bridges with integral abutments include five which have asphalt or gravel approaches, leaving 79 bridges having integral abutments, bridge end backfill, and

Table 3-2. Summary of observations at 104 structures showing occurrence of problems at integral and non-integral bridges.

| Integral | No. of Bridges | Voids under Abutment | Voids under Approach | Mudjacked | Cracking Appr. Slab | Cracking Berm | Cracking Embankment | Erosion |
|---|---|---|---|---|---|---|---|---|
| Total | 90 | 31 | 80 | 22 | 43 | 27 | 8 | 38 |
| Built Since 1980 | 84 | 28 | 76 | 20 | 40 | 25 | 7 | 37 |
| Reinforced Concrete | 85 | 31 | 78 | 20 | 42 | 27 | 8 | 33 |
| Built Since 1980 | 79 | 28 | 74 | 18 | 39 | 25 | 7 | 32 |
| Asphalt | 3 | 1 | 2 | 2 | 1 | 0 | 0 | 3 |
| Built Since 1980 | 3 | 1 | 2 | 2 | 1 | 0 | 0 | 3 |
| Gravel | 2 | 0 | 0 | 0 | 0 | 0 | 0 | 2 |
| Built Since 1980 | 2 | 0 | 0 | 0 | 0 | 0 | 0 | 2 |
| Non-Integral | No. of Bridges | Voids Under Abutment | Voids Under Approach | Mudjacked | Cracking Appr. Slab | Cracking Berm | Cracking Embankment | Erosion |
| Total | 14 | 2 | 6 | 5 | 6 | 0 | 0 | 1 |
| Built Since 1980 | 12 | 1 | 4 | 4 | 5 | 0 | 0 | 1 |
| Reinforced Concrete | 13 | 1 | 5 | 5 | 6 | 0 | 0 | 1 |
| Built Since 1980 | 12 | 1 | 4 | 4 | 5 | 0 | 0 | 1 |
| Asphalt | 1 | 1 | 1 | 0 | 0 | 0 | 0 | 0 |
| Built Since 1980 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |

reinforced concrete approach slabs. Of these 79 integral abutment bridges, voids under the approach slab have developed in 57 cases, the void ranging from loose to 10 inches thick at the abutment face. A 14-inch void was uncovered under structure #14-107-194, a bridge constructed in 1977 on SD19, 1.2 miles north of SD50. The void of 10 inches was found under structure #36-361-298, a bridge constructed in 1980 on SD44, 5.2 miles east of junction with SD73. An additional 18 of the 79 integral abutment bridges had been previously mudjacked and are assumed to have had voids. The size of the void in these situations could not be ascertained, but a void must have developed for mudjacking to be deemed necessary by maintenance personnel. Of the 79 integral abutment bridges, only four were observed in which a void was not conclusively found. Of these four, three had some structural impediment preventing the viewing of the void. Thus, only one of the 79 bridges could conclusively be determined not to have a void under the approach slab.

Photographs of typical voids observed during the field inspections are shown in Figure 3-2. The voids shown in Figure 3-2 a,b, and c occurred on approaches constructed in the mid to late 1980's with a slope on the rock backfill of 6 Horizontal to 1 Vertical (6H:1V). In each case transverse cracks developed on the approach slab just out from the end of the bridge. The approach shown in Figure 3-2d was constructed in 1980 and has a 2H:1V backslope. This approach had a chip seal coat on it which masked any cracks which may have developed. The largest void encountered during the field visits is shown in Figure 3-2e. The void was under the north approach of this structure. This approach was constructed in 1977 and has a 2H:1V backslope. The void was approximately 12 to 14 inches high for the entire abutment width and one could easily see through to the other side. The void extended back approximately eight feet. It is interesting to note that the south approach of this structure had a two to three inch void under it. This approach showed signs of longitudinal cracking, but asphalt overlays obscured much of the cracking.

Observations of the 12 non-integral abutment bridges found four with voids and eight without voids. The four with voids had been previously mudjacked, with voids

a. Structure No. 64-006-090, crosses I-29 at mile marker 38.32 in Union Co. Seven-inch void developed under north corner of east approach.

b. Structure No. 44-222-210, on SD 42 1.9 miles west of Minnehaha County line. Northeast abutment/approach interface. Approximately 4-1/2 inch void exposed, no digging required.

Figure 3-2. Photographs of void development under approach slabs.

c. Structure No. 36-071-076, on I-90 westbound at mile marker 125.53 in Jackson Co. West approach with an 8-to-10-inch void under approach exposed, no digging required.

d. Structure No. 47-111-580, on I-90 at mile marker 47.46. Six-inch void developed under approach slab.

Figure 3-2. Continued.

e. Structure No. 14-107-194, on SD 19 1.2 miles north of SD 50 in Clay Co. Void under north approach. Spade blade is 14 inches long. No digging required to expose void.

Figure 3-2. Continued

Figure 3-3. Photograph of void development under abutment, note exposure of H-pile. Structure No. 36-071-077, on I-90 eastbound at mile marker 125.53 in Jackson Co.

subsequently appearing under the grout, indicating that the mechanism causing void development continues after mudjacking. Although the sample size is smaller, the non-integral abutment bridges show less evidence of subsidence of the backfill. Statistically, there are 33% of the non-integral abutment bridges with void development compared to 95% of the integral abutment bridges. Two of the non-integral bridges were placed on a high fill on I-29 crossing I-229. The site examination on these two bridges indicate signs of embankment settlement, therefore, the problems are most likely associated with consolidation settlement of the embankment. The only non-integral bridge with asphalt approaches is structure #07-267-329 which is a pre-1980 bridge. This 1975 structure is situated on a large fill and has settlement problems associated with the approach. There are voids under the curbs and the asphalt has settled. This bridge also has problems that are indicative of consolidation settlement.

Mudjacking the void, or filling it with a cement-based fluid which hardens with time, does not necessarily solve the problem. In several instances, on both integral and non-integral abutment bridges, voids were observed to be present under approaches that have been mudjacked. The problems that may be created concerning drainage have not been addressed, and may lead to approach cracking due to a water layer freezing between mudjack and approach slab. Problems mudjacking has caused will be discussed in Chapter 4 under maintenance costs.

As the data in Table 3-2 shows, voids have developed underneath the abutment in 31 of the 90 integral abutment systems and in two of the 14 non-integral abutment systems. The void developed under the abutment itself was most often small, being an inch or less in height. It was observed that the void typically was largest near the H-piles, on the order of three to four inches in height, although one 12-inch void was observed. Figure 3-3 is a photograph of an exposed H-pile. Such exposure was not common, but where it occurred the H-piles could generally be touched through the void. In several instances a cavity extended down around the H-pile, calling into question whether the pre-bore hole through the embankment for the H-pile installation had been properly backfilled. In most cases where a void under the abutment was apparent there had been noticeable erosion problems on the berm in front of the abutment.

The bridges built prior to 1980 showed void problems in four of the six cases. The two bridges without voids were in the Rapid City region and were built in 1973 and 1959. Both of these bridges were slated for approach slab replacement. Two of the bridge approaches were mudjacked prior to inspection and were concluded to have voids. There seems to be no conclusive evidence on the pre-1980 bridges to indicate better performance than the post-1980 bridge approaches.

#### 3.1.4.3 Approach Slab Cracking

Cracking of the approach slab was also observed at many of the sites. There were two types of cracking prevalent on the approach slabs for both types of abutments. There was transverse cracking parallel to the bridge abutment approximately two feet from the approach/bridge interface. The second type of cracking was longitudinal cracking parallel to the center line of the road. The longitudinal cracking was usually perpendicular to the bridge abutment, even on skewed bridges. Figure 3-4 shows a photograph of typical crack development. The transverse cracks are readily apparent in this photo. The longitudinal cracks are more difficult to see, but are located near the centerline and between the wheel tracks in the far lane. The data in Table 3-2 indicate that approach slab cracking has developed in about one-half of the bridges visited.

This cracking is easily explained. It occurs on approach slabs where voids have developed beneath the slab. The transverse cracks develop due to a loss of support under the approach slab adjacent to the structure. The location of the transverse cracks corresponds to the end of the dowel bars from the bridge abutment. The dowel bars provide more flexural support in the first two feet of the approach slab, preventing an even distribution of flexural stresses through out the approach slab. Consequently a stress concentration develops at the end of the dowel bars, causing a transverse crack to develop in the concrete at this location. The longitudinal cracking also develops on approaches with voids under the approach slab. This cracking is most often located

Figure 3-4. Photograph of typical crack development in approach slab. Cracks in this photograph have been filled and sealed. Structure No. 52-256-401 on US 16 at mile marker 42.64 in Pennington Co.

between the wheel paths, and occurs due to a loss of flexural support due to the presence of the void.

### 3.1.4.4 Erosion and Embankment Cracking

The embankments of integral abutment bridges at several sites showed signs of distress while the non-integral abutment bridges showed no signs of embankment distress. This distress manifests itself as erosion/sloughing and cracking on the approach and abutment embankments. The data in Table 3-2 show that 37 of the 84 integral abutment bridges built since 1980 have experienced some embankment erosion. The type of erosion noted in the inspections included erosion around the wing wall, erosion of the approach embankment sides, erosion of the embankment in front of the abutment, and erosion around the end of the approach slab usually in close proximity to a drop inlet structure. The photographs in Figure 3-5 shows some of the typical erosion features observed in the field. Such erosion occurs due to scour as water collects and runs down around the wing wall. The erosional features shown in Figure 3-5b developed as water runs down around the wing wall and back around to the berm in front of the abutment.

The data in Table 3-2 show that embankment cracking has occurred in eight of the integral abutment structures visited and the berm in front of the abutment has cracked in 27 of the integral abutment structures visited. Photographs of embankment and berm cracking are shown in Figure 3-6. These photos represent some of the most severe cracking observed, but are indicative of the pattern of cracking. Cracking of the approach embankment typically developed from the contact of the wing wall and the abutment and radiated out at about a 45 degree angle. The embankment cracking shown in Figure 3-6a is likely due to a slope instability problem rather than from abutment action. Cracking of the berm in front of the abutment was a rather common observation, with the cracks out about five to eight feet from the abutment face and running parallel to the abutment until turning either upward or downward near the outer edge of the berm (Figure 3-6b).

a. Erosion around wing wall. Structure No. 48-440-253 on SD 53 in Mellette Co.

b. Erosion of berm in front of abutment. Structure No. 14-100-200 on SD 50 in Clay Co.

Figure 3-5. Photographs of erosion observed in the field.

(a) (b)

Figure 3-6. Photographs of cracking observed in the field.

a. Cracking of approach embankment. Structure No. 50-119-165 on I-90 at mile post 390.28 in Minnehaha Co.

b. Cracking of berm in front of abutment. Structure No. 62-093-230 on SD 44 at mile marker 238.66 in Tripp Co.

### 3.1.5 Discussion

The field observations indicate that void development under the approach slab is related to abutment type. Since an integral abutment is rigidly attached to the bridge, the abutment moves with the bridge during thermal cycling. A void under the approach slab was found in the majority of integral-abutment structures. Measurements made during the site visits indicate that void height is highly variable, that the void is largest next to the abutment, and the void usually tapers to zero at about five to ten feet from the abutment. A review of the data indicated that a key factor may be bridge length. Estimates of void height made during this field phase were plotted versus bridge length as shown in Figure 3-7. It can be seen that there is a great deal of scatter in the data, but the trend is one of increasing void height with increasing length. Linear regression analyses of the data are shown in the figure and are considered indicative of the trend only, not as a means of estimating void size by bridge length. Attempts to sort the data in Figure 3-7 by soil type, steel or concrete beams, embankment height, and abutment type (date of abutment detail) did not produce any clear trends. The trend shown in Figure 3-7 is consistent with thermal induced movement of the abutment. Longer bridges will tend to induce more movement in the abutment which should have a larger impact on the backfill.

Cyclic movement of the abutment can also be inferred from an additional observation made at many of the sites. As the photographs in Figure 3-8 show, gaps form in front of the abutment as the abutment cycles back and forth. Such gaps would be exposed during a period when the bridge beams are expanded. This could occur at any time of the year given the daily cycling of temperatures that can occur.

The occurrence of a void under stub abutments was fairly frequent; the data in Table 3-2 show that 28 such occurrences were observed on bridges built since 1980. The void in this area was largest around the piles, and usually formed a radial cone of depression around the pile. There appeared to be a rough correlation between the void under the approach slab and the void manifesting itself under the abutment. It is not

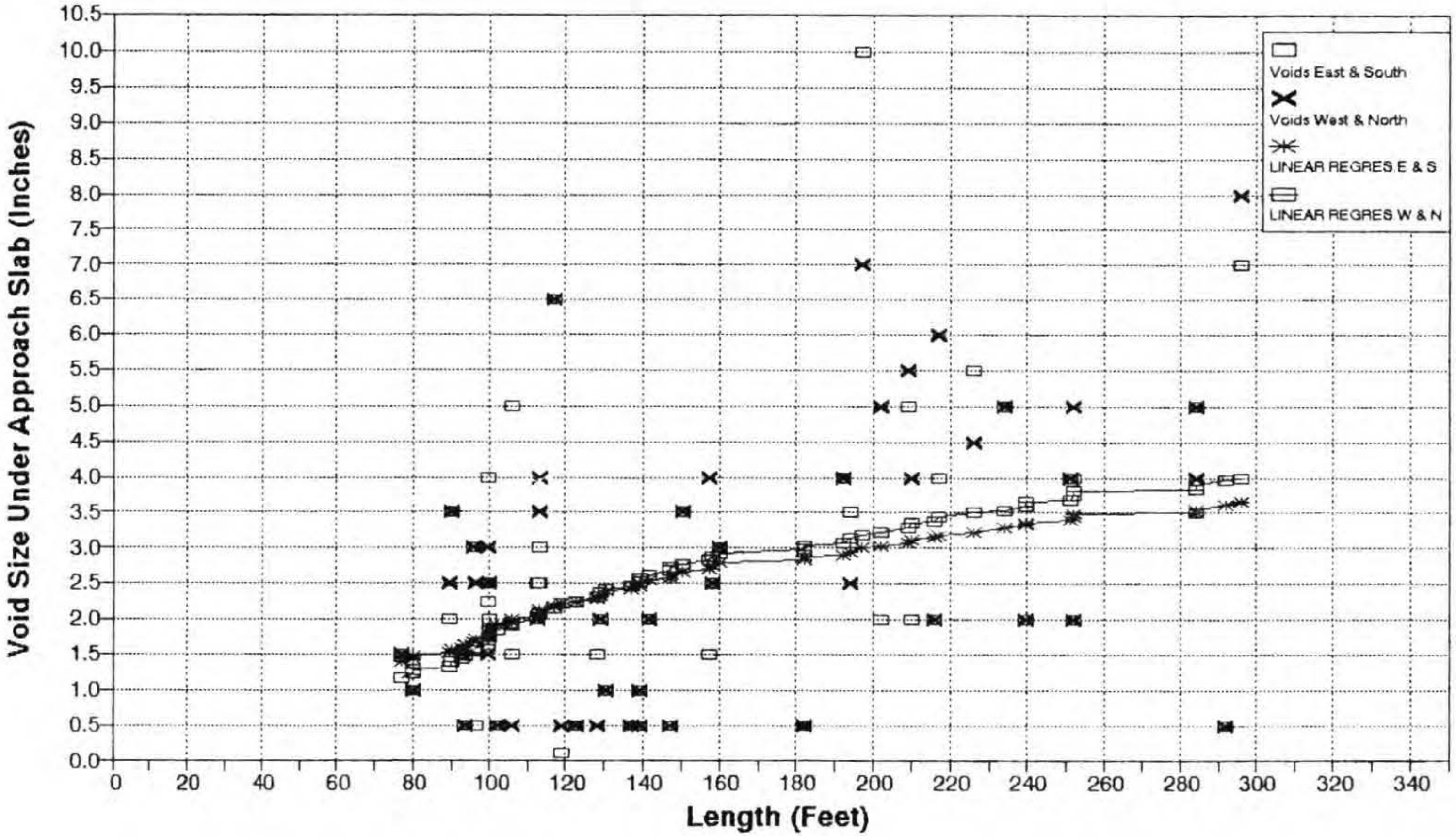



Figure 3-7. Void size under the approach slab at the abutment face versus bridge length.

(a) (b)

Figure 3-8. Photographs of gaps in front of abutment.

a. Gap formed in front of abutment wall, about one inch wide.
b. Gap formed along approach side of wing wall, about two inches wide.

known whether this void is caused by the piling action, settlement of the embankment, or other factors that may contribute to the void. Two other factors that may contribute to this void are scour and erosion. The erosion comes from water washing down from the top of the structure around the wing area. Many of the bridge sites had severe erosion around the wing walls and onto the front slope. Scour from the stream bank may also result in a washout from beneath the abutment. On stream crossings this may be a common occurrence. The sites were not previously monitored to delineate between erosion, scour, pile action, and settlement of the embankment.

Non-integral abutment bridges had a void under the abutment in only one case of the post 1980 bridges. Since only one of the non-integral abutment bridges had a void under the abutment, the void may be an integral abutment bridge phenomena.

It is interesting to note from the data in Table 3-2 that no observations of cracking of the berm or embankment were made on structures having non-integral abutments. It is possible that ground cover obscured cracks at these structures, but the total absence of observation of cracks points to abutment movement as a cause of such cracking. Cracking on the front slope berm may be due to pile action during thermal cyclic movements of the bridge. Erosion of the berm due to scour and water drainage may also create stability problems which induce this cracking. The approach embankment cracking is not easily explained. There were no specific reasons evident in the field why this was occurring.

## 3.2 Extended Field Investigations

### 3.2.1 Introduction

Following the preliminary field studies enough questions remained concerning the factors affecting void development that it was deemed necessary to conduct further field studies. These field studies were directed at obtaining more quantitative data with which to better understand the mechanisms involved in backfill subsidence. The extended field investigations included measurements of bridge length, temperature, vertical alignment, and void size.

The data base was used to select bridges that would be representative of the population as a whole. Factors involved in the selection were bridge length, date of construction, and region. Twenty-two of the bridges were integral abutment bridges built since 1980 with reinforced concrete approach slabs. Two of the bridges were non-integral abutment bridges constructed in 1989 and selected for re-examination after one year of service. Bridges were selected that had not been mudjacked. The extended surveys were conducted from the spring of 1991 through the fall of 1991.

Fifteen of the twenty-four bridge sites were inspected during this time period. Two were eliminated due to recent mudjacking of approach slabs. Three of the structures were eliminated due to traffic considerations that would have made the surveys hazardous and inconvenient for traffic. Two of the structures were slated for inspection, but scheduling problems and work on the model prevented the inspection. Structures #41-214-098/099, located on I-90 west of Rapid City, were non-integral bridges selected for re-observation after one year. These two structures were performing very well and had not developed any problems during the year of service, consequently no extended survey was conducted on them.

### 3.2.2 Evaluation Procedures

At each site the approach embankment was excavated to allow measurement of the void size. A special probe was developed to measure the area beneath the approach slab without boring through the slab. By excavating an area approximately three feet wide by three feet long and one foot deep, it was possible to insert the probe beneath the approach slab and get a reasonable estimate of the void volume. The probe was designed to consist of three sections that could be assembled in the field to a total length of 20 feet. The probe, shown in Figure 3-9, consisted of a hollow shaft that enclosed a solid shaft connected to a handle at one end and a 10-inch gauge on the other end. Drawings of the probe are shown in Appendix A. By rotating the handle the gauge would also rotate causing it to rest against the approach slab and the bottom of the void. The handle had a dial mounted to indicate the rotation of the end gauge. The dial was calibrated in the laboratory to provide a measurement of inches of height of the void. The volume could

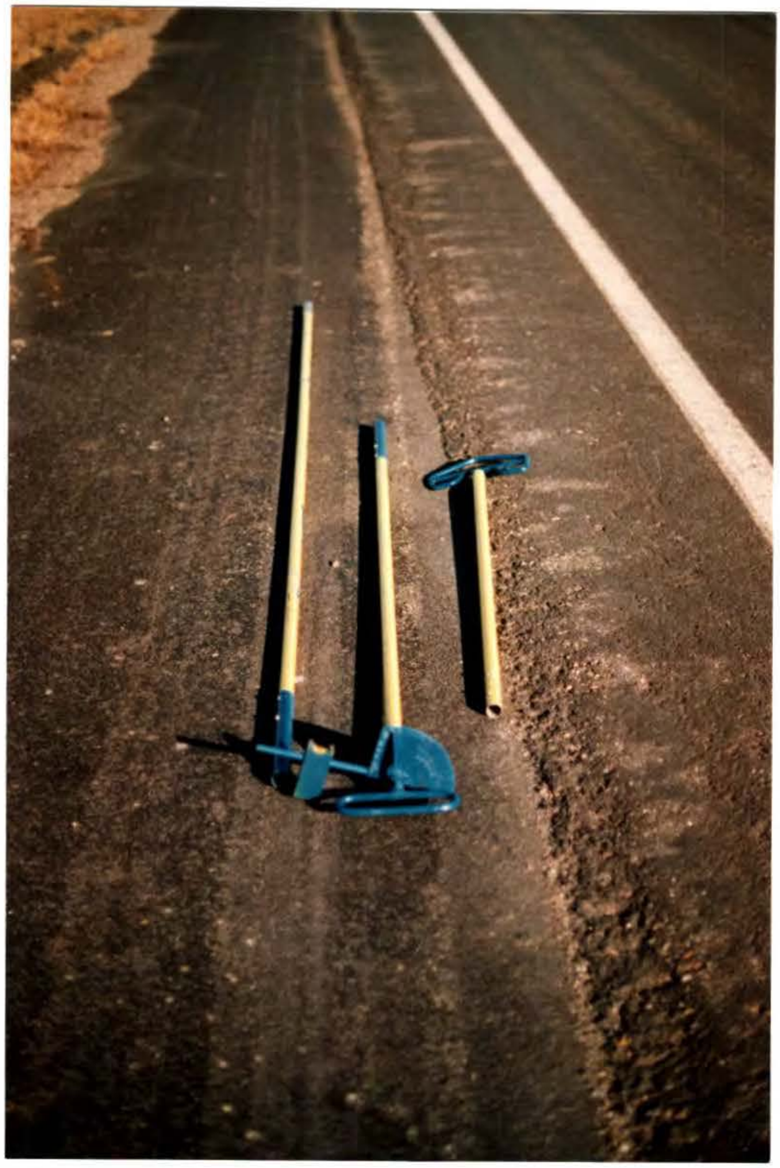


Figure 3-9. Probe developed to measure void height under approach slabs.

be estimated by positioning the probe at various locations under the approach and taking successive readings. A form was filled out at each site (Appendix A) showing the position of the probe measurement at each position.

The bridge was measured to determine the actual length of the structure and approach slabs. Temperature was also recorded to allow comparisons of measurements made at various dates. This was done by measuring down the centerline of the bridge and approaches with a 100-foot-long steel tape and adjusting for temperature. Stations were set on five-foot intervals on the roadway and approach slabs. Stations were increased to ten-foot intervals on the bridge structure itself. Stations were also set at each side of the expansion joints and the approach/bridge interface. The first station started 50 feet from the bridge/abutment interface and ended approximately 50 feet from the opposite bridge/abutment interface.

A level survey was conducted at each station and along the edge of the roadway perpendicular to each station. By leveling at each station a profile of the roadway, approach, and bridge could be made to analyze the transition. This also allowed us to determine the elevations of each approach with respect to each other. The level was set up to have an unobstructed view of the bridge and rod readings were taken to the nearest 0.01 of a foot.

### 3.2.3 Observations

The data collected from the extended field surveys are shown in Table 3-3 and include the structure number, the map number corresponding to Figure 3-1, the region, the location, the skew, the measured length, the measured void under the abutment, the measured void under the approach slab, and the temperature at the time of field survey. Additional information on these bridges is contained in Appendix A.

Voids were measured under 21 of the 30 approaches. No measurement could be made under three of the approaches due to asphalt overlays covering the embankment

Table 3-3. Data collected during extended field investigations.

| STRUCTURE NUMBER | MAP# | DATE BUILT | REGION | HIGHWAY NUMBER | LOCATION | SKEW (deg) | MEASURED LENGTH (ft) | VOIDABUT E & N (in) | VOIDABUT W & S (in) | APPRVOID W & S (cu.ft) | APPRVOID E & N (cu.ft) | ROAD TEMP (deg F) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 55-114-252 | 73 | 1981 | ABERDEEN | I 29 | 1.8 mi NW of Peever | 40 | 92.29 | 0 | 0 | 0 | 0 | 64 |
| 33-286-020 | 23 | 1985 | PIERRE | US 14 | 0.4 mi W of Blunt | 0 | 98.38 | 0 | 1.5 | 20 | 15 | 78 |
| 51-110-143 | 60 | 1986 | MITCHELL | SD 34 | 4.5 mi SW Jct SD 13 | 0 | 98.49 | 1 | 1 | 17 | 16 | 45 |
| 44-128-210 | 39 | 1983 | MITCHELL | SD42 | 1.7 mi E Jct US 81 | 0 | 98.62 | 3 | 1 | 6 | 79 | 40 |
| 44-110-156 | 38 | 1982 | MITCHELL | US 81 | 3.2 mi S I 90 | 0 | 111.69 | 0 | 0 | 20 | 83 | 38 |
| 21-424-228 | 16 | 1988 | PIERRE | US 212 | 0.6 mi E of Laplant | 0 | 117.65 | 0 | 0 | 0 | 0 | 112 |
| 59-328-274 | 88 | 1984 | PIERRE | US 14 | 7.9 mi W Jct US 83 | 0 | 127.61 | 0 | 0 | 3 | 3 | 72 |
| 48-485-271 | 45 | 1989 | PIERRE | SD 44 | 14.9 mi W Jct US 183 | 20 | 128.95 | 0 | 0 | *NM | *NM | 108 |
| 48-022-211 | 43 | 1987 | PIERRE | SD 44 | 0.3 mi E Jct SD 63 | 0 | 149.47 | 1.5 | 1.5 | 34 | 35 | 111 |
| 41-094-010 | 30 | 1986 | RAPID CITY | US 85 | Redwater N of Spearfish | 0 | 158.26 | 0 | 8 | 33 | *NM | 72 |
| 18-177-100 | 14 | 1988 | MITCHELL | SD 38 | 0.3 mi W Hanson Co. line | 0 | 176.42 | 3 | 0.5 | 0 | 0 | 58 |
| 36-361-298 | 27 | 1980 | PIERRE | SD 44 | 5.2 mi E Jct SD 73 | 0 | 195.71 | 1.5 | 1.5 | 123 | 200 | 104 |
| 36-071-076 | 25 | 1987 | RAPID CITY | I 90 | 6.3 mi W Cactus Flats | 0 | 200.91 | 0 | 3 | 58 | 8.3 | 79 |
| 28-392-038 | 20 | 1981 | PIERRE | SD 63 | 1 mi S Ziebach Haakon Co. line | 15 | 214.79 | 0 | 0 | 1 | 30 | 88 |
| 10-310-399 | 7 | 1985 | RAPID CITY | SD 79 | S of Newell over Belle Fourche | 25 | 296.02 | 2 | 0 | 107 | 82 | 86 |

* NM - Not Measured

shoulders; these are marked with a "NM" in Table 3-3. Three structures had approaches in which no void could be measured under either approach.

Structure #18-177-100, near Mitchell, did not have a measurable void under the approach slab during either the preliminary or extended field inspections. However, during the extended field surveys, voids beneath the abutment were noted. Structure #21-424-228, near Laplant, had a small void during the preliminary investigations, but on the day of the extended survey there was no void under the approach slab. This can be accounted for by the road temperature of 112° F recorded on that day. This bridge is a shorter structure and had only a small void (approximately 0.5 to 1 inch) on the west side during the first inspection. The 112° F temperature extreme during the extended inspection likely expanded the bridge sufficiently to close the small void. Structure #55-114-252, near Peever, also showed no measurable void during the extended survey. A void was found confined close to the abutment with a maximum size of one inch near the approach slab edge during the preliminary survey. This structure is also a shorter structure and the void may have been a localized effect that diminished over the year between inspections.

The voids under the approach slabs varied in size from one cubic foot to approximately 200 cubic feet, with an average size of the void under the 21 approaches of 46 cubic feet. There did not appear to be any particular pattern to the void development. At some bridges the volume under the two approaches was approximately the same, but at others the volume under one approach was much greater than that under the other. There did not appear to be any correlation with location (region), the date constructed, or the amount of void under the abutment. A little fewer than half of the approaches also experienced void development under the abutment. However, no relationship was obvious as voids under the approach occurred at ten of the locations where no void was apparent under the abutment. Conversely, at two approaches a void developed under the abutment with no void developed under the approach slab.

The preliminary field investigations showed that there was a slight trend of void height dependency upon bridge length. A plot of the void volume versus bridge length for the structures receiving extended surveys is shown in Figure 3-10. A similar trend to that previously found is noted, however, the trend is not absolute.

Bridge lengths were measured to determine changes in length with temperature variations. These data were found not to be consistent. The level surveys discounted the idea of bridge elevation influencing the void size. The elevations of the approach slabs were compared to the void sizes for their respective side. The data showed no trend for the elevation of the approach on the void size.

The level surveys did provide good cross-sections of the bridge/approach transitions. These cross-sections showed a trend that was not previously recognized. The cross-sections showed that many of the sleeper slabs have settled creating a bump at the roadway/approach slab interface. Figure 3-11 shows the cross-section measured for Structure #10-310-399, near Newell, from which it can be seen that a 0.10 inch differential has developed at each of the approach slab/roadway interfaces. The SDDOT has had this problem at other bridge sites in the state. The Mitchell DOT has mudjacked several sites to remedy these developments.

### 3.2.4 Soil Investigations

Embankment and foundation soil samples were collected from seven sites across the state to investigate the effects of variations in soil properties across the state. Surface samples of embankment materials were collected from beneath the abutment during the extended surveys. In addition, the SDDOT collected Shelby tube samples of the embankment and foundation soils. The surface samples were subjected to classification and compaction tests. The results are shown in Table 3-4. The results of these tests show that a wide variation in embankment soils is encountered across the state with the soils ranging from highly plastic clays to silty and clayey sands. The soils classified as CH, CL, SM and SC soils in the Unified Soil Classification System and as A-2-4, A-4, A-6, and A-7-5 in the AASHTO Classification System.

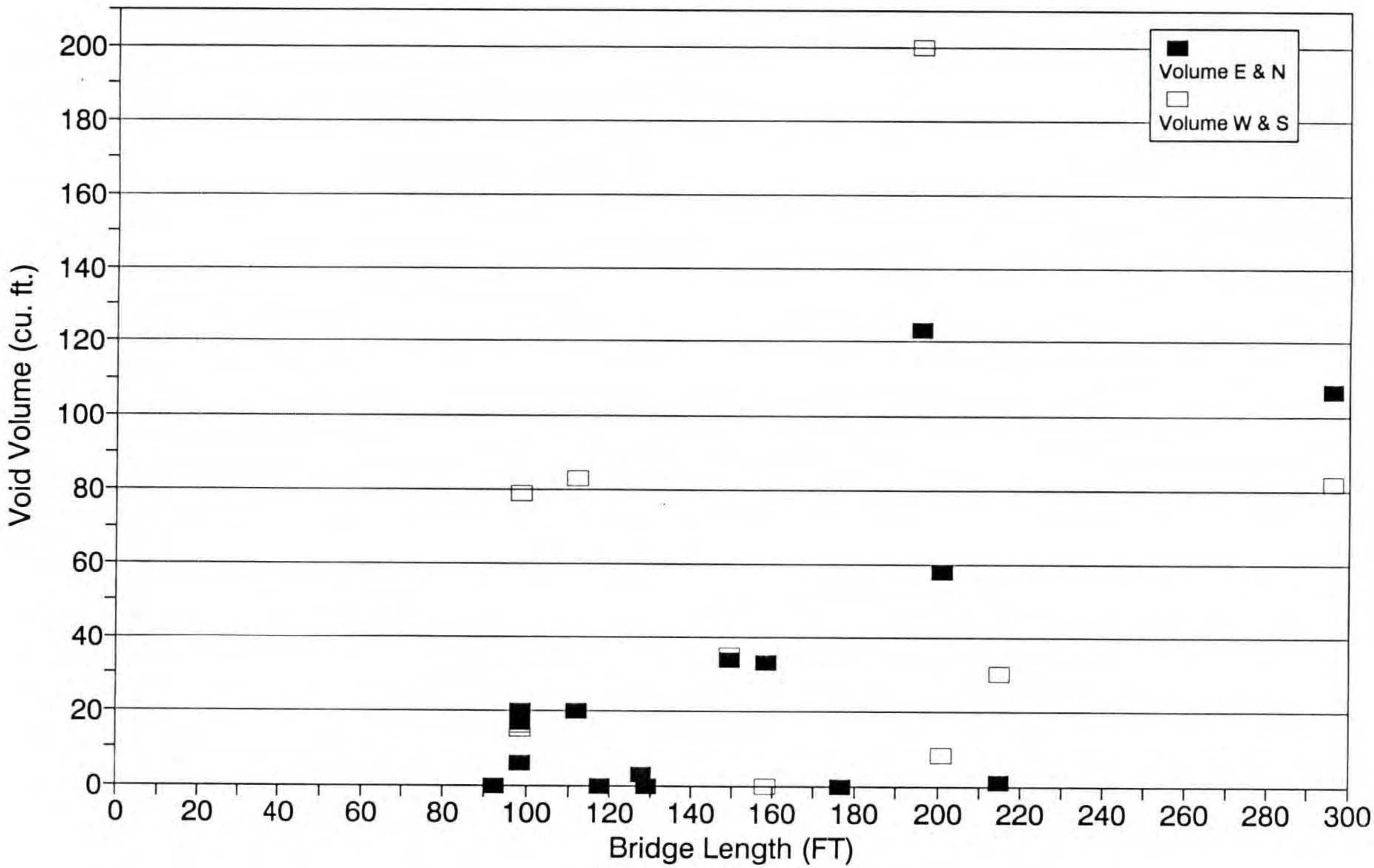



Figure 3-10. Void volume versus bridge length for extended survey bridges. Volume measured with probe.

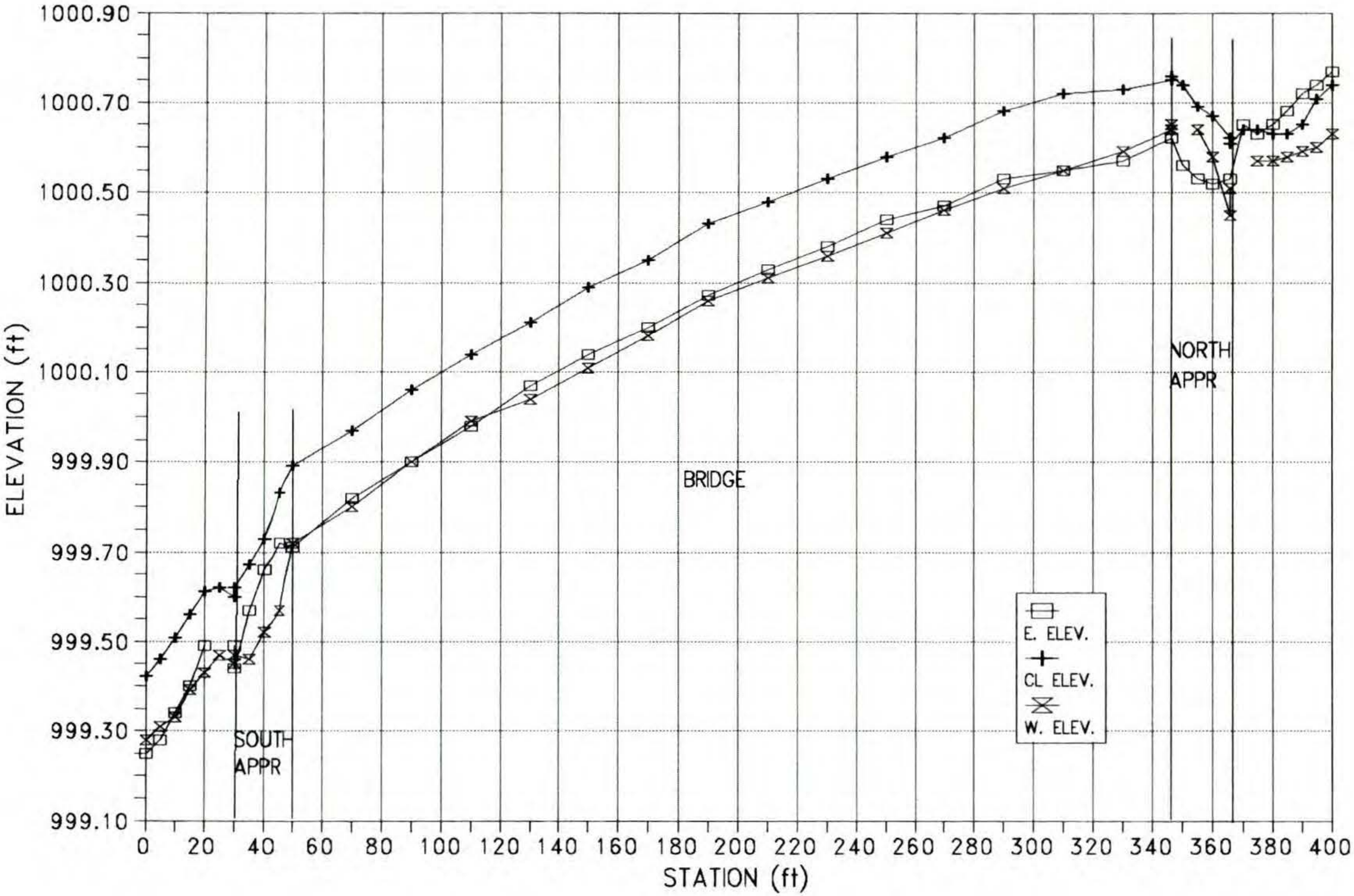



Figure 3-11. Level survey for Structure #10-310-399, near Newell.

Table 3-4. Summary of Soil Tests on Surface Samples.

| Structure Number and Location | LL | PL | PI | Specific Gravity | Percent Gravel | Percent Sand | Percent Silt | Percent Clay | USCS | AASHTO Classification | Optimum Water Content % | Maximum Dry Density pcf |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 36-071-076<br>6.3 mi. west of Cactus Flats | 36 | 22 | 14 | 2.68 | | 14 | 52 | 34 | CL | A-6 (12) | 19.1 | 102.7 |
| 33-286-020<br>0.4 mi. west of Blunt | 34 | 22 | 12 | 2.64 | 11 | 51 | 23 | 15 | SC | A-6 (1) | 14.8 | 114.6 |
| New Structure<br>West of LaPlant | 61 | 31 | 30 | 2.78 | | | | | CH | A-7-5 (31) | 22.4 | 96.8 |
| 59-390-274<br>7 mi. west of Pierre | 106 | 37 | 69 | 2.81 | 0 | 0 | 32 | 68 | CH | A-7-5 (85) | 30.7 | 85.1 |
| 52-390-278<br>Deadwood Ave.<br>Interchange by Rapid City | 31 | 25 | 6 | | | | | | SM | A-2-4 | 16.5 | 112.7 |
| 41-094-010<br>Over Redwater north of Spearfish | 25 | 22 | 3 | 2.70 | 8 | 33 | 42 | 17 | ML | A-4 | 16.6 | 106.4 |
| 55-114-256<br>1.8 mi. north of Peever | 54 | 26 | 28 | 2.77 | 3 | 18 | 35 | 44 | CH | A-7-5 (20) | 22.0 | 98.8 |

The tube samples were obtained in 3-inch and 2 1/2-inch-diameter Shelby tubes. An extruder was made to extract the samples at SDSU, but was broken during an extracting procedure. The samples were then taken to a commercial testing laboratory in Sioux Falls for extruding in their hydraulic press. Most of the samples were extruded, but the amount of disturbance was significant. Consequently, considerably less information was obtained from the tube samples than desired. The results from tests conducted on the tube samples are summarized in Table 3-5.

The samples from the Rapid City site could not be extruded. The logs on these samples show pressures from 850 to 1100 psi were required to push the samples. These pressures are an indication of the strength of the embankment material at this site. The soil is a reddish brown colored silty sand (SM and A-2-4) with a maximum dry density of 112.7 pcf at 16.5% optimum moisture.

The samples obtained from Structure 41-094-010 near Belle Fourche included two tubes that could not be extruded. The third tube, at a depth of 10 to 12 feet, was extruded, but was too disturbed for strength or consolidation testing. The sample was classified as a silty clay (CL and A-6(6)) with a PI of 11. The surface sample was a silty soil (ML and A-4) with a maximum dry density of 106.4 pcf at 16.6% optimum moisture content.

The samples from the Cactus Flats structure were the easiest to extrude. Consolidation tests were performed on a sample from a depth of 11 feet. The consolidation curve revealed an overconsolidated soil with an interpreted preconsolidation pressure, Pc, of 1.6 tsf. Using the maximum dry density of 102 pcf obtained from the surface sample, the current overburden pressures are approximately 0.9 tsf. Thus, the site soils should be capable of carrying loads higher than the current loads without experiencing settlement problems.

Table 3-5. Summary of Soil Tests on Shelby Tube Samples.

| Structure Number and Location | Depth, feet | LL | PL | PI | Specific Gravity | Percent Gravel | Percent Sand | Percent Fines | USCS | AASHTO Classification | Natural Water Content, % | Consolidation |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 36-071-076<br>6.3 mi. west of Cactus Flats | 10.0-12.6 | | | | 2.72 | | | | CL | A-6 | | Pc=1.6 tsf<br>Cc=0.32 |
| | | | | | | | | | | | | |
| 33-286-020<br>0.4 mi. west of Blunt | 15-15.5 | 32 | 14 | 18 | | 13 | 73 | 14 | SC | A-2-6 | 6.4 | |
| | 20-22.5 | 31 | 19 | 12 | | 38 | 52 | 10 | SC | A-2-6 | 14.2 | |
| | | | | | | | | | | | | |
| New Structure<br>West of LaPlant | 9.5-10 | 62 | 19 | 43 | | 0 | 8 | 92 | CH | A-7-5 (37) | 16.4 | |
| | | | | | | | | | | | | |
| 59-390-274<br>7 mi. west of Pierre | 16 | | | | 2.79 | | | | CH | A-7-5 | | Pc=2.5 tsf<br>Cc=0.37 |
| | 11 | | | | 2.89 | 0 | 3 | 97 | CH | A-7-5 | | Pc=1.45 tsf<br>Cc=0.35 |
| | | | | | | | | | | | | |
| 41-094-010<br>Over Redwater north of Spearfish | 11.3 - 12.0 | 34 | 23 | 11 | | 4 | 27 | 69 | CL | A-6 (6) | 28.7 | |
| | 15.0-17.5 | NP | | NP | | 4 | 78 | 18 | SM | A-2-4 | 17.5 | |

The samples obtained at the bridge near Blunt were difficult to extrude, but the 15 to 17.3 feet and 20 to 22.5 feet samples were extruded. The samples were too disturbed for consolidation or strength testing. The soil samples classified as clayey sands of medium plasticity (SC and A-2-6). The surface sample at this site was a clayey sand (SC and A-6(1)) with a maximum dry density of 114.2 pcf at 15% optimum moisture.

The samples from the LaPlant site were extruded, but showed considerable signs of disturbance. The log showed little difference in the soil with depth. The soil was a grey brown silty clay to a depth of 20 feet. A sample at ten feet was taken for classification. The soil classified as a silty clay with high plasticity (CH and A-7-5). The surface sample taken at the site was a silty clay material with high plasticity (CH and A-7-5) and a maximum dry density of 96.8 pcf at 22.4% optimum moisture.

The samples taken at the bridge west of Pierre were extruded and consolidation tests performed on 2 samples from depths of 11 and 16 feet. The consolidation curves for each depth showed considerable sample disturbance. The determination of Pc was therefore somewhat arbitrary on these curves. The preconsolidation values selected were 1.45 tsf and 2.5 tsf for the 11-foot and the 16-foot samples, respectively. The surface sample at this site was a silty clay of high plasticity (CH and A-7-5) with a maximum dry density of 85.1 pcf at 30.7% optimum moisture. Using the moist density the current overburden pressure is approximately 0.9 tsf. Thus, the site soils should be capable of carrying the applied loads without settlement being a problem.

The final sample was obtained from a bridge near Peever. Surface samples were obtained for classification and undisturbed samples were collected at this time at a depth of three feet below the surface of the front slope. These samples were used in an undergraduate soil mechanics class for consolidation testing. The consolidation curve showed considerable disturbance, characterized by the flat nature of the curve. The preconsolidation pressure, Pc, was estimated to be approximately 0.8 tsf. The shallow depth of the sample indicates that the soil has considerable strength before settlement would be a problem. The surface soil samples classified as silty clays with a high plasticity (CH and A-7-5). The maximum dry density was 98.7 pcf at 22% optimum moisture.

The laboratory characterization and difficulty of extruding samples on the selected bridge sites show the embankment and subsurface soils to be generally overconsolidated in nature. This was to be expected since most of the bridge approaches built since 1980 were erected on prior bridge sites. The soils have had structural and overburden loads applied for many years. The data show that consolidation settlements should not be significant at these bridge sites. The soil test results show that the soils used for embankment construction across the state are highly variable with respect to soil properties. Despite this variability, no soil related trends related to void development under the approach slab were apparent.

### 3.3 Summary

The preliminary and extended field investigations showed that void development under approach slabs has occurred at structures in all parts of the state. Voids under the approach slab were observed ranging in height from about 1/2-inch to as large as 14 inches. A preliminary field investigation was conducted at over 100 sites with information collected concerning void size, approach slab cracking, erosion, and embankment cracking. Attempts to correlate the void development with factors such as bridge length, soil type, and date of construction were mixed. Generally, the longer the bridge the larger the void, but exceptions existed. Approach slab cracking was tied closely to the presence of a void under the approach slab. The incidence of embankment cracking was more random, but was not observed at all on non-integral abutment bridges. The field observations indicated that void development is most strongly associated with integral abutment bridges.

The extended field investigations were conducted in an attempt to better understand the mechanisms of the void development. During the extended field investigations better measurement of the void space was made, level surveys were made of the alignment, and soil samples were collected for laboratory analysis. The extended surveys did not materially aid in a better understanding of the mechanisms of the void development. The development of the void space under the approach slab was so ubiquitous across the state that factors such as soil type, general geology, date of construction bore no relation to the occurrence of the void. The level surveys did reveal the "bump at the end of the bridge" has in large measure been pushed out to the end of the approach slab. The soil investigations conducted revealed that the cause of void is not likely to be consolidation settlement.

However, it must be concluded on the basis of the field studies that the key variable is the use of the integral abutment. Unfortunately the data collected from the field studies did not identify a mechanism of integral abutment/approach system interaction which would cause the void development. To further delineate the mechanism causing void development, the idea of instrumenting existing abutment/approach systems was investigated. Difficulty in identifying a location in which traffic was not a major concern and which could be easily instrumented lead to the idea of conducting a model approach/bridge system interaction study to delineate the mechanisms. At first a laboratory model study was considered. After exploring the idea further, an existing, near full-scale bridge-abutment system was discovered at the Brookings Area DOT office. This structure proved to be ideal for conducting model studies of the abutment/approach system interaction. The results of the model study are discussed in Chapter 5.

# CHAPTER 4
# REVIEW OF DESIGN PRACTICES

## 4.1 Introduction

This chapter presents a review of current design practices concerning bridge abutments, approach slabs and bridge end backfill. Included are a review of current design and performance of other states' approach system designs and an assemblage and review of existing SDDOT design, construction and maintenance data relating to bridge end backfill. To ascertain the status of bridge end backfill designs in other states, a survey of state DOTs was undertaken.

## 4.2 Survey of State Transportation Departments

### 4.2.1 General

In June of 1991 a survey was sent to the Departments of Transportation in 27 states. The survey was intended to identify the use of integral abutment bridges and approach systems in the United States. The states selected were chosen based on a previous survey conducted by Wolde-Tinsae *et al.* (1983), in which these states had indicated the use of integral abutment bridge designs. From the 27 surveys sent out, 24 responses were received. Three of the states responded contradictory to the previous survey, and responded "no" to the use of integral abutment bridges. Thus, 21 states were identified which use integral abutment bridges. Since the primary concern of this study was to determine the cause of the void beneath the approach slab, the survey was directed at the abutment-approach-backfill area of the bridge. The survey was sent to the different DOTs with the intention that completion of the form would require participation among bridge, materials, and maintenance departments. In several instances the survey was filled out by a single department without group participation. Therefore, the results, in some instances, may not provide a complete picture of what is occurring in a state. The Washington State DOT survey provided an excellent example of the differing views on bridge approach performance among the various branches of a DOT. Despite this deficiency, the survey still provides valuable insight into the current practices of other DOTs. The survey and results, along with a sample cover letter, are presented in

Appendix B. A summary of the survey is presented below for each question. Unless otherwise stated, the responses are analyzed for the number of respondents that use integral abutment bridges.

### 4.2.2 Survey Questions and Responses

**Do you use bridge designs with integral abutments and/or without expansion joints at the bridge/approach interface? yes __ no __ Primary reason why or why not:**

Twenty-one of the twenty-four states responding use integral abutments in their bridge designs. Of the affirmative responses, 15 (71.4%) cited the use of the integral abutments to eliminate the expansion devices and the maintenance thereof. Additional reasons cited for integral abutments were: reduced initial costs, reduced construction and maintenance costs, and transfer of load to embankment.

**What are the maximum length limits for integral abutment bridges, in feet, for the skew and type of bridge listed below?**
**Steel _____, Concrete _____, Pre-stressed Concrete ___**
**Are different limits imposed for skew?**

The number of respondents indicating maximum bridge lengths for the various types of structures are categorized into three length categories in Table 4-1. The survey shows that 300 ft. is a common length for steel girder composite structures, while concrete slab and prestressed concrete structures have structures greater than 300 ft. as the norm. The longest steel composite integral bridge reported was 500 feet, while the longest concrete slab and prestressed concrete reported was 927 feet. The restrictions that skew impose on bridge length varied from no restriction to 30 degrees as a length limiting factor. One respondent indicated that skewed integral abutment bridges were not allowed. The number of respondents imposing length limitations due to skew are summarized in Table 4-2.

| Table 4-1. Number of states reporting length limitations. | | | |
|---|---|---|---|
| | Length Limitations | | |
| Type of Structure | <300 FT. | 300 FT. | >300 FT. |
| Composite Steel Girder | 7 | 9 | 5 |
| Concrete Slab | 7 | 6 | 8 |
| Prestressed Concrete | 3 | 7 | 11 |

| Table 4-2. Number of states reporting skew angle limitations. | | | | | |
|---|---|---|---|---|---|
| Skew Angle | No Skew Allowed | Not Specified | No Restrictions | 15° | 30° |
| Number of States | 1 | 2 | 7 | 1 | 10 |

**What types of approach systems are currently being used?**

| | **Reinforced Concrete** | **Asphalt Concrete** |
|---|---|---|
| **Interstate** | ________________ | ______________ |
| **State Highways** | ________________ | ______________ |
| **Secondary Roads** | ________________ | ______________ |

Table 4-3 shows the responses grouped by approach system and highway system. These responses show the use of approach slabs to be a prevalent method of improving the approach-bridge transition. The respondents indicated that 18 (85.7%) use reinforced concrete approach slabs on interstate bridge systems and eight (38.1%) indicated they use asphalt approaches. The overlap indicates the use of both systems. On state highways, 14 (66.7%) use reinforced concrete approaches and 10 (47.6%) use asphalt approaches. Once again there was some overlap indicating use of both systems. Secondary roads showed a drop off of the number of reinforced concrete approach systems with only seven states indicating such use. The number using asphalt approach systems was 16, a use rate of 76.2%. Two of the respondents indicated that secondary highways were not applicable for their consideration.

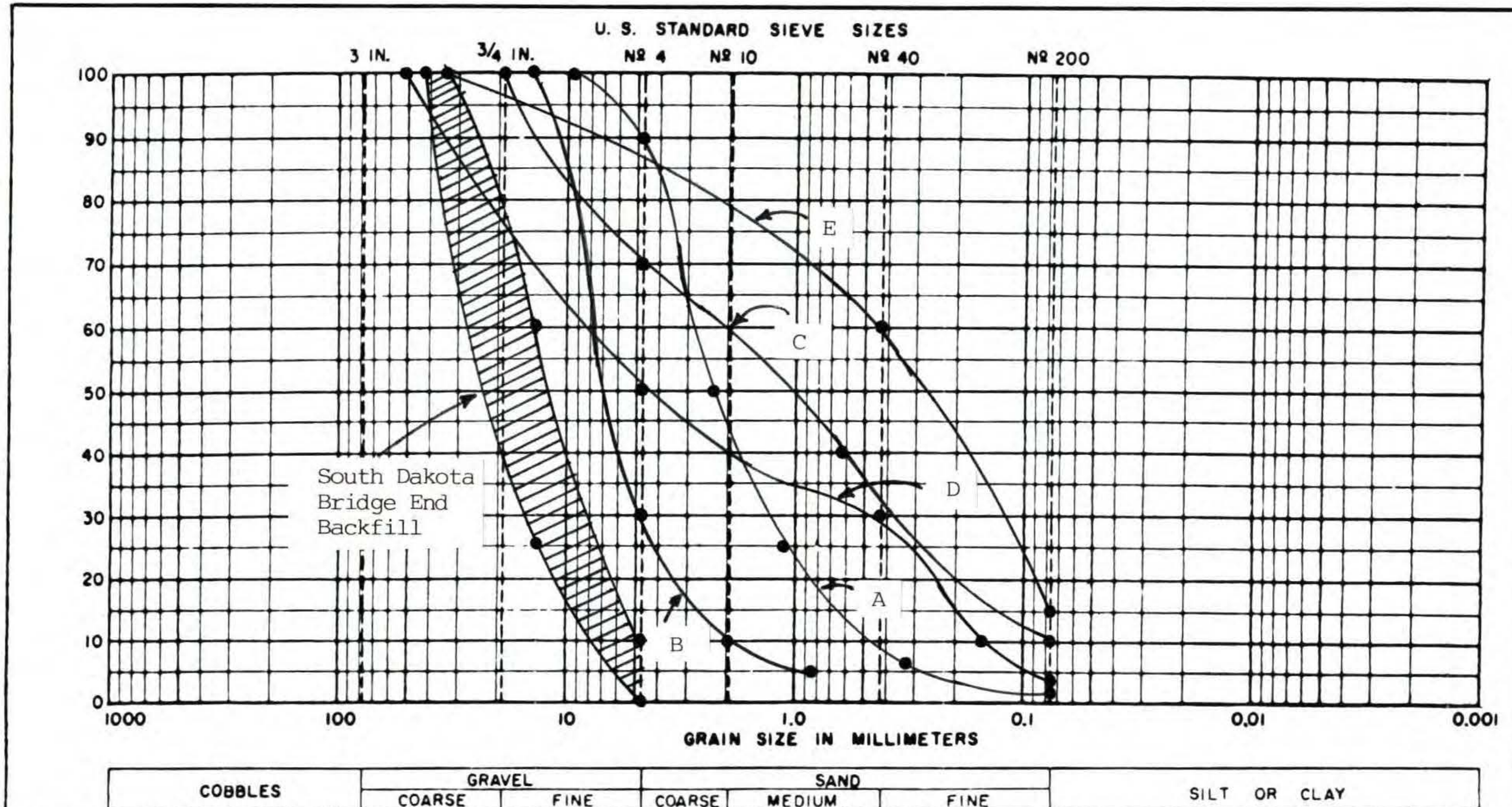
| Table 4-3. Number of states using concrete/asphalt approach systems. | | | |
|---|---|---|---|
| Highway System: Approach System | Interstate | State | Secondary |
| Reinforced Concrete | 13 | 11 | 3 |
| Asphalt | 3 | 7 | 12 |
| Both | 5 | 3 | 4 |
| N/A | 0 | 0 | 2 |

**If reinforced concrete approach slabs are used, answer the following:**
**Is slab integral with the abutment? yes __ no __**
**Is slab supported by a sleeper slab? yes __ no __**
**Is slab designed to rest on approach fill or to span its length of approach fill?**
**rest on fill __ span fill __**

The number of DOTs using reinforced concrete approach slabs tied to the abutment is nine (42.8%). The remaining DOTs surveyed do not tie the approach slab to the abutment, with one exception that does not employ the use of reinforced concrete approach slabs. The use of a sleeper slab is employed by ten (47.6%) of the states. The design of the approach slab, whether integral or not, consisted of spanning the fill for eleven (52.4%) and resting the approach on the fill for five (23.8%) of those surveyed. Several indicated that spanning some part of the fill with the approach slab was a design characteristic.

**What type of backfill material is used for the backside of the abutment? We are particularly interested in your backfill specification, i.e., material type, gradation, and compaction requirements.**

This question indicates the primary concern of the Bridge End Backfill study. Many different backfill gradations were reported by the DOTs, and none had exactly the same specification. Particle-size limits were reported by only five respondents and the range of gradations are shown in Figure 4-1 and tabulated in Table 4-4. Figure 4-1 provides an indication of the wide range of gradations used for backfill. For reference the current SDDOT backfill limits are plotted in Figure 4-1. Many of the respondents used a cutoff on the #200 sieve for the backfill specification and indicated ranges from




| Sample Nº | Elev or Depth | Classification | Nat W.C. | L.L. | P.L. | P.I. | | |
|---|---|---|---|---|---|---|---|---|
| A | | Colorado | | | | | | Project: |
| B | | New York | | | | | | |
| C | | North Dakota | | | | ≤ 12 | | |
| D | | Wyoming | | | | | | Area: |
| E | | Oklahoma | | | | ≤ 6 | | Boring Nº: |

**GRADATION CURVES** | Date:

Figure 4-1. Gradation curves reported in survey.

0.5% to 15% passing as the specification. The compaction of this area was specified by only ten of the respondents and ranged from 90% to 95% of Standard Proctor with one indicating Modified Proctor use.

| Table 4-4. Particle-size gradations reported in survey. | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Colorado | | New York | | North Dakota | | Wyoming | | Oklahoma | |
| 3/8" | 100% | 1" | 100% | 3/4" | 100% | 2" | 100% | 3" | 100% |
| #4 | 60-90% | ½" | 30-100% | #4 | 35-70% | #4 | 0-50% | 1" | 90-100% |
| #8 | 0-45% | ¼" | 0-30% | #30 | 10-40% | #40 | 0-30% | #40 | 0-60% |
| #16 | 0-25% | #10 | 0-10% | #200 | 0-10% | #100 | 0-10% | #200 | 0-15% |
| #50 | 0-6% | #20 | 0-5% | PI | ≤12 | #200 | 0-4% | PI | ≤6 |
| #200 | 0-2% | | | | | | | | |

Table 4-5 shows the responses to the compaction effort applied to the backfill area.

| Table 4-5. Compaction effort in backfill area. | | | | | |
|---|---|---|---|---|---|
| Compaction Effort | None | No Response | 90% | 95% | Modified |
| Number of Respondents | 1 | 11 | 2 | 6 | 1 |

**In the construction phase, what slope of cut is used for the embankment upon which the backfill material is placed?**

The slope of cut upon which the backfill is placed after construction correlates to the SDDOT practice. Initially the slope is cut back at a 2:1 angle during construction of the embankment and is then cut back, after abutment construction, at a slope that will encompass the length of the approach system. Responses to this question were varied, but many DOTs clarified the procedures used for their construction. Slopes varied from vertical to 6:1 with many stating not applicable to their situation. Responses of vertical and horizontal were indicative of a vertical area along the abutment where backfill was placed instead of a tapered back area. The majority of the states use a sloped cut of 1.5H:1V to 2H:1V, with one state reporting use of a 4H:1V slope and one reporting use of a 6H:1V slope. The responses are summarized in Appendix B.

**What type of drainage system is used behind the abutment? Are drainage fabrics used?**

The prevalent method of drainage in the backfill area is perforated pipe along the abutment face. Seventy three percent of the respondents use perforated pipe in the backfill area. Many of the respondents incorporate the perforated pipe along with a granular backfill strip drain system. Other responses included weep holes through the abutment and geocomposite strip drains. One state provided for no drainage in the backfill area and another specified only catch basins on the approach slab.

The use of fabrics in the backfill area was limited to only 33% of those surveyed. The primary use of fabrics was as a filter cloth or wrap and was not specified for strengthening the backfill area.

**List any problems that have arisen in the backfill area and efforts to correct the problem, i.e., voids, settlement, cracking of approach slab, cracking of abutment wall, mudjacking to fill voids.**

One of the major purposes of this survey was to see the extent of settlement in the backfill area across the nation. The respondents indicated that settlement in the backfill area is a recurring problem. Over 71% indicated problems with voids and/or settlement in the backfill area. Other problems cited were sleeper slab settlement, cracking of the approach slab, and support ledge failure. Approximately 24% of the respondents reported no problems have occurred in the backfill area. The five states that reported no problems with the backfill area are Idaho, Nebraska, New York, Vermont, and Virginia. This is one of the primary sections in the survey where the results may be tilted due to non-group participation on responding to the survey. The responses we have received from field personnel during the course of our investigation have varied from the office personnel.

The questionnaire also requested solutions for the problems encountered. The primary solution for the voids was mudjacking. This accounted for 47.6% of the solutions. The other recognized solution was asphalt overlays and wedges. Two of respondents suggested improving construction compaction and supervision as the solution.

**Has any research been performed involving the abutment and backfill interaction? yes __ no __**
**If yes please list references and or results.**

This final question shows that a number of states are trying to find solutions to the problems encountered in the backfill area. Six of the states responding indicated some research was done or is being done in this area. The Washington State DOT has also done some research into this problem and is summarized in the literature review.

### 4.2.3 Summary of the Survey

The use of integral abutment bridges has become widespread across the nation as a solution for eliminating joint problems on bridges. The length of integral abutment bridges has increased over the years as design personnel develop more confidence in their performance. The survey shows that integral abutment bridges are being built regularly with lengths up to and exceeding 400 feet. Along with longer bridges, the use of reinforced concrete approach slabs has increased. The survey responses show that nearly 80% of all bridges being built are incorporating reinforced concrete approach slabs into design. The design of the approach slab varies, but most responses indicated the approach was designed to span part or all of the fill. Approximately one-third of the approach slabs are integral with the bridge, which increases the total length of jointless design.

The approach backfill material varies, but usually consists of a coarse-grained material with little fines. Compaction of the backfill material generally is in the range of 90 to 95% of Standard Proctor (AASHTO T99). The most common drainage system incorporates a perforated pipe along the bottom of the abutment face. Geotextiles are used only one-third of the time as a wrap around the backfill to prevent washout.

The major problem identified in the backfill area was settlement of the backfill material. Several other problems were cited, eg. sleeper settlement, approach cracking, and support ledge failure, and there were several responses indicating no problems in the backfill area. The survey may be tilted in this area since the survey may have been answered by only one department. The WSDOT survey shows the variety of responses which are given to a common problem when presented to personnel in various fields.

The survey requested solutions being used to approach problems that were encountered. The void space beneath the approach slab is commonly mudjacked to fill the void. Asphalt overlays and wedges are also commonly used as an approach fix when appropriate.

The SDDOT incorporates many of the design philosophies reported by survey participants. Integral abutment design has gained widespread popularity within South Dakota as a solution to joint problems. SDDOT currently designs most of their bridges with a reinforced concrete approach system and an underlying coarse backfill gradation. Current design uses 15 feet for unsupported span length. The observations made during the field surveys show that many of the problems experienced in South Dakota are also occurring across the nation. Void development, approach cracking, and sleeper slab settlement are common problems encountered across the state and the nation as a whole. Yet fixes such as mudjacking and overlays increase maintenance costs without addressing the underlying causes of the problems.

## 4.3 Review of SDDOT Design Practices

### 4.3.1 Current Design Practices

The construction of earth embankments for integral abutment bridges is covered by the 1990 edition of the South Dakota Department of Transportation (SDDOT) Standard Specifications for Roads and Bridges (SDDOT 1990). Other information on design philosophy and design criteria was obtained through correspondence with SDDOT Bridge Design Department. The construction of integral abutment bridges will be covered in chronological order except where clarification is required for better understanding.

The embankment for the placement of integral abutment bridges shall be built to the plans provided and to Section 120 of the 1990 Standard Specifications for Roads and Bridges. Top soil shall be removed and the area thoroughly disked and recompacted. Embankment material shall be placed in lifts not to exceed a loose depth of eight inches. Lift depths exceeding eight inches may be used provided test results verify that density requirements are met the entire depth of the lift. Each lift is to be compacted with a sheepsfoot roller or other approved roller to the satisfaction of the engineer before the next lift is applied. The roadway embankment within the area bounded by the toe and extending in a line 100 feet from the bridge end is to be compacted to 97% maximum dry density as determined by Standard Proctor (AASHTO T99). The shape of the embankment shall be determined from the plans, but the embankment will be built with a bench for construction of the abutment and with a 2H:1V slope cut back from the bench.

Placement of piles for the abutment requires preboring the embankment ten feet or to natural ground, whichever is greater. This is done to minimize negative skin friction due to embankment settlement. Prebore holes for steel piles shall not be less than approximately six inches greater than the pile width. A single row of piles is installed with their weak axis orientated in the direction of longitudinal thermal movements of the bridge. This allows the piles to form a plastic hinge near the bottom of the abutment. Pile capacities are assumed to have no reduction in full AASHTO capacity of 9000 psi, providing the geometry of the bridge remains within the boundaries set by the DOT. Piles are driven to obtain bearing capacity set forth in the design and shall be determined by static load tests or pile driving formulas presented in section 510 of the 1990 standard specifications. Prebore holes are then backfilled with a coarse dry sand and compacted to prevent collapse of embankment material into the prebore holes.

The abutment is designed to act as an integral part of the bridge. Bridge connections to the abutment are detailed in Attachments 1, 2 & 3 in Appendix C. Attachments 1, 2 & 3 are for the continuous concrete, prestressed concrete girder, and the steel girder bridges respectively. These details show the integration of the bridge with the abutment forming a rigid connection. The abutment is designed to resist full Rankine passive earth pressure applied across the entire rear face. The abutment acts rigidly, but is designed as a continuous beam for flexural backwall reinforcement. The general abutment configuration is a stub abutment approximately six feet in height supported on a single row of HP 10 x 42 steel piles. This detail is subject to change to suite particular bridge geometries or requirements.

Wing walls are built into the abutment with the length determined based on a 2H:1V in-slope at the bridge end and a 2H:1V spill slope around the end of the wing down to the top of the berm. The typical orientation of the wing walls is in-line with the abutment unless wing length becomes excessive. If the length becomes excessive the wings may be placed parallel with the roadway to reduce the area subject to passive earth pressure.

The bridge length is determined using AASHTO "Standard Specifications for Highway Bridges" for structures in cold climate. This specification provides temperature ranges that can be used to limit maximum bridge length. The temperature ranges are -30° to 120° F for metal structures and a change of 80° F. for concrete structures. The AASHTO Specifications for thermal expansion coefficients are $6.0 \times 10^{-6}$/°F and $6.5 \times 10^{-6}$/°F for concrete and steel, respectively. The maximum length for concrete structures is 700 feet and 350 feet for steel structures. This would result in a maximum cyclic movement of approximately 4 inches for either type of structure. Skew of the bridge is limited to 30° for girder bridges and 45° for continuous concrete bridges.

After completion of the bridge, the 2H:1V slope cut back from the bottom of the abutment is increased to a slope that encompasses the entire length of approach system. Current design practice is to use a bridge end backfill material wrapped in a filter cloth for the approach support.

The approach slab is designed using the Load Factor Method to span an unsupported length of 15 feet. This design allows for the approach slab to function properly even if voids develop close to the abutment. The SDDOT currently uses a single approach slab of reinforced concrete attached to the abutment and supported on a sleeper slab. The approach slab is connected to the abutment by a row of #7 rebar at nine-inch spacings along the bottom of the approach slab. The sleeper is a reinforced concrete slab five feet wide and extending the width of the approach roadway. Several approach slab configurations have been used in the past and are presented in Appendix C. The most basic approach system extends approximately 20 feet from the abutment and is designed for the end to be perpendicular to the roadway. This system is supported on the free end by a sleeper slab. Another configuration of approach system may incorporate two approach slabs in series extending from the abutment. This type of system would utilize two sleeper slabs at each approach end. A strip seal expansion joint is used at each approach end and the gap is designed to accommodate the approach slab and overall bridge thermal expansion. Past configurations also utilized a one-foot-wide asphalt concrete expansion joint. Surface drainage is designed to flow off the slab end and down the in-slope in rural areas. Drop inlets are used in urban areas.

A perforated PVC drainage pipe is placed at the bottom of the cut parallel to the abutment face and extending across the abutment face including the wing walls. The PVC pipe is connected to steel pipes that drain onto the spill-slope. The fabric wrap is placed under the drain pipe and extends up the side of the cut for backfill placement. The select bridge end backfill is satisfactorily vibratory compacted in maximum one-foot lifts. Because the material is a coarse gradation, no density tests are done to determine satisfactory compaction. The backfill is placed and compacted until the configuration meets the design grade lines and is ready for the approach system.

### 4.3.2 Bridge End Backfill

The SDDOT uses bridge end backfill in their approach systems. Table 4-6 lists the changes made to the approach system since 1970. The use of select granular backfill was incorporated into the 1970 Standard Specifications for Roads and Bridges. The special provision is presented in Figure 4-2. In 1975 the specifications were changed to a slightly different gradation. Figure 4-3 shows the Special Provision for Bridge End Backfill in the 1975 Standard Specifications. There was little change in the gradation of the material, but the plasticity provision was dropped from the specification. The liquid limit was raised slightly as were the ranges for the Number 4 and 10 sieves, but the material is essentially the same. The specifications for the perforated pipe and three-inch gravel cushion under approach slabs was included in 1975.

Special Provision for Bridge End Backfill

Dated: April 1, 1969

II. Materials

A. Select Granular Backfill: This material shall be free from dirt, vegetable matter or other foreign substance. This material shall meet the following requirements.

Percent Passing 2 inch Sieve----------100

Percent Passing 1 inch Sieve----------70 - 100

Percent Passing No. 4 Sieve-----------30 - 75

Percent Passing No. 10 Sieve----------20 - 60

Percent Passing No. 40 Sieve----------10 - 35

Percent Passing No. 200 Sieve---------0 - 10

The fraction passing the No. 40 sieve shall have a liquid limit not to exceed twenty five (25) and a plasticity index not greater than six (6) as determined by AASHTO Test Methods. Abrasion loss per AASHTO T-96 shall not exceed forty five (45) percent.

Figure 4-2. 1970 Standard Specification for Bridge End Backfill.

Special Provision for Bridge End Backfill

Dated: January 10, 1975

II. Materials

A. Select Granular Backfill: This material shall be free from dirt, vegetable matter or other foreign substance. This material shall meet the following requirements:

Percent Passing 2 inch Sieve--------- 100

Percent Passing 1 inch Sieve---------70 -100

Percent Passing No. 4 Sieve----------30 - 80

Percent Passing No. 10 Sieve---------20 - 65

Percent Passing No. 40 Sieve---------10 - 35

Percent Passing No. 200 Sieve-------- 0 - 10

The fraction passing the No. 40 sieve shall have a liquid limit not to exceed thirty (30) as determined by AASHTO Test Methods. Abrasion loss per AASHTO T-96 shall not exceed forty five (45).

B. Perforated Metal Pipe: Pipe shall conform to Section 1002.1B of the Standard Specifications.

Figure 4-3. 1975 Standard Specification for Bridge End Backfill.

| Table 4-6. | Chronological listing of changes to SDDOT approach backfill requirement since 1970. |
|---|---|
| Year | Change |
| 1970 | 2:1 back slope; perforated drain pipe, perforations down; 1969 backfill gradation |
| 1975 | Backfill gradation changed slightly, allowable liquid limit increased, reference to plasticity index dropped; three-inch gravel cushion added under approach slabs |
| 1980 | Backfill gradation changed significantly to current coarse-graded material; pipe perforations placed upward |
| 1982 | Backslope decreased significantly to extend cut slope under entire length of approach slab resulting in varying backslopes; gravel cushion deleted |
| 1988 | Fabric wrapped around backfill added on select structures |

In 1980 a new specification for the bridge end backfill was adopted and later placed in the 1985 Standard Specifications. This was a dramatic change from the previous specifications as the new backfill consisted of a crushed stone with a uniform gradation. The backfill is a Class "A" aggregate of a crushed stone meeting the gradation requirements of Section 850 with the specification shown in Figure 4-4. This material is currently being used with all bridges built with reinforced concrete approach slabs. The gradation curves for the three changes are shown in Figure 4-5. The stone used is usually a Sioux Quartzite material east of the Missouri River and a Minnekahta Limestone west of the river. SDDOT tests on the two types of backfill provide a unit weight of 103.5 pcf and a friction angle of 50° for both materials. Tests conducted for this study showed a maximum density of 102 pcf, a minimum density of 83 pcf, an absorption of 0.21%, and a specific gravity of 2.44. The maximum and minimum density tests were not performed to ASTM standards, but results obtained closely resemble SDDOT results.

From 1970 to 1980 the shape of the backfill area was a wedge extending from the base of the abutment, back to the surface at a 2H:1V slope. In 1982 the shape was

Section 850 Select Granular Backfill

Dated: January 1985

This material shall be free from dirt, vegetable matter or other foreign substance.

The material shall meet the following gradation requirements by dry weight:

Percent Passing a 1½ inch Sieve.........................100

Percent Passing a 1 inch Sieve......................95 - 100

Percent Passing a ½ inch Sieve......................25 - 60

Percent Passing a No. 4 Sieve........................0 - 10

The material is to be crushed rock having at least two (2) fractured faces on material retained on the No. 4 sieve.

Abrasion loss shall not exceed forty (40) percent.

Sampling and Testing:

Sampling ..............................SD 201

Gradation ..............................SD 202

Fractured Faces ......................SD 211

Los Angeles Abrasion ..... AASHTO T-96

Figure 4-4. 1985 Standard Specifications for Bridge End Backfill.

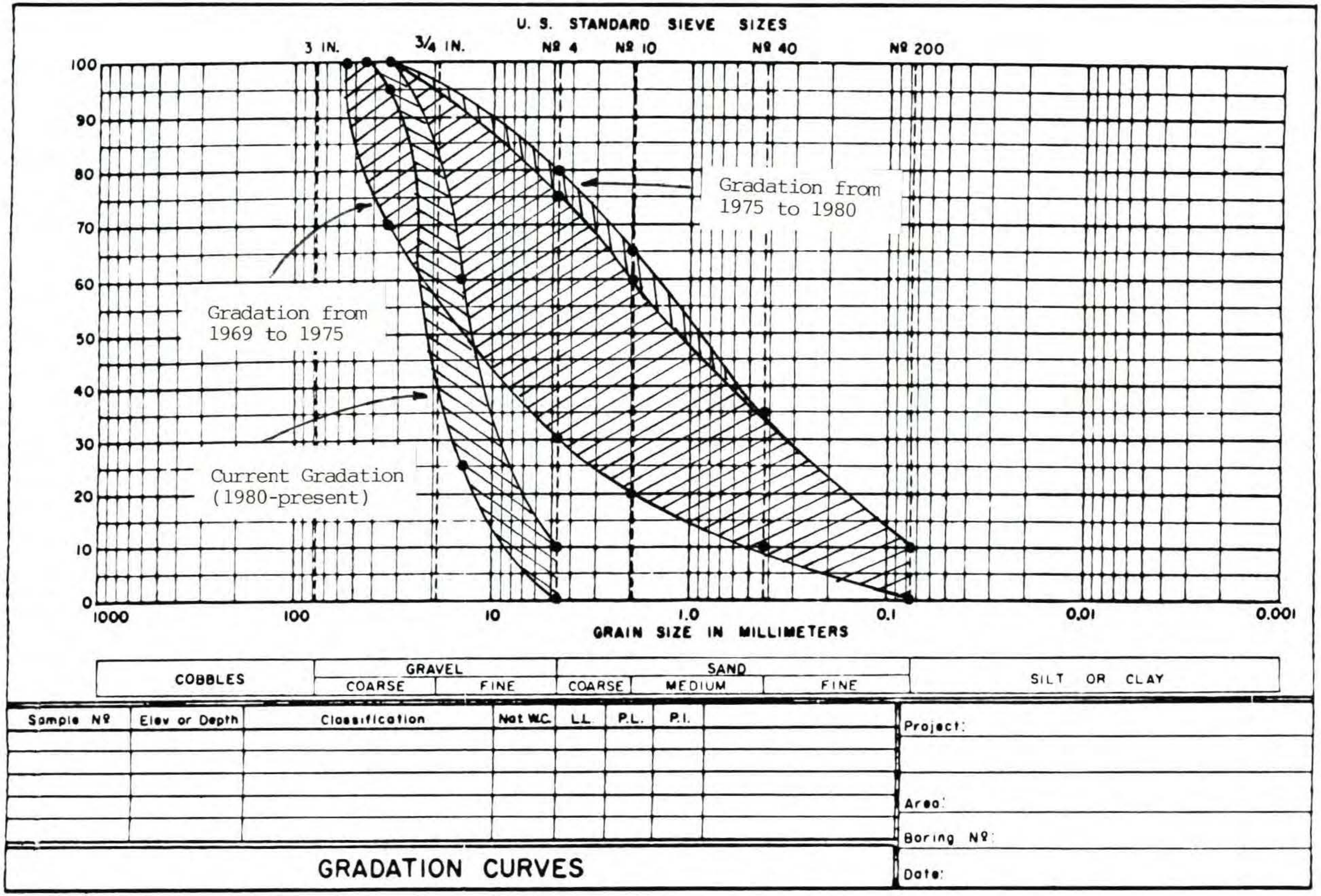


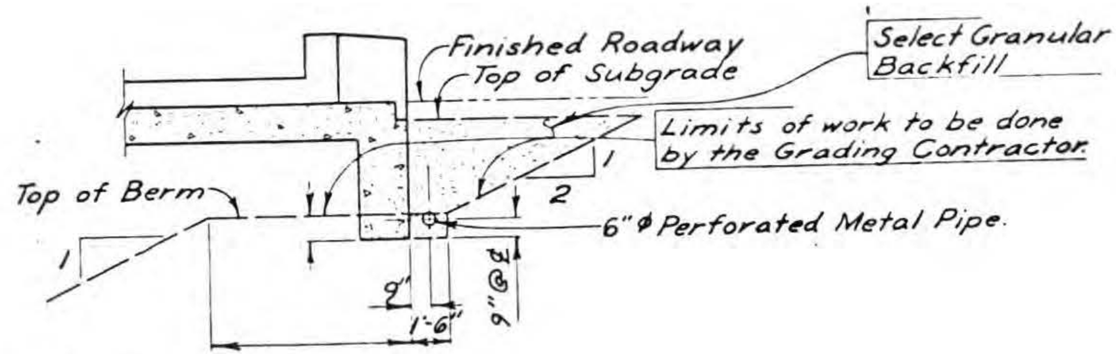

Figure 4-5. Gradation curves for select backfill used by SDDOT from 1969 to present.

changed to form a wedge that extended the length of the approach system. Typical cross-sections for these time periods are shown in Figure 4-6. The slope would then vary from 6H:1V to 10H:1V depending on the approach system. The three-inch gravel cushion was also removed from the plans in 1982. The filter wrap used in current design was added to the plans in 1988. This was incorporated into the design due to recommendations of field personnel from the Mitchell Region. The maintenance personnel were experiencing washout of backfill material and requested the wrap to prevent the washout. The intent was to use this wrap for select bridges, but has been adopted for use on all bridges with bridge end backfill. The fabric specifications are provided in the 1990 standard specifications.

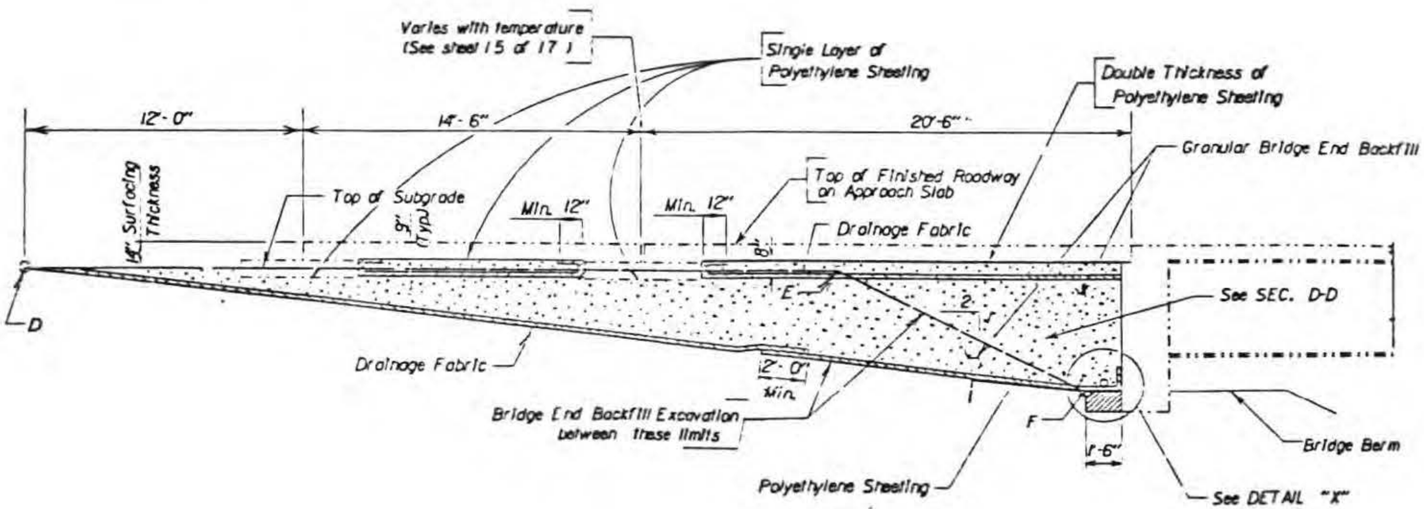
### 4.3.3 Discussion

Since 1970, the backfill used in South Dakota has become more coarse-grained. Currently, the SDDOT uses a backfill with a gradation that uses the #4 sieve as the smallest sieve for control. The survey of other DOTs showed prevalent use of granular backfill materials, but the gradations were more similar to SDDOT's 1970 and 1975 standards than the 1985 standard. Most other states specified the #200 as the control sieve with 5 to 15 percent passing. South Dakota uses the coarsest gradation of any state surveyed.

The bridge end backfill material is used for its material strength and resistance to freeze-thaw cycles. Since the material has a coarse gradation it allows for rapid drainage. Tests performed during this study showed the material to have only 0.2% absorption, which enhances its freeze-thaw resistance. Another feature of the material is its elastic potential. A granular material without fines is more elastic in nature than a material with fines. Such a material should exhibit less permanent deformation due to cyclic loading. This makes it more desirable with the integral abutment bridge. Since the bridge cycles against the backfill, an elastic material would reduce permanent deformation and thus settlement. However, at high stress levels particle breakage may become a factor causing densification.




a. System used 1970 to 1982.




b. System used 1982 to present. Slope varies from 6:1 to 10:1. Fabric wrap added in 1988.

Figure 4-6. Typical cross-section of approach systems with select backfill used by SDDOT.

The SDDOT also incorporated the use of a fabric wrap into their design. The fabric wrap encircles the backfill and prevents washout of the material. The fabric also prevents the backfill material from integrating itself into the surrounding embankment. Several other states have started to use fabrics in their backfill, primarily as a wrap and drainage fabric like the SDDOT.

The SDDOT estimates the cost of the bridge end backfill system at \$18.00/yd$^3$ with the fabric wrap and perforated pipe. The cost of the backfill FOB Brookings was quoted at \$14.22/yd$^3$. The fabric wrap FOB Brookings, S.D. was quoted at \$0.90/yd$^2$. Since the backfill costs depend significantly on quantities ordered and destination, the prices quoted FOB Brookings, S.D. are approximates for comparison.

A comparison to a granular backfill similar to the 1970 and 1975 standards was provided from a local Brookings, S.D. contractor. The price was quoted as \$5.04/yd$^3$ FOB Brookings, SD. This price quote is somewhat arbitrary as exact gradation requirements were not given. The quote was obtained only for general comparison. The cost of the 1990 bridge end backfill gradation is significantly larger than the previous specifications. The special gradation costs over twice as much as a granular material from a local pit site.

### 4.4 Approach Maintenance Costs

The SDDOT provided cost estimates for the various methods of repair on approach systems. The most common repair on SDDOT bridges is mudjacking the voids beneath the approach slab. Asphalt concrete overlays and wedges are other methods of repair on approach systems. Replacement of the approach system is the least common repair and is also the most undesirable. The three repairs are summarized individually below.

Mudjacking or slab jacking is a procedure used to fill the void beneath the approach slab. In some cases, mudjacking is used to lift the approach slab to its original elevation. Since the SDDOT uses reinforced concrete approach slabs, the procedure is usually a matter of filling voids and not lifting the slab. The procedure is done to stabilize the approach and prevent the slab from possible cracking due to support loss. The procedure involves drilling holes through the approach slab and pumping a cement/flyash slurry into the void space below. Care has to be taken to ensure that the slurry does not flow between the sleeper slab and approach slab. This could cause the approach to lift away from the sleeper resulting in a differential at the strip seal.

The costs for mudjacking approach slabs is variable, dependent on site conditions and location. Costs will vary according to structure location, roadway width, and traffic count. The Mitchell area bridge maintenance department provided actual costs for a mudjacking project in Sioux Falls, SD. The Mitchell area bridge maintenance group has their own mudjacking equipment and costs are based on actual field costs. This project included filling the void and raising the sleeper slab on the south approach slab on Structure #50-175-219, the southbound bridge on I29 between 12th street and 41st street crossing Skunk Creek. This is a high traffic area, therefore, costs for traffic control would be higher than a lower traffic bridge. The costs were broken down into employee, equipment, and material costs for the project. The project was estimated using the surface area square yardage in calculating costs per square yard. A square yard calculation can be more easily calculated since the slurry fills interparticle void spaces as well as the approach void.

Five employees were required for approximately 18 hours at a cost of $4.32/yd^2$. The employees were used for diversion of traffic, equipment mobilization, drilling holes, and actual mudjacking. The equipment costs were $3.67/yd^2$ for sign vehicles, drilling rig, personnel vehicles, and mudjacking equipment. The material costs were for the flyash and cement for the mudjacking and totaled $2.86/yd^2$. The total cost of the mudjacking was $1822.24 for 168 square yards or $10.85/yd^2$. This included costs for lifting the sleeper slab approximately 1-1/2 inches. These costs provide a good estimate for the costs to repair one side of a high traffic bridge.

Mudjacking is a common repair when there is a void present beneath the approach slab. The mudjacking process can also be used to raise the approach slab and sleeper to their original elevation. Care must be taken when mudjacking the approach slab. Several problems have been encountered by the SDDOT with mudjacking procedures. These are discussed below. As presented previously, care must be taken that the mudjacking does not lift the approach slab off the sleeper slab. This problem occurred on structure #50-175-219, the southbound bridge on I29 between 12th street and crossing Skunk Creek. The bridge was open for traffic in 1990 and experienced void development immediately. In the fall of 1991 the SDDOT bored through the approach to fill the void. There was no attempt to raise the approach slab, therefore little pressure was applied. The slurry flowed between the approach slab and sleeper slab causing a differential of approximately 1-1/2 inches on the south east corner. Ten feet from the edge the differential tapered down to zero. The bridge was allowed to sit over the winter. In the spring of 1992 the mudjacking material was chipped out from between the approach and sleeper, then the sleeper was raised to its original position. A great deal of time and effort by SDDOT personnel was required to remedy this situation.

Another problem that occurs more frequently with mudjacking is cracking of the approach slab. The SDDOT has experienced this problem on several bridge sites. During the Rapid City Region inspections, structures #24-162-058 and 52-256-401 had been mudjacked to fill voids and approach slab cracking occurred as a result of the mudjacking operations. Discussions with maintenance personnel indicated that this was a common problem that occurs when too much pressure is applied during mudjacking.

Asphalt overlays or wedges are used to repair approaches where there is a significant differential between approach and bridge. This repair is also used to smooth the transition at the approach/roadway interface. The bump at the end of the approach slab usually results from settlement of the sleeper slab. The SDDOT estimated the costs of a two-inch asphalt concrete overlay to be approximately \$7.25/ $yd^2$. Costs were based on an average density of 140 pcf for asphalt at \$40.00/ton. The common approach system is 40 feet wide by 20 feet long. Using an asphalt overlay to cover the approach and extend onto the roadway, the total yardage would be approximately 60 $yd^2$. This would translate into \$435.00 for materials for each approach slab. Equipment and employee costs were not provided by the SDDOT, but an approximate calculation would be half the cost of mudjacking an approach. Using the information provided by the Mitchell DOT for the bridge crossing Skunk Creek, an approximation for the employee costs would be \$2.16/$yd^2$ and equipment costs would be \$1.83/$yd^2$. Total cost for asphalt concrete overlay would then be \$670.00 per approach slab.

Asphalt overlays are the least expensive repair, but can only be used when there is settlement of the approach slab or sleeper. Many times the void is present beneath the approach slab with no differential settlement. The asphalt overlays would be ineffective in these situations. The asphalt overlays also have a limited life span which decreases their cost effectiveness.

Replacement costs for an approach slab are the highest of the three repairs. The SDDOT provided approximate costs for approach slab replacement (Dan Strand, personal communication, 1992). The estimate provided includes replacement of the backfill, new edge drains, reinforced concrete approach slabs, and installation costs. The Office of Materials and Surfacing provided an estimate of \$12,000.00 per approach slab for replacement costs. The concrete is estimated at \$65.00/$yd^2$ with reinforcement. The backfill with fabric is estimated at \$18.00/$yd^3$. The edge drains are estimated at \$8.00 per linear foot. A typical approach slab is 40 feet wide by 20 feet in length. This is about 37 $yd^2$ of reinforced concrete. The backfill encompasses the entire approach system with a volume of approximately 53 $yd^3$. The drains extend the length of the approach slab for a total length of 40 line feet. The materials cost for the approach slab totals \$3679.00. The balance of the costs is for equipment and installation.

The costs related to mudjacking, asphalt overlays, and replacement are \$1800.00, \$670.00, and \$12,000.00 per approach, respectively. The first two alternatives also have special considerations associated with them that must be accounted for in determining their use. The replacement of the approach slab is the least desirable alternative since the cost is prohibitively high. All three of these repairs require the closing of the structure or a lane of the structure during repair. The traffic control and safety considerations associated with this closure must be considered.

An alternative that must be considered is maintaining the status quo. If no maintenance is done on the bridge the effects of the void have to be considered. Since the approach slab is designed to span approximately 15 clear feet, a void within 10 feet of the abutment should not affect the performance of the approach slab. As cited in the bridge surveys, many of the approach slabs are cracking transversely approximately two feet from the abutment. Since these structures all had voids in the area adjacent to the structure, it can be concluded that the approach cracks because of the loss of support. A key question becomes: At what point does this cracking affect the structural integrity of the approach slab? If the cracking does not affect the structural integrity of the approach slab, the system should function satisfactorily, with only crack sealing needed. If the cracks continue to propagate because of the lack of support, then filling of the void space or replacement of the approach slab will become necessary. If the void is not stabilized, the approach will eventually have to be replaced.

# CHAPTER 5
# MODEL STUDY

## 5.1 Introduction

The purpose of the model study was to simulate the effects on bridge end backfill resulting from cyclic movements of the bridge abutment. There were several purposes for using a model to simulate these effects. The primary purpose was to control the conditions of soil type, construction sequence, construction compaction, and structure behavior. By controlling these factors it was possible to move the bridge abutment against the backfill and produce repeatable results that will parallel field thermal conditions. The ease of instrumenting a model was another key reason for a model study. The costs involved in instrumenting an existing structure in the field would far exceed the costs involved in instrumenting a model. Model development cost and time were also a concern, but were reduced in this instance because the primary components, the bridge and the abutment, were available from a previous SDDOT study and exist in close proximity to SDSU. In addition, the work involved proceeded with minimal interruption to highway traffic flow as would occur if an existing or new structure were used.

The primary goal of the model study was to compare the effects of temperature-induced expansion and contraction of the bridge/abutment system to those observed in the field. A jacking system to move the bridge against the backfill simulated thermal movement of the bridge. A series of tests were then run to simulate long-term movements and repeated cyclic movements. Access holes in the approach slab allowed the settlement of the backfill to be measured with precision and ease. Inclinometers in the backfill area provided determination of lateral and/or longitudinal expansion. Load cells were used on the abutment to determine earth pressures and compaction effects due to long-term cyclic movements.

The structure was subjected to a series of different movement patterns. Instrumentation readings were recorded for single-stage movement, short-term cyclic movements, and long-term cyclic movements. Single-stage movements consisted of movements against the backfill with inclinometer and pressure recordings at various positions of movement. Short-term cyclic movements consisted of moving the abutment against and away from the backfill for a specified period and recording the results. Long-term cyclic movements paralleled short-term movements over an increased length of time.

**5.2 Description of the Site and Existing Structure**

The structure for the model study was built for a 1973 SDDOT research project titled, "Analysis of Integral Abutment Bridges" (Lee and Sarsam 1973). This research project focused primarily on the effects of thermal expansion and contraction on the abutment and bridge girders. Soil borings at the site showed a six-inch layer of black silty sand overlying a six-inch layer of gravel. From one to nine feet a light brown sandy clay was encountered. The next five feet consisted of a light brown silty clay. A six-inch layer of sand was encountered at ten feet. Ground water was at a depth of six feet. Previous borings at this site showed a stiff, light brown silty clay to depths of 31 feet overlying a stiff mottled blue-brown silty clay to depths of about 55 feet (Bump 1970). The site soils are glacial till materials with the upper zones being weathered till.

The model bridge is located at the Brookings Area DOT office, just east of Brookings. The structure consists of a partial bridge section, jacking abutment, and integral abutment approximately 19 feet wide and 30 feet in length. The abutment is oriented east-west and is scaled down to approximately one-half width, but is full depth and pile supported. The abutment has a 12-foot approach sill and 3 1/2-foot wing walls. A schematic of the model is shown in Figure 5-1. The integral abutment is supported by two HP10 x 42 steel bearing piles placed 8'-6" on center and driven to a depth of 32 feet. The safe bearing load was calculated to be 32 tons per pile (Bump 1970). The jacking abutment is supported by 12 vertical and 4 battered timber piles. The bridge girders were 26'6" long and had a 12" x 1/2" top flange, a 38" x 5/16" web plate and a 12" x 3/4" bottom flange. A 5" x 1" x 1'5" plate was placed between the top of each piling and bottom girder flange. Additional details concerning the model bridge structure can be found in Sarsam (1972). For this project the bridge section was modified at the jacking abutment to accommodate larger jacking equipment. Otherwise no major modifications were necessary.

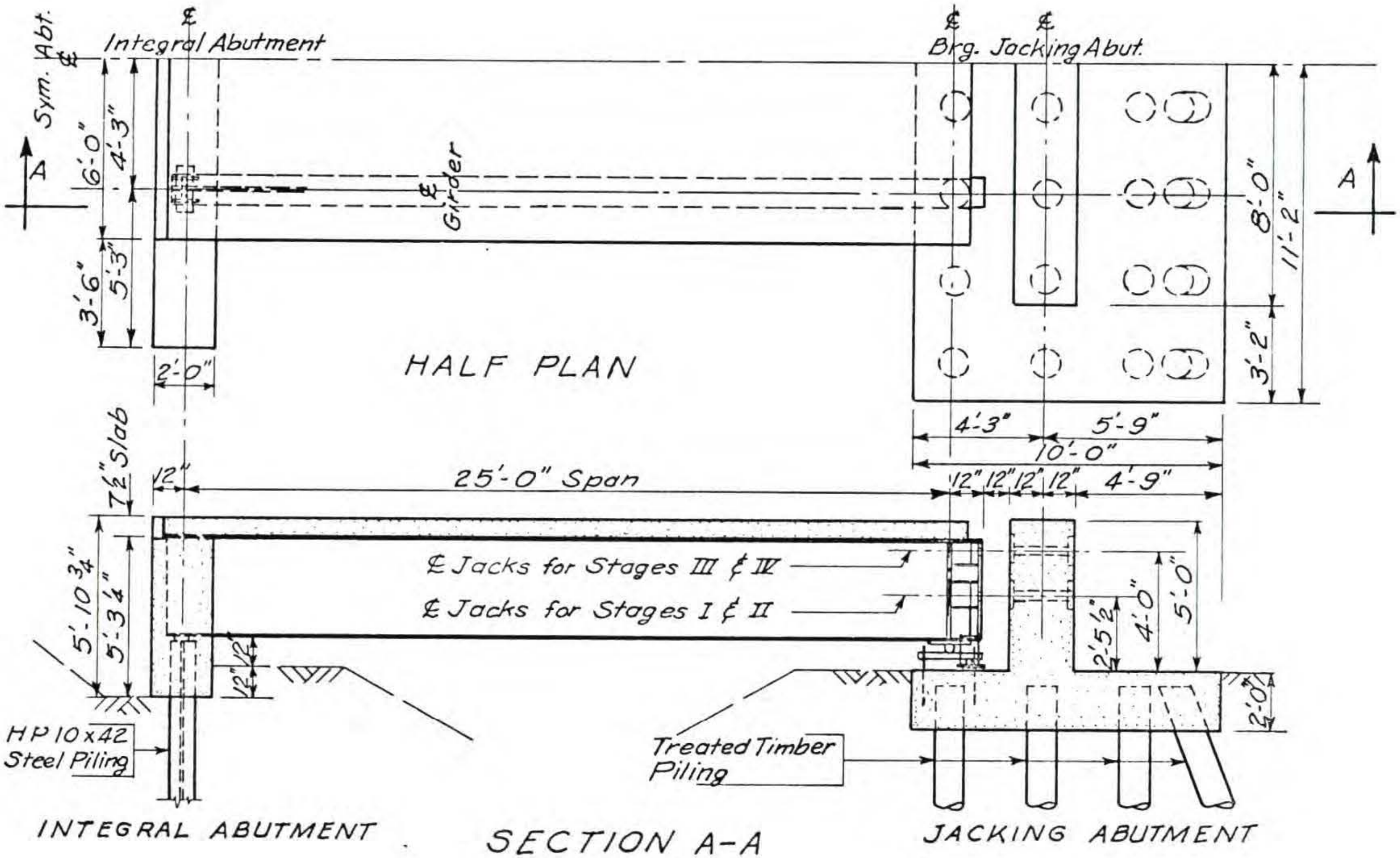



Figure 5.1 Schematic of model bridge and abutment located at Brookings Area DOT office (Sarsam 1972).

## 5.3 Description of the Model and Instrumentation

### 5.3.1 General

Initially a smaller laboratory scale model was considered, but the scaling factors involved made such a study impractical. The use of the existing structure provided scalable model/field parameters not possible at a laboratory scale. The availability of this structure was ideal for this model study as essentially a nearly full-scale approach system could be constructed. The approach system described below is full depth and full length and at 12 feet in width is about one-half of full width. It is believed that the half-scale width of the model does not appreciably affect the model tests as a half-width can be considered a representative slice of a full-width structure.

### 5.3.2 Approach Details

The approach system was designed to emulate existing DOT practices discussed in Chapter 4. A plan view of the approach system is shown in Figure 5-2. The approach slab was 12 feet wide, 20 feet long and 6 inches thick. The side shoulders of the fill were two feet wide with 2H:1V side slopes and the fill was continued out beyond the sleeper slab at approximately a 1H:1V slope. The sleeper slab was designed to be 5 feet long and 12 feet wide. The sectional views shown in Figure 5-3 represent current SDDOT design details for approach systems. These details were the basis for design of the model approach system. The fill embankment was designed to be constructed initially to a 2H:1V slope and then cut back to a 6H:1V slope in accordance with standard DOT practice (see longitudinal section in Figure 5-3). From the sections in Figure 5-3 it can be seen that the drainage fabric is wrapped entirely around the bridge end backfill. Two-foot minimum overlaps are required for the fabric. An eight-inch layer of backfill is placed on top of the fabric and overlain with a polyethylene sheet prior to placement of the approach slab. Standard DOT practice calls for a nine-inch reinforced concrete slab for the approach slab. For the model, a six-inch reinforced concrete slab was constructed to aid in handling. Details of the reinforcement can be found in Section 5.4.1.

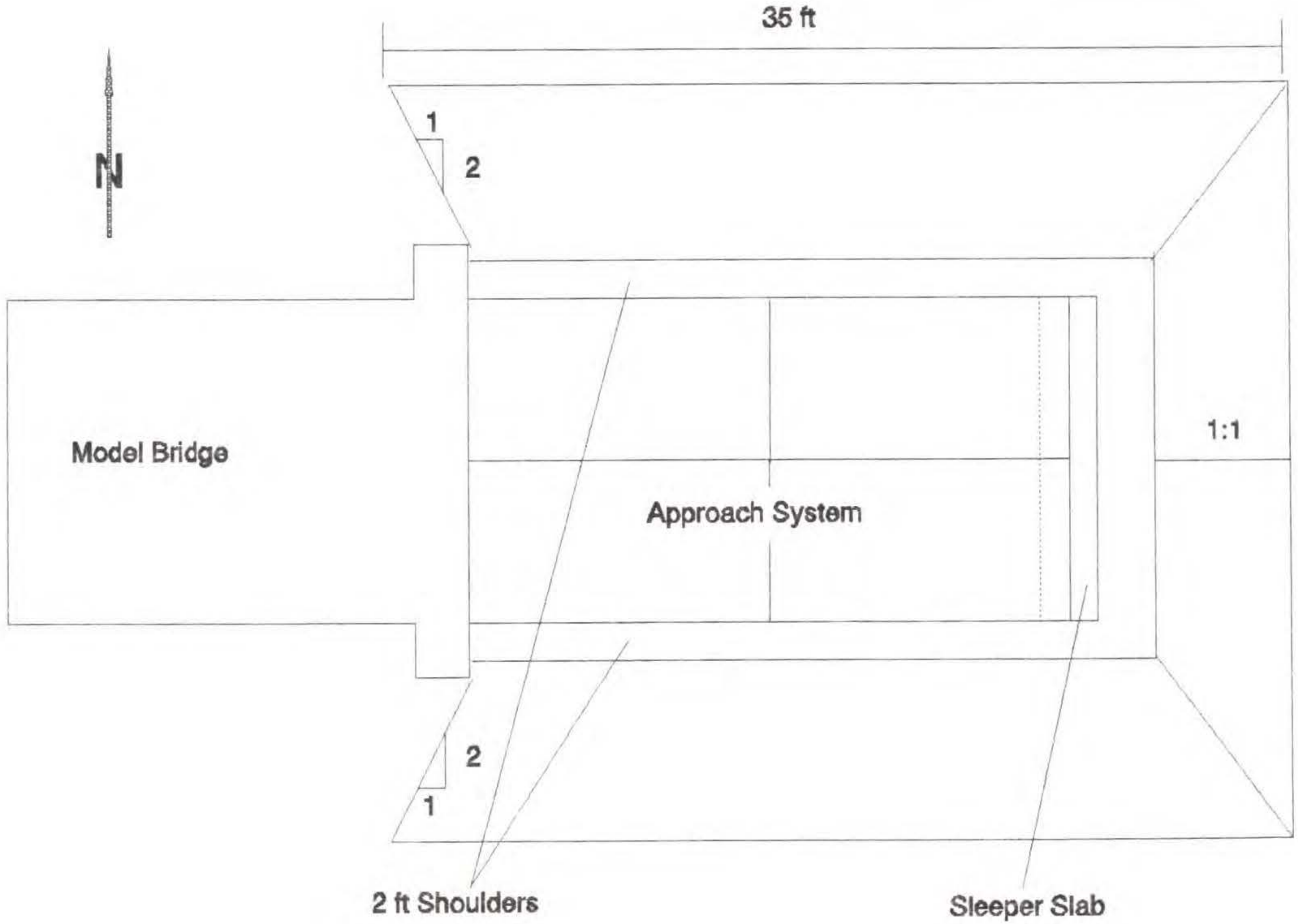
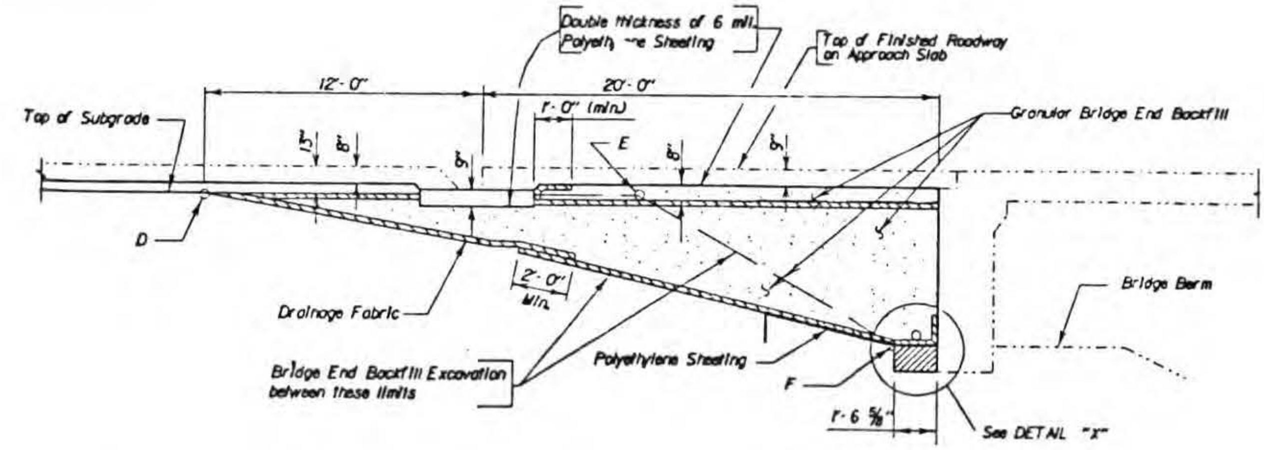
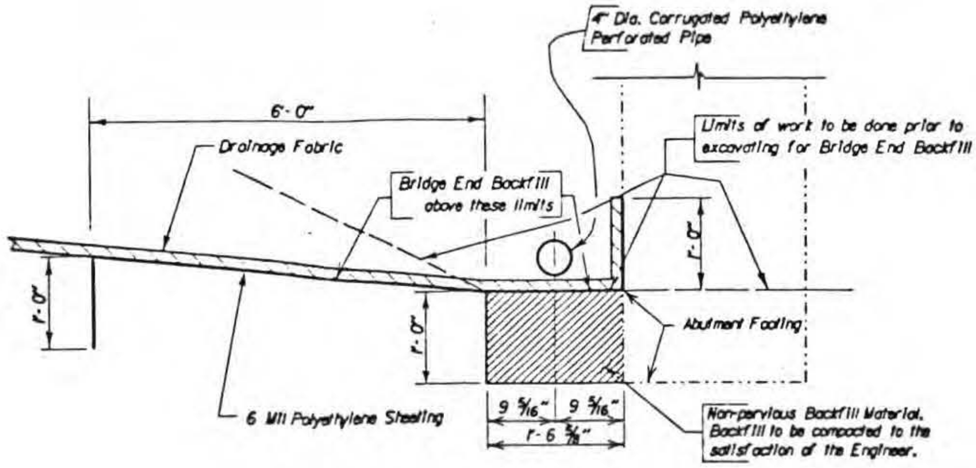


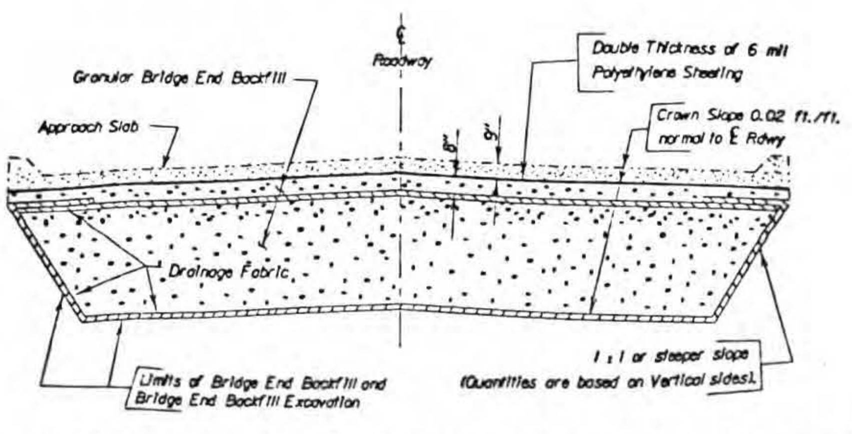

Figure 5-2. Plan view of model system.




a. Longitudinal section through approach system.




b. Detail "X" of longitudinal section of approach system.




c. Transverse section through approach system.

Figure 5-3. Sectional views of current SD DOT approach system design as used in model study.

### 5.3.3 Instrumentation

Several different methods of monitoring the backfill and the embankment were used. The backfill area was instrumented with inclinometers and pressure cells to record activity in this area. Six holes were bored through the approach slab along a line one foot from the abutment to measure void space development. Stakes were driven in the embankment fill along the approach slab and monitoring locations were set on the approach slab to enable approach slab and embankment movements to be measured with level surveys. Figure 5-4 shows a plan view of the locations of the measuring points. Dial gages were positioned in front of the abutment to monitor movement and rotation of the abutment.

Two inclinometer tubes were installed five feet behind the abutment wall and aligned longitudinally with the bridge beams. The inclinometer used was a Series 200-B instrument made by Slope Indicator Company and was supplied by the SDDOT. The instrument consists of an aluminum cylinder 2.375 inches in diameter and 15 inches in length. The sensitivity of the instrument is such to record as little as three minutes of arc. The instrument is read by inserting the probe into the inclinometer tubes with the fixed wheels pointed in the direction of recorded movement. To read in all four directions the probe must be inserted and read four times with the fixed wheels facing the direction of movement. The inclinometer is connected to the readout by a cable with one-foot reference marks on it. The readout consists of a potentiometer dial indicator that provides a reading when the circuit in the inclinometer is in balance. The inclinometer readings made in the field can be analyzed to obtain lateral deflections in the four principal directions (Slope Indicator Co. 1984).

The pressure cells used were Slope Indicator Company Model 51482 Total Pressure Cells with model 514178 Transducer. These cells are a flat pressure cell nine inches in diameter and 0.43 inches thick constructed stainless steel. The total pressure cell is filled with fluid, and soil pressure on the flat walls of the cell is converted to fluid pressure and measured by a piezometer. The standard pressure range of this cell is 1 to 300 psi. A

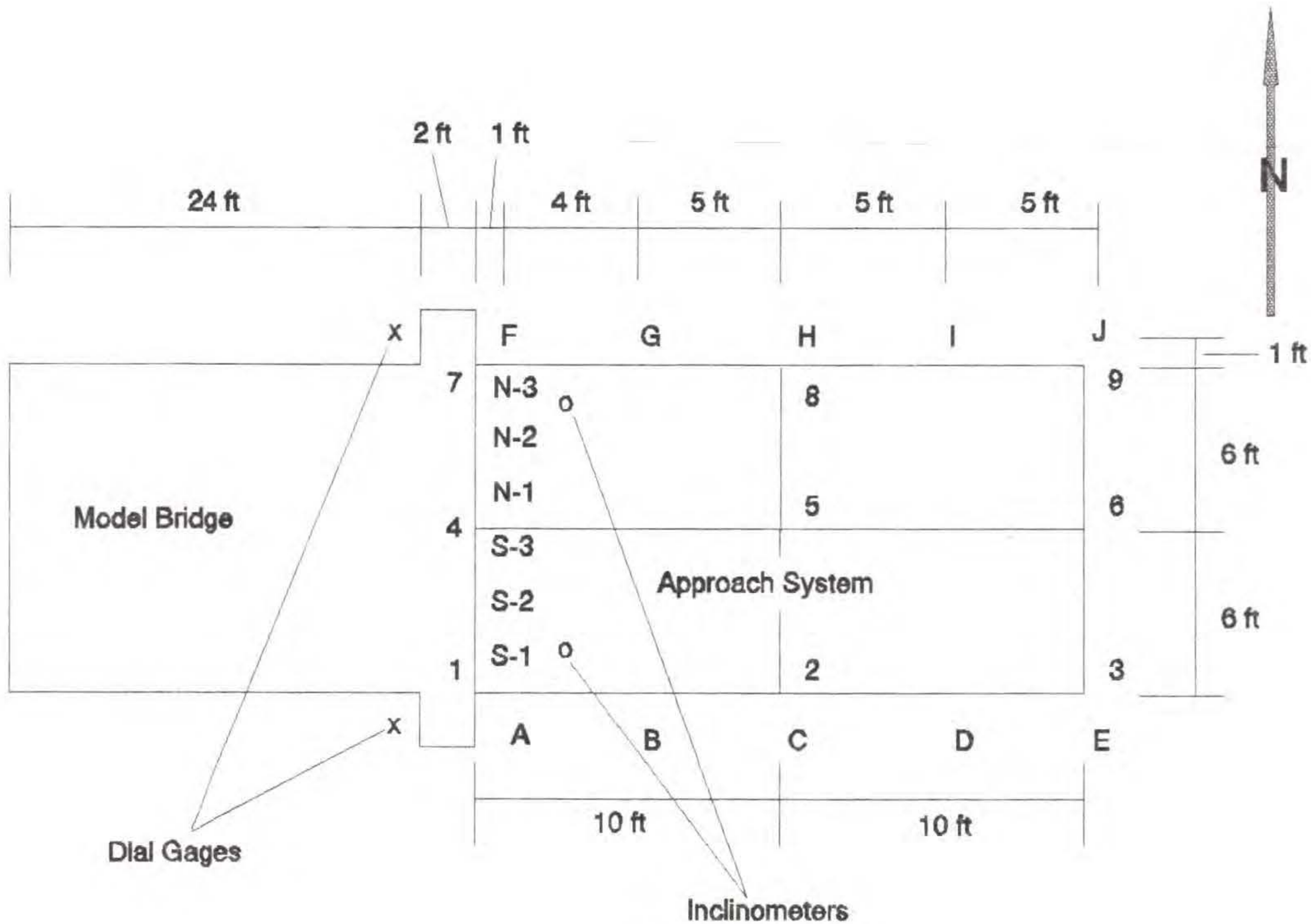



Figure 5-4. Plan view of approach system showing locations of instrumentation and measuring points. A, B, C, etc. represent embankment stake locations. S-1, N-1, etc. represent core hole locations. 1, 2, 3, etc. represent elevation points on the approach slab.

pneumatic pressure piezometer was used to measure the fluid pressure. The cell pressure was read with a Slope Indicator Model 211 pneumatic indicator with a 0.15% digital readout gauge rented from Slope Indicator Company. The pressure cells were mounted along the center of the abutment wall at depths of two and four feet from the top of the abutment. Mounting of the cells to the abutment wall is discussed in Section 5.4.1.

### 5.3.4 Description of Embankment and Backfill Soils

Several local soils were investigated for use as potential embankment materials. A cohesive material was sought for the embankment material to correspond to the soils generally encountered in the field work. The soil used for the embankment was obtained from a local Brookings site and classified as a clayey sand (SC) in the Unified Soil Classification System and an A-6 soil in the AASHTO Classification System. The soil had a liquid limit of 34.6, a plastic limit of 21.1, a plasticity index of 13.5, and a specific gravity of 2.66. The natural moisture content at the time of testing the soil was 14.0 percent. A grain-size analysis of the soil showed that the soil consisted of 10.5% gravel, 41.3% sand, 32.1% silt, and 16.1% clay. The moisture-density relationship of the material was determined in accordance with AASHTO T99, Method A, from which it was determined that the soil had a maximum dry density of 114.8 pcf at an optimum moisture of 14.3 percent. The embankment for the model required approximately 400 cubic yards of loose fill.

The special backfill material was specified in accordance with current DOT practice as shown in Figure 4-7. Approximately 25 cubic yards of compacted bridge end backfill material was needed to construct the approach system. The backfill material was obtained from the L.G. Everist quarry in Dell Rapids and was a Sioux Quartzite material.

The drainage fabric for encapsulating the special backfill was ordered to meet the specifications of Section 831 - Drainage Fabric of the 1990 Standard Specifications for Roads and Bridges (SDDOT 1990). Approximately 50 square yards of the drainage fabric was needed.

## 5.4 Construction Sequence

### 5.4.1 Fall 1991

Approval to perform the model tests was provided in September 1991 (D. Strand, written communication, 1991). Work on the project began immediately. The first step was the solicitation of material bids. Due to State procurement rules at least three bids were required on most of the materials. Shortly after the bids were awarded construction of the embankment was started.

The embankment material was compacted to 97% or better of its maximum density at natural moisture content as required for bridge embankments by Section 120 of the Standard Specifications for Roads and Bridges (SDDOT 1990). The material was placed in lifts of six to eight inches and compacted with the tires of the loader. This method was used since the size and location of the embankment did not allow use of normal compaction equipment (sheepsfoot roller). With the bucket of the loader filled with soil, the lift of soil was compacted by successive passes of the wheels with the loader. Soil density was tested by the balloon method. If the density did not meet the specifications, additional passes of the loader were made to raise the density to at least 97% of maximum density. Initial density tests showed that compaction requirements were met with three passes of the loader. The embankment was built up to grade with a 2H:1V slope from the bottom of the abutment to the top of grade. After this was accomplished, the slope for the backfill was excavated. This was approximately at a 6H:1V grade from the abutment base to a point 30 feet back. The embankment construction required one week to complete.

The next step involved the installation of monitoring equipment. Before the pressure cells arrived, the weather took a turn for the worse. A snow storm the end of October stopped construction of the embankment. Subsequent below freezing temperatures prevented further earth work at this time and efforts were directed to making the modular approach slab.

The approach slab was designed to use C6x8.2 channel iron to form four 6-foot by 10-foot boxes. The four boxes could then be assembled at the site and filled with concrete to form a rigid 12-foot by 20-foot approach slab. The boxes were attached to one another by 3/4-inch bolts through holes punched in the web of the channels. The channels were fabricated on campus and the pieces taken to the site for assembly. The approach slabs formed by the channel were doubly reinforced with the use of #6 and #4 rebar. The #6 rebar was used for structural support for spanning the ten feet between boxes. The #4 rebar was used in the top for flexural support in lifting the slab during removal. The rebar spacings were at 12-inch centers for both the upper and lower rebar.

In mid-November the SDDOT Office of Materials and Surfacing sent a drill crew to Brookings to install the inclinometer tubes. The tubes were provided by the SDDOT, as was the inclinometer. The tubes were three-inch aluminum tubes in ten-foot sections. Two ten-foot sections were connected together to provide a sealed twenty-foot tube. Each tube has slots oriented at 90° to each other. The inclinometer has guide wheels that ride in the slots to measure deformation in the slot direction. To install the north inclinometer it was necessary to level part of the south embankment and fill part of the cut made for the backfill. This allowed the drill rig to back in parallel to the abutment to position the north inclinometer. The north inclinometer tube was inserted into the boring and grouted into position with a cement-flyash slurry. The tube was aligned in the hole with the slots parallel and perpendicular to the abutment. After the north inclinometer was set, the cut for the approach fill was cleaned and graded with the loader.

The south inclinometer was positioned by backing in parallel to the abutment on the leveled area of the south embankment. The south inclinometer tube was inserted into the boring and grouted into position with a cement-flyash slurry. The tube was aligned in the hole with the slots parallel and perpendicular to the abutment. After setting the south inclinometer, the truck was driven forward ten feet. The crew drilled into the embankment and obtained soil samples for testing. The drill rig pushed three-inch Shelby tubes at depths of one, five, and ten feet. The rig was unable to push a sample at 15 feet. The test hole and the north inclinometer hole both had water levels at a depth of five feet after drilling. The south inclinometer hole was dry for its entire depth, indicating the variability of the soil strata at the site.

Both inclinometer tubes were located approximately five feet from the abutment face, and set to a depth of twenty feet below the top of the abutment. The tubes were aligned longitudinally with the bridge beams, with a distance of eight and one-half feet between the tubes. This allowed approximately two feet between the tube and embankment on either side.

While the drill crew set the inclinometer tubes, the bridge beams were modified to accommodate the jacks for pushing the abutment. A section of the bridge beam was cut out and iron was welded in a U-shape for the jack to sit in. The jacks were placed in position to determine if further modification was necessary. Since the jacks could be positioned correctly, they were wrapped in plastic until needed. This was the only modification to the bridge structure required. The jacks used on the main bridge beams each have a 200-ton capacity with a 10-inch stroke. Sweetman Construction Company of Sioux Falls provided the jacks and the hydraulic system for operating the jacks.

The pressure cells were received November 1, 1991 from Slope Indicator Company of Seattle, Washington. The cells are described in Section 5.3.3. The cells were attached to the abutment using a configuration suggested by Patrick Smith of Slope Indicator Company (Smith, personal communication, 1991). Since the backfill material was a crushed rock, the cell had to be protected to prevent puncture of the cell by the jagged edges of the rock. The cells were glued to the abutment using a rubber cement with the sensitive side towards the concrete. This was done for two reasons: the first was to protect the cell as described above, and the second was to provide a uniform contact area on the cell itself. Since the rock would produce point contacts on the cell, the reading would be an average of those points. Such a reading would not be as accurate as that for a finer-grained soil with more contact points. The pressure cells were glued into position at depths of two and four feet from the top of the abutment. A silicone caulk was applied around the edges to prevent rock grains from wedging between the cell and abutment. Additional protection was provided to the cells by covering them with a scrap piece of conveyor belt. The polyethylene tubing was encased in PVC conduit to protect it from crushing. The total system was then sealed with silicone caulk to prevent rock and moisture from entering. The PVC casing was extended out the side of the embankment along the abutment face where the pressure readout was positioned.

### 5.4.2 Spring 1992

The weather allowed construction of the backfill area to continue in early April of 1992. L.G. Everist Inc. of Dell Rapids, S.D. donated 50 tons of aggregate used for the Section 850 select granular backfill. The fabric wrap had been donated and received in November 1991 from Contech Construction Products, Inc. of Minneapolis, MN. Construction began on April 3 by positioning the fabric wrap on the 6H:1V grade cut from the bottom of the abutment. The wrap was placed parallel with the abutment with the wrap coming up the sides of the cut. The wrap was placed with two-foot overlaps utilizing three strips to encompass the entire slope of the cut. The perforated PVC pipe was placed on the fabric in a position one foot from the abutment, running parallel to the abutment and out the north side in front of the wing wall. Backfill was placed around the pipe and up the slope to hold the fabric in place for subsequent layers. On April 4 the weather once again turned and snowed on the site.

Construction of the backfill began again on April 9. The backfill was placed in six-to-eight-inch lifts and compacted with a Ground Pounder vibratory packer. There is no specification for compaction of this material, therefore a system of passes with the vibratory compactor was used to provide uniform compaction. Three passes over the backfill with the ground pounder provided a firm surface with no noticeable improvement with further effort. It was noted that during compaction there was significant particle breakage in the backfill material. This would increase the density of the material by providing more fines to fill interparticle spaces.

The embankment was built up concurrent with each layer of backfill to provide a confining boundary to the backfill. The soil was placed using the same compactive effort as used in the initial embankment construction. Readings were taken on the earth pressure cells during construction to monitor stresses induced during construction. When the level of backfill reached a point nine inches below the abutment approach sill, the fabric was wrapped over the top of the backfill and a strip was laid over the top of the backfill. The final backfill layer was a nine-inch layer on top of the fabric and below the approach slab. The backfill and embankment were brought up to grade, except where the sleeper was to be poured. The sleeper slab was formed using a box made from dimensioned lumber to form a five-foot long by twelve-foot wide by nine-inch thick slab. A standard 3000 psi mix concrete was used for the sleeper slab and the approach slab. The sleeper slab was poured on April 15 and allowed to set overnight.

The sleeper slab forms were removed the next day and the area filled and compacted with backfill. The approach system was carried onto the graded backfill and the four sections assembled. A layer of polyethylene plastic was placed below the approach system to prevent concrete from flowing into the backfill. The approach slab was then poured and allowed to set over a weekend.

During the week of April 20 to 24, 1992, the embankment was brought up to the level of the approach slab and the embankment was shaped into the final configuration with 2 1/2-foot shoulders and 2H:1V side slopes. The beam for pulling the bridge to simulate thermal contraction was positioned on the west side of the jacking abutment and the tension rods were attached. The beam for pulling the bridge was designed using the Load Resistance Design Method. A 60- ton jack was used in the center of the beam with tension rods attached to the bridge through the abutment at each end. The beam was initially designed for the full 60-ton capacity, but the beam size became impractical. The beam was sized down to handle a 36 ton capacity jack. If the pressure exceeded the beam capacity during testing, provisions were made to add stiffeners and a tension rod on the back of the beam. Neither modification was necessary. The tension rods were designed to be 1-1/2 inch rods, but one-inch rods were substituted to use threaded rod on hand. Figure 5-5 shows the jacking system used to expand and contract the bridge. At this time the jacking systems were connected to hydraulic pumps and tested for movement action. Both systems appeared to work efficiently with no apparent problems.

a. Jacking system at west end of reaction abutment to pull abutment wall away from the approach fill.

b. Hydraulic jack placement on main bridge beams. Jacks used to push abutment into approach backfill.

Figure 5-5. Jacking systems used in the model.

The supports for the dial indicators were positioned and concreted into the ground. The dial indicators were positioned on the abutment to provide movement and rotation of the abutment. The dial gages were positioned on both sides of the bridge with the top dial gauge 6 inches below the bridge deck and the bottom dial gauge 33 inches below the top gauge. The stakes were driven in the embankment fill at locations shown in Figure 5-4. The six holes bored through the approach slab were made after the first cycle of testing was performed.

### 5.5 Temperature-Induced Cyclic Movements

To determine the amount of movement to induce in the model, a review of temperature variations around the state was made. Temperature data, including daily and monthly maximum, minimum, and average temperatures and monthly maximum daily temperature differences, were obtained from National Oceanic and Atmospheric Administration publications for several locations throughout South Dakota for the period of July 1989 to July 1991. An example of the type of data obtained is shown in Figure 5-6 which shows the range in maximum, minimum and normal temperatures for Brookings, SD during this period. This time period was selected because it is coincident with the study and several bridge approach systems constructed during this period have developed problems. Examples of these structures are #50-175/176-219 on I-29 crossing Skunk Creek in Sioux Falls, SD; #18-180-100 on SD 38 crossing the James River east of Mitchell, SD; and #50-119-165/166 on I-90 crossing SD38. All these bridges exhibited problems before they were open to traffic and the first two have had to be mudjacked.

A review of the temperature data from around the state showed that while large temperature variations occurred throughout the state at different times, the maximum variations in temperature experienced at different locations were very similar. Because the maximum variations around the state are similar and the model site is located in Brookings, the Brookings maximum temperature change data shown in Figure 5-6 were selected for use in computing the cyclic movements. These data were used to calculate movements corresponding to temperature changes with respect to normal, high and low

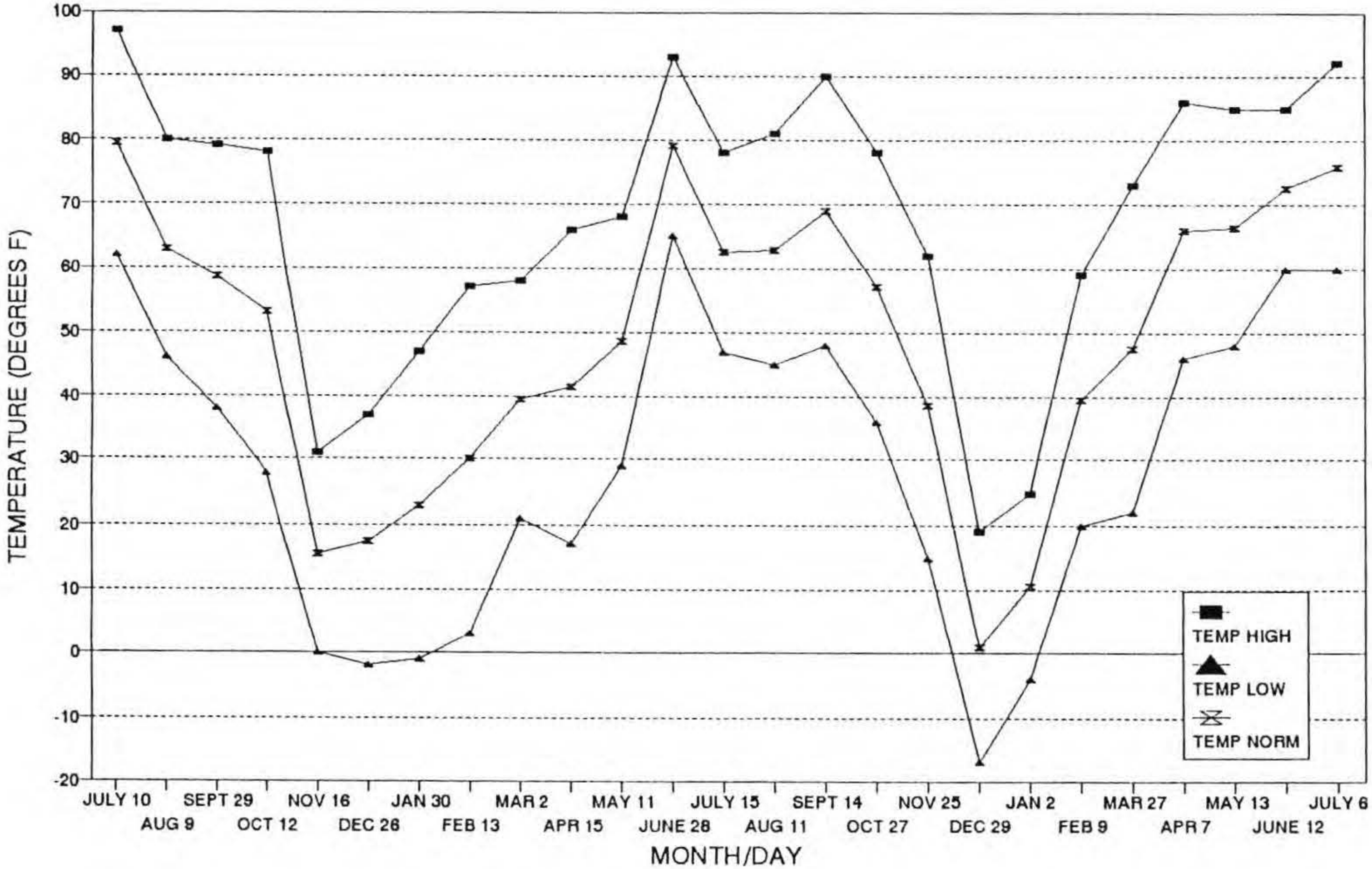



Figure 5-6. Range of maximum, minimum and normal monthly temperatures for Brookings, SD for period July 1989 to July 1991.

temperature. Standard coefficients of thermal expansion of 6.5 x $10^{-6}$ in/in/°F for steel and 6.0 x $10^{-6}$ in/in/°F for concrete were used. An average bridge length of 300 feet was assumed and movements at one abutment were calculated for one-half of this length. To calculate movements corresponding to temperature changes the movement had to be referenced to an initial starting point. Therefore, the initial movement cycle was initialized using the May normal temperature and this point was arbitrarily assigned as zero movement. The resulting pattern of movement is shown in Figure 5-7 for the 1989-1991 period. The movements shown in Figure 5-7 represent maximum monthly movements expected due to temperature changes occurring during that month.

In conducting the model test it was desired to conduct a one-year cycle of movements to simulate the effects of the four seasons. This was effected by determining 12 cycles of movement based on the data in Figure 5-7. In conducting the model test a one-day cycle of movement was developed for each month. Each cycle represents movement from the normal, to the high, to the low of a month, and then to the normal of the next month. The resulting one-year pattern of movement is shown in Figure 5-8. During the early stages of testing it was found that the contraction stage of the movement was effectively limited to -0.7 inches. Therefore the late fall movements, which in Figure 5-7 can be seen to be in the negative range, were adjusted upward using -0.7 as a lowerlimit of movement. Thus the movement cycles shown in Figure 5-8 have been adjusted upward, but the range of movement was kept the same.

### 5.6 Test Procedure

Movement of the abutment was controlled using hydraulic jacks placed on each bridge girder and a single jack on the opposite side of the jacking abutment (Figure 5-5). Control was established using a theodolite and initial movements were recorded with the dial gages. Plates were later placed on the bridge deck for movements to be recorded with the theodolite (Figure 5-9). Readings were taken of the inclinometers, stakes, voids, and approach slab level at each step of the cycle, while the dial indicators and pressure cells were read at each 0.10 inch of abutment movement. The jacks provided a constant pressure and movement was maintained at a rate of approximately 0.40 inches per hour.

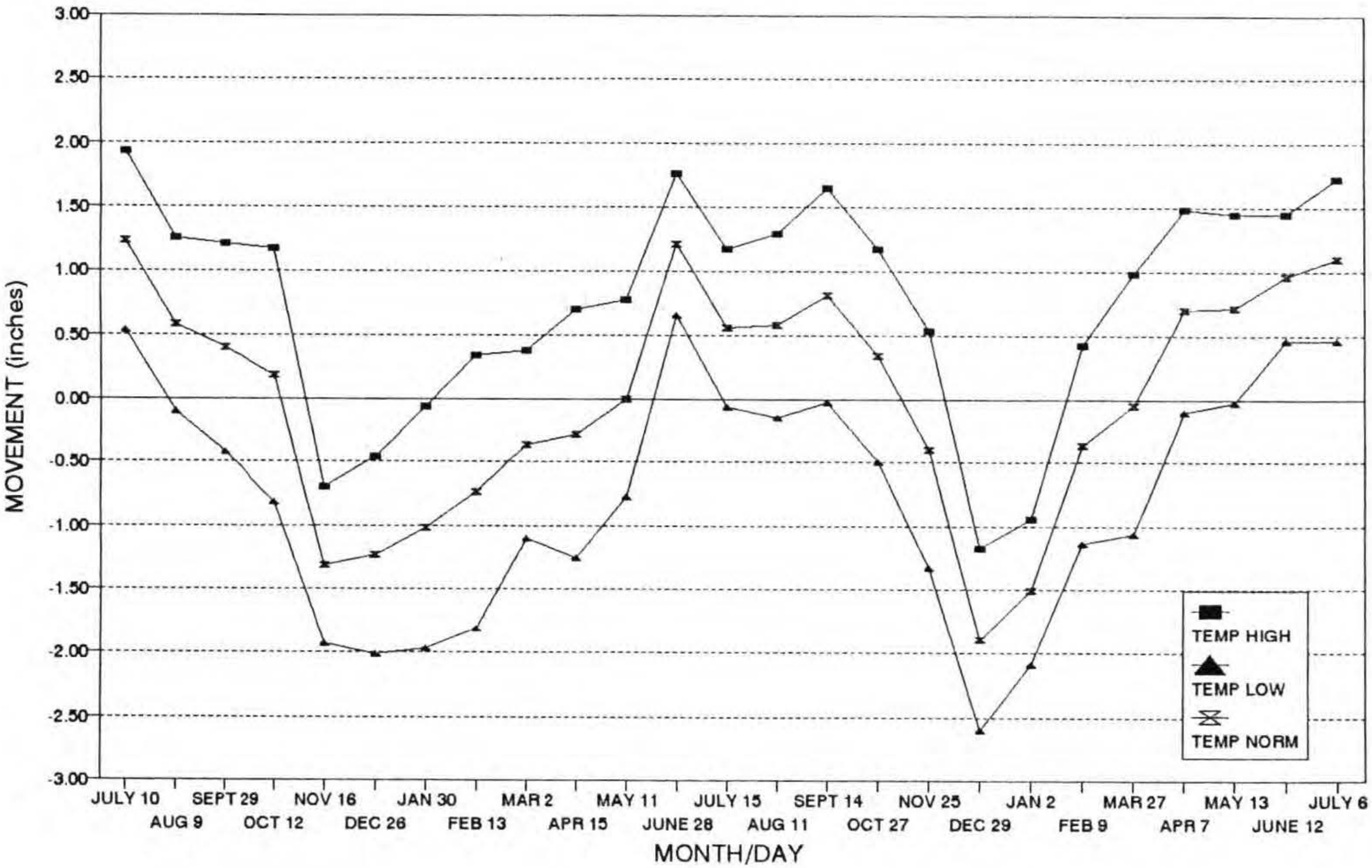



Figure 5-7. Pattern of abutment movement due to cyclic temperature variation shown in Figure 5-6.

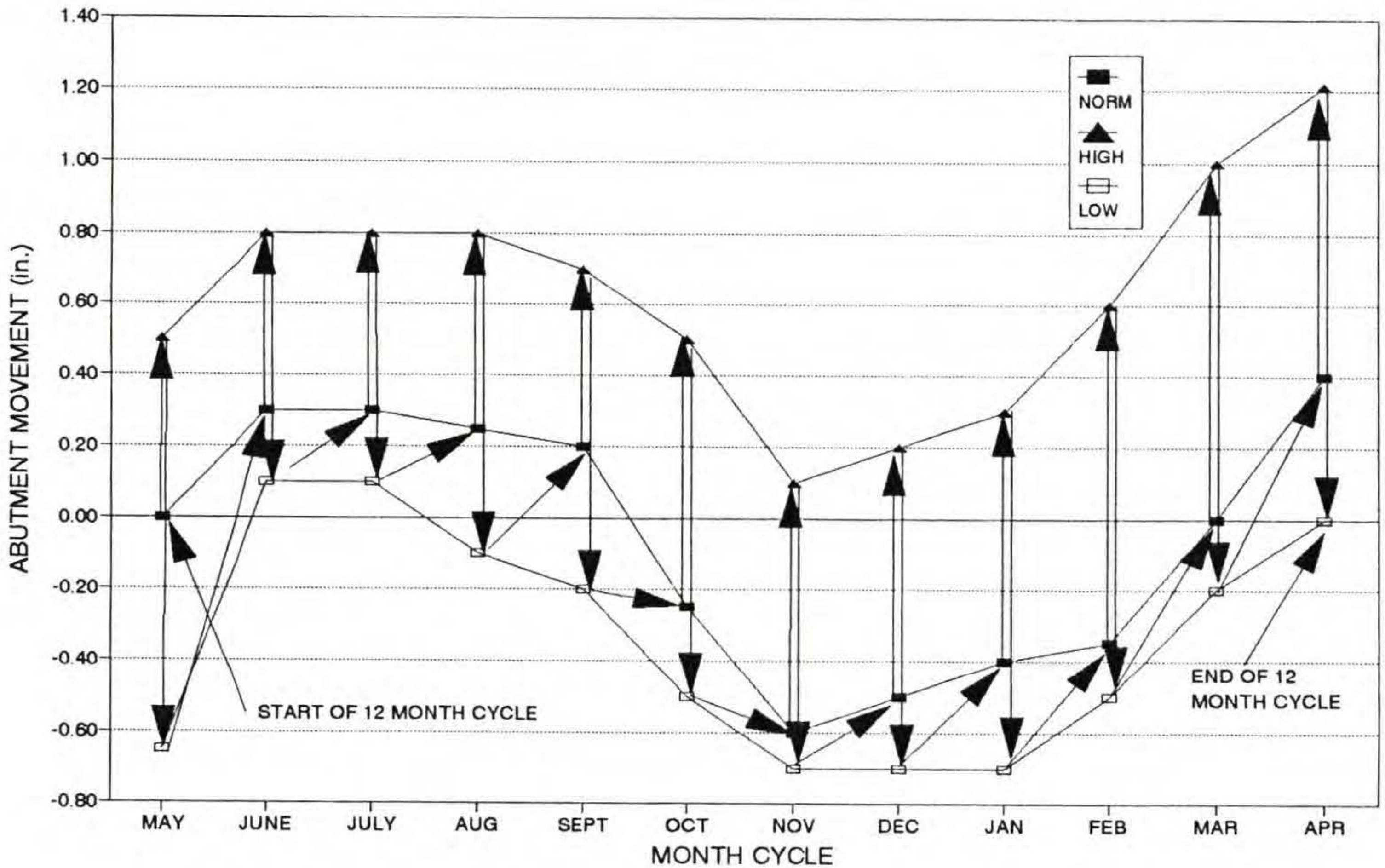



Figure 5-8. Monthly abutment movement cycles developed from temperature data for May 1990 through April 1991 to simulate a one-year cycle of abutment movement.

a. Dial gages positioned in front of abutment to measure movement and rotation of the abutment.

b. Plate used for monitoring abutment movement with a theodolite.

Figure 5-9. Systems used to measure abutment movements.

To effect a one-year pattern of movements, twelve daily cycles of movement were induced to the model. Following the first two cycles of movement, it became apparent that it would not be possible to run the test in twelve days due to the amount of time necessary to collect and reduce the instrumentation data. The testing started on May 20, 1992 and concluded on June 22, 1992. During the testing period daily temperatures ranged from the 30's to the 90's. Approximately six inches of rain occurred during the testing period.

### 5.7 General Observations

The month of May was used as the starting point for the first movement cycle, as shown in Figure 5-8, which also shows actual thermal cycles to which the bridge was subjected. Starting at 0.00 abutment movement, the first movement induced was a positive movement towards the backfill simulating thermal expansion. The movement was done in 0.10 inch increments every 15 minutes. The May high movement simulated was +0.50 inches and was achieved without incident. The next movement was contracting to the normal position. Problems arose at this time as the pulling jack was difficult to control and modifications had to be made to provide better pressure control to the jack. Consequently, the bridge was allowed to set overnight before the contraction to the May low point.

Significant problems appeared during the movement step from the normal to the low. The dial gages in front of the abutment were connected to posts cemented into the ground approximately three feet back from the abutment. Movement of the abutment in the negative direction moved the posts cemented into the ground, and thus the dial gages showed little movement on the south side. The north side showed some movement of the abutment and it was assumed that the jacking system was not pulling straight. A theodolite was subsequently used to check and determine the movement of the bridge abutment. As control was set prior to movement, it was easy to determine that both sides of the abutment were moving together. Recorded movement on the north side was influenced by the abutment movement, which resulted in a contraction of -0.65 inches instead of the

-0.50 inch movement desired. Pressure was released off the pulling jack and the bridge was allowed to set over a long weekend.

The reason the dial gages recorded little movement on the south was attributed to the stiffness of the soil between the abutment and the posts concreted into the ground. Consequently, as the abutment contracted, the soil moved, pushing the posts back also. The north side had a smaller berm than the south and the soil was softer. Thus the soil likely tolerated the abutment movement with less effect on the dial gage post. New plates were installed on the bridge at this time to allow the bridge movements to be monitored with a theodolite (Figure 5-9). The theodolite was used to control movements since the abutment could still influence the posts during contractions below the zero point.

The June cycle of movements proceeded smoothly. Cracking of the embankment on the south side was noted at +0.60 inches of movement. The cracks are shown in Figure 5-10. The cracks were oriented diagonally down the embankment from the south wing wall corner. The cracks increased in size to approximately 1/8-inch in width and extended two to two and one-half feet from the wing wall corner. The cracks closed during the contraction step. During the contraction step, new cracks opened in the upper part of the embankment in front of the north wing wall and extended from the approach slab diagonally to the wing wall. These cracks appeared at +0.20 inches of movement and increased as movement continued to the June low of +0.10 inches (see Figure 5-11). A similar pattern of cracking developed on the south side at +0.10 inches of movement. Similar cracking patterns developed during each cycle of movement, with the crack width increasing with each cycle. During the September expansion, the south embankment developed a failure wedge that is illustrated in Figure 5-12. The area outlined by chalk appears to have failed completely from the rest of the embankment due to the expansion of the abutment. The diagonal cracks emanating from the abutment corner were up to 1/4-inch in width.

The cycles continued without trouble until the contraction step to the November low. The jack pulling the bridge reached its maximum movement at -0.70 inches. The

Figure 5-10. Diagonal cracks in south embankment near abutment at +0.60 inches of movement.

Figure 5-11. Cracking of north embankment during first contraction step, movement at +0.20 inches.

Figure 5-12. Outline of a wedge of failed embankment. Failure occurred during September expansion step.

Figure 5-13. Crack development in front of north wing wall during winter month cycles.

hydraulic line pressure became significantly larger than the recommended line pressure for the hoses. It became necessary to change the cycle of movement at this time to compensate for the jacking limitations. Since the jacks had a much greater pushing capacity, it was determined that the movement cycle could be shifted to a more positive movement cycle. This was easily achieved by adjusting the cycle of abutment movement for November on to a maximum negative value of -0.70 inches. Figure 5-8 reflects the adjusted cycles of abutment movement.

During the winter cycle months the cracks in front of the wing walls became quite large as shown in Figure 5-13. The cracks in the picture are approximately one inch wide. The embankments had become quite dry at this time and a one-half inch rain improved site conditions greatly. The spring cycles proceeded to the March normal with similar crack pattern development. The embankment alongside the approach slab settled approximately two inches next to the abutment tapering back on the north side, as shown in Figure 5-14. The south embankment had settled one to one and one-half inches next to the abutment, as shown in Figure 5-15.

Before the March high expansion step could be made, a severe thunderstorm poured three inches of rain on the site. This rainstorm caused some minor erosion around the wing walls and collapsed the soil on the north side into the void that had developed under the approach slab which is shown in Figure 5-16. The void was probed with a metal tape and met no resistance clear across. The void appeared to be two to two and one-half inches in height on the north side for an undetermined distance. The rain also caused other erosion development. An example is the wing wall erosion on the outer edges of the wing wall and on the embankments as shown in Figure 5-17. The rain thoroughly saturated the site and caused all cracks that had developed to seal. The site experienced approximately six to seven inches of rain over the next several days. The final cycles of movement were completed during this time.

The crack patterns, void development, and erosion features shown in Figures 5-10 through 5-17 are very similar to those observed during the field studies. For example,

Figure 5-14. Settlement of north embankment next to approach slab and wing wall. February cycle.

Figure 5-15. Settlement of south abutment next to approach slab and wing wall. February cycle.

Figure 5-16. Void development exposed on north embankment after a March rainstorm.

Figure 5-17. Erosion around wing wall following March rainstorm.

Figure 3-2c shows a developed void which required no digging to expose. The slope and settlement pattern is similar to that shown in Figure 5-16 from the model studies. The crack patterns noted in the model embankments were found on many of the structures visited during the field investigations. Figure 3-8 shows the development of gaps in front of the abutment wall and along a wing wall. Such gap development occurred during the first few cycles of movement in the model test.

### 5.8 Test Results

The results of the model tests are discussed below. Selected test results are shown to illustrate key points. Complete test results are provided in Appendix D.

#### 5.8.1 Void Development

The primary reason for conducting the model study was to determine if thermal expansion and contraction directly influenced the development of a void underneath the approach slab. From the very first movement a void began to develop under the approach slab and grew consistently larger with each cycle of movement. Figures 5-18 to 5-20 show the development of the void throughout the 12 month cycle for the outside, intermediate, and middle holes on the approach slab, located one-foot back from the abutment face. (See Figure 5-4 for location.) These graphs show that a larger void developed near the outside than on the inside. As discussed in Chapter 3, such a pattern was prevalent in field observations and measurements.

Figures 5-18 to 5-20 also show that a cyclic increase/decrease in the void height occurred as the abutment contracted and expanded. It follows that the size of a void measured in the field will be dependent on the expansion/contraction stage of the bridge structure. As noted in Chapter 3, there were sites where no void was measured during extremely high temperatures at sites where a void had been previously found. The cyclic behavior of the void height suggests a direct relationship between void development and abutment movement.

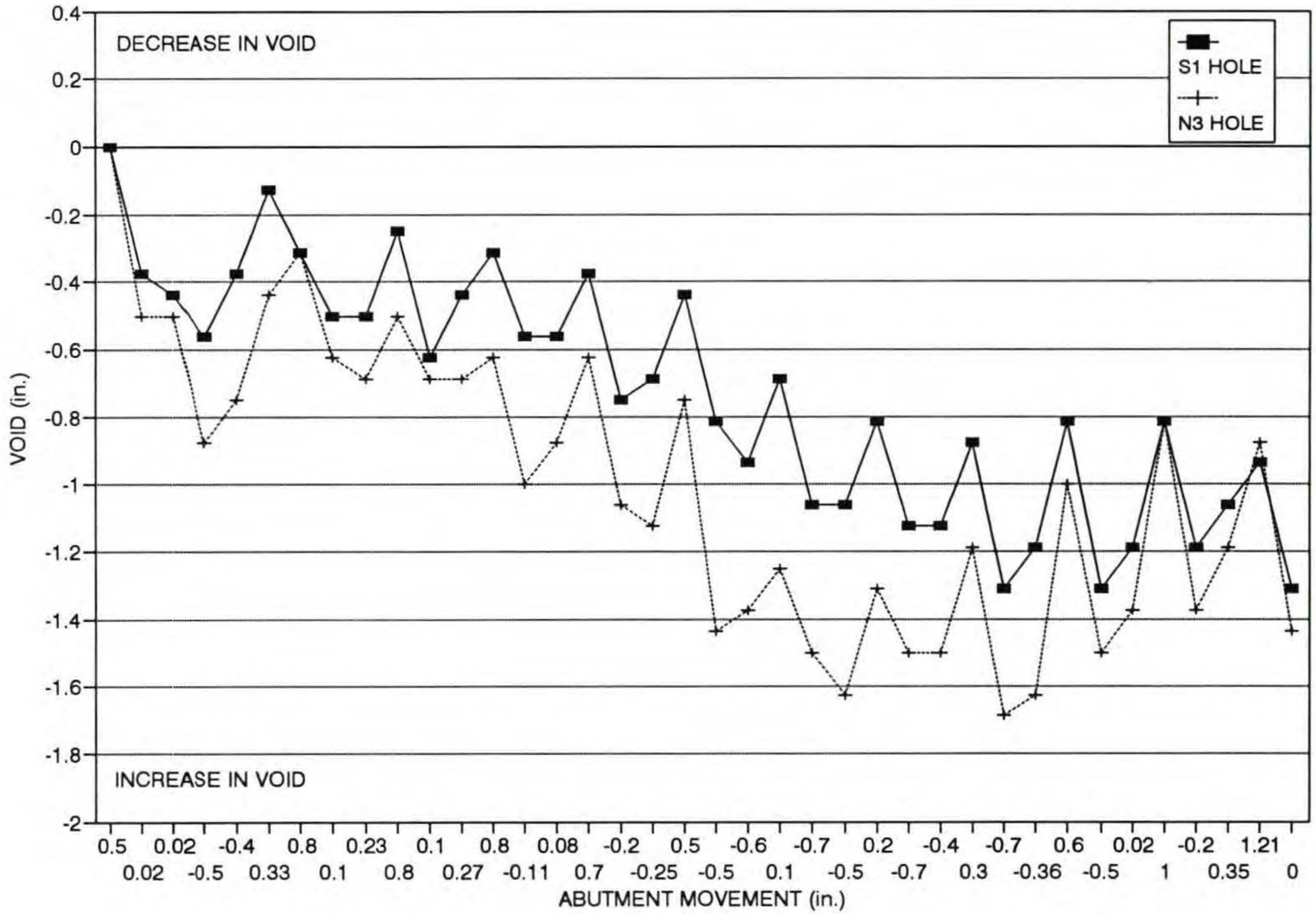



Figure 5-18. Void development under the approach slab at locations S-1 and N-3.

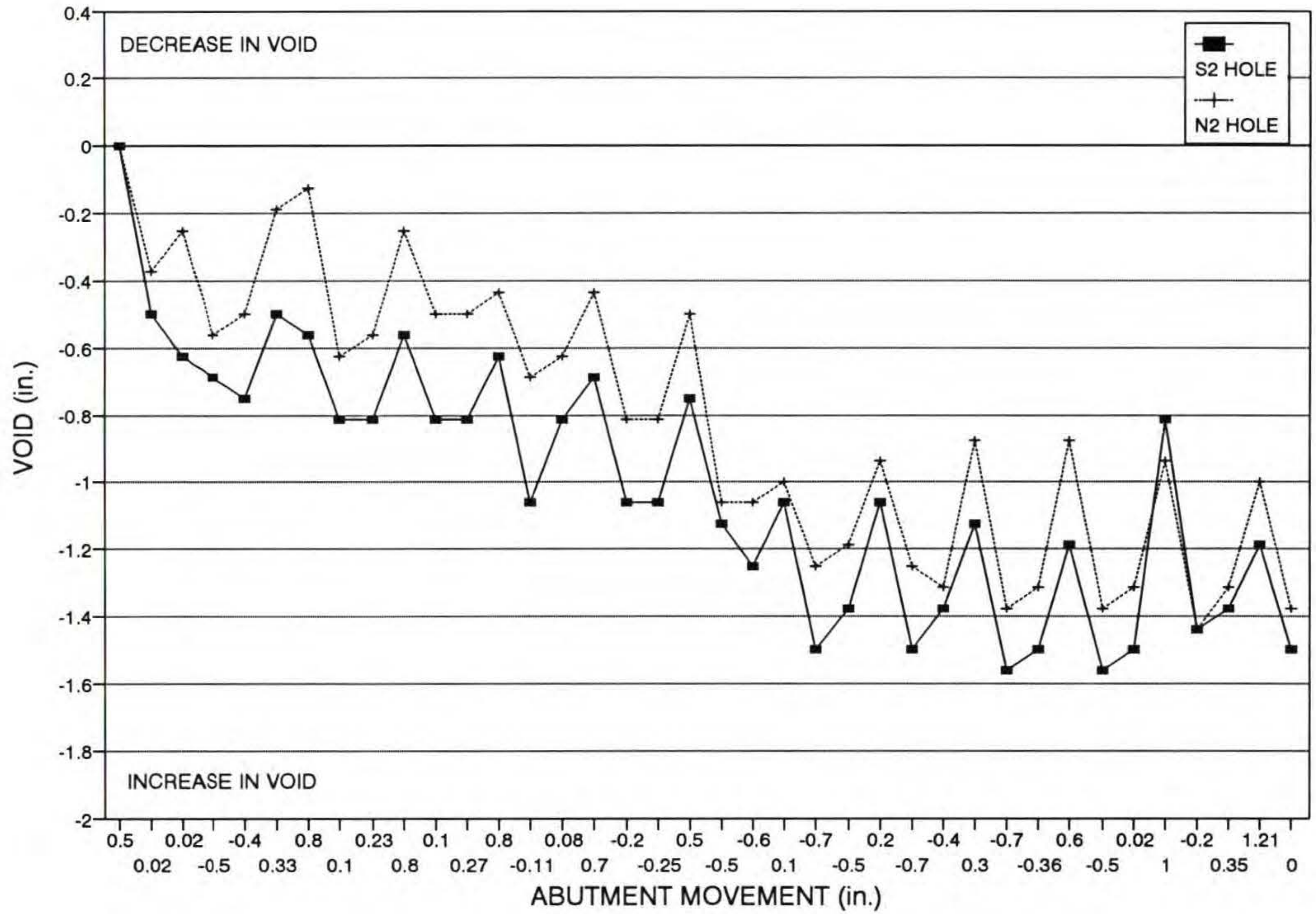



Figure 5-19. Void development under the approach slab at the S-2 and N-2 locations.

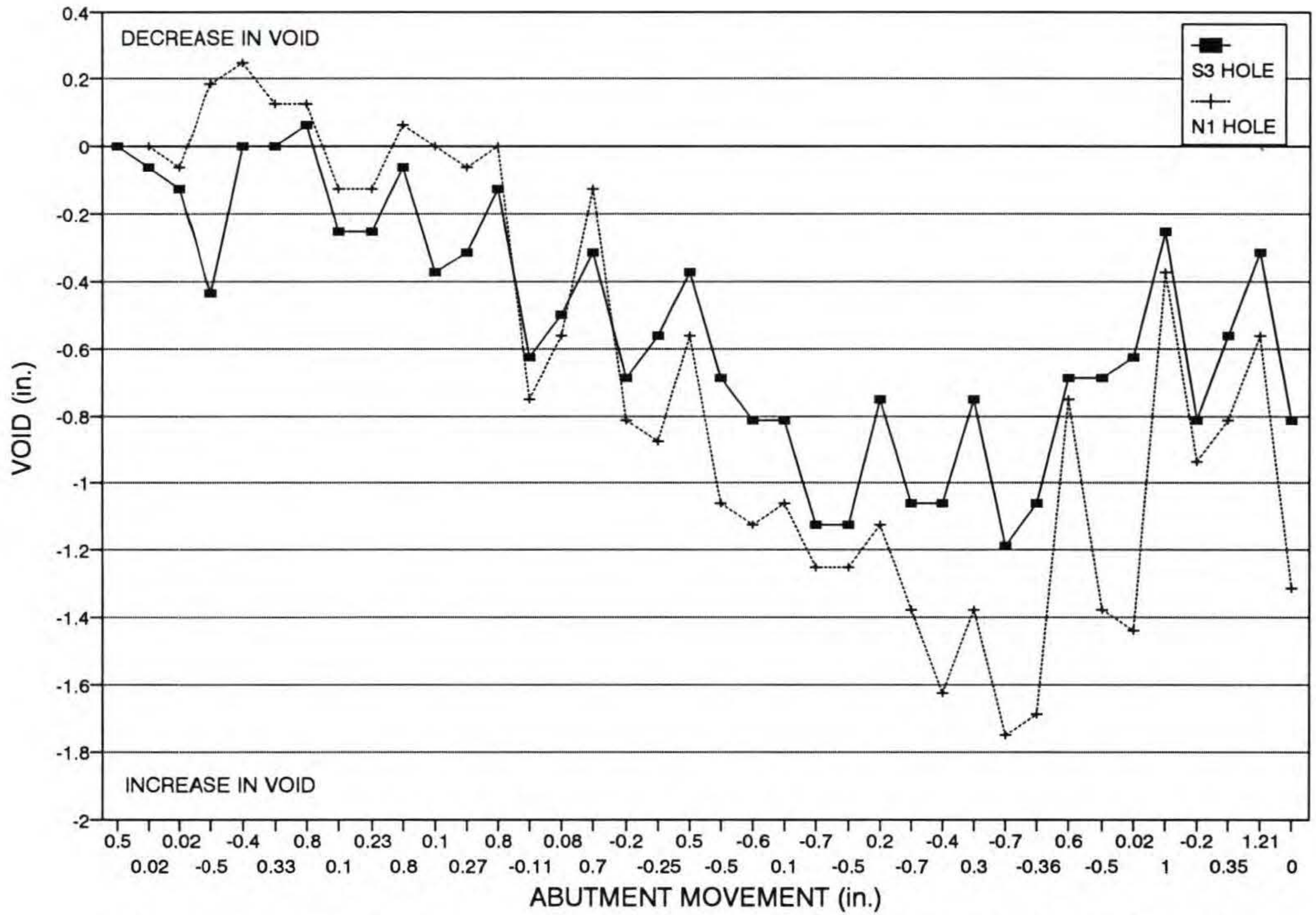



Figure 5-20. Void development under the approach slab at the S-3 and N-1 locations.

The void was measured with the field probe to determine the extent of the void and to determine the total volume of the void developed. The void was found to be two inches in height at the abutment face and extending approximately two feet back from the abutment. The volume of the void was calculated to be approximately four cubic feet. The field probe measurements are summarized in Table 5-1.

| Table 5-1. Measured void heights using field probe at end of test. | | | | | | |
|---|---|---|---|---|---|---|
| Measurement Location | | | | | | |
| Probe Position | S1 in. | S2 in. | S3 in. | N1 in. | N2 in. | N3 in. |
| Abutment | 2 | 1.5 | 1.5 | 2 | 2 | 2.25 |
| 1 ft. back | 1.3 | 1.5 | .8 | 1.3 | 1.4 | 1.4 |
| 2 ft. back | 0 | 0 | 0 | 0 | 0 | 0 |

### 5.8.2 Earth Pressure Development

The model tests also measured the development of earth pressures in the backfill due to thermal bridge movements. Inclinometers and pressure cells were placed in the backfill area to determine movements and earth pressures. The pressure cells recorded the pressure developed on the pressure cells during the cycles of movement. The pressure data recorded were reduced to determine the differential pressures acting on the abutment wall relative to the initial, premovement pressure valves. The top and bottom cells were positioned at depths of two and four feet, respectively, from the top of the bridge deck. Figures 5-21 and 5-22 show the pressure differential from the initial reading for the top and bottom cells, respectively, at different abutment movements for the entire twelve-month test. The graphs clearly show a relationship exists between the abutment movements and changes in earth pressure. In general, the band of high pressures to the right of zero abutment movement developed during expansion steps as the wall was moved into the backfill. The band of pressures below the high pressures developed as the wall was moved away from the backfill. As will be shown below, a hysteretic pressure change develops during the cyclic loading/unloading of the backfill.

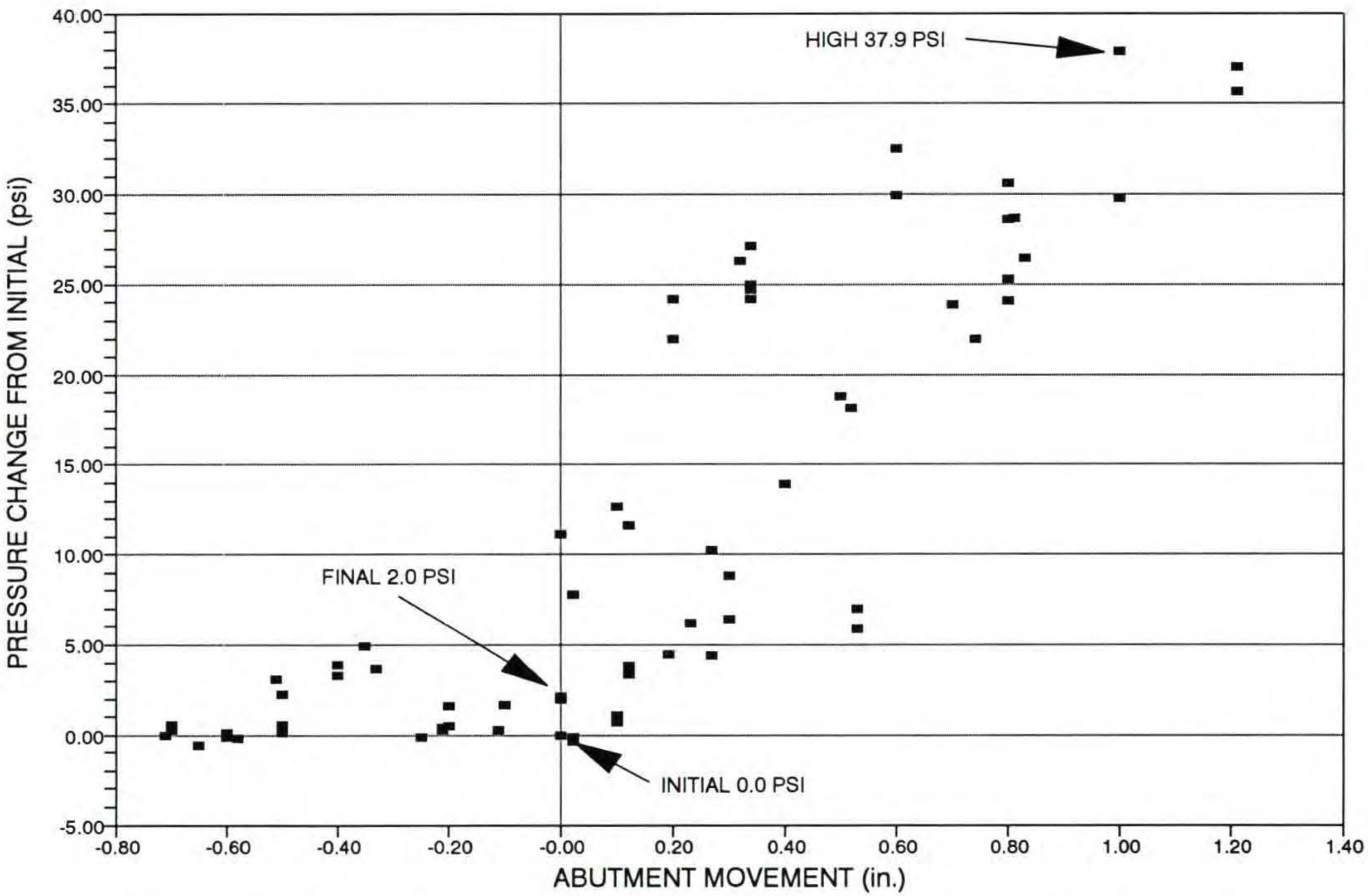



Figure 5-21. Change in pressure of top cell versus abutment movement. Values for entire twelve-month test.

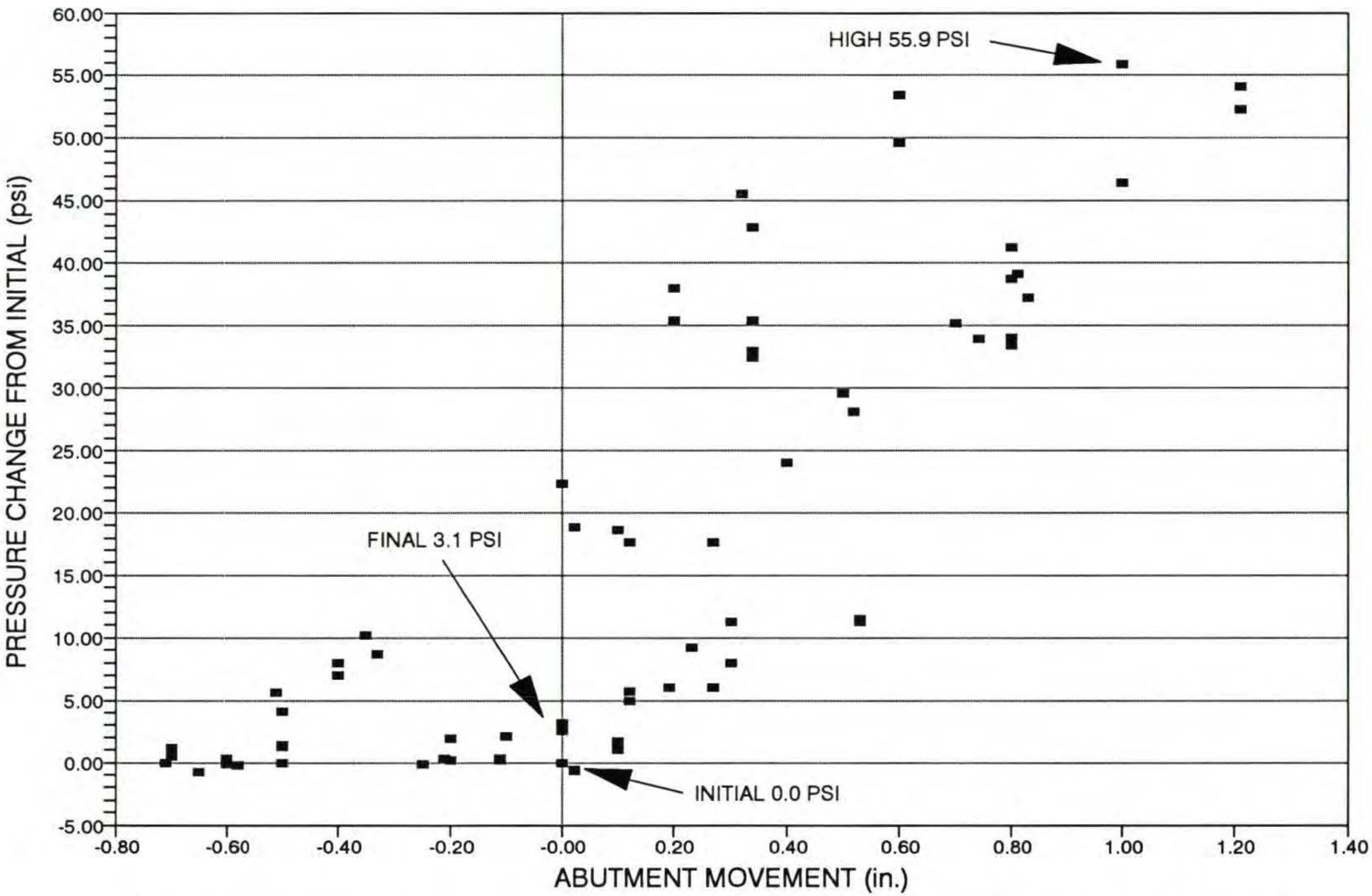



Figure 5-22. Change in pressure of bottom cell versus abutment movement. Values for entire twelve-month cycle.

Highlighted on Figures 5-21 and 5-22 are the initial, maximum, and final differential cell pressures for the twelve-month test simulation. The top cell reached a maximum value of 37.9 psi which corresponds to an earth pressure development of 5460 psf two feet below the top of the abutment. The bottom cell reached a maximum value of 55.9 psi which corresponds to an earth pressure development of 8050 psf four feet below the top of the abutment. Clough and Duncan (1991) summarized the available data on approximate magnitudes of movements required to reach maximum passive earth pressure conditions. For a dense sand the value of wall movement necessary to reach these conditions is approximately 0.01 times the wall height. With a wall height of five feet, the required wall displacement is about 0.6 inches to reach maximum passive earth pressure conditions. In addition, theoretical passive earth pressures for the two and four foot depths can be calculated using logarithmic spiral earth pressure theory. The resulting earth pressure is dependent upon the density of the backfill, the friction angle of the backfill, and the wall friction angle between the concrete abutment wall and the backfill. A backfill density of 100 pcf was assumed, which is slightly less than the maximum value obtained in the laboratory. The friction angle of the compacted backfill is likely between 40 and 50 degrees. The coefficient of passive earth pressure is highly dependent upon the value of wall friction selected. The use of a conservative value of wall friction of zero results in theoretical earth pressures at two and four feet of much less than the measured values. The use of wall friction values of about two-thirds of the backfill friction angle results in theoretical values of earth pressure reasonably close to the measured values. Thus, based on the measured earth pressure values, it is likely that the granular backfill reached a state of passive failure at wall movements above about 0.6 inches from the zero point.

Figures 5-21 and 5-22 also show that final differential pressures of 2.0 psi and 3.1 psi were recorded for the top and bottom cells, respectively. These residual pressures are consistent with plastic behavior of the backfill.

Figures 5-23 and 5-24 show the earth pressure response of the backfill during a summer cycle and a spring cycle. The earth pressure response during the August cycle (summer) is shown in Figure 5-23, where it can be seen that the backfill exhibits an

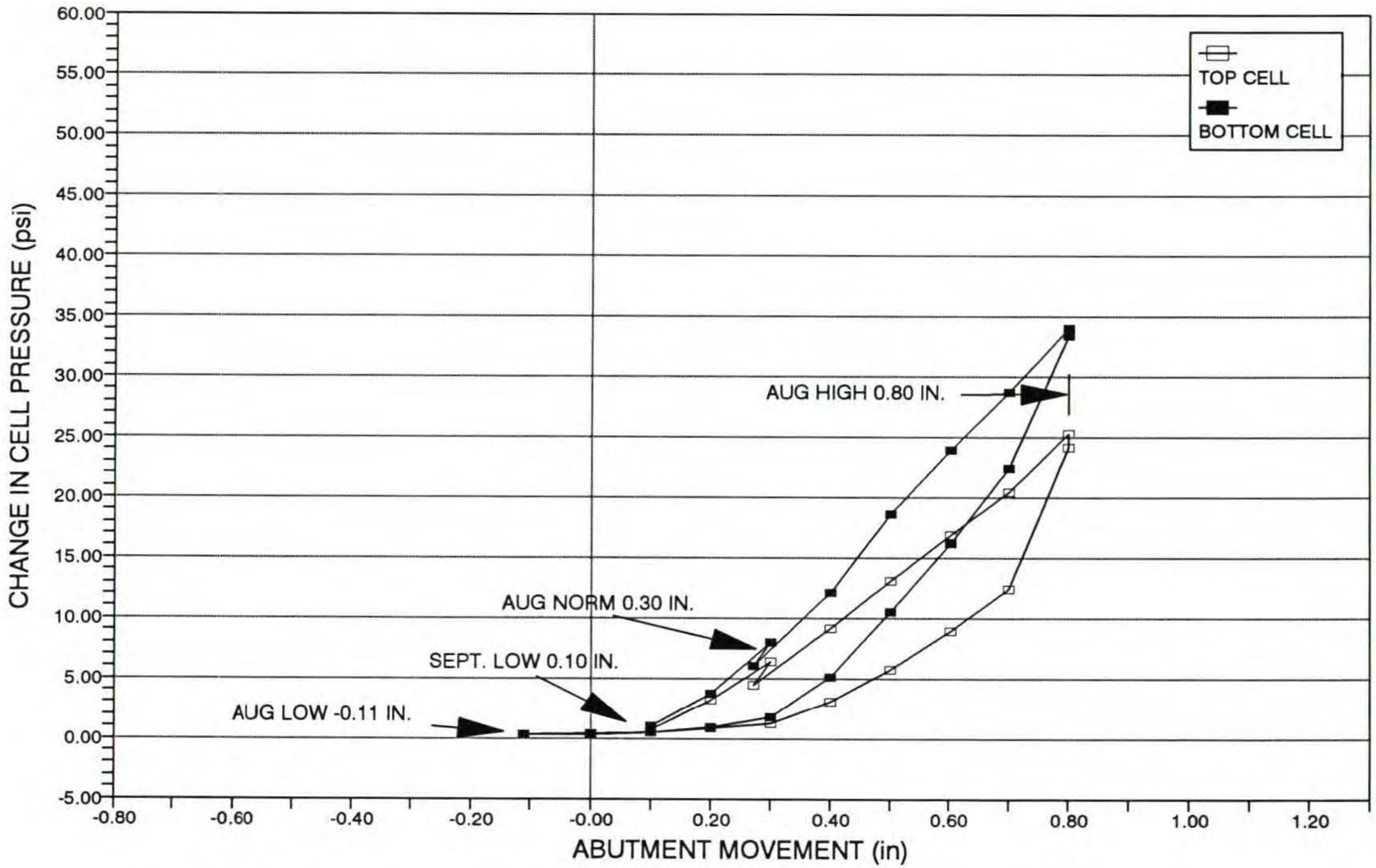



Figure 5-23. Example of elastic behavior in the material with bridge movement.

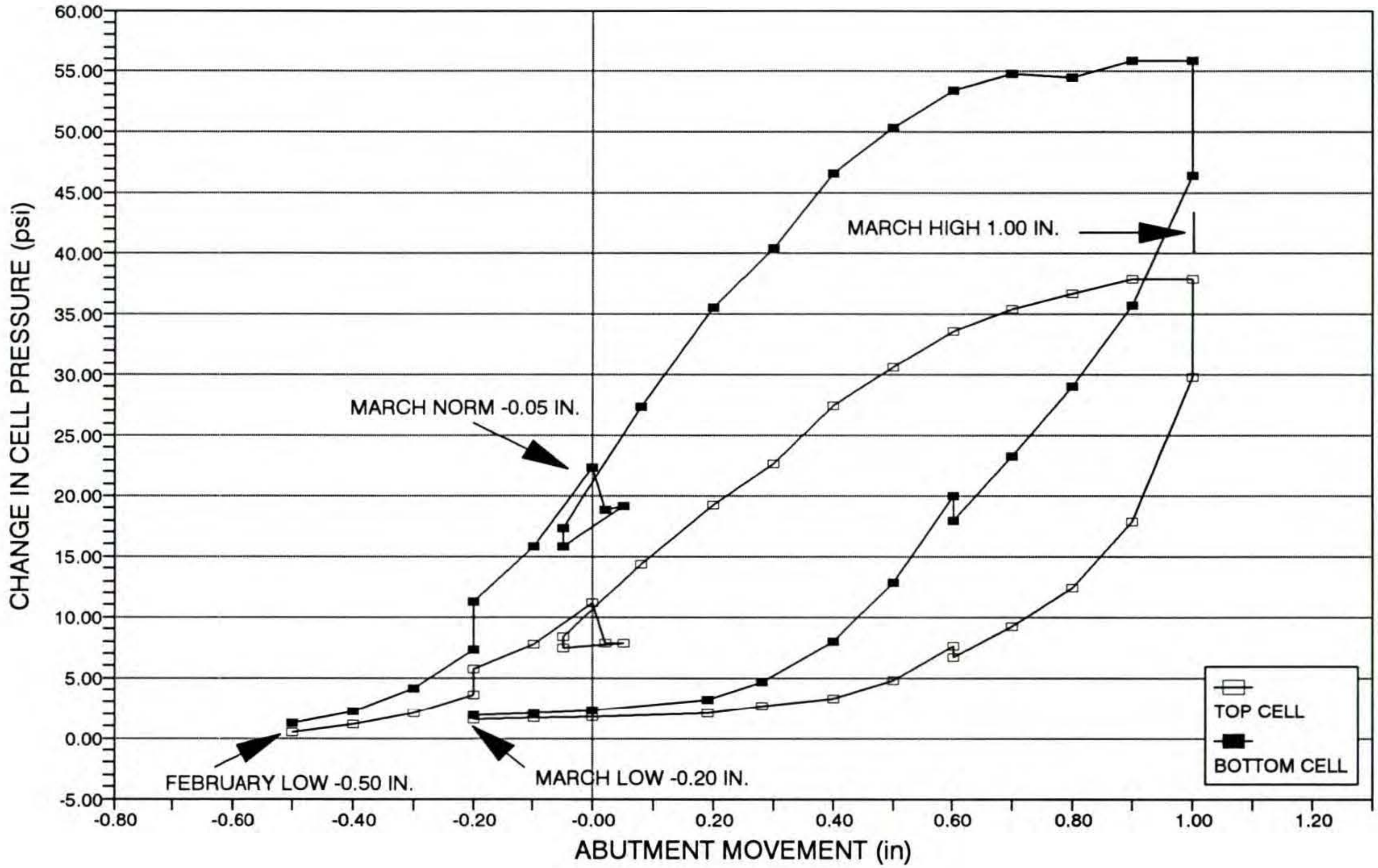



Figure 5-24. Example of plastic deformation in material with movement cycle.

elastic, although hysteretic, material response when stressed with a +0.70 inch movement. The upper branch of the curve represents the path as the abutment expands and the lower branch represents the path as the abutment contracts. It can be seen that as the abutment returns to a zero movement, the earth pressures essentially reduced to zero.

The earth pressure response during the March cycle (spring) is shown in Figure 5-24, where it can be seen that the backfill exhibits an elastic behavior up to a movement level of about +0.60 inches followed by plastic behavior. It is important to note that this expansion cycle started at the February low of -0.50 inches and ended at the March high of +1.00 inches, for a total abutment movement of 1.50 inches for this expansion step. The plastic behavior occurred after abutment movement of about 1.1 to 1.2 inches.

The pressure cell responses also showed that earth pressure redistribution occurs during the abutment movement steps. For example, during the movement step to the October high, pressure cell readings were recorded immediately after each 0.10 inch movement, and again following a 10 minute delay. This procedure was repeated for the movement steps from the October low and to the November norm. Figures 5-25 and 5-26 show the data for the top and bottom pressure cells, respectively. The data show that a pressure decrease occurs during the ten minute delay. The pressure decrease is largest during the expansion step. During the contraction step the pressure decrease occurred during the first two 0.10 inch increments. The pressure changes occur due to a redistribution of stresses occurring in the backfill due to the loading. It is also interesting to note that additional movement of the abutment occurred at the October high as the system was allowed to equilibrate.

### 5.8.3 Inclinometer Movements

The inclinometers in the backfill were oriented to record the lateral and longitudinal movement of the backfill with respect to the initial recordings. The inclinometers were positioned approximately five feet away from the abutment and were positioned in line with the integral abutment piles, to include possible pile movement effects on the backfill.

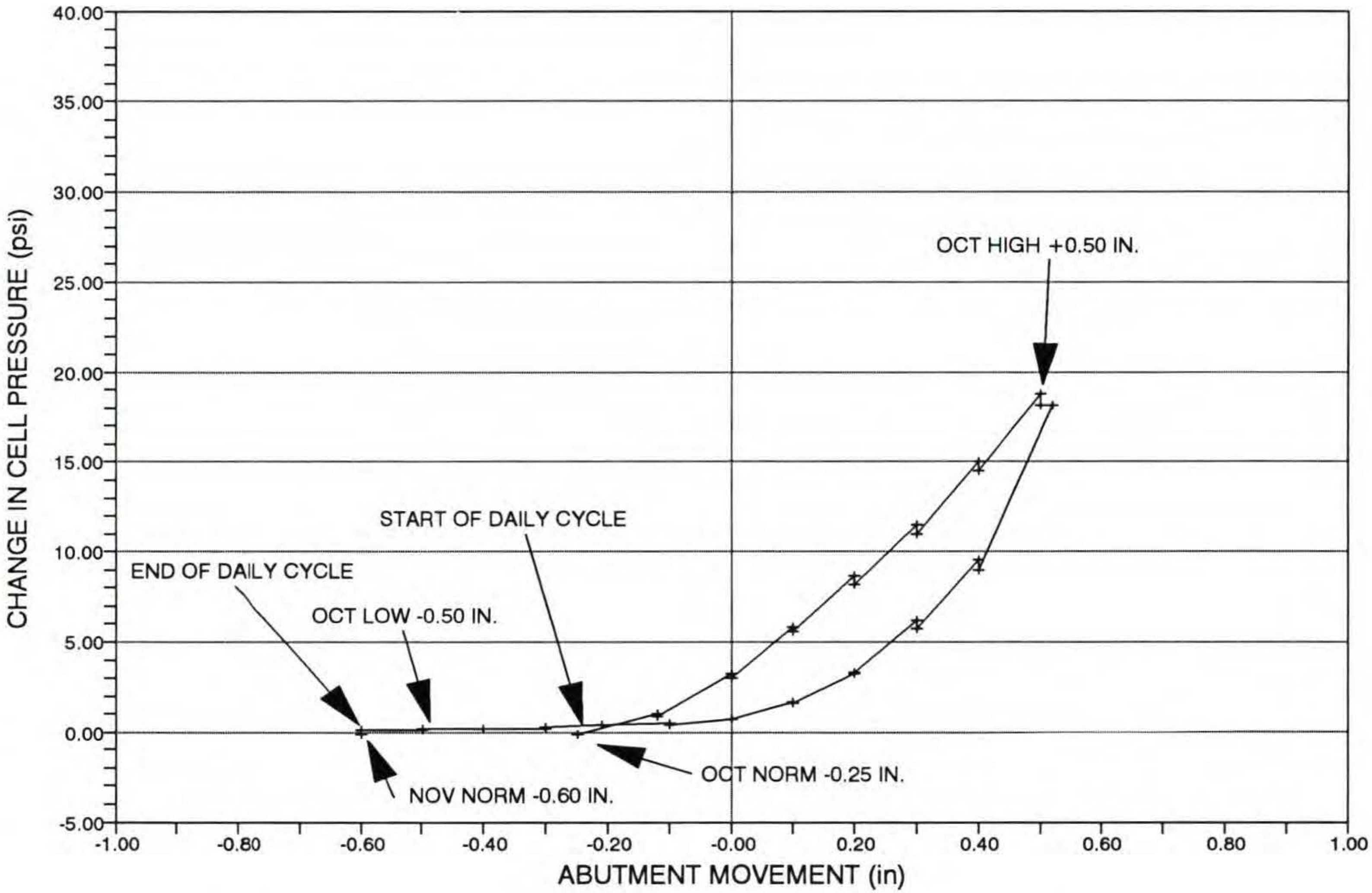



Figure 5-25. Pressure changes versus abutment movement for October cycle, top cell. Pressure readings taken before and after a 10 minute relaxation period.

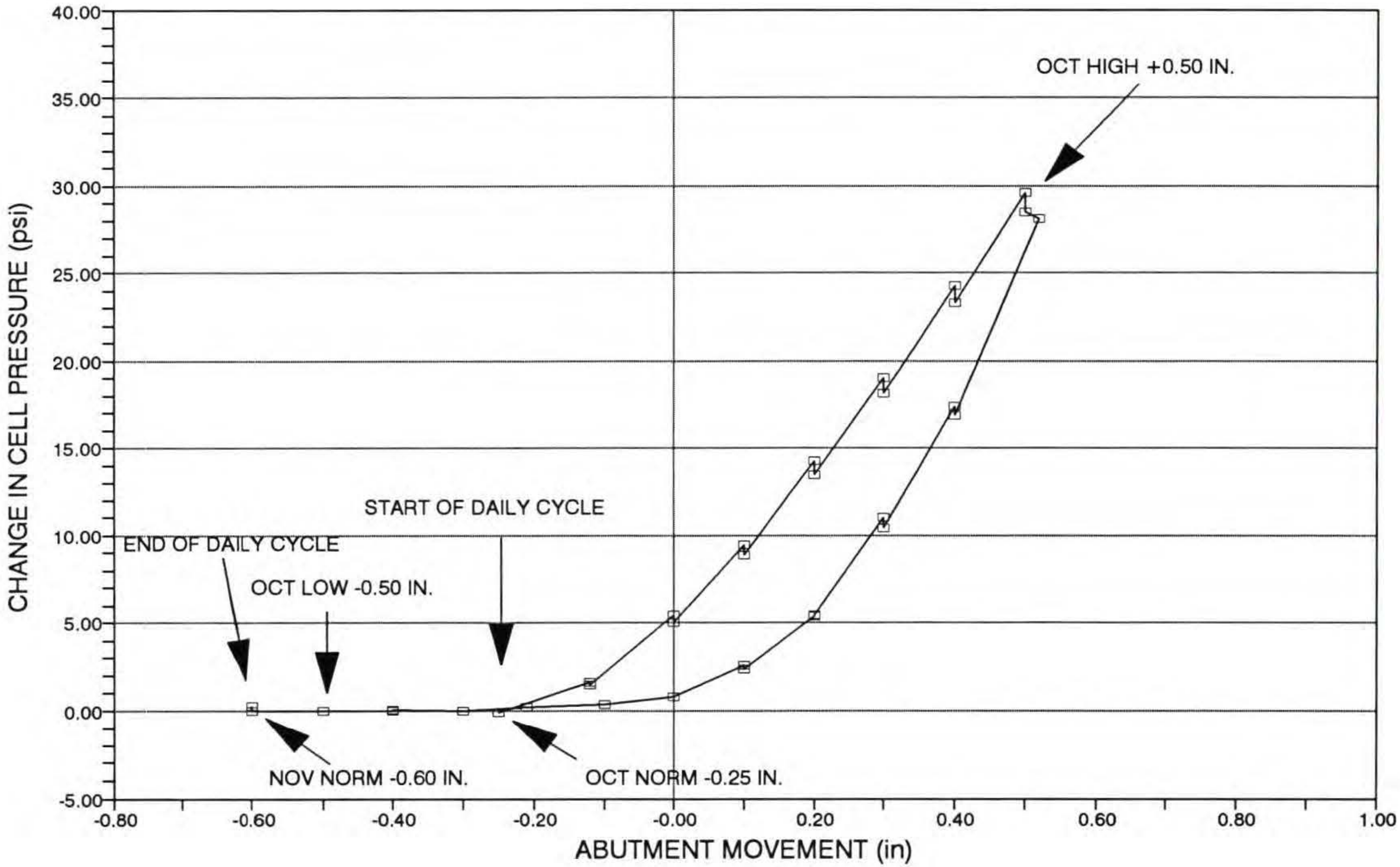



Figure 5-26. Pressure changes versus abutment movement for October cycle, bottom cell. Pressure readings taken before and after a 10 minute relaxation period.

Figures 5-27 and 5-28 show the final results for lateral deflection, with intermediate results presented in Appendix D. It can be seen that little movement occurred in the lateral direction. The north inclinometer recorded a final lateral expansion of only 0.08 inches while the south inclinometer recorded slightly more with a final lateral expansion of 0.19 inches. Although small, these movements show that the backfill is expanding the embankment outward with the cyclic movement. More lateral movement may have been recorded in the area closer to the abutment, but the inclinometer tubes had to be positioned some distance from the piles. The approach slab may have also influenced the movement of the inclinometer tubes in the lateral direction. Figure 5-28 shows the maximum movement at a depth of five feet, with movement decreasing at the approach slab interface. The slot made in the approach slab could accommodate up to one inch of movement, and aggregate may have wedged itself between the tube and the approach slab to prevent further movement. The same trend is visible in the north inclinometer tube to a lesser degree (Figure 5-27). Movement reaches a maximum at a depth of two feet but then decreases at the approach slab interface.

The graphs also show deflection of the inclinometer tubes occurred in the embankment below the backfill, indicating that pile movement due to cyclic movements of the bridge may affect the embankment. The pile action has the effect of expanding the embankment outward below the backfill which would contribute to backfill settlement.

The inclinometers also recorded deflection longitudinally with the bridge and approach. Figures 5-29 and 5-30 show the longitudinal deflection of the inclinometers at the completion of the twelve-month cycle. The longitudinal deflections were significantly larger than the lateral movements. The north inclinometer had a final longitudinal deflection of 1.9 inches away from the abutment. The south inclinometer had a final longitudinal deflection of 2.0 inches away from the abutment. The results for each monthly cycle are presented in Appendix D. The maximum longitudinal deflections were recorded after the April high abutment expansion of 1.2 inches. The two inclinometers recorded approximately 2.6 inches of longitudinal deflection away from the abutment after

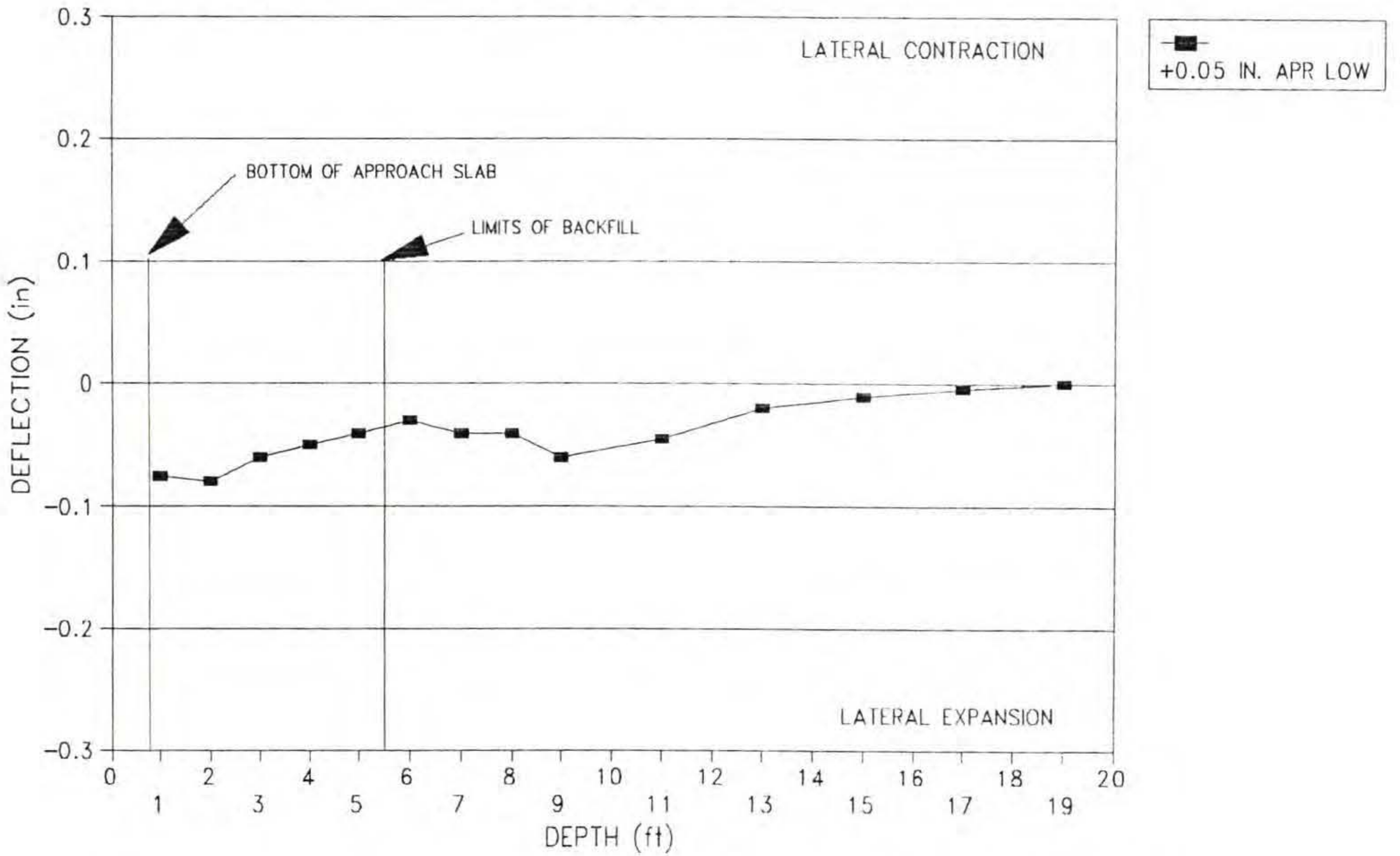



Figure 5-27. Lateral movements of the north inclinometer at the completion of the test.

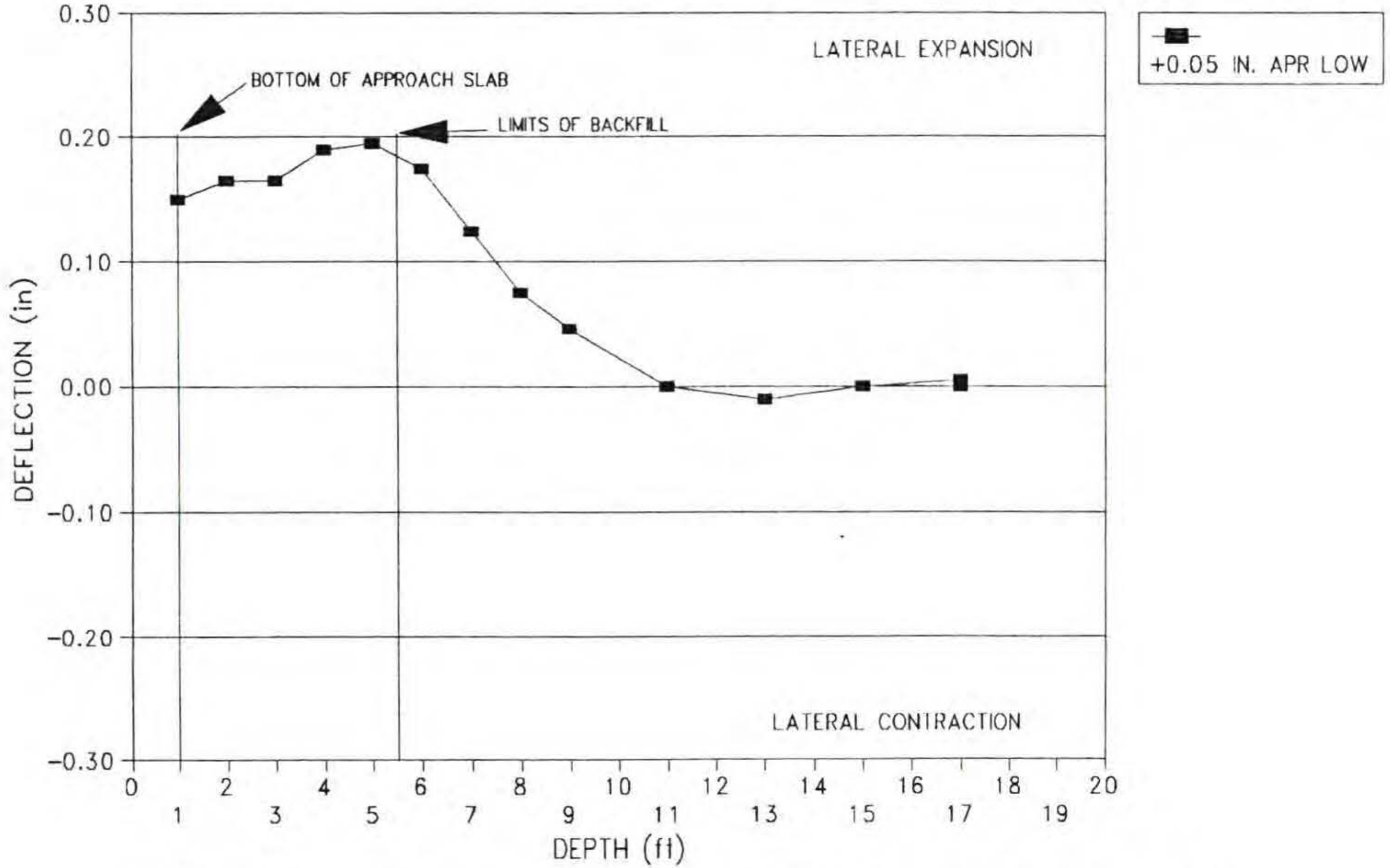



Figure 5-28. Lateral movement of the south inclinometer at the completion of the test.

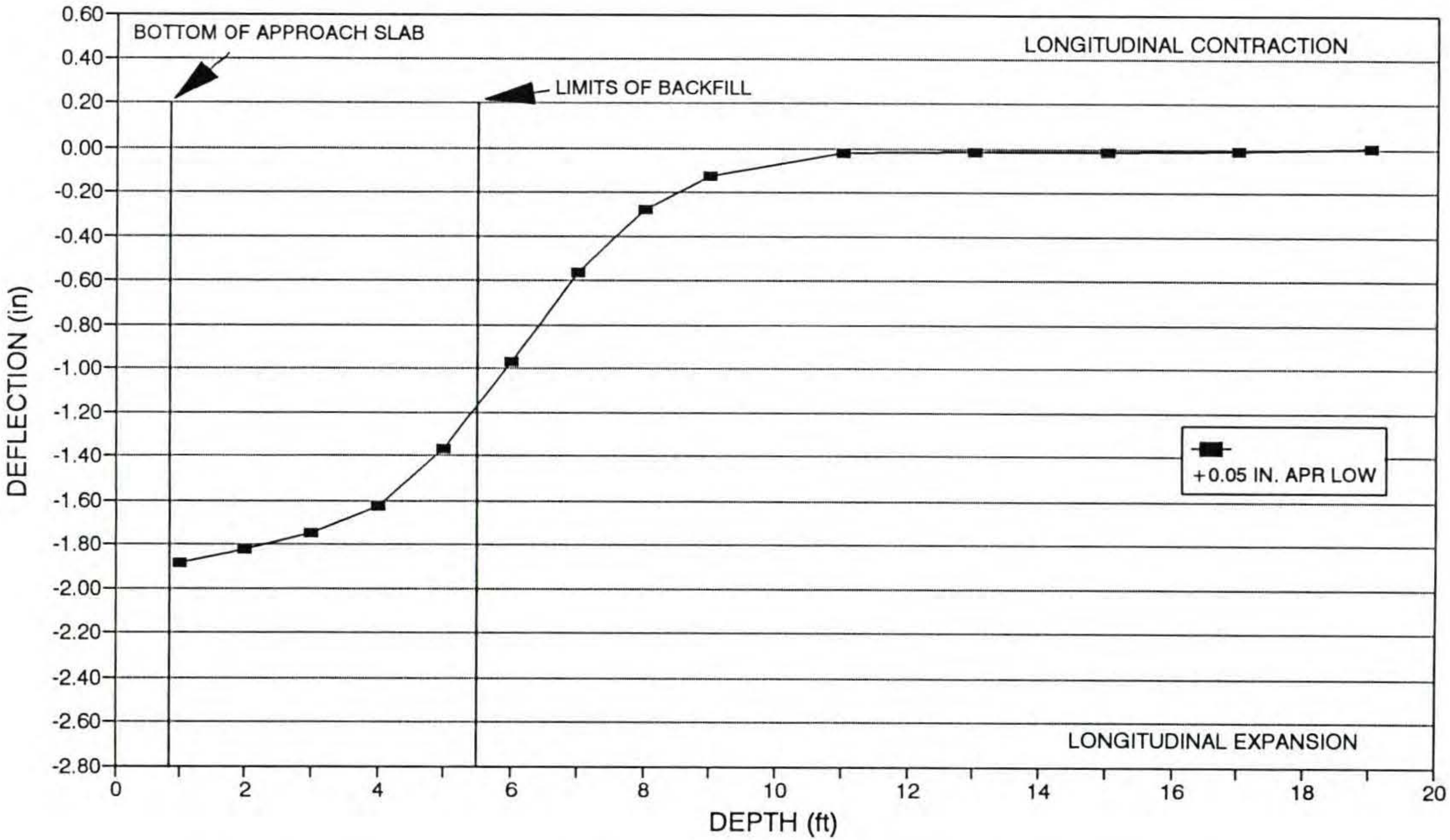



Figure 5-29. Longitudinal movement of the north inclinometer at the completion of the test.

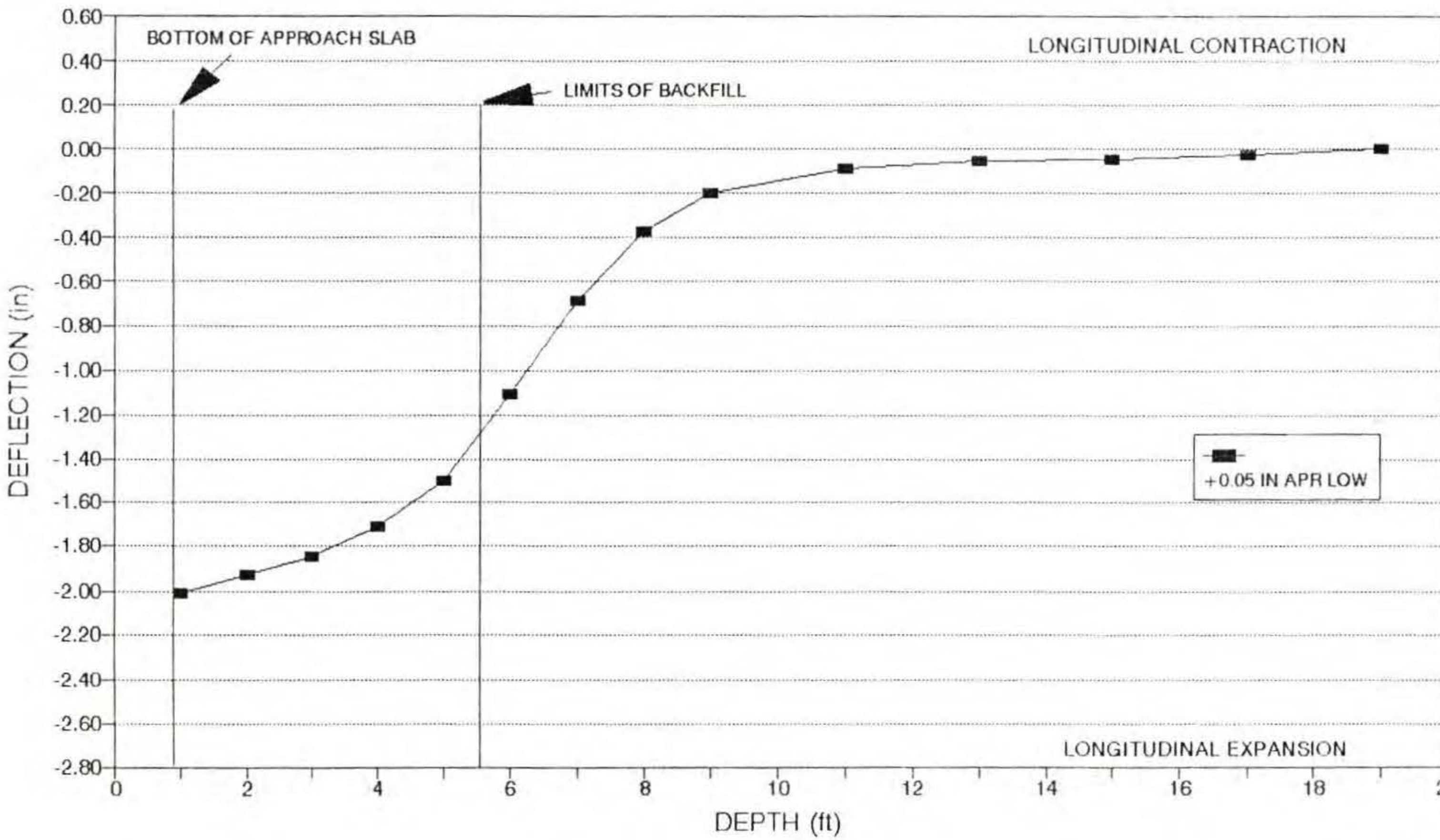



Figure 5-30. Longitudinal movement of the south inclinometer at the completion of the test.

this cycle. The cyclic nature of the longitudinal movements can be seen in Figure 5-31 where the longitudinal movements of the north inclinometer are plotted versus sequential abutment movement. It can be seen that the longitudinal movements cycled as the abutment expanded and contracted. At a depth of one foot the cyclic movement was about 0.4 inches with permanent deformation of 0.4 to 0.5 inches. At depths of five and six feet the cyclic movement was about the same but the permanent deformations were somewhat larger. For most of the test the movements cycled in a narrow band. Abutment expansion would induce longitudinal expansion and abutment contraction would induce recovery of a portion of the expansion. A permanent set was quickly achieved at each level. Even large contractions would not induce much recovery of the permanent lateral set. Near the end of the test much larger abutment movements were induced to model large spring-time temperature changes. These larger abutment movements induced significant additional cyclic and permanent deformation in the backfill and embankment soils.

### 5.8.4 Approach Slab Movements

The movement of the approach slab was monitored by level survey at stations 1 through 9 shown in Figure 5-4. The elevation changes versus abutment movement are shown in Figures 5-32, 5-33, and 5-34. The accuracy of the level survey is approximately 0.01 foot or about one-eighth of an inch. Therefore elevation changes of about 0.12 inches may simply reflect the accuracy of the measurements. As can be seen in Figures 5-32 through 5-34, most of the measured elevations changes are about 0.12 inch. However, a couple of interesting trends are apparent on these graphs. On the edges of the approach slab only the center stations showed much elevation change, with increases of one-quarter to one-half an inch. These elevations are associated with the high abutment expansion steps, indicating that upward movement of the backfill is taking place during expansion. The approach slab elevations cycled with abutment movement, although with smaller changes than other measurements. No apparent permanent change in the approach slab elevations occurred near the abutment or the sleeper slab. An apparent permanent change of about one-quarter inch occurred for the middle stations. Such small vertical changes represent sizeable volume changes for the backfill when considered over the approach slab area. For example, a 0.02 foot vertical elevation change over a 10-foot longitudinal

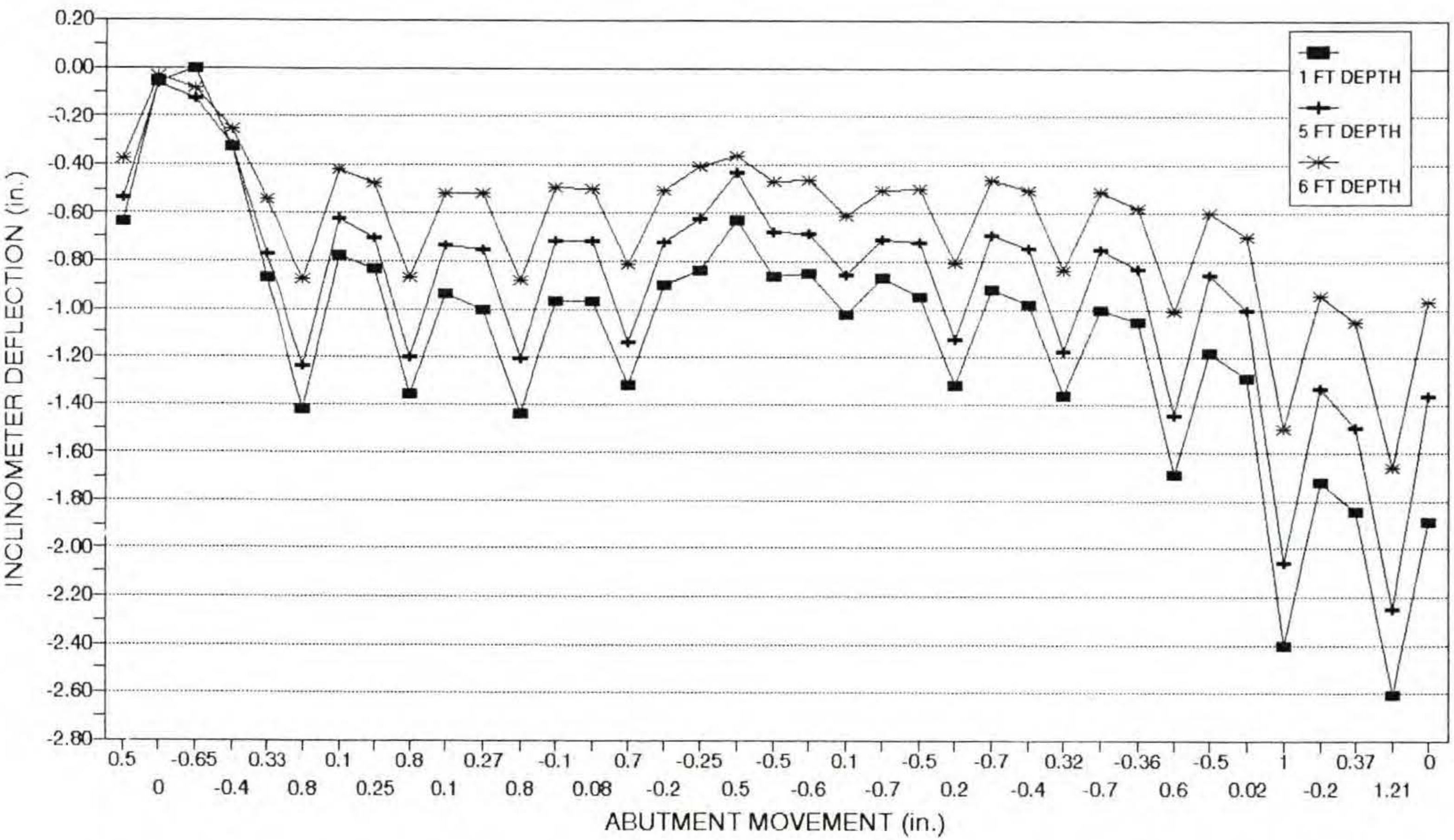



Figure 5-31. Longitudinal movement of the north inclinometer versus abutment movement.

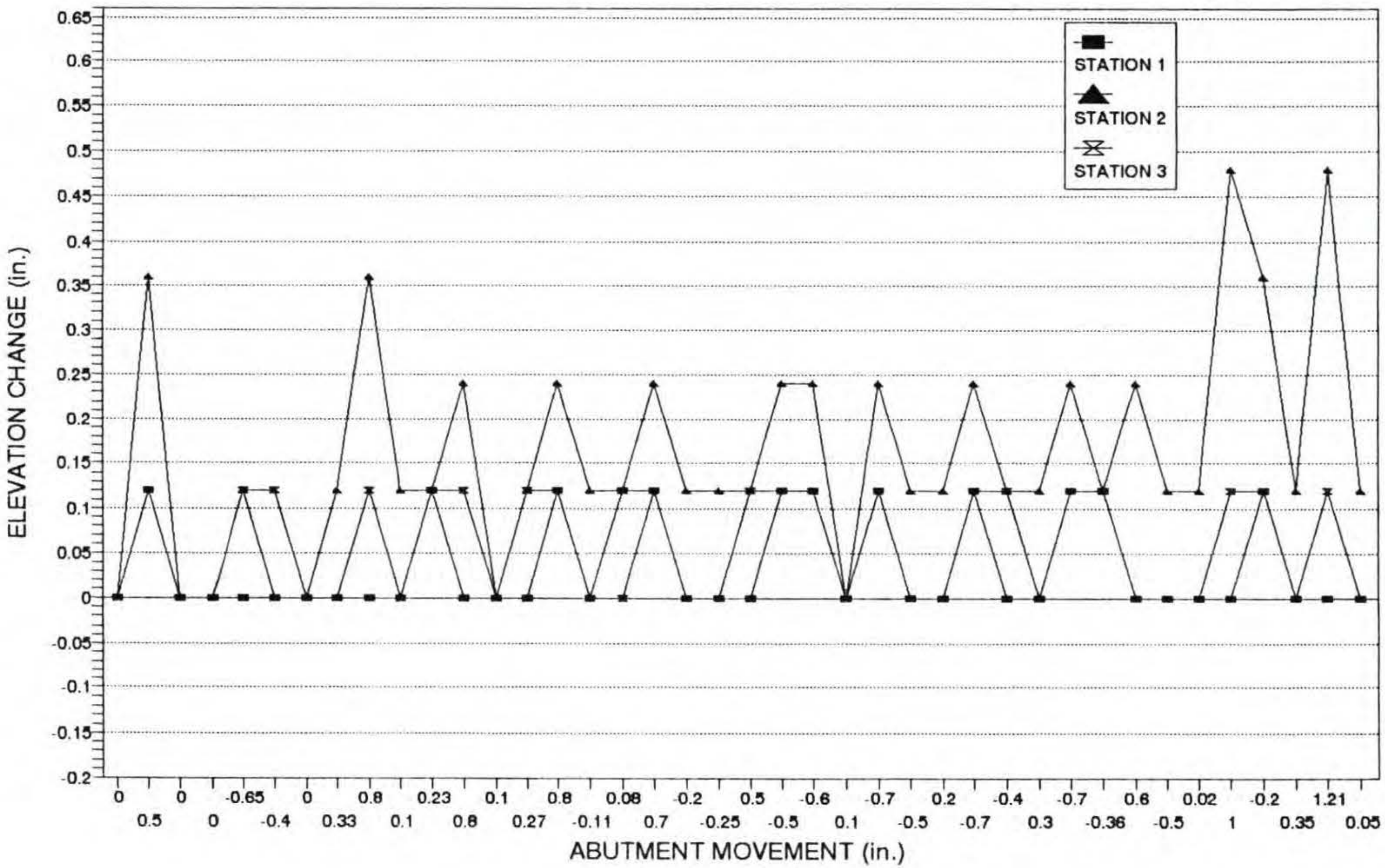



Figure 5-32. Change in approach slab elevation versus abutment movement for stations 1, 2, and 3.

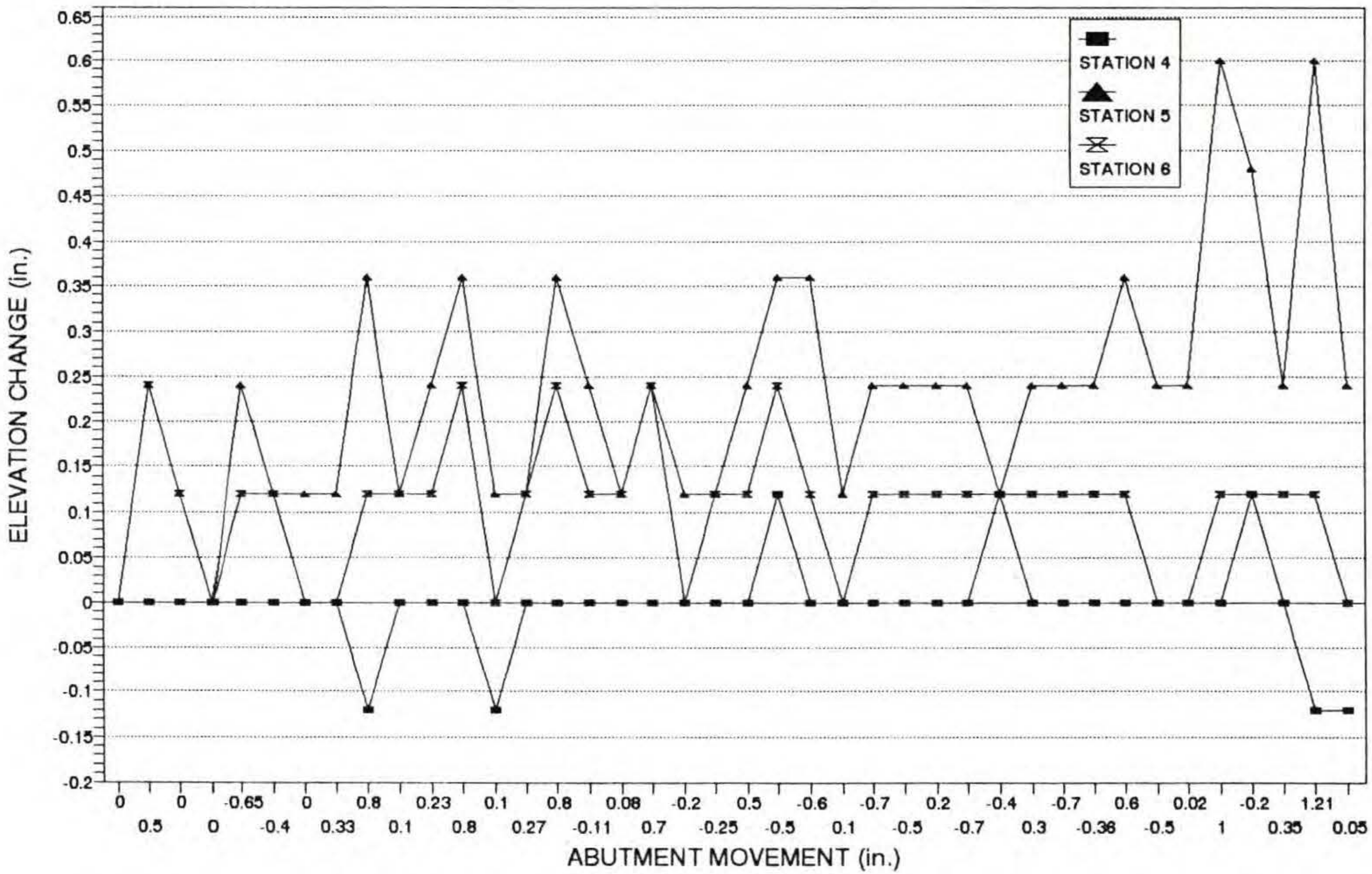



Figure 5-33. Change in approach slab elevation versus abutment movement for stations 4, 5, and 6.

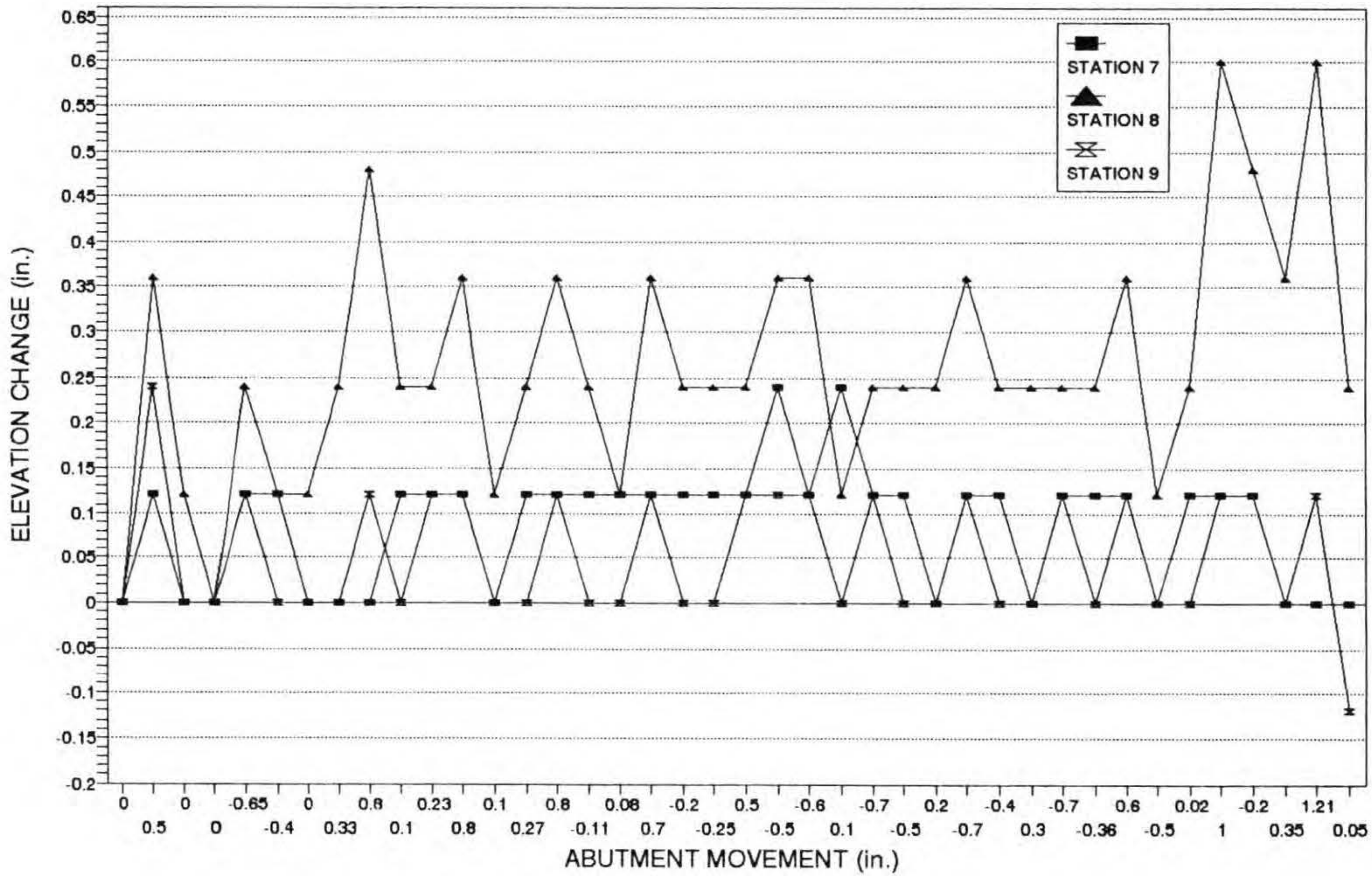



Figure 5-34. Change in approach slab elevation versus abutment movement for stations 7, 8, and 9.

section of the approach slab (12 feet wide) represents a volume change of about seven cubic feet.

### 5.8.5 Embankment Stake Movements

The movements of stakes driven into the embankment shoulders were measured using a tape measure. The locations of the stakes are shown in Figure 5-4. Measurements were made of the longitudinal and lateral movement of the stakes as the abutment was expanded and contracted.

The longitudinal movement of the embankment stakes on the south and north sides of the approach slab are shown in Figures 5-35 and 5-36, respectively. It can be seen that cyclic movement of the stakes occurs as the abutment movement cycles. On the south side of the approach slab, the two stakes (stakes A and B) closest to the abutment show a longitudinal expansion and contraction that is consistent with the expansion and contraction of the abutment. The middle stake (stake C) shows very little movement. The two stakes farthest from the abutment (stakes D and E) show slight movement towards the abutment. On the north side of the approach slab the stakes closest to the abutment (stakes F, G, and H) show cyclic movements as the abutment moves. However, the stake closest to the abutment (stake F) shows movement generally towards the abutment, in contrast to stake A on the south side, where movement was generally away from the abutment. The two stakes farthest from the abutment show contrasting movements, with stake I indicating movement towards the abutment and stake J indicating movement away from the abutment. Although these longitudinal movements show cyclic compatibility with abutment movement in a directional sense, the magnitudes of the movements are much smaller than the abutment movements.

The lateral movements of the embankment stakes on the south and north side of the approach slab are shown in Figures 5-37 and 5-38, respectively. With the exception of the two stakes closest to the abutment (stakes A and F), only a small amount of lateral expansion occurred. Stakes A and F show markedly different behavior, with 0.25 inches of contraction for stake A and 0.6 inches of contraction for stake F. Such behavior is

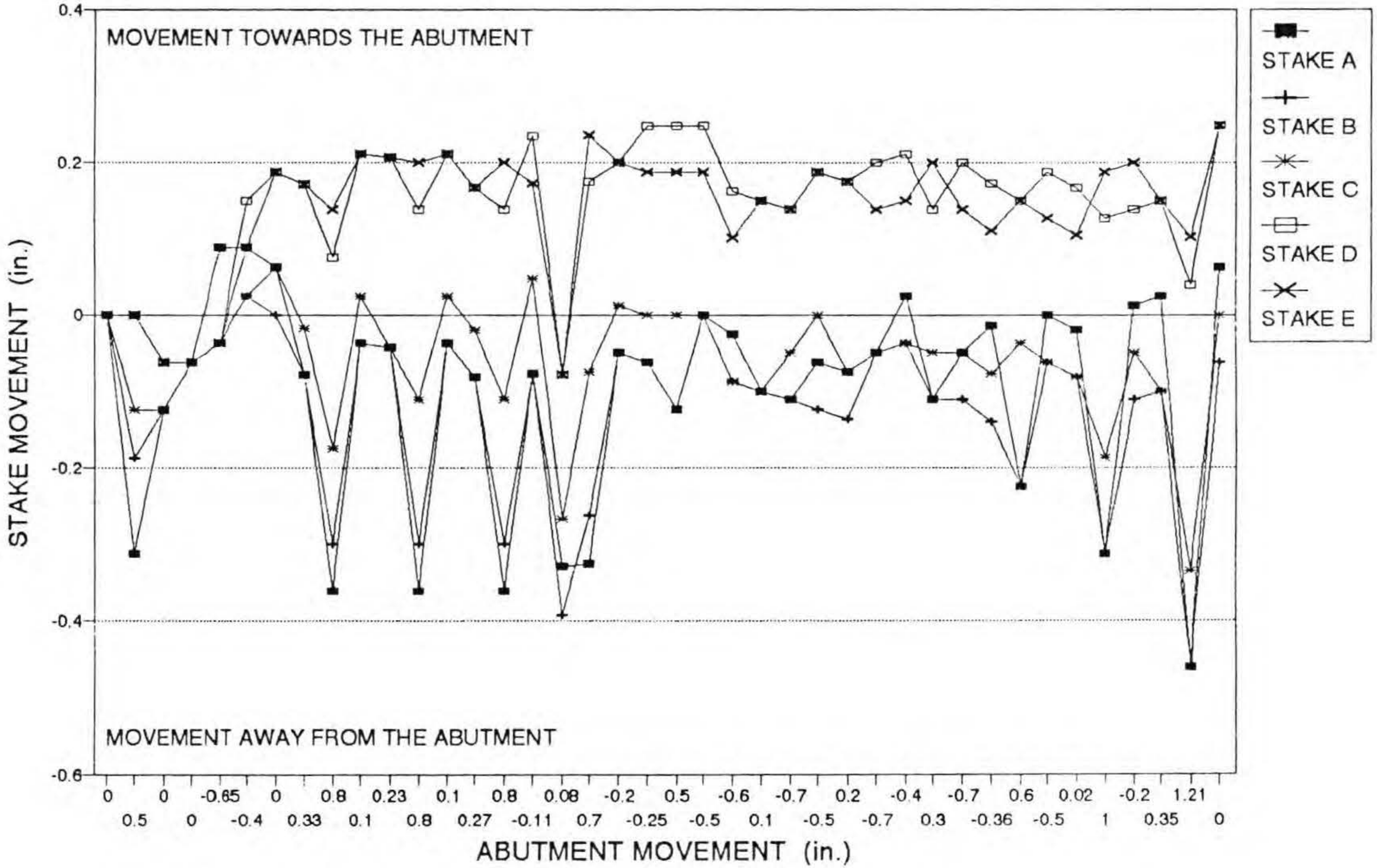



Figure 5-35. Longitudinal movement of embankment stakes on south side of approach slab.

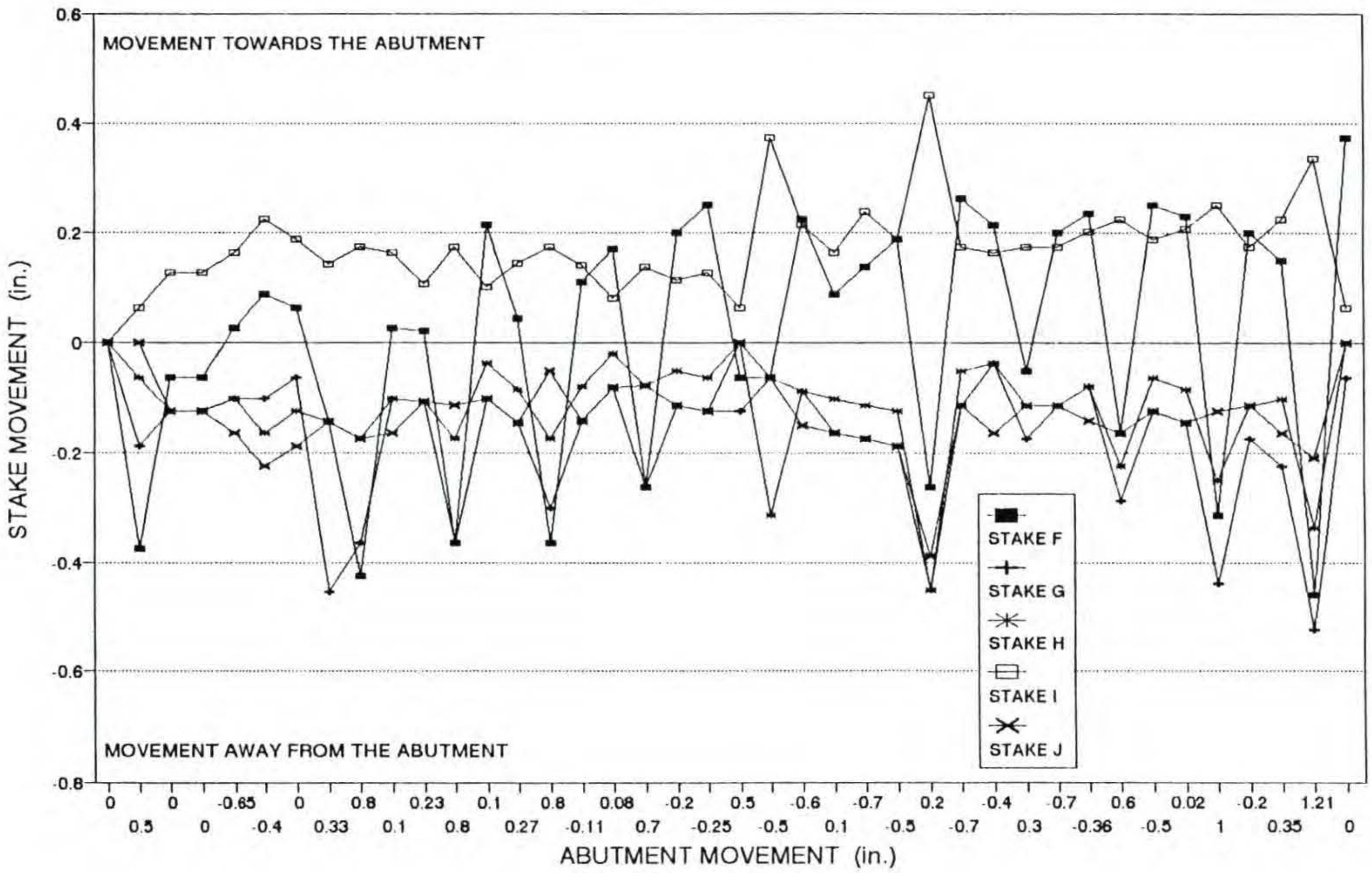



Figure 5-36. Longitudinal movement of embankment stakes on north side of approach slab.

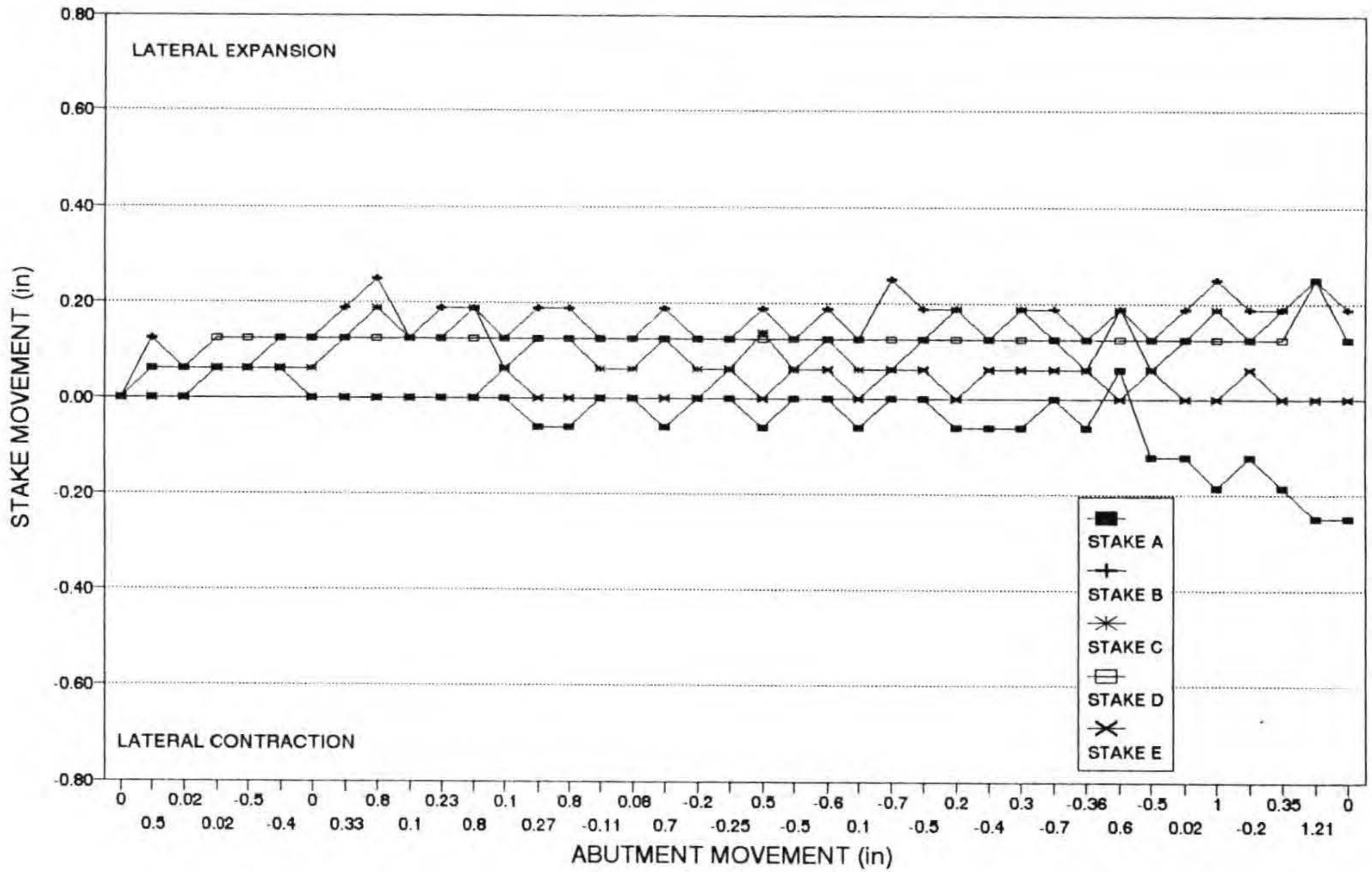



Figure 5-37. Lateral movement of embankment stakes on south side of approach slab.

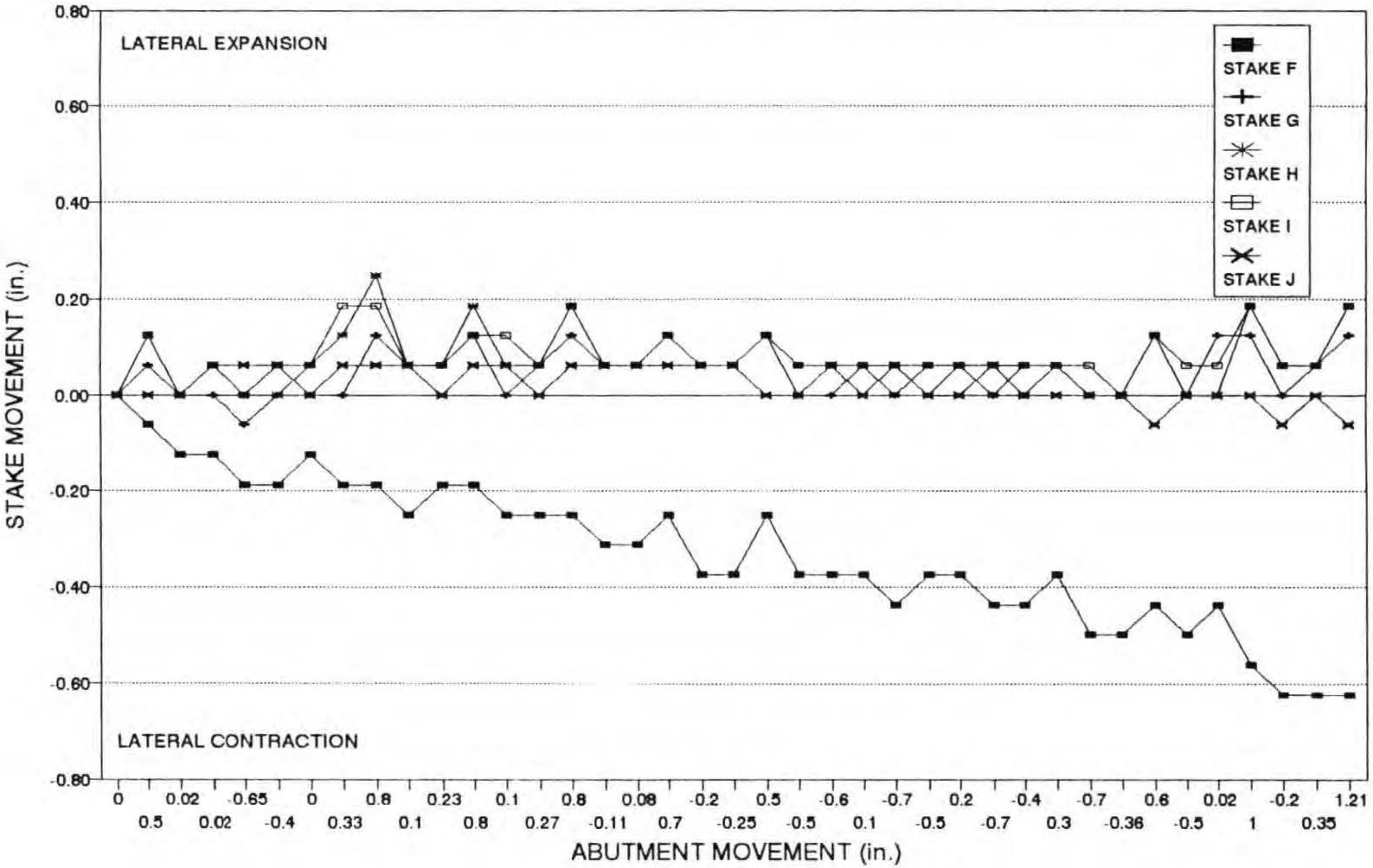



Figure 5-38. Lateral movement of embankment stakes on north side of approach slabs.

consistent with the observed behavior of the embankment near these stakes, where the embankment cracked and appeared to slough into the void created near the abutment. The longitudinal movement of stake F towards the abutment combined with large movement towards the approach slab reflects the conditions shown in Figure 5-13 where the large crack developed provides an opportunity for the stake to make such movements.

In general, the observed stake movements show the embankment soils to have laterally expanded during the test. However, the lateral expansion was relatively small and calculations of the volume due to expansion show less than one cubic foot of volume expansion could be attributed to embankment expansion.

### 5.9 Discussion and Summary

The model test results clearly show that cyclic movement of the abutment causes development of void space under the approach slab. The measurements of void development as the abutment cycles show that the void begins to develop immediately with cycling of the abutment. The size of the void cycles as the abutment movement expands and contracts, and the size of the void increases as the cycles continued. These data are consistent with field observations which show void development under almost all approach systems associated with integral abutment bridges. It also explains the observations made by DOT personnel and during this study of the development of void space under approach slabs before a structure is opened to traffic. These test results also show that void space development will likely continue, even after mudjacking. This is consistent with field observations of void space under approach slabs which have been mudjacked.

The earth pressure measurements show that the pressures induced to the backfill due to the temperature-induced abutment movement are sufficient to cause passive failure in the backfill. This justifies the design assumption of the use of passive pressures for designing the abutment. The development of passive failure zones in the backfill can lead to large displacements within the backfill. The residual pressures developed at the end of the test are consistent with plastic behavior of the backfill. The redistribution of stresses noted during the tests may be responsible for some of the void development as the embankment and foundation soils move in response to loading from the backfill and deform to adjust to the increased pressures.

The earth pressure measurements also show that large pressures develop in the backfill, pressures that can lead to particle breakage. Upon excavating the embankment and backfill material following the tests, it was noted that in the vicinity of the abutment and the inclinometer tubes, the backfill material appeared to be significantly finer, as though particle breakage had occurred. Several samples of the backfill were taken and gradation tests conducted on the material. The gradation tests show that backfill after the tests is indeed finer grained, but comparison of these results with the original pre-test gradation reveals that the post-test gradations are still within the original specification. The post-test gradation results show that about a five percent increase in the percent passing of each limit occurs during the test. This amount of increase is not considered significant.

The inclinometer movements show that little lateral deformation occurred during the cyclic tests. However, the small movements recorded do indicate that the lateral movements of the backfill occur, and may induce lateral bulging of the embankment soils. The small lateral movements recorded indicate that lateral expansion of the backfill may contribute some to the void development, but the contribution is likely small. The longitudinal inclinometer movements show significant levels of permanent deformation, on the order of two inches. Such magnitudes of movement would contribute substantially to the volume of void space developed under the approach slab. The cyclic results show that during each cycle some portion of the longitudinal deflection is recoverable, but that even large contraction cycles did not induce much recovery of the longitudinal deflection. The longitudinal deflection measured is compatible with the development of passive failure conditions in the backfill. The implications of no recovery of the longitudinal deflection are that cyclic behavior will continue to induce longitudinal deflection and consequent development of the void space. This is consistent with the observations of void development under approaches which have been mudjacked.

The approach slab movements appear to be fairly elastic in nature, with the upward movements induced during abutment expansion. During the last cycles the approach slab movement appears to become permanent. Even small movements upward of the approach slab yield a large volume of potential space under the approach slab which could translate into a void.

The embankment stake movement measurements showed that only small lateral expansion of the embankment soil takes place. This movement is consistent with the lateral movement measured within the backfill. Calculation of the volume represented by the measured movements show the volume to be relatively small at about one cubic foot.

The cracking patterns in embankment slopes along approach slabs noted during the field investigations were evident during the model test. Based on the field observation that this cracking was observed only at structures having integral abutments, and the occurrence of cracking during the model test, it is concluded that such cracking is related to the movements induced in the embankment soils due to the cycle movement of the abutment.

The patterns of void development, embankment distress, and erosion noted in the field occurred during the model study in nearly identical forms. The initial subsidence of the soil at the abutment/approach slab interface as the embankment soils attempted to fill the void created was noted during the model study. The embankment cracking detailed in Figures 5-12 to 5-15 are very similar to crack patterns observed in the field. The erosion patterns which developed at the model following heavy rains are also similar to erosion patterns observed in the field. Based on comparisons of model test results to observed field behavior, the model test is considered an accurate representation of field behavior. Thus, conclusions drawn from the model test are considered applicable to field conditions.

The measurements made during the cyclic tests show that lateral bulging of the embankment occurred, the approach slab moved upward, longitudinal movement of the backfill material occurred, and passive failure of the backfill material occurred, all in direct response to the cyclic movement of the abutment. Although the volume contribution of each of these effects might be small, cumulatively the total volume of the void developing under the approach slab can easily be accounted for by these effects. Thus, the void development under the approach slab in the model tests is a direct consequence of the effects of the abutment movement. The abutment movement is a direct consequence of temperature-induced deformation of the bridge beams which cause lateral expansion and contraction of the bridge abutments. Thus, the development of voids beneath approach slabs and distress to embankment soils at structures having integral abutments is a direct consequence of the integral nature of the bridge beams and the abutment.

# CHAPTER SIX
# SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

## 6.1 Summary

The results of the field studies, the review of the design methodology of the SDDOT and other Departments of Transportation, and the results of the model study provide a wealth of information. Below is a synopsis of the findings.

### 6.1.1 Field Studies

The field studies showed that the occurrence of void spaces under approach slabs is widespread at bridge structures in South Dakota having integral abutments. The occurrence of these voids was found in all geographic regions in the state, in very different climatological and geological environments, and at structures having very heavy and very light traffic loadings. Attempts to correlate void occurrence to a number of factors resulted in two key observations: the voids occurred primarily in structures having integral abutments and the void size increased as the length of the bridge increases. Void space under approach slabs at non-integral abutment bridges was noted, but much less frequently than at integral abutment structures. The relation of length of structure to void size was not absolute, as other factors affect the size of the void measured, including the time of measurement. However, in general, a longer structure would lead to the development of a larger void under approach slab.

Other problems at approaches were noted in the field studies: approach slab cracking, voids under the abutment, surface erosion, and cracking of the approach embankment and berm under the bridge. The approach slab cracking correlated highly to the occurrence of voids under the slab, leading to the conclusion that the cracking occurs as a result of loss of support from the underlying backfill. The berm and embankment cracking was noted as occurring only at sites having integral abutments.

The field studies showed that mudjacking as a remedial measure does not solve the problem, as many instances of voids occurring under approaches that had been mudjacked were observed. Indeed, during the course of the study it became apparent that mudjacking often leads to additional problems as excessive mudjacking pressures would raise the approach slab or affect the sleeper slab.

The extended field studies conducted did not shed additional light on the mechanisms causing the void space development. Attempts to correlate the behavior to soil properties were not successful. The soil studies did show that the approach foundation soils are generally overconsolidated and it is unlikely that the void development problems are a result of consolidation settlement. The level surveys conducted showed that the "bump at the end of the bridge" has largely been pushed out to the end of the approach, a fact which other transportation departments noted in the survey conducted.

### 6.1.2 Review of Design Practices

A review of design practices was conducted by surveying selected Departments of Transportation in the United States and by obtaining design information from SDDOT personnel. This review revealed that over 20 states currently use integral abutment bridge structures with approach slab systems to eliminate primarily joint maintenance problems. A common length of such structures is 300 feet, although lengths up to nearly 800 feet were reported. Most DOTs use concrete approach slabs on the primary systems. Settlement in the approach area has been a recurring problem for most states. There was much disagreement over the cause of the settlement and how to best prevent or deal with it. The causes of settlement were not readily delineated in the survey responses, which made it difficult to categorize the problems. The problem of void space under approach slabs was noted by several states, but its likely cause, if known, was not mentioned.

The present approach system which the SDDOT employs is typical of that used by other states. Only five respondents listed the gradation of the backfill used and these gradations correspond to the gradation used in South Dakota prior to 1982. The current South Dakota gradation is the coarsest in use. Little information was obtained on compaction requirements of the backfill, although it is usually listed as an item of concern.

### 6.1.3 Model Study

The model study showed that the development of void space under the approach slab is a direct consequence of the thermal-induced movement of the abutment system. The cyclic movement induced to the approach backfill system resulted in a void developing from the first expansion step and continuing as the cycling continued. The expansion-contraction steps were calibrated to temperature cycles for the 1989-1991 time period, resulting in varying amounts of movement of the abutment. Under the induced cyclic movements high earth pressures were measured in the backfill, large longitudinal movements were measured in the backfill, lateral movement of the embankment soils was measured, cracks developed in the approach embankments, and the approach slab raised up. The earth pressure cycles showed that passive earth pressure failure likely occurred in the backfill. All the measured effects could be correlated to the induced movements of the abutment.

One of the most important aspects of the model study is the degree of correspondence between the effects of the abutment movement on the backfill and embankments and the observation of those same features in the field studies. Such correspondence provides a large degree of confidence that the model study indeed replicated the field behavior.

## 6.2 Conclusions

The occurrence of voids under approach slabs in bridges in South Dakota is very high. Field studies conducted showed that the voids varied from less than 1 inch to as large as 14 inches. The field studies also showed a high correlation between void development and an integral abutment structure. A model study was conducted to determine the mechanism causing void development. Based on the results of the field studies and the model study, the mechanism of void space development under approach slabs is thermal-induced movements of integral abutments tied to bridge beams. This mechanism is also responsible for the development of cracks in berms and approach embankments as the abutment expands and contracts.

The void development is not isolated to one mechanism resulting from the abutment movement; it is a result of the cumulative effects of embankment bulging as the backfill deforms, approach slab uplift, backfill densification as particle breakage occurs, and backfill deformation as passive failure occurs in the backfill. The relative contribution of each of these mechanisms to void development is difficult to discern. The largest increases in void size occurred when passive failure likely occurred and this mechanism is probably the most important one. However, each of the effects contributes to the void development. The backfill material used is a strong and resilient material and the void development is not a result of improper backfill material. Thus, there is nothing inherently wrong with the material used as backfill.

The cracking of approach slabs is a direct consequence of the loss of support under the approach slab. Although the slab is designed to support traffic loads, the loss of support affects the flexural stresses induced in the slab. The transverse crack development is related to the large amount of reinforcement present at the abutment end of the approach slab, which allows stress concentrations to develop at that location.

Erosion of the approach embankments exacerbates the problems around the approach, but does not contribute significantly to void development. Erosion near the abutment/approach interface often allows water to flow into the void. The fabric wrap used around the backfill provides benefits by not allowing mixing of the backfill material with the embankment soils.

Mudjacking does not affect the mechanism causing the void development but only provides temporary support to the approach slab. Void development will continue under approach slabs which have been mudjacked until such time as the system comes to equilibrium with the cyclic movement.

### 6.3 Recommendations

The void development under approach slabs identified during this study would be most easily eliminated in future structures by returning to the use of non-integral abutment structures. However, this is unlikely because of the proven engineering and cost saving benefits provided by an integral abutment design. Therefore, the SDDOT must recognize that the elimination of one set of problems through the use of integral abutments can introduce another set of problems. There is virtually no way to eliminate the development of a void space under an approach slab when an integral abutment is part of the system. Thus, integral abutment bridge approach systems must be used with the recognition of increased maintenance costs associated with problems identified in this study. Below are outlined recommendations for maintenance measures at existing structures and changes to current bridge end backfill design practices to minimize problems at existing and new structures.

In existing structures the integrity of the approach embankments should receive increased attention to reduce the amount of erosion occurring. Specifically, the shoulder areas of the approach embankments should be capped with an asphalt concrete cover to reduce erosion and water flow into the backfill material. During the course of this study several SDDOT Area Offices were undertaking such measures. The cracking induced to approach embankments and berms by the expansion and contraction cycle provides a location for the formation of erosion rills. Such cracks will need increased attention to prevent serious erosion of these soils from occurring.

It must be recognized that the use of mudjacking does not alleviate the mechanism causing the formation of the void and that a void may form under the mudjacked approach. The development of a void at an existing structure should not automatically require that mudjacking be undertaken. Rather, the decision of whether to mudjack an approach should be based on the extent of the void and the amount and severity of traffic loadings. As noted in Chapter 4, an approach slab is designed to span an unsupported length of 15 feet without underlying support. Thus, it is recommended that mudjacking should occur when the void extends back 10 feet from the abutment. Such a distance provides a factor of safety of 1.5 on the unsupported length design. This distance is also compatible with the failure wedge observed in the backfill in the model tests. Mudjacking is also recommended if the void reaches a height of greater than four inches. This height is extrapolated from the model test results, which show a relationship between void development, and movements of the backfill and slab uplift. Those approaches not meeting these criteria should not be mudjacked as long as the approach shoulders are in good condition and the approach slab has not cracked. If an approach slab has cracked then mudjacking would be appropriate to provide support to minimize additional cracking. During mudjacking operations extreme care should be exercised to avoid the mudjack from flowing to the sleeper slab area. Those approach slabs which are in high traffic and high loading areas should be considered for mudjacking when the void space becomes larger than two inches in height, to reduce the likelihood of cracking to the slab due to heavy loading.

New approach systems in which integral abutments are used will develop voids under the approach slabs. The following recommendations are made to reduce the impact of such voids, reduce the cost of the approach system and reduce the maintenance costs of the approach system.

One of the major problems with the approach systems is the tendency of the slab to crack, primarily transversely about two to three feet from the abutment. The present reinforcement design of the approach slab allows stress to concentrate at this location. Therefore, the reinforcement of the approach slab should be redesigned to minimize the transverse cracking which occurs near the abutment/approach slab interface. Either the dowel bars tying the approach to the abutment should be decreased in size or a smoother transition of reinforcement across the approach slab should be designed to even out the stresses. Reducing the likelihood of crack development reduces the need to mudjack slabs to provide underlying support, thereby reducing future maintenance costs.

The model test results showed that void development is primarily a function of abutment movement and affects the approach embankment soils and the backfill material in close proximity to the abutment. Additionally, several problems were noted with sleeper slabs placed on the select backfill. Therefore, it is recommended that the slope of the cut made for backfill placement be changed to between 4H:1V to 2H:1V. The model test results showed the failure wedge and backfill movements to be confined to an area of about 1H:1V to 1.5H:1V behind the abutment, indicating little stability benefit is gained by flatter slopes. The steeper slope reduces the amount of select material required, places the sleeper slab on embankment material, and based on the results of the model test, should not affect the performance of the approach system. The impact of using steeper slopes is believed to be minimal; however, the impact could be verified by additional model testing.

The present material used as select backfill is a strong, resilient, free-draining coarse rock. While the material possesses good engineering properties, the mechanism which causes the void development is not affected significantly by these properties. It is believed that other free-draining materials available in the state could be used as backfill with similar results. Therefore, it is recommended that the gradation of the backfill material be changed to a slightly finer, more well-graded material, and the requirement of fractured faces be dropped.

The gradation of the material should be along the lines of the Colorado and New York gradation curves, with tight control on the amount of fines. The use of rounded, well-graded materials will provide a backfill having the necessary drainage properties as well as the needed elastic and strength properties. The use of alternative granular materials would greatly reduce the cost of the approach system by reducing the 'first' cost of the material itself and by opening the item up to bidding by additional materials producers.

Estimated potential savings of the present design versus the recommended design changes is shown below. The calculations assume an 8H:1V slope for the present geometry, a 3H:1V slope for the recommended geometry, and average unit prices described in Chapter 4. This recommendation is made on the basis of judgment considering the results of the model test and the expected behavior of other granular materials. The effect of the use of such materials could be verified by model testing. It is recommended that the use of the filter wrap be continued to prevent erosion and raveling of the granular materials and as a separator for future mudjacking.

| | Volume (cu yds.) | $/Bridge |
|---|---|---|
| Present geometry and present material specification | 430 | 7,800 |
| Recommended geometry and present material specification | 165 | 3,000 |
| Present geometry and recommended material specification | 430 | 3,100 |
| Recommended geometry and recommended material specification | 165 | 1,200 |

Finally, the recommendations concerning the backslope of the select backfill material and the material used for select backfill are based on engineering judgment based on the results of the field studies and the model test. It is recommended that additional testing using the model be conducted with the slope configuration suggested and with other backfill materials. Such testing would verify the acceptability of the recommended changes.

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# APPENDIX A

# FIELD INVESTIGATION DATA

SOUTH Dakota
Department of Transportation
700 Broadway Avenue East
Pierre, South Dakota 57501
MEMORANDUM

To: Region Engineers
Region Bridge Maintenance Engineers
Region Operations Engineers

From: Daniel Strand, Office of Research

Date: May 25, 1990

Subject: "Bridge End Backfill Study"

Early this winter a panel developed a Research Problem Statement for a study entitled "Bridge End Backfill Study" and recommended that the study be performed by an outside contractor. Around New Year's the Office of Research invited consulting firms and universities to submit proposals for this study. The panel for the research project met on March 8 to evaluate the proposals and select the researcher(s) to perform the work. With the panel's recommendation, the Research Review Board accepted Dr. Vernon R. Schaefer's proposal from SDSU.

Enclosed is the copy of the Problem Statement sent to each of the prospective researchers. Please read this to familiarize yourself with the problem.

Dr. Schaefer wants to start on the project by meeting with personnel from each region. In these meetings Dr. Schaefer wants to:

1. Identify some bridges that have bridge end backfill problems.

2. Identify some bridges that do not have bridge end backfill problems.

3. Identify any new construction, approach slab rehabilitation or approach maintenance that will take place this construction period.

4. Discuss construction and maintenance procedures.

Dr. Schaefer will possibly contact you within the next week to schedule a meeting. Please be thinking of any prospective bridges that may be used in this study. Contact me at 773-3871 if you have any questions.

con:beb007

**South Dakota Bridge End Backfill Study**

**Inspection Form**

Date _______________

Bridge Location

Structure Number ______________________________

Highway or Street No. ______________________________

Mile Marker Reference ______________________________

DOT Region, Area ______________________________

County ______________________________

Personnel on Inspection

______________________________

______________________________

______________________________

______________________________

1. Geology and Geography of the Site.

   Physiographic Division ____________________________________________

   General Geologic Description ________________________________________

   Description of Crossing: Wide valley _____ Narrow Valley _____ Uplands ______

   Stream Crossing: Wide _____ Narrow _____ Grade Separation _____

   Other____________________________________________________

   Brief Description of Landscape and Setting: (Are photos attached?_____ )

2. Type of Bridge ____________________________________________________

   General Condition of Bridge____________________ Drainage________________

   Brief Description and Sketch of Bridge: (Are photos attached? _____ )

3. Type of Abutment

   Stub Column _____ Pile End Bent _____ Open End Column _____ Step Tapered _____

   Integral _____ Spread Footing _____ Other _____

   Expansion Joint _____ Describe ______________________________________________

   Method of Drainage ______________________________________________

   Brief Description and Sketch of Abutment: (Are photos attached?_____ )

4. Foundation Soils.

   Soils Site Investigation: Yes _____ No _____ If yes, date: ________

   Swelling Soil: Yes _____ No _____ Treatment ____________________

   Any special soil improvement recommended at site? _____ If yes, describe:

   Description of foundation soils (classification, thickness, extent, etc.):

5. Approach Embankment Soils

   Soil Type ______________________________________________

   Fill or Cut Section ______________________________________________

   Height or thickness of fill above natural ground ______________________________

   Side slopes and condition ______________________________________________

   Compaction Specification ______________________________________________

   Lift Thickness ________ Compaction Equipment ______________________________

   Compaction Verified: Yes _____ No _____ Describe Method __________________

Special Back End Backfill: Yes _____ No _____ Gradation ____________________

Backfill Compaction Specification ______________________ Verified __________

Brief Description and Sketch of Embankment: (Are photos attached?_____ )

6. Approach Slab

   Type: Reinforced Concrete _____ Non-reinforced Concrete _____ Asphalt Concrete _____

   Sleeper Slab _____ Other ________________________________________

   Thickness ____________ Length __________________________________

   Condition of Approach Slab: (Describe crack patterns, dips and sags, measure differential movement between approach slab and bridge abutment and end of approach slab and pavement).

7. Pavement and Subgrade

   Pavement Type: JRCP _____ CRCP _____ ACP _____ Other _____

   Thickness of Pavement ________________________________________

   Description of Subgrade Layers ________________________________

   Description of Condition of Pavement ___________________________

8. Maintenance History

Frequency of Inspection________________________________________

Bridge Performance Evaluation__________________________________

Last Rehabilitation____________________________________________

Type of Rehabilitation_________________________________________

9. Traffic Design Criteria

Average Daily Traffic: Design __________ Actual __________ Date of ADT __________

Percent Truck ______ Percent Car ______ Speed Limit _______

| Struct# | Map # | YearBlt | YearPlans | DOTRegion | Highway | Location | BackType | ApprRate |
|---|---|---|---|---|---|---|---|---|
| 03-100-133 | 1 | 1980 | | Aberdeen | SD 14 | 2mi NW of Wolsey | 1 | 7 |
| 06-116-151 | 2 | 1979 | | Aberdeen | US 14 | 1.7mi E of Volga | 0 | |
| 07-125-330 | 3 | 1989 | | Aberdeen | SD 12 | 1.4mi E Jct US 281N | 0 | 9 |
| 07-222-329 | 4 | 1975 | | Aberdeen | SD 12 | 0.5mi S & 2.7mi E of Bath | 0 | 7 |
| 07-267-329 | 5 | 1975 | | Aberdeen | SD 12 | 3.2mi W Jct SD 37 | 0 | 6 |
| 09-284-080 | 6 | 1987 | 1986 | Mitchell | SD 34 | 0.7mi W Jct SD 45 | 2 | 7 |
| 10-310-399 | 7 | 1985 | | Rapid City | SD 79 | S of Newell over Belle Fouche | 2 | 6 |
| 14-100-200 | 8 | 1981 | 1979 | Mitchell | SD 50 | 1mi NW of Vermillion | 1 | 7 |
| 14-107-194 | 9 | 1977 | 1975 | Mitchell | SD 19 | 1.2mi N Jct SD 50 | 1 | 6 |
| 16-580-075 | 10 | 1980 | 1974 | Pierre | SD 63 | 1.8mi N of McLaughlin | 1 | 7 |
| 16-580-084 | 11 | 1980 | 1974 | Pierre | SD 63 | 0.9mi N of McLaughlin | 1 | 7 |
| 17-404-025 | 12 | 1986 | 1985 | Rapid City | SD 40 | 1.5mi W of Hermosa | 1 | 8 |
| 17-411-027 | 13 | 1986 | 1985 | Rapid City | SD 40 | 1.5mi W of Hermosa | 1 | 9 |
| 18-177-100 | 14 | 1988 | 1986 | Mitchell | SD 38 | 0.3mi W Hanson Co. Line | 3 | 8 |
| 18-180-100 | 15 | 1988 | 1986 | Mitchell | SD 38 | Davidson Hanson Co Line | 3 | 7 |
| 21-424-228 | 16 | 1988 | 1985 | Pierre | US 212 | 0.6mi E of Laplant | 2 | 9 |
| 24-162-058 | 17 | 1982 | | Rapid City | US 18 | 3.0mi S Jct 18 & 89 | 2 | 7 |
| 26-290-068 | 18 | 1982 | | Aberdeen | SD 15 | 0.7mi N Jct US 12 | 2 | 9 |
| 26-373-023 | 19 | 1983 | | Aberdeen | SD 109 | 0.1mi N Jct US 12 | 2 | 8 |
| 28-392-038 | 20 | 1981 | 1971 | Pierre | SD 63 | 1mi S Ziebach Haakon Co. Line | 1 | 7 |
| 31-001-106 | 21 | 1988 | | Mitchell | I 90 | Davidson Co. Line | 0 | 8 |
| 31-001-107 | 22 | 1988 | | Mitchell | I 90 | Davidson Co. Line | 0 | 8 |
| 33-286-020 | 23 | 1985 | 1984 | Pierre | US 14 | 0.4mi W of Blunt | 2 | 8 |
| 33-302-015 | 24 | 1985 | 1984 | Pierre | US 14 | 1.4mi NE of Blunt | 2 | 7 |
| 36-071-076 | 25 | 1987 | 1985 | Rapid City | I 90 | 6.3mi W of Cactus Flats | 2 | 7 |
| 36-071-077 | 26 | 1973 | | Rapid City | I 90 | 6.3mi W of Cactus Flats | 0 | 6 |
| 36-361-298 | 27 | 1980 | 1975 | Pierre | SD 44 | 5.2mi E Jct SD 73 | 1 | 8 |
| 36-366-300 | 28 | 1980 | 1975 | Pierre | SD 44 | 5.7mi E Jct SD73 | 1 | 8 |

| Struct# | Map # | YearBlt | YearPlans | DOTRegion | Highway | Location | BackType | ApprRate |
|---|---|---|---|---|---|---|---|---|
| 39-177-117 | 29 | 1988 | 1985 | Aberdeen | US 14 | 3.1mi E Jct SD 25 | 2 | 8 |
| 41-094-010 | 30 | 1986 | 1984 | Rapid City | US 85 | Redwater N of Spearfish | 2 | 8 |
| 41-214-098 | 31 | 1989 | 1988 | Rapid City | I 90 | 1mi E of Whitewood | 3 | 8 |
| 41-214-099 | 32 | 1989 | 1988 | Rapid City | I 90 | 1mi E of Whitewood | 3 | 8 |
| 42-064-093 | 33 | 1982 | 1981 | Mitchell | I 29 | 2.7mi N SD 44 Int. | 2 | 7 |
| 42-065-093 | 34 | 1982 | 1981 | Mitchell | I 29 | 2.7mi N SD 44 Int. | 2 | 7 |
| 42-066-006 | 35 | 1982 | 1980 | Mitchell | I 29 | Over I 229 Sioux Falls | 2 | 6 |
| 42-067-006 | 36 | 1982 | 1980 | Mitchell | I 29 | Over I 229 Sioux Falls | 2 | 6 |
| 42-162-140 | 37 | 1980 | 1977 | Mitchell | US 18 | 0.7mi W of Canton | 1 | 7 |
| 44-110-156 | 38 | 1982 | | Mitchell | US 81 | 3.2mi S I 90 | 1 | |
| 44-128-210 | 39 | 1983 | | Mitchell | SD 42 | 1.7mi E Jct US 81 | 1 | |
| 44-222-210 | 40 | 1984 | | Mitchell | SD 42 | 1.9mi W Minnehaha Co Line | 0 | 7 |
| 47-111-580 | 41 | 1980 | 1979 | Rapid City | I 90 | Stagebarn Canyon Interchange | 1 | 8 |
| 48-013-210 | 42 | 1980 | | Pierre | SD 44 | 0.6mi W Jct SD 63 | 1 | 8 |
| 48-022-211 | 43 | 1987 | 1985 | Pierre | SD 44 | 0.3mi E Jct SD 63 | 2 | 8 |
| 48-440-253 | 44 | 1989 | | Pierre | SD 53 | 0.6mi N Jct SD 44 | 2 | 8 |
| 48-485-271 | 45 | 1989 | 1988 | Pierre | SD 44 | 14.9mi W Jct US 183 | 3 | 8 |
| 49-168-130 | 46 | 1986 | | Mitchell | SD 34 | 6.3mi W Jct US 81 | 2 | |
| 50-020-141 | 47 | 1987 | 1986 | Mitchell | I 90 | SD 19 Interchange | 3 | 8 |
| 50-020-142 | 48 | 1987 | 1986 | Mitchell | I 90 | SD 19 Interchange | 3 | 8 |
| 50-119-165 | 49 | 1989 | 1987 | Mitchell | I 90 | SD 38 & I 90 Int. | 3 | 8 |
| 50-119-166 | 50 | 1989 | 1987 | Mitchell | I 90 | SD 38 & I 90 Int. | 3 | 7 |
| 50-125-168 | 51 | 1986 | | Mitchell | SD 38 | 0.6mi SE I90 & SD 38 Int. | 2 | |
| 50-161-210 | 52 | 1982 | 1981 | Mitchell | SD 42 | 1.4mi W I29 | 0 | 6 |
| 50-175-219 | 53 | 1989 | 1988 | Mitchell | I 29 | 0.9mi S SD 42 | 3 | 7 |
| 50-176-219 | 54 | 1989 | 1988 | Mitchell | I 29 | 0.9mi S SD 42 | 3 | 8 |
| 50-180-140 | 55 | 1989 | 1988 | Mitchell | FASC-6364 | 2.2mi N I90 Int. | 3 | 8 |
| 50-210-167 | 56 | 1986 | 1986 | Mitchell | I 90 | SD 115 Interchange | 2 | 8 |

| Struct# | Map # | YearBlt | YearPlans | DOTRegion | Highway | Location | BackType | ApprRate |
|---|---|---|---|---|---|---|---|---|
| 50-210-168 | 57 | 1986 | 1986 | Mitchell | I 90 | SD 115 Interchange | 2 | 8 |
| 50-286-180 | 58 | 1988 | | Mitchell | SD 264 | 0.5mi E Jct SD 11 | 3 | |
| 50-335-181 | 59 | 1989 | 1987 | Mitchell | SD 264 | 0.5mi W Minn. State Line | 3 | 8 |
| 51-110-143 | 60 | 1986 | | Mitchell | SD 34 | 4.5mi SW Jct SD 13 | 2 | |
| 51-129-130 | 61 | 1986 | | Mitchell | SD 34 | 2.1mi W Jct SD 13 | 2 | |
| 52-256-401 | 62 | 1987 | | Rapid City | US 16 | 1.2mi E of Hill City | 2 | 6 |
| 52-383-263 | 63 | 1973 | | Rapid City | I 90 | 0.6mi E Meade Co Line | 0 | 7 |
| 52-383-264 | 64 | 1959 | | Rapid City | I 90 | 0.6mi E Meade Co Line | 0 | 7 |
| 52-390-278 | 65 | 1983 | 1981 | Rapid City | I 90 | Deadwood Ave. Exchange | 2 | 7 |
| 52-424-285 | 66 | 1984 | | Rapid City | X I90 | Lacrosse Str. Interchange | 2 | 7 |
| 52-485-275 | 67 | 1982 | | Rapid City | I 90 | 0.6mi W of Box Elder Int. | 0 | 8 |
| 52-485-276 | 68 | 1982 | | Rapid City | I 90 | 0.6mi W of Box Elder Int. | 0 | 8 |
| 52-486-275 | 69 | 1982 | | Rapid City | I 90 | 0.5mi W of Box Elder Int. | 0 | 8 |
| 52-486-276 | 70 | 1982 | | Rapid City | I 90 | 0.5mi W of Box Elder Int. | 0 | 8 |
| 54-160-224 | 71 | 1989 | 1987 | Pierre | US 83 | 1.5mi N Sully Potter Co. line | 3 | 9 |
| 55-114-241 | 72 | 1982 | 1982 | Aberdeen | I 29 | 5.1mi S Jct SD 10 | 2 | 9 |
| 55-114-252 | 73 | 1981 | | Aberdeen | I 29 | 1.8mi NW of Peever | 1 | 9 |
| 55-115-220 | 74 | 1982 | 1981 | Aberdeen | X I 29 | 3mi S Jct 10 & 29 | 2 | 9 |
| 55-115-241 | 75 | 1982 | 1982 | Aberdeen | I 29 | 5.1mi S Jct SD 10 | 2 | 7 |
| 55-115-252 | 76 | 1981 | | Aberdeen | I 29 | 1.8mi NW of Peever | 1 | 9 |
| 55-115-256 | 77 | 1981 | | Aberdeen | I 29 | 1.4mi NW of Peever | 1 | 9 |
| 55-116-190 | 78 | 1982 | 1981 | Aberdeen | SD 10 | Jct of SD 10 & I90 | 2 | 8 |
| 55-116-256 | 79 | 1981 | | Aberdeen | I 29 | 1.4mi NW of Peever | 1 | 9 |
| 55-118-183 | 80 | 1982 | 1981 | Aberdeen | I 29S | 0.8mi N Jct 10 | 2 | 9 |
| 55-119-183 | 81 | 1982 | 1981 | Aberdeen | I 29N | 0.8mi N Jct 10 | 2 | 9 |
| 55-124-170 | 82 | 1982 | 1981 | Aberdeen | X I 29 | 2mi N Jct SD 10 & I29 | 2 | 9 |
| 55-139-140 | 83 | 1980 | | Aberdeen | I 29S | 5.5mi NE SD 10 | 1 | 7 |
| 55-140-140 | 84 | 1980 | | Aberdeen | I 29N | 5.5mi NE SD 10 | 1 | 9 |

| Struct# | Map # | YearBlt | YearPlans | DOTRegion | Highway | Location | BackType | ApprRate |
|---|---|---|---|---|---|---|---|---|
| 55-144-130 | 85 | 1980 | | Aberdeen | X I 29 | 6.5mi NE Jct SD 10 & I90 | 1 | 8 |
| 55-160-100 | 86 | 1980 | | Aberdeen | I 29 | 4.4mi S Jct SD 127 | 1 | 8 |
| 55-161-100 | 87 | 1980 | | Aberdeen | I 29 | 4.4mi S Jct SD 127 | 1 | 7 |
| 59-328-274 | 88 | 1984 | 1983 | Pierre | US 14 | 7.9mi W Jct US 83 | 2 | 8 |
| 59-398-295 | 89 | 1987 | 1985 | Pierre | US 83 | Within Fort Pierre | 2 | 8 |
| 59-493-328 | 90 | 1981 | 1979 | Pierre | SD 1806 | 10mi SE Jct US 83 | 1 | 7 |
| 61-182-044 | 91 | 1989 | 1984 | Pierre | US 18 | 9.2mi W Jct US 83N | 3 | 8 |
| 61-300-061 | 92 | 1981 | | Pierre | US 83 | 0.3mi S Jct US 18 | 1 | 8 |
| 62-093-230 | 93 | 1988 | 1987 | Pierre | SD 44 | 1.7mi W US183 | 3 | 8 |
| 62-235-518 | 94 | 1983 | | Pierre | US 183 | 2.2mi N Nebraska State Line | 0 | 8 |
| 63-140-034 | 95 | 1989 | 1988 | Mitchell | SD 19 | 3.7mi N Jct SD 44 | 3 | 8 |
| 64-006-090 | 96 | 1988 | 1986 | Mitchell | X I 29 | 9.0mi S Jct SD 46 & I29 | 2 | 6 |
| 64-060-241 | 97 | 1983 | | Mitchell | SD 11 | 1.9mi S Jct SD 50 | 1 | 7 |
| 64-154-385 | 98 | 1981 | | Mitchell | I 29 | N Sioux City Int. | 0 | 6 |
| 64-155-385 | 99 | 1981 | | Mitchell | I 29 | N Sioux City Int. | 0 | 6 |
| 68-180-199 | 100 | 1976 | 1973 | Mitchell | SD 50E | 6.2mi E Jct US 81 | 1 | 7 |
| 68-180-200 | 101 | 1985 | 1984 | Mitchell | SD 50E | 6.2mi E Jct US 81 | 2 | 8 |
| 69-220-289 | 102 | 1984 | 1983 | Pierre | SD 65 | 0.2mi N Jct US 212 | 2 | 8 |
| 69-260-092 | 103 | 1984 | 1983 | Pierre | SD 65 | 3.1mi S Jct SD 20 | 2 | 7 |
| 69-390-535 | 104 | 1981 | 1971 | Pierre | SD 63 | Ziebach Haakon County Line | 0 | 6 |

| Struct# | ApprType | AbutType | Skew | Length | ADT | VoidAbut | VoidSizeAbut |
|---|---|---|---|---|---|---|---|
| 03-100-133 | Reinforced Concrete | Integral | 25.000 | 128.20 | 2300 | No | Some settlement at piles N. abut. |
| 06-116-151 | Reinforced Concrete | Integral | | 100.00 | | No | Cracking and sloughing both toes |
| 07-125-330 | Reinforced Concrete | Integral | 30.000 | 119.30 | 13443 | No | None uncovered |
| 07-222-329 | Reinforced Concrete | Integral | 15.000 | 240.10 | 1931 | Yes | Loose to 2 inches, sloughing or erosion |
| 07-267-329 | Asphalt | Non-Integral | 0.000 | 831.00 | 1610 | Yes | W. 2 to 4" w/11"x3' hole,E. loose |
| 09-284-080 | Reinforced Concrete | Integral | 0.000 | 209.00 | 210 | Yes | W. 1 to 3",E. 0,settlement on berms |
| 10-310-399 | Reinforced Concrete | Integral | 25.000 | 296.00 | 605 | Yes | S. 1 to 2",N. none uncovered |
| 14-100-200 | Reinforced Concrete | Non-Integral | 0.000 | 372.00 | 1535 | Yes | SE corner due to erosion |
| 14-107-194 | Reinforced Concrete | Integral | 0.000 | 267.00 | 1405 | Yes | Exposed piles N. Abut.,S. loose to 1" |
| 16-580-075 | Reinforced Concrete | Integral | 38.000 | 194.00 | 346 | No | Steep toes w/ minor erosion |
| 16-580-084 | Reinforced Concrete | Integral | 20.000 | 158.00 | 346 | No | Erosion on toes severe in front of WW |
| 17-404-025 | Reinforced Concrete | Integral | 0.000 | 102.00 | 190 | No | Full depth abutments |
| 17-411-027 | Reinforced Concrete | Integral | 45.000 | 210.00 | 190 | Yes | SW corner 2 to 3" |
| 18-177-100 | Reinforced Concrete | Integral | 0.000 | 178.00 | 1765 | Yes | E. 2 to 4",W. large near piles |
| 18-180-100 | Reinforced Concrete | Integral | 0.000 | 308.00 | 1790 | Yes | 6 to 12" both sides largest around piles |
| 21-424-228 | Reinforced Concrete | Integral | 0.000 | 119.00 | 365 | No | Toes steep and loose |
| 24-162-058 | Reinforced Concrete | Non-Integral | 0.000 | 804.00 | 1637 | No | Steep toes good condition |
| 26-290-068 | Reinforced Concrete | Integral | 25.000 | 106.00 | 985 | Yes | Visible under Rip Rap Piles visible 4" |
| 26-373-023 | Reinforced Concrete | Integral | 0.000 | 205.00 | 900 | Yes | Void by buried water main |
| 28-392-038 | Reinforced Concrete | Integral | 15.000 | 216.00 | 270 | No | Sloughing E. toe,Cracking W. toe |
| 31-001-106 | Reinforced Concrete | Non-Integral | 0.000 | 753.00 | 3530 | | |
| 31-001-107 | Reinforced Concrete | Non-Integral | 0.000 | 753.00 | 3530 | | |
| 33-286-020 | Reinforced Concrete | Integral | 0.000 | 99.50 | 1172 | Yes | W. 1 to 2",E. none,Sloughing E. end |
| 33-302-015 | Reinforced Concrete | Integral | 0.000 | 112.50 | 856 | No | Movement between berm & abut. 1/2 to 1" |
| 36-071-076 | Reinforced Concrete | Integral | 0.000 | 202.00 | 1720 | Yes | E. loose,W. 1.5 to 4",cracking on toe |
| 36-071-077 | Reinforced Concrete | Integral | 0.000 | 230.00 | 1720 | Yes | E. 4", W. 6", Piles visible both sides |
| 36-361-298 | Reinforced Concrete | Integral | 0.000 | 197.00 | 170 | Yes | Both sides 1 to 2" largest around piles |
| 36-366-300 | Reinforced Concrete | Integral | 0.000 | 117.00 | 170 | Yes | Both sides 1 to 2" largest around piles |

| Struct# | ApprType | AbutType | Skew | Length | ADT | VoidAbut | VoidSizeAbut |
|---|---|---|---|---|---|---|---|
| 39-177-117 | Reinforced Concrete | Integral | 20.000 | 96.40 | 1555 | No | Toes good,tight against abut. |
| 41-094-010 | Reinforced Concrete | Integral | 0.000 | 160.00 | 4000 | Yes | 8 inches SE corner |
| 41-214-098 | Reinforced Concrete | Non-Integral | 45.000 | 136.00 | 3710 | No | New rehab |
| 41-214-099 | Reinforced Concrete | Non-Integral | 45.000 | 136.00 | 3710 | No | New rehab |
| 42-064-093 | Reinforced Concrete | Integral | 0.000 | 192.00 | 4745 | Yes | In center both sides 1'x2' wide,Erosion? |
| 42-065-093 | Reinforced Concrete | Integral | 0.000 | 192.00 | 4745 | Yes | Both sides east corners 1 to 3",Erosion? |
| 42-066-006 | Reinforced Concrete | Non-Integral | 0.000 | 365.30 | 6090 | No | Steep,asphalt on toes |
| 42-067-006 | Reinforced Concrete | Non-Integral | 0.000 | 365.30 | 6090 | No | Steep,asphalt on toes |
| 42-162-140 | Reinforced Concrete | Integral | 0.000 | 132.00 | 4990 | Yes | E. 1 to 2",W. none uncovered |
| 44-110-156 | Reinforced Concrete | Integral | 0.000 | 113.00 | | No | Cracking S. toe,settlement apparent |
| 44-128-210 | Reinforced Concrete | Integral | 0.000 | 99.50 | | Yes | Voids W. can feel with rod,E. 2-5" |
| 44-222-210 | Reinforced Concrete | Integral | 0.000 | 106.00 | 397 | Yes | East end,2 to 3 in. large hole NE corner |
| 47-111-580 | Reinforced Concrete | Integral | 36.183 | 234.00 | 33 | No | Cracking on toes |
| 48-013-210 | Reinforced Concrete | Integral | 25.000 | 265.00 | 118 | No | Cracking & Sloughing on W. toe |
| 48-022-211 | Reinforced Concrete | Integral | 0.000 | 150.50 | 390 | Yes | Both sides 1 to 2" |
| 48-440-253 | Gravel | Integral | 0.000 | 83.30 | 50 | No | Erosion on outer reaches of toe |
| 48-485-271 | Reinforced Concrete | Integral | 20.000 | 130.10 | 99 | No | Minor erosion on toes |
| 49-168-130 | Reinforced Concrete | Integral | | 96.00 | | No | Settlement along abut.,cracking both toes |
| 50-020-141 | Reinforced Concrete | Non-Integral | 45.000 | 189.30 | 3785 | No | Hot tar stabilization on toes,erosion |
| 50-020-142 | Reinforced Concrete | Non-Integral | 45.000 | 189.30 | 3785 | No | Hot tar stabilization on toes,erosion |
| 50-119-165 | Reinforced Concrete | Integral | 35.000 | 239.30 | 4200 | No | Slope failure on toes and appr. emb. |
| 50-119-166 | Reinforced Concrete | Integral | 35.000 | 239.30 | 4200 | No | Slope failure on toes and appr. emb. |
| 50-125-168 | Reinforced Concrete | Integral | | 192.00 | | No | Minor erosion on toes |
| 50-161-210 | Reinforced Concrete | Integral | 0.000 | 113.00 | 4660 | No | Full depth abutment on east |
| 50-175-219 | Reinforced Concrete | Integral | 25.000 | 284.20 | 11025 | Yes | Both sides 2 to 3" at ends |
| 50-176-219 | Reinforced Concrete | Integral | 25.000 | 284.20 | 11025 | Yes | Both sides 2 to 3" at ends |
| 50-180-140 | Reinforced Concrete | Integral | 10.000 | 252.00 | 1062 | No | Made Integral (1989),sloughing on toes |
| 50-210-167 | Reinforced Concrete | Integral | 0.000 | 182.00 | 6700 | No | Concrete west berm,erosion east toe |

| Struct# | ApprType | AbutType | Skew | Length | ADT | VoidAbut | VoidSizeAbut |
|---|---|---|---|---|---|---|---|
| 50-210-168 | Reinforced Concrete | Integral | 0.000 | 182.00 | 6700 | No | Concrete west berm,erosion east toe |
| 50-286-180 | Reinforced Concrete | Integral | 0.000 | 217.00 | | No | Cracking on E. toe,settement along abut. |
| 50-335-181 | Reinforced Concrete | Integral | 0.000 | 100.00 | 1215 | Yes | W. large under corners,E. none found |
| 51-110-143 | Reinforced Concrete | Integral | | 100.00 | | Yes | Both sides 1-2" largest at piles |
| 51-129-130 | Reinforced Concrete | Integral | | 226.00 | | No | Rip Rap high |
| 52-256-401 | Reinforced Concrete | Integral | 2.082 | 107.10 | 3925 | Yes | 2 to 8 inches,larger on uphill side |
| 52-383-263 | Reinforced Concrete | Integral | 0.000 | 306.00 | 4840 | No | None |
| 52-383-264 | Reinforced Concrete | Integral | 0.000 | 306.00 | 4840 | No | None |
| 52-390-278 | Reinforced Concrete | Integral | 35.000 | 251.00 | 5150 | Yes | 1 1/2 to 2 inches,sloughing & cracking |
| 52-424-285 | Reinforced Concrete | Integral | 0.000 | 223.00 | 13779 | No | Cracking on toes |
| 52-485-275 | Reinforced Concrete | Integral | 0.000 | 242.00 | 7515 | No | Concrete breakup on front slope |
| 52-485-276 | Reinforced Concrete | Integral | 0.000 | 242.00 | 7515 | No | Concrete breakup on front slope |
| 52-486-275 | Reinforced Concrete | Integral | 30.000 | 162.00 | 7515 | No | Steep toes good condition |
| 52-486-276 | Reinforced Concrete | Integral | 30.000 | 162.00 | 7515 | No | Steep toes good condition |
| 54-160-224 | Reinforced Concrete | Integral | 15.000 | 89.80 | 1115 | No | N. toe cracking,low fill |
| 55-114-241 | Reinforced Concrete | Integral | 10.000 | 137.00 | 1110 | No | Toes good condition |
| 55-114-252 | Reinforced Concrete | Integral | 40.000 | 94.00 | 1110 | No | Cracking on toes |
| 55-115-220 | Reinforced Concrete | Integral | 0.000 | 292.00 | 68 | No | Minor erosion on toes,vaulted abut. |
| 55-115-241 | Reinforced Concrete | Integral | 10.000 | 137.00 | 1110 | No | Cracking both toes |
| 55-115-252 | Reinforced Concrete | Integral | 40.000 | 94.00 | 1110 | No | Cracking on toes |
| 55-115-256 | Reinforced Concrete | Integral | 40.000 | 94.00 | 1110 | No | Cracking on toes |
| 55-116-190 | Reinforced Concrete | Integral | 19.577 | 342.00 | 960 | No | Good condition |
| 55-116-256 | Reinforced Concrete | Integral | 40.000 | 94.00 | 1110 | No | Cracking on toes |
| 55-118-183 | Reinforced Concrete | Integral | 15.000 | 147.00 | 925 | No | Settlement around piles |
| 55-119-183 | Reinforced Concrete | Integral | 15.000 | 147.00 | 925 | No | Settlement around piles NW corner |
| 55-124-170 | Reinforced Concrete | Integral | 18.150 | 330.00 | 16 | Yes | Settlement both abut. & 2" voids E. abut. |
| 55-139-140 | Reinforced Concrete | Integral | 0.000 | 80.00 | 925 | No | Low fill embankments good |
| 55-140-140 | Reinforced Concrete | Integral | 0.000 | 80.00 | 925 | No | Low fill embankments good |

| Struct# | ApprType | AbutType | Skew | Length | ADT | VoidAbut | VoidSizeAbut |
|---|---|---|---|---|---|---|---|
| 55-144-130 | Reinforced Concrete | Integral | 26.000 | 354.00 | 79 | Yes | Settlement both abut., 6" void E. Abut. |
| 55-160-100 | Reinforced Concrete | Integral | 29.000 | 139.00 | 960 | Yes | Settlement along abut., cracking on toe |
| 55-161-100 | Reinforced Concrete | Integral | 29.000 | 139.00 | 960 | No | Settlement along abutment |
| 59-328-274 | Reinforced Concrete | Integral | 0.000 | 129.00 | 950 | No | Cracking on W. toe,some erosion on both |
| 59-398-295 | Reinforced Concrete | Non-Integral | 0.000 | 434.00 | 4865 | No | Slight erosion on toes |
| 59-493-328 | Gravel | Integral | 20.000 | 119.00 | 70 | No | Erosion on toes |
| 61-182-044 | Reinforced Concrete | Integral | 0.000 | 141.50 | 465 | Yes | Both abut. 2 to 3" |
| 61-300-061 | Reinforced Concrete | Integral | 0.000 | 112.00 | 1095 | No | Reinforced toes both sides |
| 62-093-230 | Reinforced Concrete | Integral | 0.000 | 90.00 | 365 | No | Settlement on both berms, cracking W. toe |
| 62-235-518 | Reinforced Concrete | Integral | 25.000 | 122.70 | 295 | No | Concrete berm N. side |
| 63-140-034 | Reinforced Concrete | Integral | 30.000 | 157.30 | 780 | No | Sloughing both toes,cracking S. toe |
| 64-006-090 | Reinforced Concrete | Integral | 0.000 | 252.00 | 105 | No | Some erosion & cracking on F.slopes |
| 64-060-241 | Asphalt | Integral | 30.000 | 70.00 | 640 | No | Some settlement & erosion |
| 64-154-385 | Asphalt | Integral | 0.000 | 126.00 | 4960 | Yes | Large hole S. Abut.,settlement visible |
| 64-155-385 | Asphalt | Integral | 0.000 | 126.00 | 4960 | No | Settlement visible along abut. face |
| 68-180-199 | Reinforced Concrete | Non-Integral | 20.000 | 328.00 | 1785 | No | Long front slope berms, 30 ft. |
| 68-180-200 | Reinforced Concrete | Integral | 20.000 | 326.10 | 1785 | No | Cement treated front berms |
| 69-220-289 | Reinforced Concrete | Integral | 0 | 77.00 | 200 | No | Sloughing S. toe, cracking both toes |
| 69-260-092 | Reinforced Concrete | Integral | 25.000 | 93.00 | 200 | No | Steep toes,minor erosion |
| 69-390-535 | Reinforced Concrete | Non-Integral | 0.000 | 2109.00 | 295 | No | Both toes good |

| Struct# | VoidAppr | VoidE&S | VoidW&N | ApprComments | Mudjacked |
|---|---|---|---|---|---|
| 03-100-133 | Yes | 1.5 | 0.5 | Long.cracks both sides & 2nd appr.slab | No |
| 06-116-151 | Yes | 0 | 0 | Long. & trans. cracks | Yes |
| 07-125-330 | No | 0 | 0 | Crack E. end CL per. to skew | No |
| 07-222-329 | Yes | 6 | 1.5 | Minor cracking both ends | No |
| 07-267-329 | Yes | 2 | 4 | Asphalt settlement | No |
| 09-284-080 | Yes | 5 | 5.5 | H/L cracks developing | No |
| 10-310-399 | Yes | 7 | 8 | Good condition no cracks | No |
| 14-100-200 | Yes | 0 | 0 | Excessive grout,trans. & Long Cracking | Yes |
| 14-107-194 | Yes | 4 | 14 | Hole clear through under N. appr. | No |
| 16-580-075 | Yes | 3.5 | 2.5 | Diagonal cracks N & S appr. | No |
| 16-580-084 | Yes | 2.5 | 2.5 | Long. cracks both sides | No |
| 17-404-025 | Yes | 0.5 | 0.5 | Both sides loose to 1" | No |
| 17-411-027 | Yes | 2 | 4 | Good condition no cracks | No |
| 18-177-100 | No | 0 | 0 | Long. cracks both sides | No |
| 18-180-100 | Yes | 0 | 0 | filled,Appr.cracks long. & trans. | Yes |
| 21-424-228 | Yes | 0 | 0.5 | Good condition no cracking | No |
| 24-162-058 | Yes | 0 | 0 | Mudjacked,approx. 8" mudjack NE corner | Yes |
| 26-290-068 | Yes | 1.5 | 0.5 | N. looser at edges,long. crack,S. no cracks | No |
| 26-373-023 | No | 0 | 0 | Voids unknown due to sidewalk | No |
| 28-392-038 | Yes | 2 | 2 | No cracking,appr. embank. settled 6" | No |
| 31-001-106 | No | 0 | 0 | Trans & long cracks,to be inspected | No |
| 31-001-107 | No | 0 | 0 | Trans & long cracks,to be inspected | No |
| 33-286-020 | Yes | 2.25 | 3 | Long. crack E. end WB lane CL | No |
| 33-302-015 | Yes | 2.5 | 2 | Good condition no cracking | No |
| 36-071-076 | Yes | 2 | 5 | Long. cracking W. end,pavement blowup E.end | No |
| 36-071-077 | Yes | 0 | 0 | Mudjacked,Trans. cracking both sides | Yes |
| 36-361-298 | Yes | 10 | 7 | HL long. crack CL W. appr. both lanes | No |
| 36-366-300 | Yes | 6.5 | 6.5 | HL trans. crack CL both sides | No |

| Struct# | VoidAppr | VoidE&S | VoidW&N | ApprComments | Mudjacked |
|---|---|---|---|---|---|
| 39-177-117 | Yes | 0.5 | 2.5 | Trans. cracks both sides,long. crack W | No |
| 41-094-010 | Yes | 3 | 3 | No cracking, expansion jnt. swell | No |
| 41-214-098 | No | 0 | 0 | New rehab good condition | No |
| 41-214-099 | No | 0 | 0 | New rehab good condition | No |
| 42-064-093 | Yes | 0 | 0 | Mudjacked no cracks | Yes |
| 42-065-093 | Yes | 1 | 0 | Mudjacked,no cracks,void SE corner | Yes |
| 42-066-006 | Yes | 0.5 | 1 | Under MJ 1" close to abut. | Yes |
| 42-067-006 | Yes | 0.5 | 1 | Under MJ 1" close to abut. | Yes |
| 42-162-140 | Yes | 0 | 0 | Mudjacked,cracking,voids under sidewalk | Yes |
| 44-110-156 | Yes | 2.5 | 4 | No cracks,settlement along appr. emb. | No |
| 44-128-210 | Yes | 4 | 1.5 | No cracks,embankments good | No |
| 44-222-210 | Yes | 5 | 0.5 | Long. cracks E.,Trans. crack W. | No |
| 47-111-580 | Yes | 5 | 5 | Good condition chip seal on appr. | No |
| 48-013-210 | Yes | 1.5 | 0.5 | Voids under MJ,trans. crack W. appr. | Yes |
| 48-022-211 | Yes | 3.5 | 3.5 | W. diagonal crack,E. trans & long. cracks | No |
| 48-440-253 | No | 0 | 0 | Gravel appr. heavy erosion | No |
| 48-485-271 | Yes | 1 | 1 | HL crack perpendicular skew E. | No |
| 49-168-130 | Yes | 3 | 3 | Long. crack CL E.,No cracks W. | No |
| 50-020-141 | No | 0 | 0 | Diagonal cracking N. lane E. & W. | No |
| 50-020-142 | No | 0 | 0 | Diagonal cracking S. lane W. & N. lane E. | No |
| 50-119-165 | Yes | 2 | 2 | Voids before open to traffic | No |
| 50-119-166 | Yes | 2 | 2 | Voids before open to traffic,diag. cracking | No |
| 50-125-168 | Yes | 4 | 4 | Voids visible,long. cracking | No |
| 50-161-210 | Yes | 3 | 3.5 | Trans. cracking both sides | No |
| 50-175-219 | Yes | 5 | 4 | Good condition no cracking | No |
| 50-176-219 | Yes | 3.5 | 5 | Good condition no cracking | No |
| 50-180-140 | Yes | 2 | 2 | Asphalt seal on appr. emb.,estimated void | No |
| 50-210-167 | Yes | 0.5 | 0.5 | Loose to 1" no cracking | No |

| Struct# | VoidAppr | VoidE&S | VoidW&N | ApprComments | Mudjacked |
|---|---|---|---|---|---|
| 50-210-168 | Yes | 0.5 | 0.5 | Loose to 1" no cracking | No |
| 50-286-180 | Yes | 4 | 6 | Long cracks both sides | No |
| 50-335-181 | Yes | 2.5 | 2.5 | H/L long. cracks both sides | No |
| 51-110-143 | Yes | 2 | 2.5 | No cracks | No |
| 51-129-130 | Yes | 5.5 | 4.5 | Appr./pavement joint rough | No |
| 52-256-401 | Yes | 0 | 0 | Long. and trans. cracking both sides | Yes |
| 52-383-263 | No | 0 | 0 | Scheduled for appr. slab replacement | No |
| 52-383-264 | No | 0 | 0 | Scheduled for appr. slab replacement | No |
| 52-390-278 | Yes | 4 | 4 | Trans. cracks S. appr.,chip seal on appr. | No |
| 52-424-285 | Yes | 0 | 0 | Mudjacked | Yes |
| 52-485-275 | Yes | 0 | 0 | Mudjacked,minor cracks | Yes |
| 52-485-276 | Yes | 0 | 0 | Mudjacked,minor cracks | Yes |
| 52-486-275 | Yes | 0 | 0 | filled,cracks | Yes |
| 52-486-276 | Yes | 0 | 0 | filled,cracks | Yes |
| 54-160-224 | Yes | 2 | 2.5 | S. long crack SB lane | No |
| 55-114-241 | Yes | 0.5 | 0.5 | Good condition no cracks | No |
| 55-114-252 | No | 0 | 0 | Long. crack N.appr. | No |
| 55-115-220 | Yes | 0.5 | 0.5 | Tightens as probe goes in,no cracking | No |
| 55-115-241 | Yes | 0 | 0 | Looks like sleeper settled | Yes |
| 55-115-252 | Yes | 0 | 0 | Cracking trans & long. second appr. slab | Yes |
| 55-115-256 | Yes | 0.5 | 0.5 | No cracks,backfill loose | No |
| 55-116-190 | Yes | 1 | 0 | Mudjacked w/ concrete?, no cracking | Yes |
| 55-116-256 | Yes | 1 | 0 | Long. crack N. appr.,Void under SE corner | Yes |
| 55-118-183 | Yes | 0.5 | 0.5 | Good condition no cracking | No |
| 55-119-183 | Yes | 0.5 | 0.5 | Good condition no cracking | No |
| 55-124-170 | Yes | 0.5 | 0 | Mudjacked,loose 4 ft back on S. appr. | Yes |
| 55-139-140 | Yes | 1 | 1 | Tightens as rod goes in, no cracking | No |
| 55-140-140 | Yes | 1 | 1 | Tightens as rod goes in, no cracking | No |

| Struct# | VoidAppr | VoidE&S | VoidW&N | ApprComments | Mudjacked |
|---|---|---|---|---|---|
| 55-144-130 | Yes | 0 | 0 | Mudjacked,no cracks | Yes |
| 55-160-100 | Yes | 0.5 | 0.5 | Random cracking,void at outer edges | No |
| 55-161-100 | Yes | 1 | 1 | Random cracking,extensive void S. side | No |
| 59-328-274 | Yes | 2 | 2 | No cracking, void at edges tight 5' in | No |
| 59-398-295 | No | 0 | 0 | Able to dig one corner no voids found | No |
| 59-493-328 | No | 0 | 0 | Gravel appr. freshly graded | No |
| 61-182-044 | Yes | 2 | 2 | Asphalt on shoulder could estimate with rod | No |
| 61-300-061 | No | 0 | 0 | Unable to dig voids not apparent | No |
| 62-093-230 | Yes | 3.5 | 3.5 | Good condition no cracks | No |
| 62-235-518 | Yes | 0.5 | 0.5 | Long. cracking CL both sides | No |
| 63-140-034 | Yes | 1.5 | 4 | Good condition no cracking | No |
| 64-006-090 | Yes | 4 | 5 | Trans. crack E.,HL long. crack W.NB CL | No |
| 64-060-241 | No | 0 | 0 | S.- Both gutters broke and settled | No |
| 64-154-385 | Yes | 0 | 2 | Under curb,Small void visible | Yes |
| 64-155-385 | Yes | 3 | 8 | Under curbs,large void NE curb | Yes |
| 68-180-199 | Yes | 0 | 0 | Random cracking, excessive grout on shoulder | Yes |
| 68-180-200 | Yes | 0 | 0 | Cracks around grout holes,excessive grout | Yes |
| 69-220-289 | Yes | 1.5 | 1.5 | No cracking,voids taper off furthur in | No |
| 69-260-092 | Yes | 1.5 | 1.5 | No cracking,void localized close to abut. | No |
| 69-390-535 | No | 0 | 0 | Gravel backfill tight under appr. | No |

| Struct# | SoilType | Erosion |
|---|---|---|
| 03-100-133 | Glacial till | Severe around wing walls |
| 06-116-151 | Sandy silt clay | No significant erosion |
| 07-125-330 | Glacial till | No significant erosion |
| 07-222-329 | Sandy clay | Severe on W. toe |
| 07-267-329 | Sandy clay | No significant erosion |
| 09-284-080 | Brown silt clay/clay sand/gravel sand | Around wing walls |
| 10-310-399 | Clayey Sand/Sandy Clay | Severe around wing walls |
| 14-100-200 | Dk. brown silt clay | Severe SE corner,toes &wing walls |
| 14-107-194 | Brown silt-clay | Severe around wing walls |
| 16-580-075 | Brown Sand-Silt | Around wing walls |
| 16-580-084 | Brown/Redish Brown Sandstone | Severe around wing walls |
| 17-404-025 | Clayey Sand/Sandy Clay | Minor around wing walls |
| 17-411-027 | Clayey Sand/Sandy Clay | No significant erosion |
| 18-177-100 | Brown-gray sand clay/sand | No significant erosion |
| 18-180-100 | Brown sand silt/clay | No significant erosion |
| 21-424-228 | Brown Silt-Clay | Minor around wing walls |
| 24-162-058 | Clayey Sand/Sandy Clay | Minor around wing walls |
| 26-290-068 | Silty Clay | No significant erosion |
| 26-373-023 | Silty Clay | No significant erosion |
| 28-392-038 | Brown Silt-Clay | Severe around wing walls |
| 31-001-106 | | |
| 31-001-107 | | |
| 33-286-020 | Brown Silt Clay | Severe around wing walls |
| 33-302-015 | Brown Silt Clay | Minor around wing walls |
| 36-071-076 | Brown Silt Clay | Minor around wing walls |
| 36-071-077 | Brown Silt Clay | Minor around wing walls |
| 36-361-298 | Brown Silt-Clay/Brown Fine Sand | Severe past drops & wing walls |
| 36-366-300 | Brown Silt-Clay | Severe past drops & wing walls |

| Struct# | SoilType | Erosion |
|---|---|---|
| 39-177-117 | Black silty clay | No significant erosion |
| 41-094-010 | Clayey Sand/Sandy Clay | Erosion around wing walls |
| 41-214-098 | Red Clayey Sand | No significant erosion |
| 41-214-099 | Red Clayey Sand | No significant erosion |
| 42-064-093 | Brown Silt/Clay | No significant erosion |
| 42-065-093 | Brown Silt/Clay | Some erosion on berms |
| 42-066-006 | Brown sandy clay | Minor around wing walls and appr. emb. |
| 42-067-006 | Brown sandy clay | Minor around wing walls and appr. emb. |
| 42-162-140 | Brown sand silt clay | Erosion around wing walls |
| 44-110-156 | Brown sand silt clay | Erosion around wing walls |
| 44-128-210 | Brown sand silt clay | No significant erosion |
| 44-222-210 | Brown sand silt clay | Severe around wing walls |
| 47-111-580 | Red Silty/Clay | Severe around wing walls,Bad on NW co |
| 48-013-210 | Sandy stoney clay | Severe around wing walls & emb. |
| 48-022-211 | Brown Silt-Clay | Erosion around wing walls |
| 48-440-253 | Brown Silt-Clay | Erosion around wing walls |
| 48-485-271 | Brown Silt-Clay | Minor erosion around wing walls |
| 49-168-130 | Black silty clay | Erosion around wing walls |
| 50-020-141 | Brown silty/reddish brown silty clay | Minor erosion around wing walls |
| 50-020-142 | Brown silty/reddish brown silty clay | Minor erosion around wing walls |
| 50-119-165 | Brown silt clay | Erosion around wing walls |
| 50-119-166 | Brown silt clay | Erosion around wing walls |
| 50-125-168 | Brown silt clay | Minor around wing walls and appr. emb. |
| 50-161-210 | Brown silt clay/clay gravel | No significant erosion |
| 50-175-219 | Brown silt clay | Minor erosion around wing walls |
| 50-176-219 | Brown silt clay | Minor erosion around wing walls |
| 50-180-140 | Brown silt clay | Erosion around wing walls & appr. emb. |
| 50-210-167 | Brown sandy clay | Erosion confined to front slopes |

| Struct# | SoilType | Erosion |
|---|---|---|
| 50-210-168 | Brown sandy clay | Erosion confined to front slopes |
| 50-286-180 | Sandy silt clay | Erosion around wing walls |
| 50-335-181 | Brown silt clay | Erosion around wing walls |
| 51-110-143 | Sandy silt clay | Erosion around wing walls |
| 51-129-130 | Sandy silt clay | No significant erosion |
| 52-256-401 | Rock Fill | No significant erosion |
| 52-383-263 | Red Clay | No significant erosion |
| 52-383-264 | Red Clay | No significant erosion |
| 52-390-278 | Red Clay | Erosion around wing walls |
| 52-424-285 | Red Clayey Sand | Erosion around wing walls |
| 52-485-275 | Red Clayey Sand | No significant erosion |
| 52-485-276 | Red Clayey Sand | No significant erosion |
| 52-486-275 | Red Clayey Sand | No significant erosion |
| 52-486-276 | Red Clayey Sand | No significant erosion |
| 54-160-224 | Brown Silt-Clay | No significant erosion |
| 55-114-241 | Brown Silt-Clay | Erosion around wing walls and toes |
| 55-114-252 | Brown Silt-Clay | Minor erosion wing walls & appr. emb. |
| 55-115-220 | Brown Silt-Clay | Minor erosion around wing walls and toe |
| 55-115-241 | Brown Silt-Clay | Erosion around wing walls and toes |
| 55-115-252 | Brown Silt-Clay | Minor erosion wing walls & appr. emb. |
| 55-115-256 | Brown Silt-Clay | Minor erosion wing walls |
| 55-116-190 | Brown Silt-Clay | No significant erosion |
| 55-116-256 | Brown Silt-Clay | Minor erosion wing walls |
| 55-118-183 | Brown Silt-Clay | No significant erosion |
| 55-119-183 | Brown Silt-Clay | No significant erosion |
| 55-124-170 | Brown Silt-Clay | No significant erosion |
| 55-139-140 | Brown Silt-Clay | No significant erosion |
| 55-140-140 | Brown Silt-Clay | No significant erosion |

| Struct# | SoilType | Erosion |
|---|---|---|
| 55-144-130 | Brown Silt-Clay | No significant erosion |
| 55-160-100 | Brown Silt-Clay | Minor erosion wing walls |
| 55-161-100 | Brown Silt-Clay | Minor erosion wing walls |
| 59-328-274 | Pierre Shale | Severe erosion around wing walls |
| 59-398-295 | Pierre Shale | Minor erosion wing walls |
| 59-493-328 | Brown Silt-Clay | Severe erosion around SE wing wall |
| 61-182-044 | Brown Silt-Clay | Severe erosion around wing walls |
| 61-300-061 | Brown Silt-Clay | Erosion around wing walls |
| 62-093-230 | Brown Silt-Clay | Minor erosion around wing walls |
| 62-235-518 | Brown Silt-Clay | No significant erosion |
| 63-140-034 | Brown Silt-Clay | Minor erosion around wing walls |
| 64-006-090 | Sandy silt/clay | Erosion around wing walls |
| 64-060-241 | Brown Silt-Clay | Erosion around wing walls & appr. emb. |
| 64-154-385 | Brown Silt-Clay | Severe erosion around wing walls |
| 64-155-385 | Brown Silt-Clay | Severe erosion around wing walls |
| 68-180-199 | Brown silt clay | Minor erosion around wing walls |
| 68-180-200 | Brown silt clay | Minor erosion around wing walls |
| 69-220-289 | Brown Silt-Clay | Minor erosion around wing walls |
| 69-260-092 | Brown Silt-Clay | Erosion around wing walls |
| 69-390-535 | Brown Silt-Clay | Minor erosion around wing walls |

| Struct# | Comments |
|---|---|
| 03-100-133 | Diff. settlement at slabs on north end |
| 06-116-151 | Long. cracks at 1st sleeper both slabs on E. appr. |
| 07-125-330 | Rehab with widening of structure and appr. slabs |
| 07-222-329 | West toe eroded and sluffing down,some cracking on appr. slabs |
| 07-267-329 | Large new fill approx. 40 ft.,large settlement of embankments |
| 09-284-080 | 1989 Embank. washout 4 to 8 in., settlement along appr. embankment |
| 10-310-399 | Settlement along appr. embankment voids showing |
| 14-100-200 | Chip seal coming off |
| 14-107-194 | Settlement & cracking along embankment |
| 16-580-075 | large eroded hole under drain,appr. emb. settlement |
| 16-580-084 | Appr. emb. settlement,steep toes |
| 17-404-025 | Embankments good condition |
| 17-411-027 | Embankments good condition |
| 18-177-100 | Embankments good condition |
| 18-180-100 | Embankments good condition,grout on embankment |
| 21-424-228 | Embankments good condition |
| 24-162-058 | Appr. emb. settlement |
| 26-290-068 | Embankment settlement close to abut |
| 26-373-023 | Settlement of sidewalks and under abut by water main |
| 28-392-038 | Appr. Emb. settled minor erosion |
| 31-001-106 | Noted; trans. and long. cracking on approach slabs |
| 31-001-107 | Noted; trans. and long. cracking on approach slabs |
| 33-286-020 | Appr. emb. settled, voids visible |
| 33-302-015 | Small amount of settlement on appr embkmt |
| 36-071-076 | Large voids under appr. & settlement along abut |
| 36-071-077 | Appr. emb. settlement |
| 36-361-298 | Settlement along appr. embankment,Drops not handling flow |
| 36-366-300 | Minor sloughing on east toe, drops not carrying flow |

| Struct# | Comments |
|---|---|
| 39-177-117 | Good condition embankments look good |
| 41-094-010 | Settlement along appr. emb., asphalt seal on embankment |
| 41-214-098 | Rehab new appr. 1989 |
| 41-214-099 | Rehab new appr. 1989 |
| 42-064-093 | Mudjacked appr. in 1987, voids under abutments |
| 42-065-093 | Mudjacked appr. in 1987,Berm erosion |
| 42-066-006 | Erosion on top of appr. emb. and settlement, high fill |
| 42-067-006 | Erosion on top of appr. emb. and settlement, high fill |
| 42-162-140 | Settlement of appr. emb., voids visible under sidewalks |
| 44-110-156 | Settlement of appr. emb.,some cracking |
| 44-128-210 | Low fill,voids under abutment largest at piles |
| 44-222-210 | Settlement on appr. emb. voids visible east side |
| 47-111-580 | Settlement & cracking on embank., high fill |
| 48-013-210 | Settlement along appr. emb. |
| 48-022-211 | Settlement of appr. emb. |
| 48-440-253 | New bridge, gravel appr. |
| 48-485-271 | Slight settlement along appr. emb.,asphalt sealed |
| 49-168-130 | Little settlement along appr. emb. |
| 50-020-141 | Minor settlement of appr. embank.,Tied back abutments |
| 50-020-142 | Minor settlement of appr. embank.,Tied back abutments |
| 50-119-165 | Major slope failure & settlement along appr. emb. |
| 50-119-166 | Major slope failure & settlement along appr. emb. |
| 50-125-168 | Erosion on top of appr. emb. and settlement |
| 50-161-210 | Settlement on appr. emb. & of sleeper |
| 50-175-219 | Bridge end backfill settlement immediately after opening |
| 50-176-219 | Bridge end backfill settlement immediately after opening |
| 50-180-140 | Minor settlement on appr. emb. |
| 50-210-167 | Minor settlement on appr. emb. |

| Struct# | Comments |
|---|---|
| 50-210-168 | Minor settlement on appr. emb. |
| 50-286-180 | Erosion around WW,settlement along appr. emb. |
| 50-335-181 | Appr. emb. good condition |
| 51-110-143 | Minor settlement along appr. emb.,wet under appr.slabs |
| 51-129-130 | Settlement along appr. emb. |
| 52-256-401 | Appr. emb. look good |
| 52-383-263 | Rubberized asphalt chip seal,Erosion at sill |
| 52-383-264 | Scheduled for appr. rehab |
| 52-390-278 | Settlement along appr. emb., some cracking on emb. |
| 52-424-285 | Backfill settled immed. after rehab. |
| 52-485-275 | Breakup on front slope |
| 52-485-276 | Breakup on front slope |
| 52-486-275 | Steep side slopes |
| 52-486-276 | Steep side slopes |
| 54-160-224 | Minor cracking N. toe,embankments good |
| 55-114-241 | Cracking & settlement on appr. emb. |
| 55-114-252 | Embankments good condition |
| 55-115-220 | Steep toes, emb. good condition |
| 55-115-241 | Settlement along appr. emb.,possible sleeper settlement |
| 55-115-252 | Embankments good condition |
| 55-115-256 | Settlement along appr. emb. |
| 55-116-190 | Settlement along appr. emb. concrete on shoulders |
| 55-116-256 | Settlement along appr. emb. |
| 55-118-183 | Embankments good condition |
| 55-119-183 | Embankments good condition |
| 55-124-170 | Settlement along appr. emb. concrete on shoulders |
| 55-139-140 | Embankments good, low fill |
| 55-140-140 | Embankments good, low fill |

| Struct# | Comments |
|---|---|
| 55-144-130 | Settlement along appr. emb. concrete on shoulders |
| 55-160-100 | Embankments good condition |
| 55-161-100 | Embankments good condition |
| 59-328-274 | Settlement along appr. emb. approx. 6" |
| 59-398-295 | Settlement along appr. emb. voids visible under sidewalk |
| 59-493-328 | Embankments good condition except for SE wing wall |
| 61-182-044 | Settlement along appr. emb. |
| 61-300-061 | Embankments good condition |
| 62-093-230 | Settlement along appr. emb. |
| 62-235-518 | Embankments good condition, Sand Backfill |
| 63-140-034 | Appr. emb. cracking NE corner |
| 64-006-090 | Settlement along appr. emb., voids visible |
| 64-060-241 | Curbs settled and broken |
| 64-154-385 | Embankments settled approx. 9" voids under curb visible |
| 64-155-385 | Embankments settled approx. 9" voids under curb visible |
| 68-180-199 | Settlement along shoulders asphalt filled & stabilized |
| 68-180-200 | Settlement along shoulders asphalt filled & stabilized |
| 69-220-289 | Settlement along appr. emb. approx. 4" |
| 69-260-092 | Settlement along appr. emb. approx. 4" |
| 69-390-535 | Embankments good condition slightly loose on top |

# Approach Slab/Abutment/Bridge Investigation

**Structure Number:** ____________________

**Date of Inspection:** ____________________

**Temperature:** ______________

**Measured By:** ______________

______________

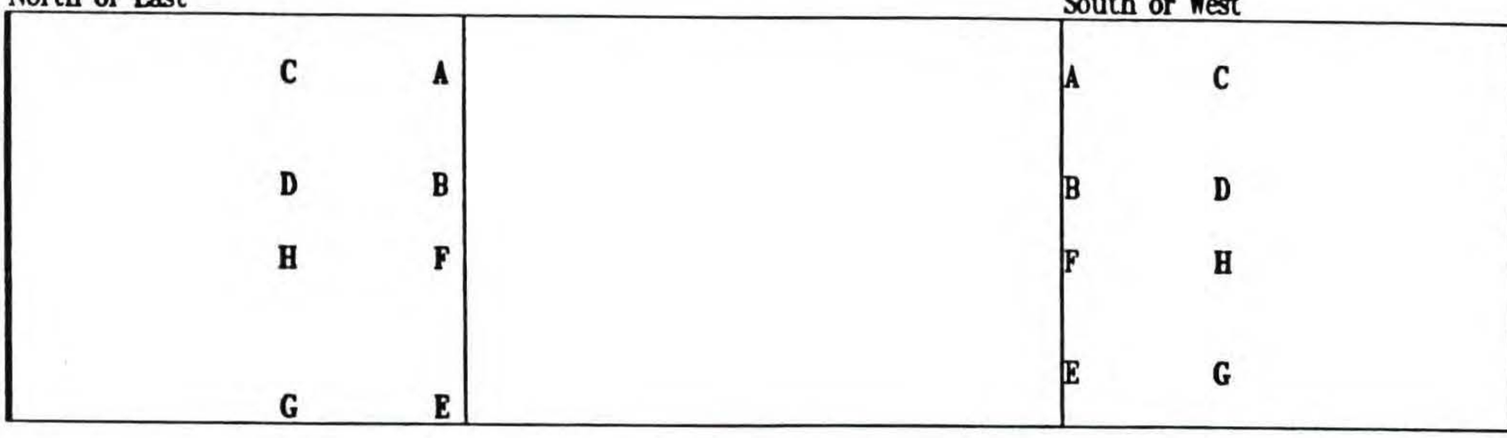
______________

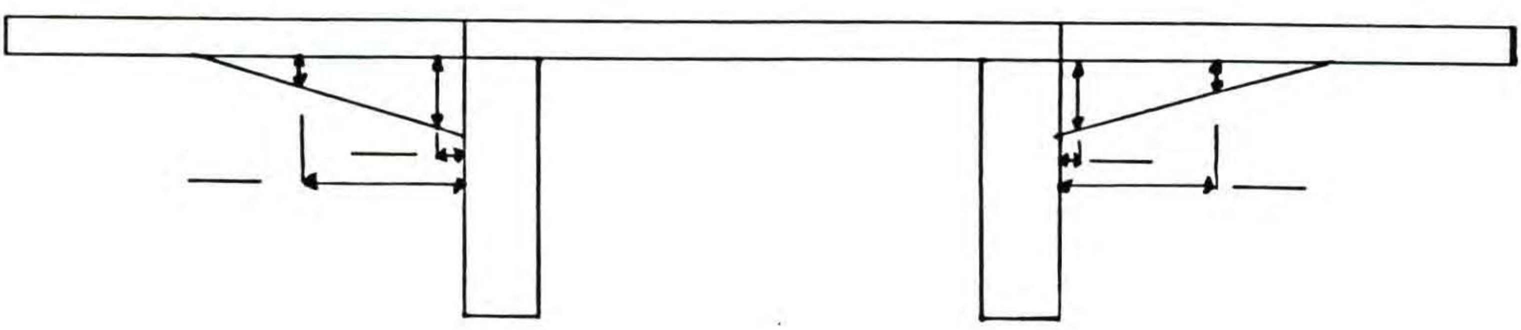
**Location:** ____________________

**Region:** ____________________

**County:** ____________________

**North or East** | **South or West**



| North or East | |
|---|---|
| Position | Void (in) |
| A | |
| B | |
| C | |
| D | |
| E | |
| F | |
| G | |
| H | |

| South or West | |
|---|---|
| Position | Void (in) |
| A | |
| B | |
| C | |
| D | |
| E | |
| F | |
| G | |
| H | |

**Expansion Joint Measurement Centerline**

**North or East** __________ ( inches )

**South or West** __________ ( inches )

**Bridge Length CL:** ________________

**Approach Length CL:**

**South or East** ________________

**North or West** ________________

Selected bridges for extended surveys.

| Structure# | Map # | DateBlt | DatePln | Region | Highway# | Location | BackType | ApprRate | ApprType |
|---|---|---|---|---|---|---|---|---|---|
| 03-100-133 | 1 | 1980 | | Aberdeen | SD 14 | 2mi NW of Wolsey | 1 | 7 | Reinforced Concrete |
| 10-310-399 | 7 | 1985 | | Rapid City | SD 79 | S of Newell over Belle Fouch | 2 | 6 | Reinforced Concrete |
| 18-177-100 | 14 | 1988 | 1986 | Mitchell | SD 38 | 0.3mi W. Hanson County Lin | 3 | 8 | Reinforced Concrete |
| 21-424-228 | 16 | 1988 | 1985 | Pierre | US 212 | 0.6mi E of Laplant | 2 | 9 | Reinforced Concrete |
| 28-392-038 | 20 | 1981 | 1971 | Pierre | SD 63 | 1mi S Ziebach Haakon Co. L | 1 | 7 | Reinforced Concrete |
| 33-286-020 | 23 | 1985 | 1984 | Pierre | US 14 | 0.4mi W. Blunt | 2 | 8 | Reinforced Concrete |
| 36-071-076 | 25 | 1987 | 1985 | Rapid City | I 90 | 6.3mi W of Cactus Flats | 2 | 7 | Reinforced Concrete |
| 36-071-077 | 26 | 1973 | | Rapid City | I 90 | 6.3mi W of Cactus Flats | 0 | 6 | Reinforced Concrete |
| 36-361-298 | 27 | 1980 | 1975 | Pierre | SD 44 | 5.2mi E Jct SD 73 | 1 | 8 | Reinforced Concrete |
| 41-094-010 | 30 | 1986 | 1984 | Rapid City | US 85 | Redwater N of Spearfish | 2 | 8 | Reinforced Concrete |
| 44-110-156 | 38 | 1982 | | Mitchell | US 81 | 3.2mi S I 90 | 1 | | Reinforced Concrete |
| 44-128-210 | 39 | 1983 | | Mitchell | SD 42 | 1.7mi E Jct US 81 | 1 | | Reinforced Concrete |
| 48-022-211 | 43 | 1987 | 1985 | Pierre | SD 44 | 0.3mi E Jct SD 63 | 2 | 8 | Reinforced Concrete |
| 48-485-271 | 45 | 1989 | 1988 | Pierre | SD 44 | 14.9mi W Jct US 183 | 3 | 8 | Reinforced Concrete |
| 50-119-165 | 49 | 1989 | 1987 | Mitchell | I 90 | SD 38 & I 90 Int. | 3 | 8 | Reinforced Concrete |
| 50-119-166 | 50 | 1989 | 1987 | Mitchell | I 90 | SD 38 & I 90 Int. | 3 | 7 | Reinforced Concrete |
| 51-110-143 | 60 | 1986 | | Mitchell | SD 34 | 4.5mi SW Jct SD 13 | | | Reinforced Concrete |
| 52-390-278 | 65 | 1983 | 1981 | Rapid City | I 90 | Deadwood Ave. Exchange | 2 | 7 | Reinforced Concrete |
| 55-114-252 | 73 | 1981 | | Aberdeen | I 29 | 1.8mi NW of Peever | 1 | 9 | Reinforced Concrete |
| 55-115-252 | 76 | 1981 | | Aberdeen | I 29 | 1.8mi NW of Peever | 1 | 9 | Reinforced Concrete |
| 55-118-183 | 80 | 1982 | 1981 | Aberdeen | I 29S | 0.8mi N Jct 10 | 2 | 9 | Reinforced Concrete |
| 59-328-274 | 88 | 1984 | 1983 | Pierre | US 14 | 7.9mi W. Jct US 83 | 2 | 8 | Reinforced Concrete |
| 41-214-098/099 | 31&32 | 1989 | 1988 | Rapid City | I 90 | 1mi E of Whitewood | 3 | 8 | Reinforced Concrete |

Selected bridges for extended surveys.

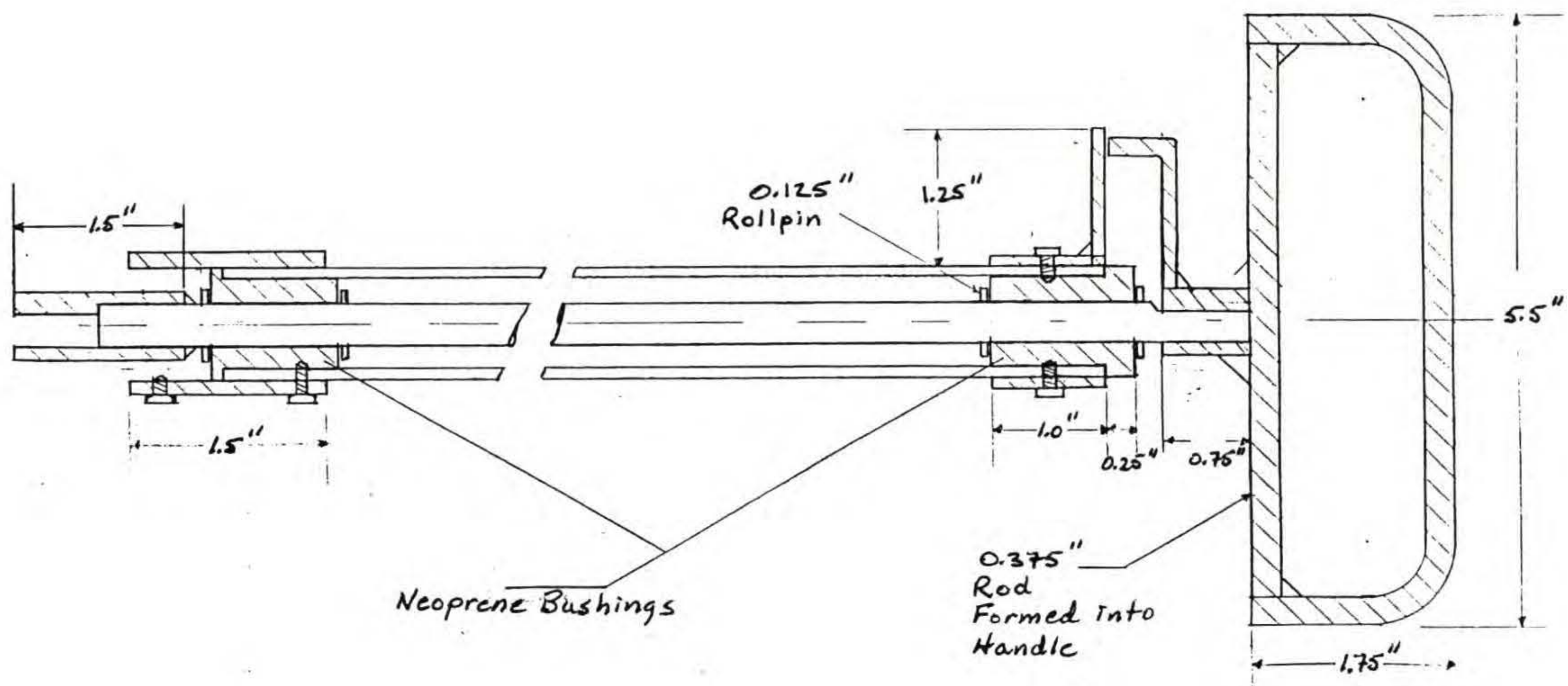
| Structure# | AbutType | Skew | Length | ADT | VoidAbut | Abut Comments | VoidAppr |
|---|---|---|---|---|---|---|---|
| 03-100-133 | Integral | 25.000 | 128.20 | 2300 | No | Depressions at piles along abut. | Yes |
| 10-310-399 | Integral | 25.000 | 296.00 | 605 | Yes | S. 1 to 2",N. none uncovered | Yes |
| 18-177-100 | Integral | 0.000 | 178.00 | 1765 | Yes | E.- 2 to 4", W.- large near piles | No |
| 21-424-228 | Integral | 0.000 | 119.00 | 365 | No | Toes steep and loose | Yes |
| 28-392-038 | Integral | 15.000 | 216.00 | 270 | No | Sloughing E. toe,Cracking W. toe | Yes |
| 33-286-020 | Integral | 0.000 | 99.50 | 1172 | Yes | W.- 1 to 2 inches,E.- none | Yes |
| 36-071-076 | Integral | 0.000 | 202.00 | 1720 | Yes | E. loose,W. 1.5 to 4" largest at piles | Yes |
| 36-071-077 | Integral | 0.000 | 230.00 | 1720 | Yes | E. 4", W. 6", Piles visible both sides | Yes |
| 36-361-298 | Integral | 0.000 | 197.00 | 170 | Yes | Both sides 1 to 2" largest around piles | Yes |
| 41-094-010 | Integral | 0.000 | 160.00 | 4000 | Yes | 8 inches SE corner | Yes |
| 44-110-156 | Integral | 0.000 | 113.00 | | No | Cracking S. toe,settlement apparent | Yes |
| 44-128-210 | Integral | 0.000 | 99.50 | | Yes | Voids W. can feel with rod,E. 2-5" | Yes |
| 48-022-211 | Integral | 0.000 | 150.50 | 390 | Yes | both sides 1 to 2" | Yes |
| 48-485-271 | Integral | 20.000 | 130.10 | 99 | No | Minor erosion on toes | Yes |
| 50-119-165 | Integral | 35.000 | 239.30 | 4200 | No | Slope failure on toes and appr. emb. | Yes |
| 50-119-166 | Integral | 35.000 | 239.30 | 4200 | No | Slope failure on toes and appr. emb. | Yes |
| 51-110-143 | Integral | 0.000 | 100.00 | | Yes | W. & E. 1" all the way across, 2" at piles | Yes |
| 52-390-278 | Integral | 35.000 | 251.00 | 5150 | Yes | 1 1/2 to 2 inches,H/L cracks | Yes |
| 55-114-252 | Integral | 40.000 | 94.00 | 1110 | No | Good condition | Yes |
| 55-115-252 | Integral | 40.000 | 94.00 | 1110 | No | Good condition | Yes |
| 55-118-183 | Integral | 15.000 | 147.00 | 925 | No | Settlement around piles evident | Yes |
| 59-328-274 | Integral | 0.000 | 129.00 | 950 | No | Cracking on W toe,some erosion on both | Yes |
| 41-214-098/099 | Non-Integral | 45.000 | 136.00 | 3710 | No | New rehab | No |

Selected bridges for extended surveys.

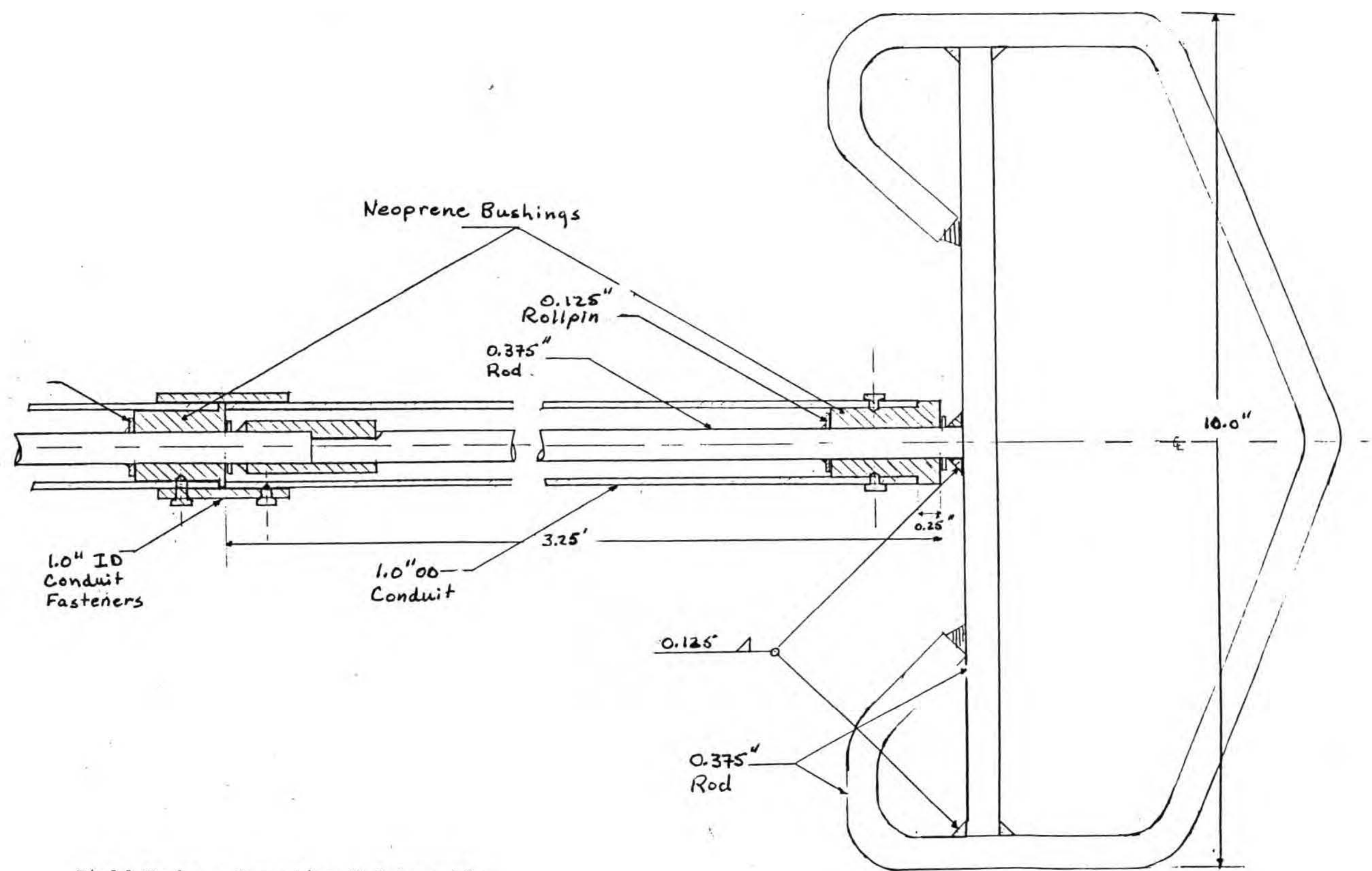
| Structure# | VoidE&S | VoidW&N | Appr Comments | Mudjacked | Soil Type |
|---|---|---|---|---|---|
| 03-100-133 | 1.5 | 0.5 | Long.cracks both sides & 2nd appr.slab | No | Glacial till |
| 10-310-399 | 6 | 6 | Good condition no cracks | No | Clayey Sand/Sandy Clay |
| 18-177-100 | 0 | 0 | Long. crachs both ends | No | Brown-gray sand clay/sand |
| 21-424-228 | 0 | 0.5 | Good condition no cracking | No | Brown Silt-Clay |
| 28-392-038 | 2 | 2 | No cracking,appr. embank. settled 6" | No | Brown Silt-Clay |
| 33-286-020 | 4 | 4 | Long. crack E. end | No | Brown Silt Clay |
| 36-071-076 | 2 | 9 | Long. cracking W. end | No | Brown Silt Clay |
| 36-071-077 | 0 | 0 | Mudjacked,Trans. cracking both sides | Yes | Brown Silt Clay |
| 36-361-298 | 7 | 3 | Voids W. end estimated,unable to measure | No | Brown Silt-Clay/Brown Fine Sand |
| 41-094-010 | 3 | 2 | Good condition expansion joint swell | No | Clayey Sand/Sandy Clay |
| 44-110-156 | 4 | 3.5 | No cracks,settlement along appr. emb. | No | Brown sand silt clay |
| 44-128-210 | 2.5 | 1.5 | No cracks,embankments good | No | Brown sand silt clay |
| 48-022-211 | 4 | 2 | Estimated W. appr.,cracking E. appr. | No | Brown Silt-Clay |
| 48-485-271 | 1 | 1 | Estimated voids due to settlement | No | Brown Silt-Clay |
| 50-119-165 | 2 | 2 | Voids before open to traffic | No | Brown silt clay |
| 50-119-166 | 2 | 2 | Voids before open to traffic | No | Brown silt clay |
| 51-110-143 | 2.5 | 2.5 | Extensive voids both sides | No | Sandy Silt/Clay |
| 52-390-278 | 4 | 4 | Good condition chip seal on appr. | No | Red Clay |
| 55-114-252 | 0.5 | 0.5 | Tightens as probe goes in | No | Brown Silt-Clay |
| 55-115-252 | 0 | 0 | Mudjacked,minor cracking trans & long. | Yes | Brown Silt-Clay |
| 55-118-183 | 0.5 | 0.5 | Good condition no cracking | No | Brown Silt-Clay |
| 59-328-274 | 3.5 | 0.5 | Good condition no cracking | No | Brown Silt-Clay |
| 41-214-098/099 | 0 | 0 | New rehab good condition | No | Red Clayey Sand |

Selected bridges for extended surveys.

| Structure# | Comments |
|---|---|
| 03-100-133 | Severe erosion around WW,embankments good |
| 10-310-399 | Cited as a problem bridge |
| 18-177-100 | Long. cracks W & E ends no signs of mudjacking |
| 21-424-228 | None |
| 28-392-038 | Long. cracks both abut. |
| 33-286-020 | Long. crack w. bound lane |
| 36-071-076 | Long. cracking,very large voids under appr. & Abut |
| 36-071-077 | Transverse cracking both appr. |
| 36-361-298 | No problems cited |
| 41-094-010 | Cited as a bridge with no problems |
| 44-110-156 | Little erosion,piles to measure from present |
| 44-128-210 | Low fill,voids under abutment largest at piles |
| 48-022-211 | Bridge berms faced with min. 3' of impervious mat'l LL>40 & PI>18 |
| 48-485-271 | New bridge |
| 50-119-165 | Major slope failure, dia. & long. cracking |
| 50-119-166 | Major slope failure, dia. & long. cracking |
| 51-110-143 | Erosion around WW,very wet under appr. slabs |
| 52-390-278 | Cited as a problem bridge,Tran. cracking noted 10/83 |
| 55-114-252 | No problems cited |
| 55-115-252 | No problems cited |
| 55-118-183 | Reported that appr. slabs are undermined by settlement and/or animals |
| 59-328-274 | Settlement along appr. embank.,erosion around WW |
| | |
| 41-214-098/099 | Rehab new appr. 1989 |




Field Probe - Handle Assembly




Field Probe - Insertion End Assembly

# APPENDIX B

## SURVEY OF TRANSPORTATION DEPARTMENTS

SDSU

South Dakota State University
Box 2219
Brookings, SD 57007-0495

Civil Engineering Department
College of Engineering
(605) 688-5427

June 7, 1991

Harry A. Reed
Transportation Plng. Div. Asst Dir.
Transportation Department
206 S. 17th Avenue, Room 100A
Phoenix, AZ 85007

Dear Mr. Reed:

We are presently engaged in a study of problems associated with bridge end backfill for the South Dakota Department of Transportation. Our study is primarily focused on details of approach slab/abutment design, gradation and compaction of the backfill, the slope of the embankment under the backfill, and void development under the approach slab.

To aid us in assessing how other states design such systems and the performance obtained, we have enclosed a survey concerning key issues we are examining. Could you take a few minutes to let us know what your state's procedures are?

Please return the survey to:

Vernon R. Schaefer
Civil Engineering Department
South Dakota State University
Brookings, SD 57007-0495

I thank you in advance for your time.

Sincerely,

Vernon R. Schaefer, Ph.D., P.E.
Assistant Professor of Civil Engineering

VRS:atm

| STATE | DATE | INTEGRAL ABUTMENT | REASON FOR INTEGRAL USE | STEEL LENGTH (ft) | CONCRETE LENGTH (ft) | PRESTRESS LENGTH (ft) | SKEW (deg) |
|---|---|---|---|---|---|---|---|
| ARKANSAS | 6/15/91 | YES | TO TRANSFER LOADS TO EMBANKMENT | 0 | 150 | 0 | 0 |
| COLORADO | 7/5/91 | YES | MINIMIZE BUMP | 300 | 400 | 400 | 0 |
| CONNECTICUT | 7/5/91 | NO | AASHTO HAS NOT ADEQUATELY ADDRESSED INTEGRAL ABUTMENTS | N/A | N/A | N/A | N/A |
| GEORGIA | 6/24/91 | NO | N/A | N/A | N/A | N/A | N/A |
| IDAHO | 7/1/91 | YES | ELIMINATION OF JOINTS | 0 | 300 | 300 | 30 |
| INDIANA | 6/24/91 | YES | TO RETARD DRAINAGE THRU JOINT | 250 | 200 | 300 | 30 |
| IOWA | 6/24/91 | YES | ELIMINATION OF JOINTS | 300 | 500 | 500 | 30 |
| KANSAS | 6/26/91 | YES | ELIMINATION OF JOINTS | 300 | 475 | 400 | 0 |
| KENTUCKY | 6/24/91 | YES | ELIMINATION OF JOINTS | 300 | 400 | 400 | 30 |
| MISSOURI | 6/24/91 | YES | ELIMINATION OF JOINTS | 500 | 0 | 600 | 0 |
| MONTANA | 7/24/91 | YES | NONE PROVIDED | 300 | 0 | 350 | 0 |
| NEBRASKA | 6/14/91 | YES | ELIMINATION OF JOINTS | 350 | 0 | 800 | 30 |
| NEW YORK | 7/29/91 | YES | ELIMNINATION OF JOINTS AND REDUCE CONSTR. COSTS | 400 | 400 | 400 | 30 |
| NORTH DAKOTA | 6/15/91 | YES | ELIMINATION OF JOINTS | 400 | 400 | 400 | 30 |
| OHIO | 6/19/91 | YES | REDUCE CONSTRUCTION & MAINTENANCE COSTS | 300 | 300 | 300 | 30 |
| OKLAHOMA | 8/12/91 | YES | HAVE NOT FOUND DURABLE EXPANSION JOINT | 0 | 300 | 300 | 1 |
| OREGON | 6/26/91 | NO | N/A | N/A | N/A | N/A | N/A |
| TENNESSEE | 6/19/91 | YES | ELIMINATION OF JOINTS,PROTECTION FROM SEISMIC EVENTS | 416 | 927 | 927 | 0 |
| UTAH | 6/24/91 | YES | ELIMINATION OF JOINTS | 300 | 300 | 300 | 0 |
| VERMONT | 7/5/91 | YES | ELIMINATION OF JOINTS | 90 | 50 | 0 | 0 |
| VIRGINIA | 6/26/91 | YES | ELIMINATION OF JOINTS,LOWER INITIAL COSTS | 200 | 200 | 0 | 30 |
| WASHINGTON | 7/5/91 | YES | ELIMINATION OF JOINTS | 300 | 400 | 450 | 30 |
| WISCONSIN | 6/24/91 | YES | NONE PROVIDED | 150 | 300 | 300 | 15 |
| WYOMING | 6/16/91 | YES | ELIMINATION OF JOINTS | 300 | 300 | 300 | 0 |

| STATE | INTERSTATE APPROACH | STATE HIGHWAY APPROACH | SECONDARY APPROACH | APPROACH INTEGRAL | SLEEPER SALB | APPR DESIGN |
|---|---|---|---|---|---|---|
| ARKANSAS | REINFORCED CONCRETE | ASPHALT | ASPHALT | NO | YES | REST ON FILL |
| COLORADO | BOTH | ASPHALT | ASPHALT | YES | YES | SPAN FILL |
| CONNECTICUT | N/A | N/A | N/A | N/A | N/A | N/A |
| GEORGIA | N/A | N/A | N/A | N/A | N/A | N/A |
| IDAHO | BOTH | BOTH | ASPHALT | YES | YES | SPAN 1/2 |
| INDIANA | REINFORCED CONCRETE | REINFORCED CONCRETE | REINFORCED CONCRETE | YES | YES | SPAN FILL |
| IOWA | REINFORCED CONCRETE | REINFORCED CONCRETE | BOTH | NO | YES | REST ON FILL |
| KANSAS | REINFORCED CONCRETE | REINFORCED CONCRETE | BOTH | NO | NO | REST ON FILL |
| KENTUCKY | REINFORCED CONCRETE | REINFORCED CONCRETE | REINFORCED CONCRETE | NO | NO | SPAN FILL |
| MISSOURI | REINFORCED CONCRETE | REINFORCED CONCRETE | N/A | NO | NO | SPAN FILL |
| MONTANA | ASPHALT | ASPHALT | ASPHALT | N/A | N/A | N/A |
| NEBRASKA | REINFORCED CONCRETE | REINFORCED CONCRETE | ASPHALT | YES | YES | SPAN FILL |
| NEW YORK | REINFORCED CONCRETE | ASPHALT | ASPHALT | YES | YES | SPAN 10' |
| NORTH DAKOTA | REINFORCED CONCRETE | REINFORCED CONCRETE | N/A | NO | NO | SPAN FILL |
| OHIO | REINFORCED CONCRETE | REINFORCED CONCRETE | ASPHALT | YES | NO | SPAN FILL |
| OKLAHOMA | REINFORCED CONCRETE | REINFORCED CONCRETE | ASPHALT | YES | NO | REST ON FILL |
| OREGON | N/A | N/A | N/A | N/A | N/A | N/A |
| TENNESSEE | BOTH | ASPHALT | ASPHALT | NO | NO | SPAN FILL |
| UTAH | BOTH | BOTH | ASPHALT | NO | YES | SPAN FILL |
| VERMONT | ASPHALT | ASPHALT | ASPHALT | NO | YES | SPAN FILL |
| VIRGINIA | ASPHALT | ASPHALT | ASPHALT | NO | NO | N/A |
| WASHINGTON | BOTH | BOTH | BOTH | YES | NO | SPAN 2/3 |
| WISCONSIN | REINFORCED CONCRETE | REINFORCED CONCRETE | BOTH | NO | NO | SPAN FILL |
| WYOMING | REINFORCED CONCRETE | REINFORCED CONCRETE | REINFORCED CONCRETE | YES | YES | REST ON FILL |

| STATE | BACKFILL | COMPACTION | SLOPE | DRAINAGE |
|---|---|---|---|---|
| ARKANSAS | STANDARD EMBANKMENT MATERIAL | No Response | N/A | NONE |
| COLORADO | FINE GRAINED UNIFORM GRADATION | NONE | 2:1 | PERFORATED PIPE |
| CONNECTICUT | N/A | N/A | N/A | N/A |
| GEORGIA | N/A | N/A | N/A | N/A |
| IDAHO | GRANULAR BACKFILL MAX .5% PASSING #200 | 95 % AASHTO T-99 | 2:1 | PERFORATED PIPE |
| INDIANA | GRANULAR BACKFILL MAX 8% PASSING #200 | No Response | 1:4 | GRANULAR BACKFILL ALONG ABUT FACE AND/OR PERFORATED PIPE |
| IOWA | GRANULAR BACKFILL MAX 10% PASSING #200 | No Response | N/A | PERFORATED PIPE,GRANULAR BACKFILL,FILTER FABRICS ON TOP |
| KANSAS | GEOCOMPOSITE STRIP DRAIN | No Response | 1:1 | GEOCOMPOSITE STRIP DRAIN & PERFORATED PIPE |
| KENTUCKY | GRANULAR BACKFILL MAX 5% PASSING #100 | No Response | 1:1 | VERTICAL STRIP DRAIN,PERFORATED PIPE,FABRICS POSSIBLE |
| MISSOURI | GRANULAR BACKFILL | No Response | 6:1 | STRIP DRAIN WITH DRAINAGE PIPE $ FABRIC ALONG ABUT. FACE |
| MONTANA | GRANULAR BACKFILL | AASHTO T-180 | 2:1 | GRANULAR BACKFILL,FABRIC WRAP,PERFORATED PIPE |
| NEBRASKA | GRANULAR BACKFILL WASHED CONCRETE SAND/GRAVEL | 95 % AASHTO T-99 | 2:1 | PERFORATED PIPE |
| NEW YORK | GRANULAR BACKFILL MAX 15% PASSING #200 | 90% AASHTO T-99 | 1.5:1 | 1' WIDE GRAN. FILTER ALONG ABUT. WALL MAX 5% PASS. #20 |
| NORTH DAKOTA | GRANULAR BACKFILL MAX 10% PASSING #200 | No Response | 4:1 | SEEPAGE TRENCHES OR PERFORATED PIPE |
| OHIO | GRANULAR BACKFILL 65% RETAINED #200 | 95 % AASHTO T-99 | N/A | POLYETHELENE PIPE,BACKFILL,WEEP HOLES |
| OKLAHOMA | GRANULAR BACKFILL MAX 15% PASSING #200 | 95 % AASHTO T-99 | 2:1 | PERFORATED PIPE |
| OREGON | N/A | N/A | N/A | N/A |
| TENNESSEE | GRANULAR BACKFILL | No Response | 2:1 | PERFORATED PIPE,GRANULAR BACKFILL |
| UTAH | GRANULAR BACKFILL | No Response | N/A | CATCH BASINS ON APPR SLAB |
| VERMONT | GRANULAR BACKFILL MAX 6% PASSING #200 | 90% AASHTO T-99 | VERT | WEEP HOLES IN ABUTMENT |
| VIRGINIA | NONE | 95 % AASHTO T-99 | 1.5:1 | POUROUS BACKFILL WITH FILTER CLOTH |
| WASHINGTON | GRAVEL BORROW MAX 7% PASSING #200 | 95 % AASHTO T-99 | 1.5:1 | POLYETHELENE PIPE,SOME FABRICS |
| WISCONSIN | GRANULAR BACKFILL | No Response | 2:1 | PERFORATED PIPE,GRANULAR BACKFILL |
| WYOMING | GRANULAR BACKFILL MAX 4% PASSING #200 | No Response | HORZ | PERFORATED PIPE,GRANULAR BACKFILL,REINFORCED FILTER FABRICS |

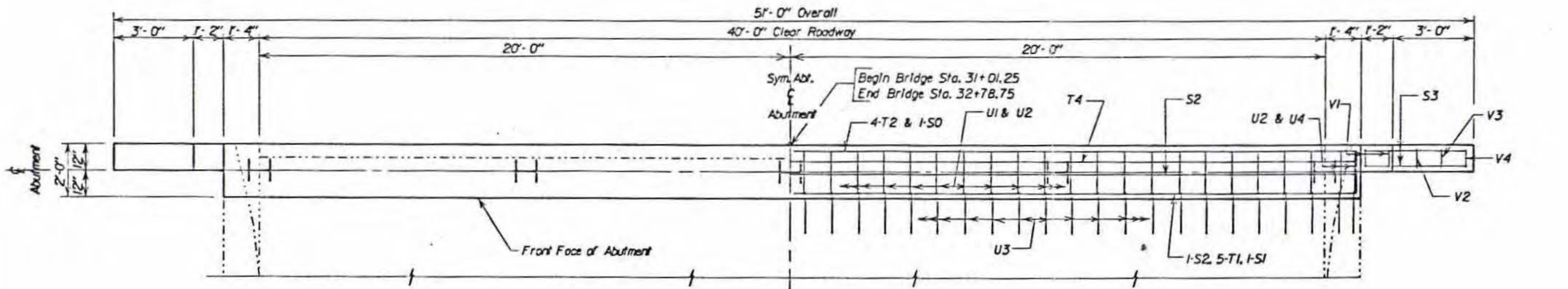
| STATE | PROBLEMS | SOLUTIONS | RESEARCH |
|---|---|---|---|
| ARKANSAS | SETTLEMENT AND CRACKING | NONE SPECIFIED | NONE |
| COLORADO | VOIDS,SETTLEMENT OF SLEEPER | MUDJACK VOIDS AND SLEEPER | YES |
| CONNECTICUT | N/A | N/A | N/A |
| GEORGIA | N/A | N/A | N/A |
| IDAHO | NONE | NONE | NONE |
| INDIANA | VOIDS,SETTLEMENT | MUDJACKING | NONE |
| IOWA | SETTLEMENT, VOIDS, CRACKING | MUDJACKING | NONE |
| KANSAS | VOIDS | MUDJACKING | YES |
| KENTUCKY | SETTLEMENT, VOIDS, CRACKING | MUDJACKING(HOLES PROVIDED)&ASPH.WEDGING | NONE |
| MISSOURI | VOIDS | MUDJACKING HOLES PROVIDED | NONE |
| MONTANA | SETTLEMENT, VOIDS, CRACKING | MUDJACKING,ASPHALT OVERLAYS,AND REPLACEMENT | NONE |
| NEBRASKA | NONE NEWER SYSTEM 2 YEARS OLD | NONE | NONE |
| NEW YORK | NONE CITED | NONE | NONE |
| NORTH DAKOTA | SUPPORT LEDGE FAILURE | REDESIGN SUPPORT LEDGE | NONE |
| OHIO | SETTLEMENT | ATTRIBUTED TO POOR CONSTRUCTION | YES |
| OKLAHOMA | SETTLEMENT AT PAVEMENT/APPR. EDGE | ASPHALT OVERLAYS | YES |
| OREGON | N/A | N/A | N/A |
| TENNESSEE | SETTLEMENT, VOIDS, CRACKING | SITES BETTER COMPACTION AS SOLUTION | NONE |
| UTAH | SETTLEMENT | MUDJACKING | NONE |
| VERMONT | NONE | NONE | NONE |
| VIRGINIA | NONE | NONE | NONE |
| WASHINGTON | SETTLEMENT, VOIDS, CRACKING | MUDJACKING AND OVERLAYS | YES |
| WISCONSIN | SETTLEMENT, VOIDS, CRACKING | MUDJACKING AND MORE RC APPR SLABS | NO |
| WYOMING | VOIDS,SETTLEMENT | NONE PROVIDED | YES |

# APPENDIX C

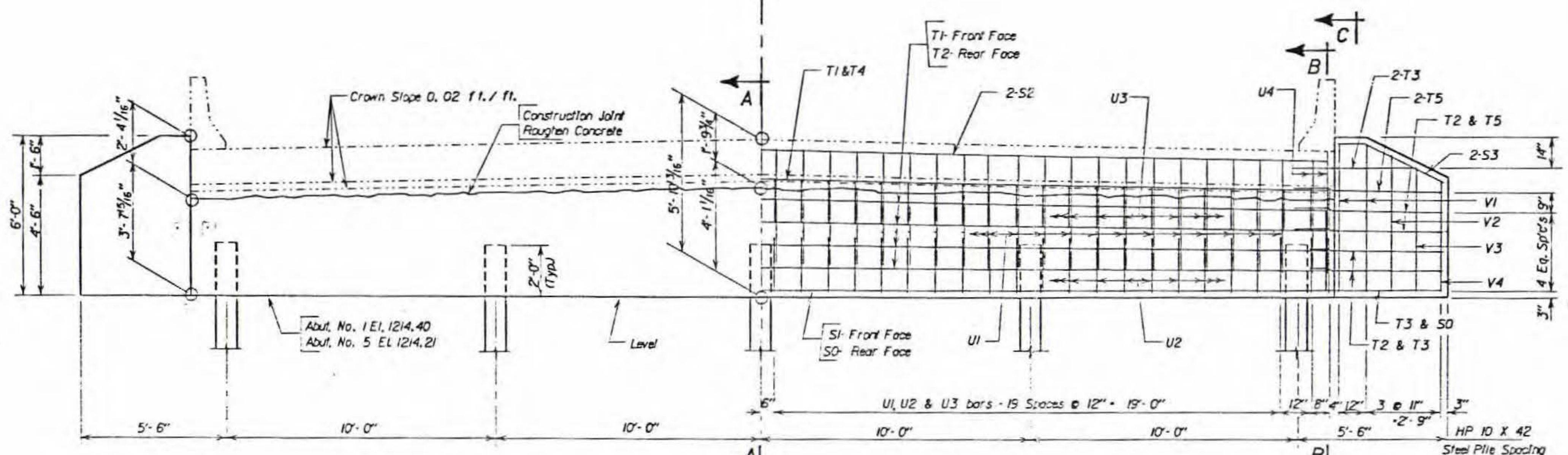
# BRIDGE CONNECTION ATTACHMENTS
# AND
# APPROACH SLAB CONFIGURATIONS

Attachment 1

| FED.HWY. ADMIN.NO. | STATE OF | PROJECT | SHEET NO. | TOTAL SHEETS |
|---|---|---|---|---|
| 8 | S.D. | BRF0038(9)302 | 21 | 53 |




PLAN

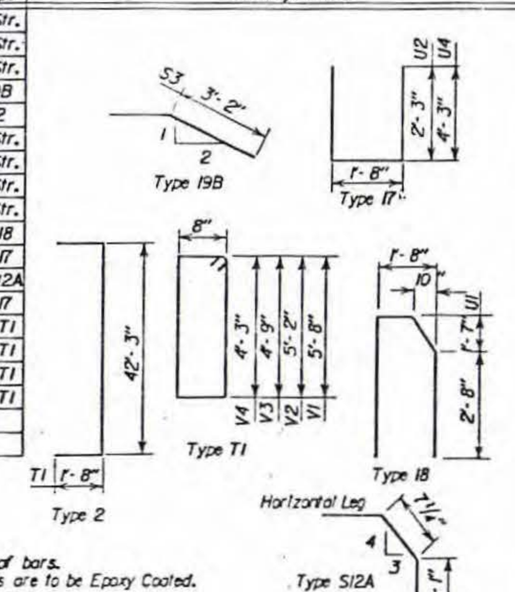



ELEVATION

## REINFORCING SCHEDULE
(For One Abutment)

| | MK. | No. | Size | Length | Type |
|---|---|---|---|---|---|
| | So | 1 | 9 | 50'- 8" | Str. |
| | S1 | 1 | 9 | 42'- 4" | Str. |
| *△ | S2 | 2 | 9 | 42'- 4" | Str. |
| | S3 | 4 | 9 | 4'- 2" | 19B |
| * | T1 | 5 | 5 | 45'- 7" | 2 |
| * | T2 | 4 | 5 | 50'- 8" | Str. |
| | T3 | 10 | 5 | 4'- 6" | Str. |
| * | T4 | 1 | 5 | 42'- 4" | Str. |
| | T5 | 8 | 5 | 5'- 6" | Str. |
| △ | U1 | 40 | 6 | 9'- 6" | 18 |
| | U2 | 44 | 4 | 6'- 2" | 17 |
| △ | U3 | 44 | 4 | 2'- 10" | S12A |
| △ | U4 | 4 | 6 | 10'- 2" | 17 |
| | V1 | 4 | 4 | 13'- 5" | T1 |
| | V2 | 2 | 4 | 12'- 5" | T1 |
| | V3 | 2 | 4 | 11'- 7" | T1 |
| | V4 | 2 | 4 | 10'- 7" | T1 |

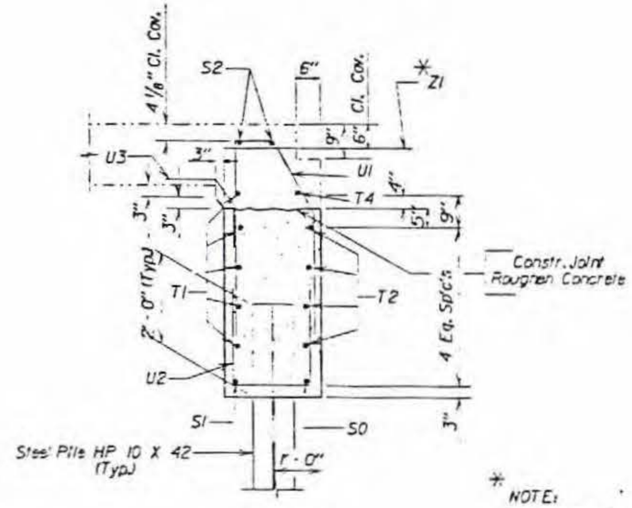
Bending Details




NOTE:
All dimensions are out to out of bars.
△ All S2, U1, U3, and U4 bars are to be Epoxy Coated.
* Bend in field as necessary to fit.

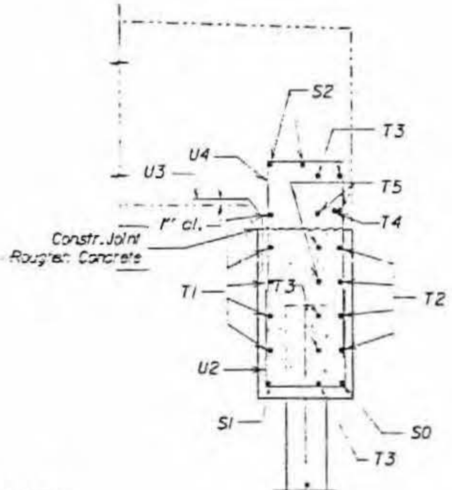
## ESTIMATED QUANTITIES
(For Two Abutments)

| ITEM | | UNIT | QUANTITY |
|---|---|---|---|
| Class "A45" Concrete Bridge | | Cu. Yds. | 27.8 |
| Reinforcement, Concrete Masonry | | Lbs. | 2445 |
| Epoxy Coated Reinforcement for Concrete Masonry | | Lbs. | 2006 |
| Structure Excavation-Bridge | | Cu. Yds. | 19.4 |
| Furnish Steel Test Pile | HP 10 X 42 | L.F. | 2 @ 72' = 144' |
| Drive Steel Test Pile | HP 10 X 42 | L.F. | 2 @ 72' = 144' |
| Furnish Steel Pile | HP 10 X 42 | L.F. | 8 @ 67' = 536' |
| Drive Steel Pile | HP 10 X 42 | L.F. | 8 @ 67' = 536' |
| Prebore Piling | | L.F. | 10 @ 10' = 100' |

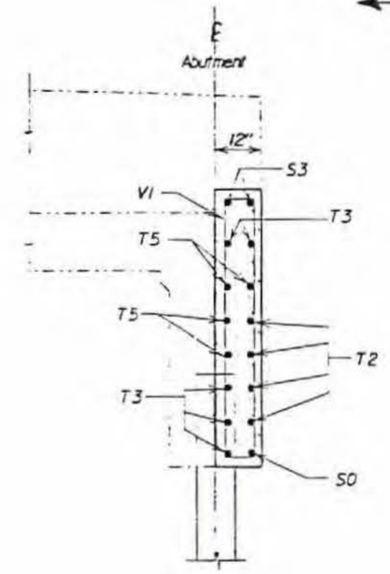



SEC. A-A

* NOTE:
Z1 bars are listed and included in superstructure Quantities




SEC. B - B



SEC. C - C

ABUTMENT DETAILS
FOR
177'-6" CONTINUOUS CONCRETE BRIDGE
40'-0" ROADWAY — SEC. 24/25-T103N-R60W
OVER FIRESTEEL CREEK — 0° SKEW
STA. 31+01.25 TO 32+78.75 — BRF0038(9)302
STR. NO. 18-177-100 — HS20-44 (& ALT.)

DAVISON COUNTY

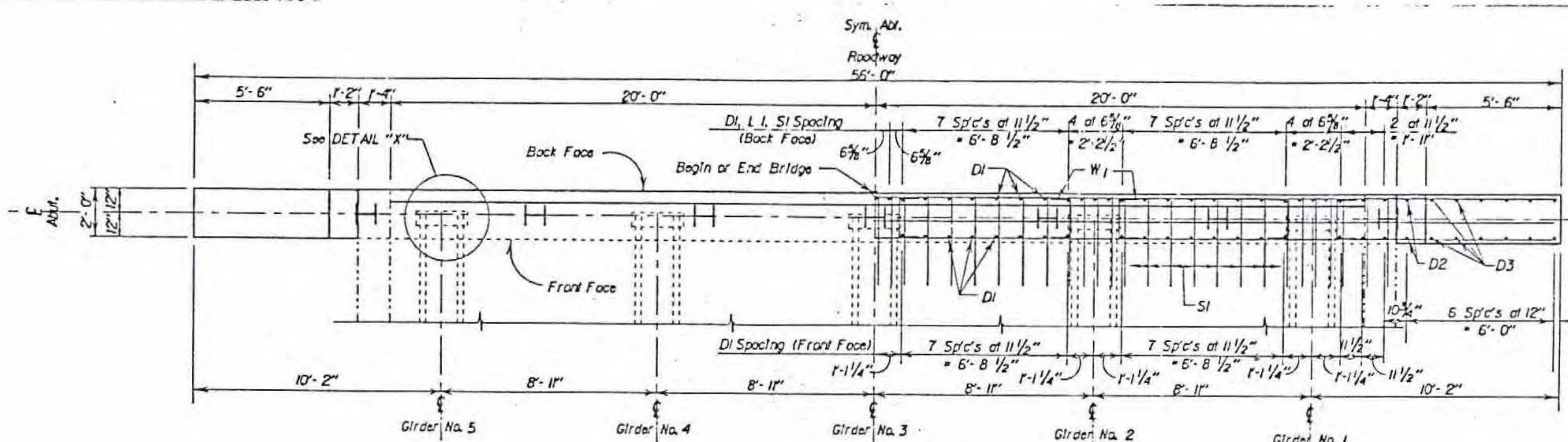
S. D. DEPT. OF TRANSPORTATION

APRIL 1986 — ④ OF ⑫

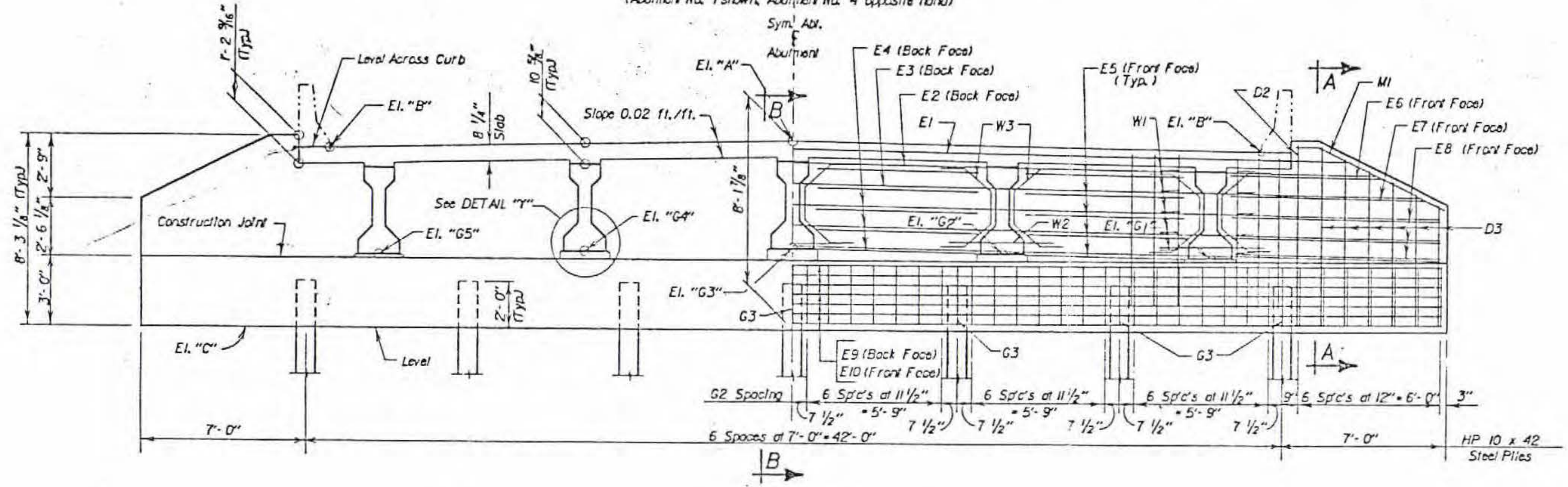
| DESIGNED BY | DRAWN BY | CHECKED BY | APPROVED |
|---|---|---|---|
| P.K./T.R. | L.M. | P.K./K.G. | |

Attachment 2

| FED. HWY. ADMIN. NO. | STATE OF | PROJECT | SHEET NO. | TOTAL SHEETS |
|---|---|---|---|---|
| 8 | S.D. | BRS3264(01)425 | 34 | 64 |




PLAN
(Abutment No. 1 shown, Abutment No. 4 opposite hand)

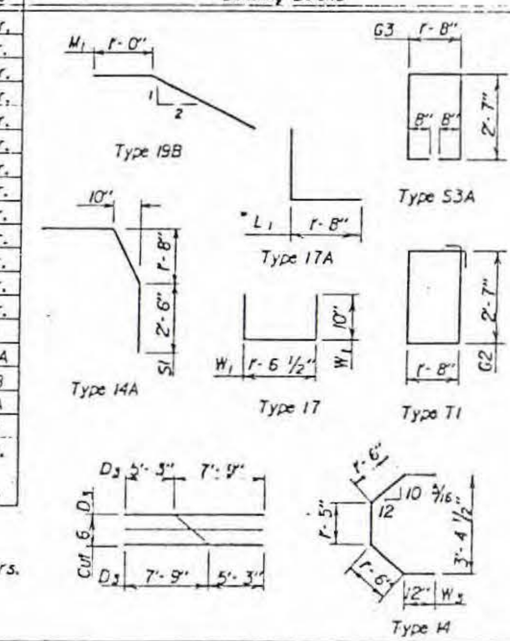



ELEVATION

## REINFORCING SCHEDULE
(For One Abutment)

| | Mk. | No. | Size | Length | Type |
|---|---|---|---|---|---|
| | $D_1$ | 91 | 5 | 6'-6" | Str. |
| | $D_2$ | 4 | 5 | 7'-10" | Str. |
| # | $D_3$ | 12 | 5 | 13'-0" | Str. |
| △ | $E_1$ | 2 | 5 | 47'-9" | Str. |
| | $E_2$ | 1 | 8 | 50'-8" | Str. |
| | $E_3$ | 1 | 8 | 54'-3" | Str. |
| | $E_4$ | 3 | 8 | 55'-8" | Str. |
| | $E_5$ | 20 | 5 | 6'-10" | Str. |
| | $E_6$ | 2 | 5 | 6'-7" | Str. |
| | $E_7$ | 2 | 5 | 8'-4" | Str. |
| | $E_8$ | 6 | 5 | 9'-0" | Str. |
| | $E_9$ | 7 | 9 | 55'-8" | Str. |
| | $E_0$ | 7 | 6 | 55'-8" | Str. |
| | $G_2$ | 56 | 4 | 9'-3" | T1 |
| | $G_3$ | 7 | 5 | 8'-2" | S3A |
| | $M_1$ | 4 | 5 | 7'-0" | 19B |
| △ | $S_1$ | 53 | 5 | 8'-0" | 13A |
| | $W_1$ | 50 | 5 | 3'-3" | 17 |
| | $W_2$ | 5 | 5 | 4'-6" | Str. |
| | $W_3$ | 10 | 4 | 6'-5" | 14 |
| | $L_1$ | 40 | 4 | 3'-4" | 17A |

Bending Details




NOTE:
All dimensions are out to out of bars.
△ Bars to be Epoxy Coated.
# See Cutting Diagram.

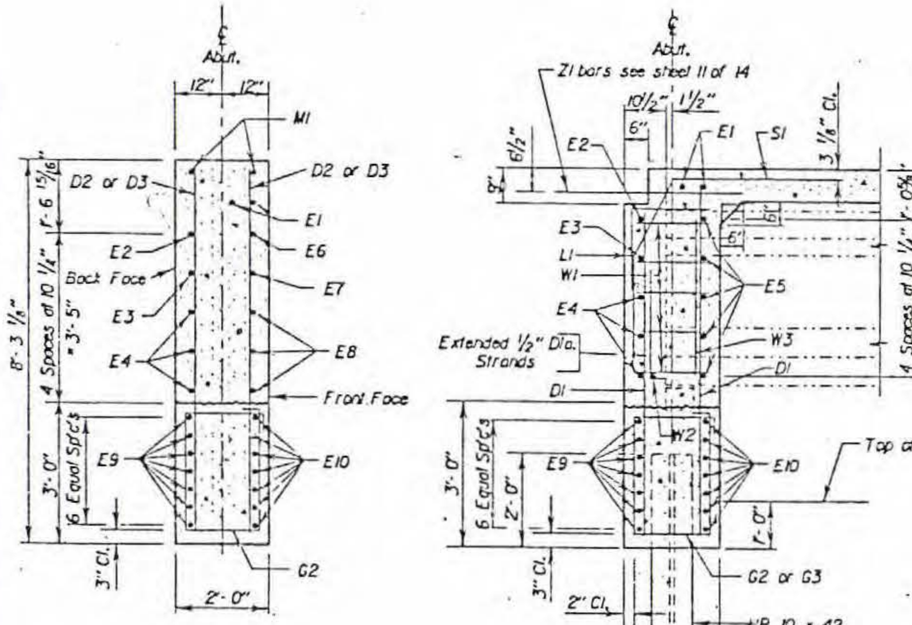
## ESTIMATED QUANTITIES

| | ITEM | UNIT | QUANTITY Abut. No. 1 | QUANTITY Abut. No. 4 |
|---|---|---|---|---|
| # | Class "A45" Concrete, Bridge | Cu. Yd. | 12.5 | 12.5 |
| | Reinforcement for Concrete Masonry | Lb. | 4438 | 4438 |
| | Structure Excavation, Bridge | Cu. Yd. | 10.9 | 10.9 |
| | Furnish Steel Test Pile (HP 10 x 42) | L. F. | 1 @ 71' = 71' | 1 @ 77' = 77' |
| | Drive Steel Test Pile (HP 10 x 42) | L. F. | 1 @ 71' = 71' | 1 @ 77' = 77' |
| | Furnish Steel Pile (HP 10 x 42) | L. F. | 6 @ 66' = 396' | 6 @ 72' = 432' |
| | Drive Steel Pile (HP 10 x 42) | L. F. | 6 @ 66' = 396' | 6 @ 72' = 432' |
| | Preboring Piling | L. F. | 7 @ 10' = 70' | 7 @ 10' = 70' |
| | Epoxy Coated Reinforcement for Conc. Masonry | Lb. | 542 | 542 |

# Includes 0.1 cu. yds. for grout pads.

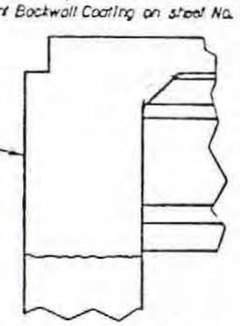
Note: Abutment Backwalls are placed with and included in quantity for Class "A45" Concrete Bridge Deck on sheet 6 of 14.

Note: Concrete shall be placed in the space under the beams (within the backwall width) during the backwall pour. Care shall be taken to get the concrete vibrated into this area. If upon form removal the space is not completely filled and consolidated, the contractor shall grout in the remaining voids.



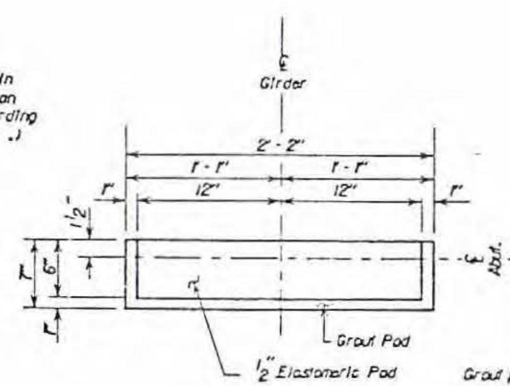

SEC. A - A

SEC. B - B

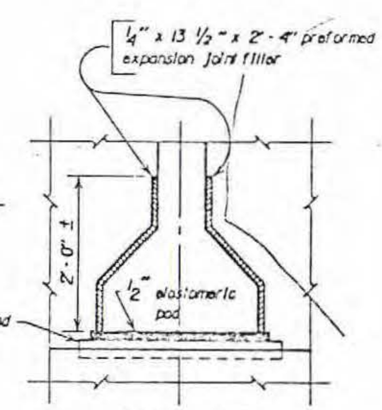


This face on the back side of the abutment within the r.o.w. width shall be thoroughly coated with an approved water proof sealant. (See Notes Regarding Abutment Backwall Coating on sheet No. 2 of 14.)

ABUT. BACKWALL COATING DETAILS




DETAIL "X"




DETAIL "Y"
(Typical at girder ends; abutments only)

## TABLE OF ELEVATIONS

| ABUTMENT | EL. "A" | EL. "B" | EL. "C" | EL. "G1" | EL. "G2" | EL. "G3" | EL. "G4" | EL. "G5" |
|---|---|---|---|---|---|---|---|---|
| NO. 1 | 1316.57 | 1316.17 | 1308.41 | 1311.54 | 1311.72 | 1311.89 | 1311.72 | 1311.54 |
| NO. 4 | 1322.75 | 1322.35 | 1314.59 | 1317.72 | 1317.90 | 1318.08 | 1317.90 | 1317.72 |

NOTE-
Elevations "A" and "B" are top of slab at centerline of abutment.
Elevations "G1", "G2", "G3", "G4" AND "G5" are top of grout pad at centerline of abutment.

ABUTMENT DETAILS
FOR
217'-0" CONT. COMP. PRESTR. GIRDER BRIDGE
40'-0" ROADWAY SEC. 35/2-T102/101N-R48W
OVER SPLIT ROCK CREEK 0° SKEW
STA. 5+13.00 TO 7+30.00 BRS3264(01)425
STR. NO. 50-286-180 HS20-44 (& ALT.)

MINNEHAHA COUNTY

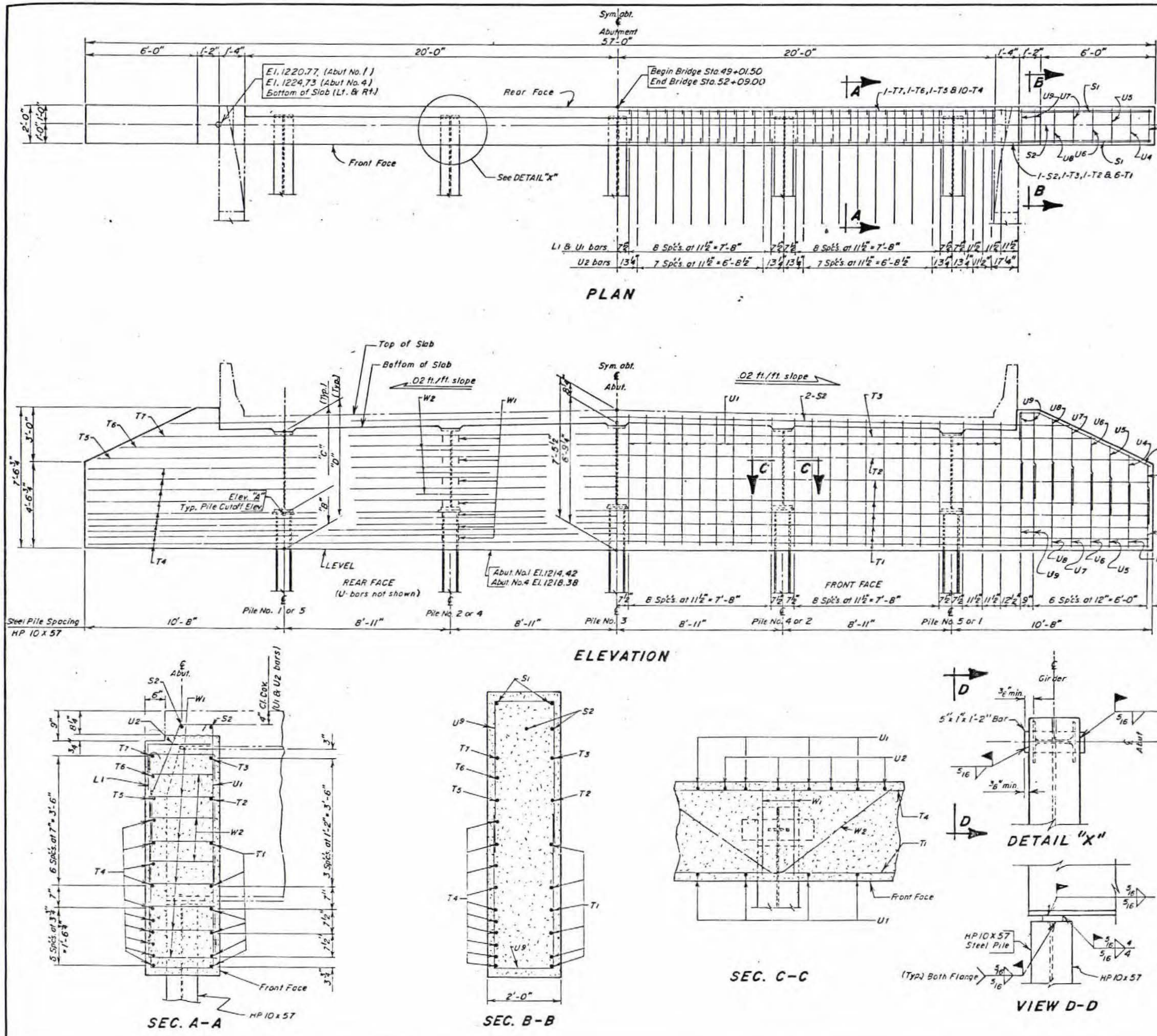
S. D. DEPT. OF TRANSPORTATION

MARCH 1987 4 OF 14

| DESIGNED BY | DRAWN BY | CHECKED BY | APPROVED |
|---|---|---|---|
| KG / GF | LAK | KG / GF | BRIDGE ENGINEER |

Attachment 3

| FED. HWY. ADMIN. NO. | STATE OF | PROJECT | SHEET NO. | TOTAL SHEETS |
|---|---|---|---|---|
| 8 | S.D. | BRF0038 (10) 302 | 34 | 53 |

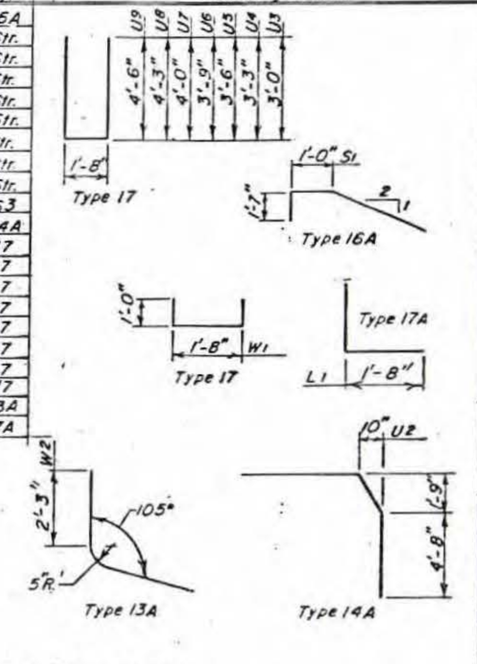


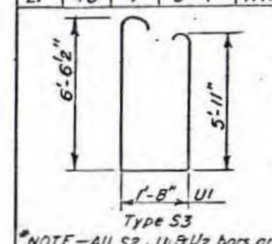

## REINFORCING SCHEDULE

(For One Abutment)

| Mk. | No. | Size | Length | Type |
|---|---|---|---|---|
| S1 | 4 | 9 | 9'-1" | 16A |
| *S2 | 2 | 9 | 48'-2" | Str. |
| T1 | 6 | 7 | 55'-8" | Str. |
| T2 | 1 | 7 | 55'-11" | Str. |
| T3 | 1 | 7 | 51'-1" | Str. |
| T4 | 10 | 9 | 56'-8" | Str. |
| T5 | 1 | 9 | 55'-11" | Str. |
| T6 | 1 | 9 | 53'-6" | Str. |
| T7 | 1 | 9 | 51'-1" | Str. |
| *U1 | 42 | 6 | 15'-5" | S3 |
| *U2 | 41 | 6 | 11'-8" | 14A |
| U3 | 4 | 6 | 7'-8" | 17 |
| U4 | 4 | 6 | 8'-2" | 17 |
| U5 | 4 | 6 | 8'-8" | 17 |
| U6 | 4 | 6 | 9'-2" | 17 |
| U7 | 4 | 6 | 9'-8" | 17 |
| U8 | 4 | 6 | 10'-2" | 17 |
| U9 | 8 | 6 | 10'-8" | 17 |
| W1 | 70 | 6 | 3'-8" | 17 |
| W2 | 15 | 8 | 5'-0" | 13A |
| L1 | 40 | 4 | 3'-4" | 17A |

Bending Details




*NOTE—All S2, U1 & U2 bars are to be Epoxy Coated. All dimensions are out to out of bars.

## ESTIMATED QUANTITIES

| ITEM | UNIT | QUANTITY Abutment No. 1 | QUANTITY Abutment No. 4 |
|---|---|---|---|
| Class "A45" Concrete Bridge | Cu. Yds. | 27.4 | 27.4 |
| Reinforcement for Concrete Masonry | Lbs. | 4634 | 4634 |
| Epoxy Coated Reinforcing Steel | Lbs. | 2019 | 2019 |
| Structure Excavation—Bridge | Cu. Yds. | 11.1 | 11.1 |
| Furnish Steel Test Pile HP 10 x 57 | L.F. | 1@ 92' = 92' | 1@ 96' = 96' |
| Drive Steel Test Pile HP 10 x 57 | L.F. | 1@ 92' = 92' | 1@ 96' = 96' |
| Furnish Steel Piles HP 10 x 57 | L.F. | 4@ 87' = 348' | 4@ 91' = 364' |
| Drive Steel Piles HP 10 x 57 | L.F. | 4@ 87' = 348' | 4@ 91' = 364' |
| Preboring Piling | L.F. | 5@ 10' = 50' | 5@ 10' = 50' |

## TABLE OF ELEV. & DIMENSIONS

| | Abutment No. 1 | | | | Abutment No. 4 | | | |
|---|---|---|---|---|---|---|---|---|
| Pile No. | Elev. "A" | "B" | "C" | "D" | Elev. "A" | "B" | "C" | "D" |
| 1 and 5 | 1216.421 | 2'-0" | 4'-3 1/4" | 6'-3 1/4" | 1220.380 | 2'-0" | 4'-3 1/4" | 6'-3 1/4" |
| 2 and 4 | 1216.599 | 2'-2 1/8" | 4'-3 1/4" | 6'-5 3/8" | 1220.559 | 2'-2 1/8" | 4'-3 1/4" | 6'-5 3/8" |
| 3 | 1216.777 | 2'-4 1/4" | 4'-3 1/4" | 6'-7 1/2" | 1220.737 | 2'-4 1/4" | 4'-3 1/4" | 6'-7 1/2" |

DETAILS OF ABUTMENT NO.1 & NO.4

FOR

# 307'-6" CONT. COMP. GIRDER BRIDGE

40'-0" ROADWAY — 0° SKEW

OVER JAMES RIVER — SEC. 24/25-T103N-R60W

STA. 49+01.50 TO 52+09.00 — BRF0038 (10) 302

STR. NO. 18-180-100 — HS 20-44 (& ALT.)

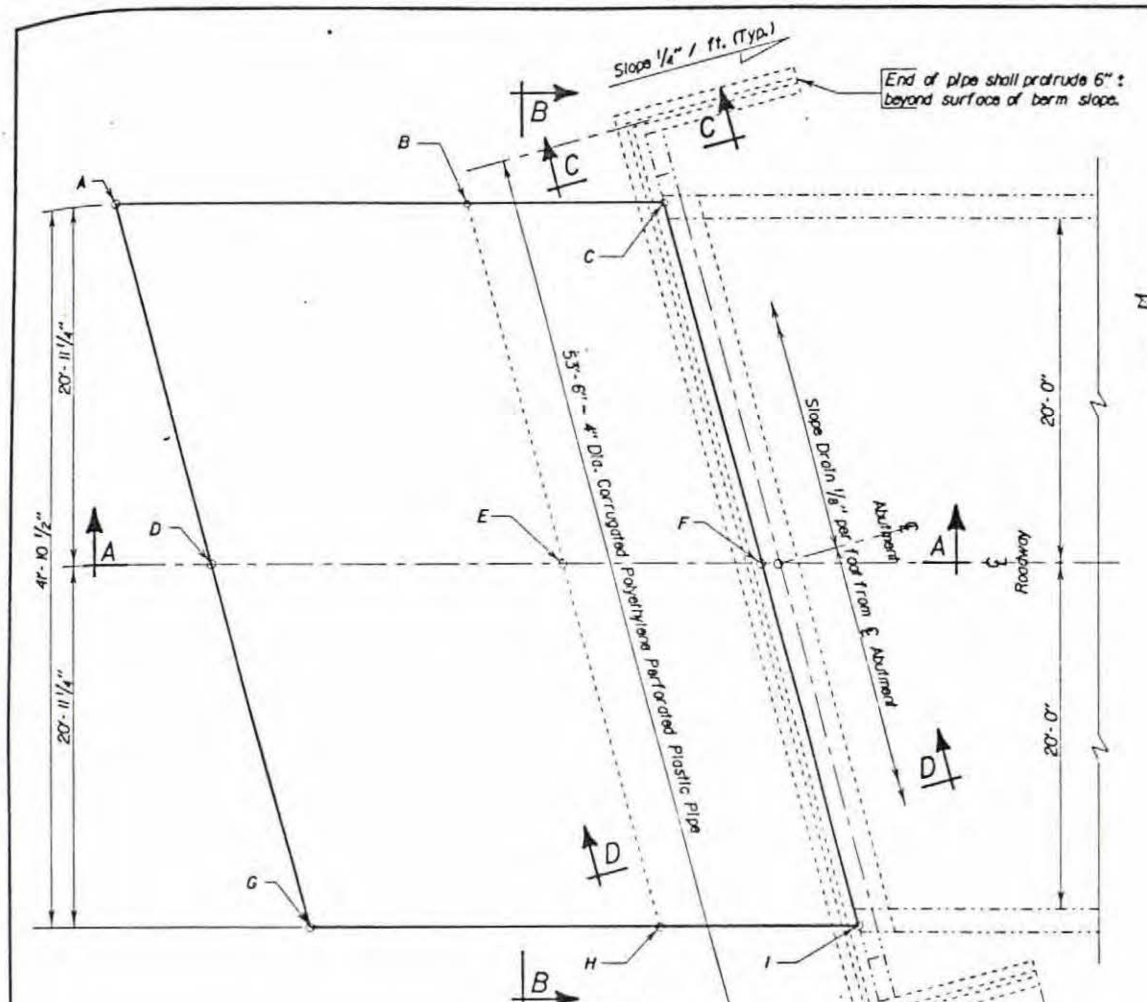
DAVISON COUNTY

S.D. DEPT. OF TRANSPORTATION

MAY 1986 — 5 OF 17

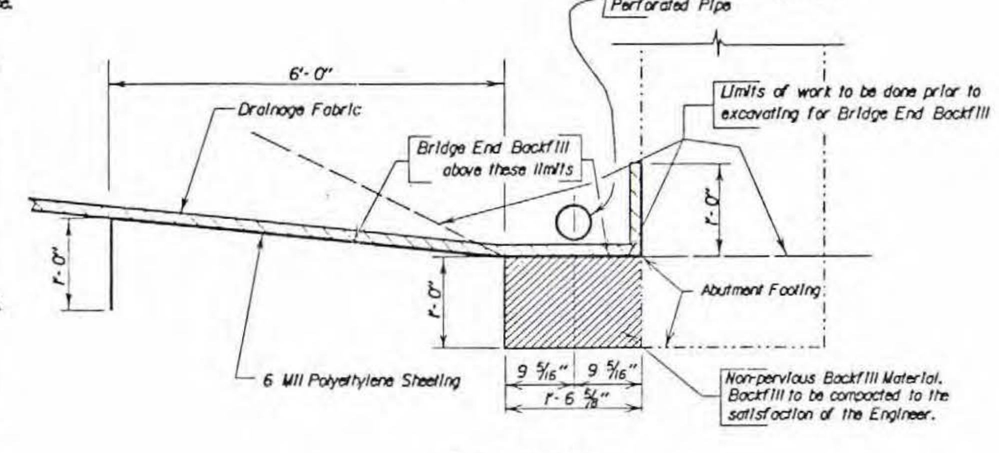
| DESIGNED BY | DRAWN BY | CHECKED BY | APPROVED |
|---|---|---|---|
| KG / SJ. | f.a.k. | | |

| FED.HWY. ADMIN.NO. | STATE OF | PROJECT | SHEET NO. | TOTAL SHEETS |
|---|---|---|---|---|
| 8 | S.D. | F0083(32)166 | 56 | 129 |

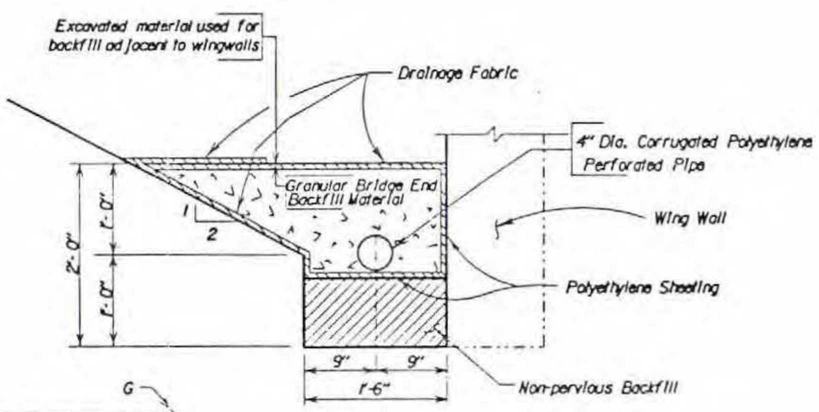



PLAN
(Abut. No. 1)

NOTE:
Bridge End Backfill shown adjacent to Abut. No. 1, Abut. No. 4 will be similar by rotation except as shown.



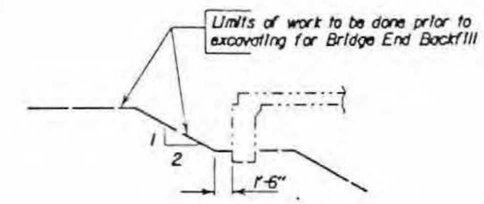

DETAIL "X"
(at ℄ Roadway)



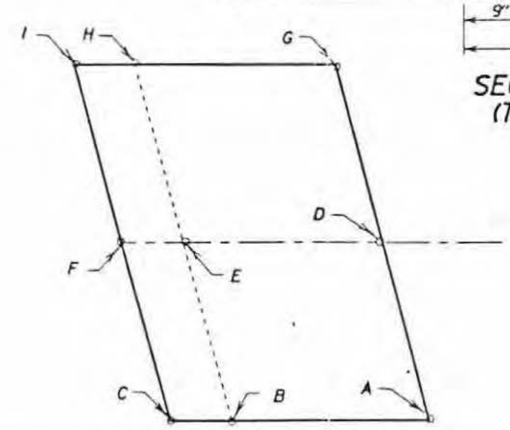

SEC. C-C
(TYP.)

# NOTES REGARDING BRIDGE END BACKFILL

1. For Informational purposes only, the total length of 4" Dia. Corrugated Polyethylene Perforated Plastic Pipe or Tubing for 2 abutments is 107 feet.
2. For Informational purposes only, the total length of 4" Dia. Corrugated Polyethylene Plastic Pipe or Tubing for 2 abutments is 20 feet.
3. For Informational purposes only, the total estimated length of 4" Dia. Standard Black Steel Pipe for 2 abutments is 20 feet.
4. The total length of Bridge End Backfill Underdrain Pipe for 2 abutments is 147 feet, Including 4" Dia. Corrugated Polyethylene Plastic Pipe or Tubing, Perforated or Non-perforated and 4" Dia. Standard Black Steel Pipe.
5. Bridge End Backfill Excavation will not be measured. Plans quantity payment will be full compensation for this item. The estimated quantity for 2 abutments is 128.6 cu. yds. This quantity is based on a vertical sided excavation full width of the approach slab (See Section B - B ).
6. The total estimated theoretical volume of Ganular Bridge End Backfill for 2 abutments is 260.7 cu. yds. This quantity is based on a vertical sided excavation full width of the approach slab (See Section B - B). This quantity has been increased by a "Shrinkage" factor of 1.3 to convert final compacted volume to loose volume.
7. Granular Bridge End Backfill shall be placed to, but not exceed, the full width of the approach slab as shown in Section B - B.
8. Granular Bridge End Backfill material shall also be placed behind the abutment wing walls as shown in Section C - C.
9. Excavated material not used for backfill of abutments shall be used to build the spill cones around the ends of the wingwalls as directed by the Engineer.
10. The Drainage Fabric shall envelop the Granular Bridge End Backfill as shown on this sheet and shall meet the requirements of Section 831 of the Standard Specifications. All joints in the fabric shall be lapped a minimum of two feet. On slopes, the upper piece of fabric shall be lapped over the lower one (See Section A - A).
11. Polyethylene Sheeting shall be placed as detailed on this sheet and each layer shall be a minimum of 6 mils in thickness. Care shall be taken so as not to puncture the Polyethylene Sheeting during backfill operations.
12. All cost involved in furnishing and placing drainage fabric and polyethylene sheeting shall be included in the Bid price for Granular Bridge End Backfill.
13. For Informational purposes only, the estimated quantity of 6 mil polyethylene sheeting required is 4837 sq. ft. for 2 abutments.
14. For Informational purposes only, the estimated quantity of drainage fabric required is 5583 sq. ft. for 2 abutments.



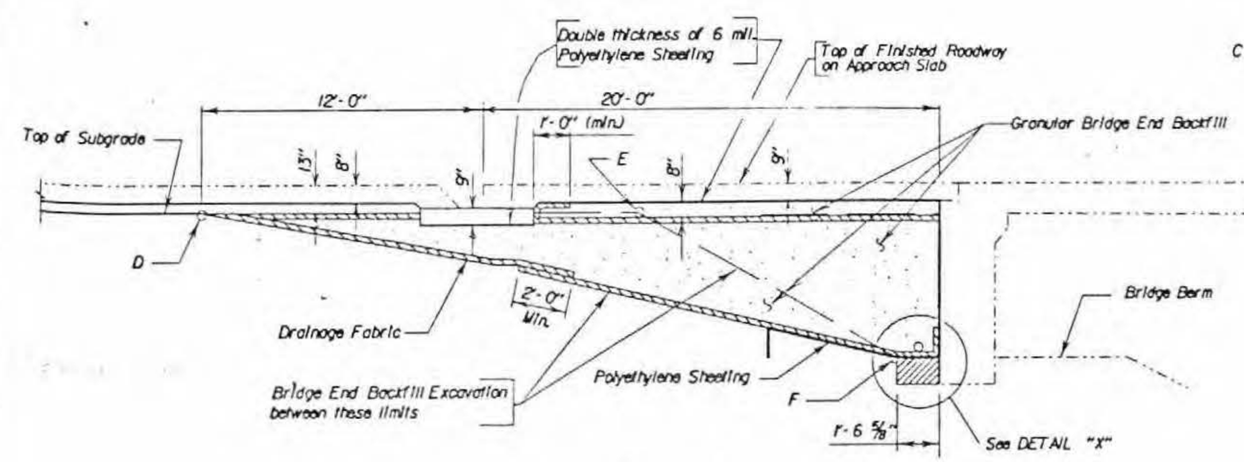

SEC. D-D
(TYP.)



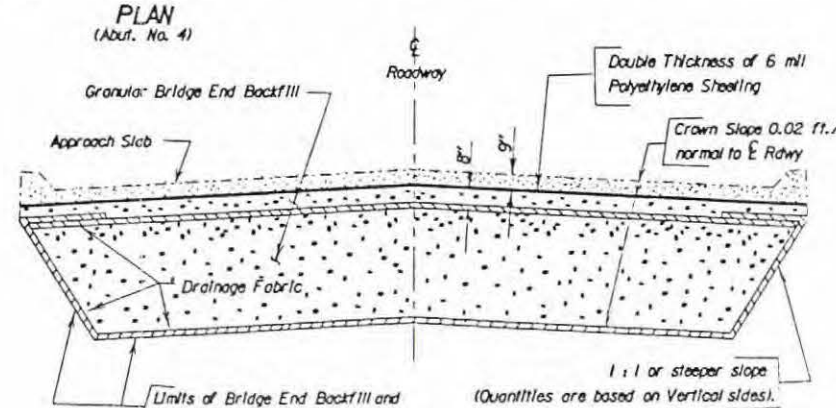
PLAN
(Abut. No. 4)

TABLE OF STATIONS AND ELEVATIONS

| POINT | | A | B | C | D | E | F | G | H | I |
|---|---|---|---|---|---|---|---|---|---|---|
| Begin Bridge | Station | 342+67.48 | 342+90.85 | 342+97.93 | 342+73.09 | 342+95.59 | 343+03.54 | 342+78.70 | 343+02.09 | 343+09.15 |
| | Elevation | 1882.44 | 1882.41 | 1878.99 | 1882.85 | 1882.83 | 1878.99 | 1882.43 | 1882.40 | 1878.99 |
| End Bridge | Station | 344+32.52 | 344+09.09 | 344+02.07 | 344+26.91 | 344+04.35 | 343+96.46 | 344+21.30 | 343+97.89 | 343+90.85 |
| | Elevation | 1882.27 | 1882.29 | 1878.90 | 1882.69 | 1882.71 | 1878.90 | 1882.28 | 1882.30 | 1878.90 |




SEC. A-A
(at ℄ Roadway)

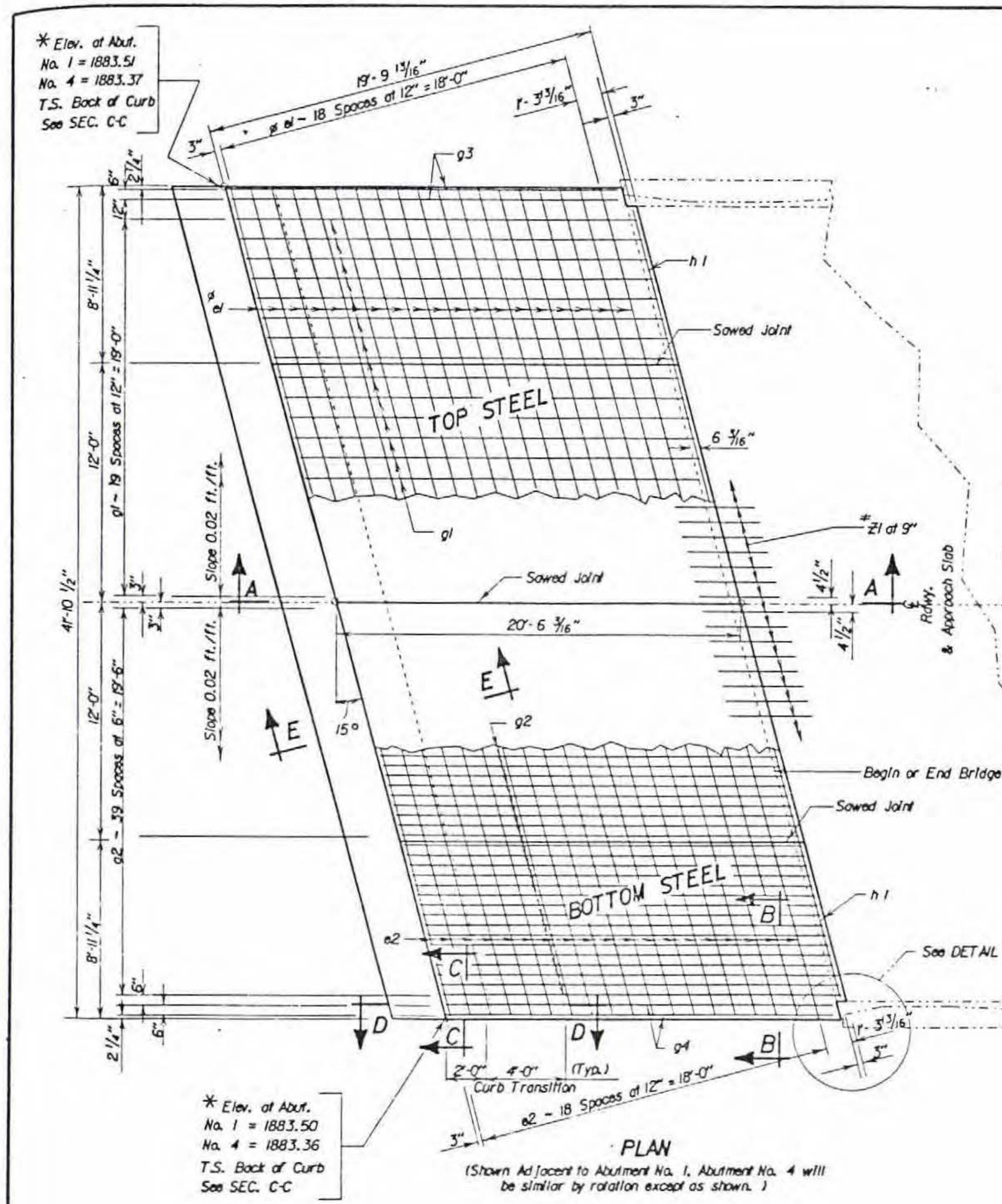



SEC. B-B

DETAILS OF BRIDGE END BACKFILL
FOR
89'-9 7/8" CONTINUOUS CONCRETE BRIDGE
40'- 0" ROADWAY 15° SKEW R.H.F.
OVER ARTICHOKE CREEK SEC. 26/25-T117N-R77W
STA. 343 + 05.09 TO 343 + 94.91 F0083(32)166
STR. NO. 54-160-224 HS20-44 (& ALT.)
POTTER COUNTY
S. D. DEPT. OF TRANSPORTATION
NOV. 1987 8 OF 11

| DESIGNED BY | DRAWN BY | CHECKED BY | APPROVED |
|---|---|---|---|
| S. J. & J.A. | L.M. | S. J. & J.A. | BRIDGE ENGINEER |

| FED.HWY. ADMIN.NO. | STATE OF | PROJECT | SHEET NO. | TOTAL SHEETS |
|---|---|---|---|---|
| 8 | S.D. | F0083(32)166 | 57 | 129 |



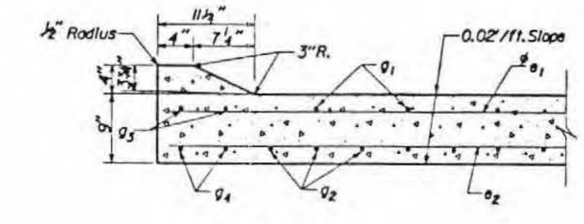

PLAN
(Shown Adjacent to Abutment No. 1, Abutment No. 4 will be similar by rotation except as shown.)

*NOTE:
Elevation Top of Slab Back of Curb at this point.

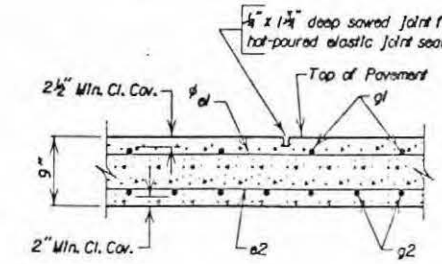
△ NOTE:
Stations true only at 70 Degrees F.

ø NOTE:
Min. Lap of 12" on all e1 bars.

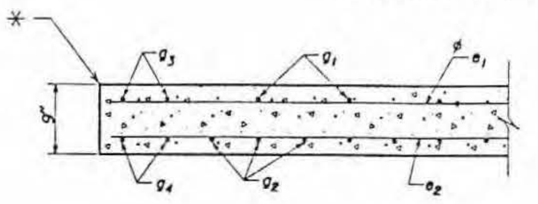
ẑ NOTE:
ẑ1 bars are listed and included in superstructure quantities.



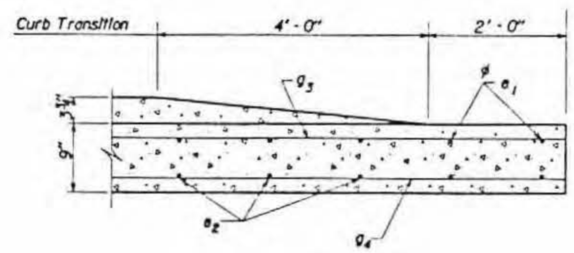

SEC. B - B



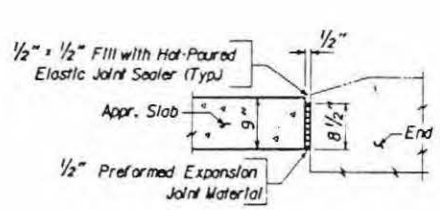

SAWED LONGITUDINAL JOINT



SEC. C - C
( Sleeper Slab Not Shown )




SEC. D - D
( Sleeper Slab Not Shown )

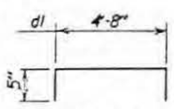



SEC. F - F

## REINFORCING SCHEDULE
(For Two Approach Slabs)

| Mk. | No. | Size | Length | Type |
|---|---|---|---|---|
| ø e1 | 76 | 3 | 22'- 1" | Str. |
| e2 | 38 | 6 | 43'- 1" | Str. |
| | | | | |
| ø g1 | 80 | 3 | 20'- 2" | Str. |
| g2 | 160 | 8 | 20'-2" | Str. |
| ø g3 | 8 | 3 | 19'-8" | Str. |
| g4 | 8 | 8 | 19'-8" | Str. |
| ø h1 | 4 | 6 | 41'- 1" | Str. |
| | | | | |
| c2 | 28 | 5 | 43'- 1" | Str. |
| d1 | 164 | 4 | 5'-6" | 2 |

Bending Details



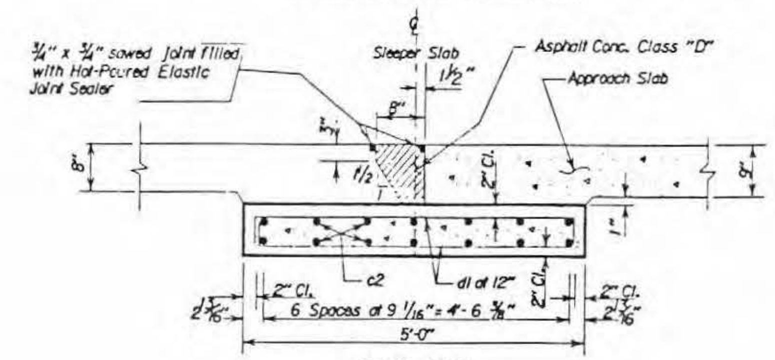

NOTE -
ø Indicates Bars to be Epoxy Coated.
All dimensions are out to out of bars.

## ESTIMATED QUANTITIES
(For Two Approach Slabs)

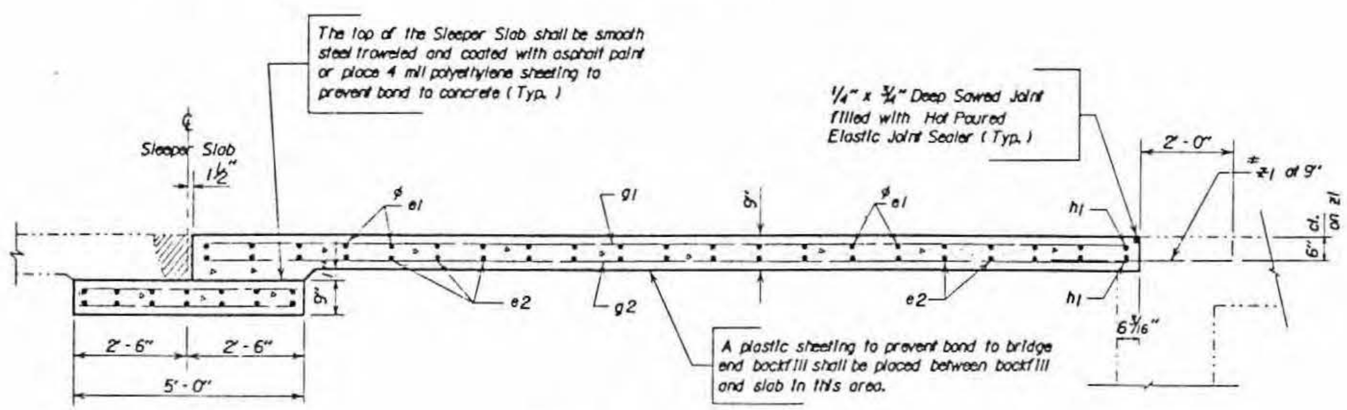
| ITEM | UNIT | QUANTITY |
|---|---|---|
| Conc. Approach Slab Adjacent to Bridge | Sq. Yd. | 190.7 |
| Bridge Approach Sleeper Slab | Sq. Yd. | 48.2 |

1. 1544 Lbs. Epoxy Coated Re-Steel in Approach Slabs.
2. 48.6 Cu. Yds. Concrete in Approach Slabs.
3. 11494 Lbs. Re-Steel in Approach Slabs.
4. 12.1 Cu. Yds. Concrete in Sleeper Slabs.
5. 1861 Lbs. Re-Steel in Sleeper Slabs.

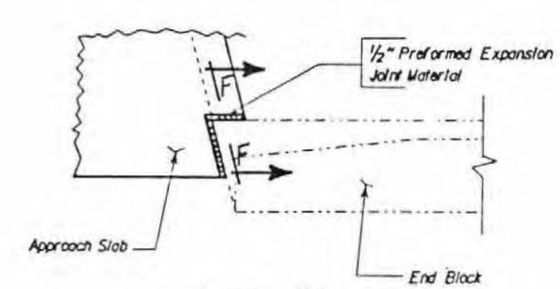
Items 1 thru 5 are approximate quantities contained in the above bid items and are for information only.




SEC. E-E




SEC. A - A



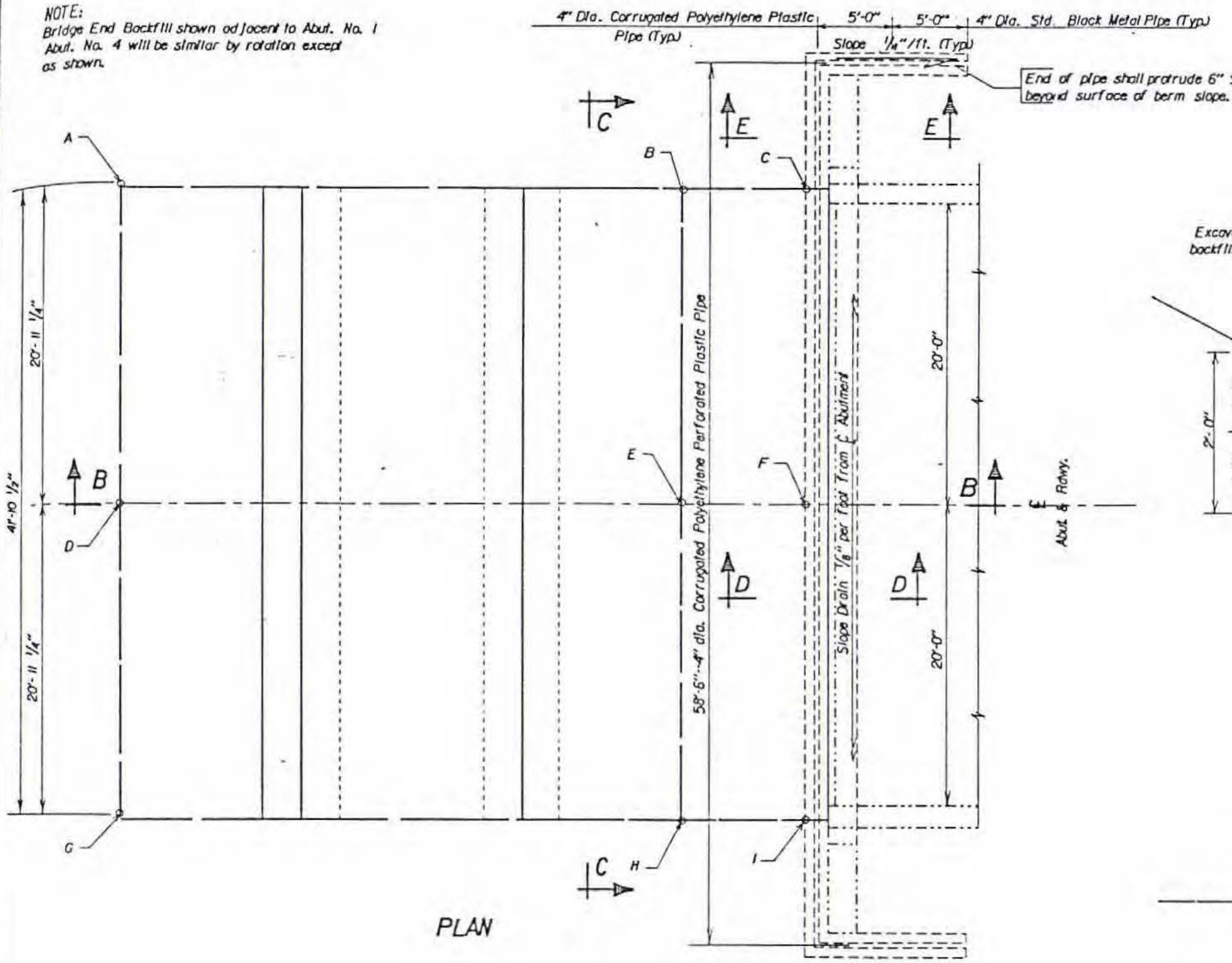

DETAIL "X"

DETAILS OF APPROACH SLAB ADJACENT TO BRIDGE
FOR
89'-9 7/8" CONTINUOUS CONCRETE BRIDGE
40'- 0" ROADWAY 15° SKEW R.H.F.
OVER ARTICHOKE CREEK SEC. 26/25-T117N-R77W
STA. 343+05.09 TO 343+94.91 F0083(32)166
STR. NO. 54-160-224 HS20-44 (& ALT.)
POTTER COUNTY
S. D. DEPT. OF TRANSPORTATION
NOV. 1987 9 OF 11

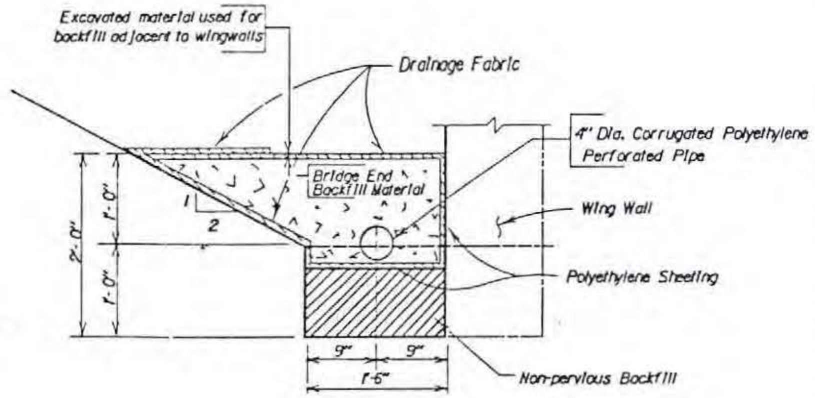
| DESIGNED BY | DRAWN BY | CHECKED BY | APPROVED |
|---|---|---|---|
| S.J. & J.A. | L.M. | S.J. & J.A. | BRIDGE ENGINEER |

| FED.HWY. ADMIN.NO. | STATE OF | PROJECT | SHEET NO. | TOTAL SHEETS |
|---|---|---|---|---|
| 8 | S.D. | BRF0038(10)302 | 42 | 53 |

NOTE:
Bridge End Backfill shown adjacent to Abut. No. 1
Abut. No. 4 will be similar by rotation except
as shown.



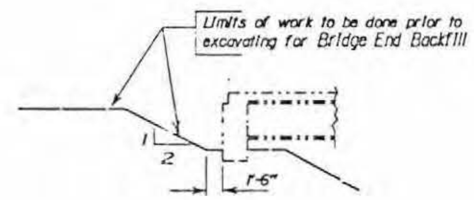

PLAN



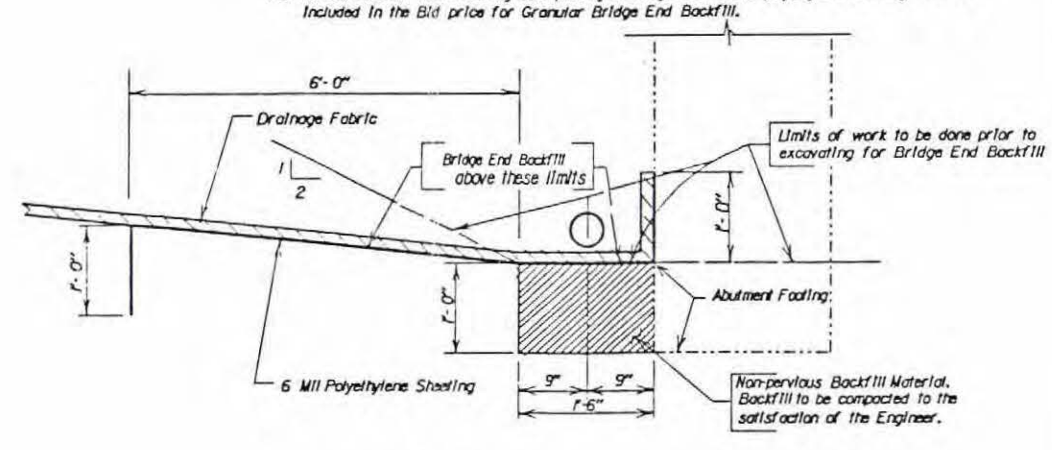

SEC. E-E
(TYP.)

## NOTES REGARDING BRIDGE END BACKFILL

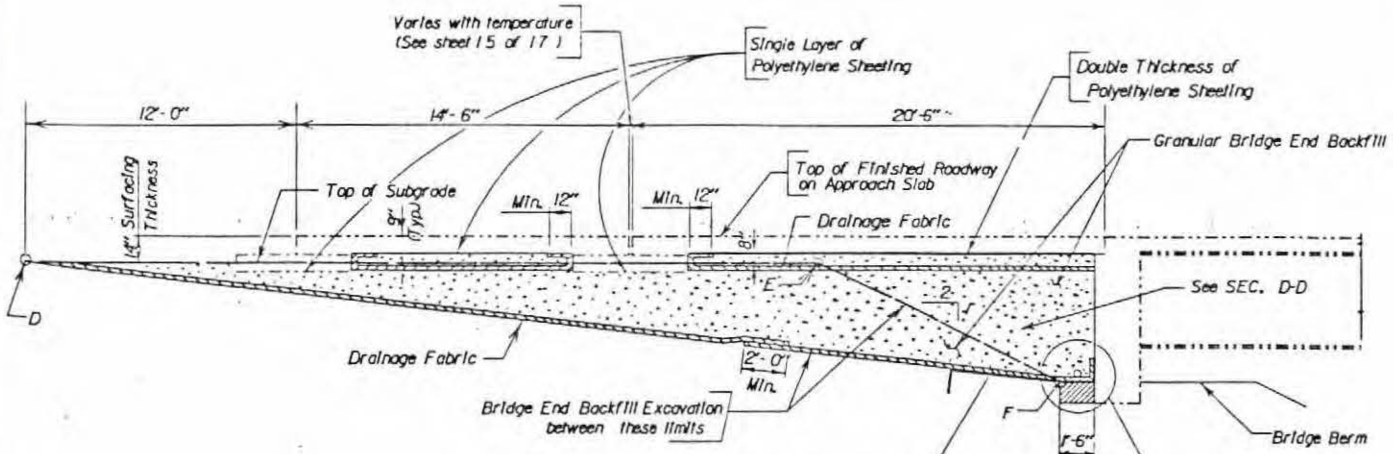
1. For Informational purposes only, the total length of 4" Dia. Corrugated Polyethylene Perforated Plastic Pipe of Tubing for 2 abutments is 117 feet.
2. For Informational purposes only, the total length of 4" Dia. Corrugated Polyethylene Plastic Pipe or Tubing for 2 abutments is 20 feet.
3. For Informational purposes only, the total estimated length of 4" Dia. Standard Black Steel Pipe for 2 abutments is 20 feet.
4. The total length of Bridge End Backfill Underdrain Pipe for 2 abutments is 157 feet, including 4" Dia. Corrugated Polyethylene Plastic Pipe or Tubing, Perforated or Non-perforated and 4" Dia. Standard Black Steel Pipe.
5. Bridge End Backfill Excavation will not be measured. Plans quantity payment will be full compensation for this item. The estimated quantity for 2 abutments is 276 cu. yds. This quantity is based on a vertical sided excavation full width of the approach slab (See Section C - C ).
6. The total estimated theoretical volume of Granular Bridge End Backfill for 2 abutments is 532 cu. yds. This quantity is based on a vertical sided excavation full width of the approach slab (See Section C - C). The quantity has been increased by a "Shrinkage" factor of 1.3 to convert final compacted volume to loose volume.
7. Granular Bridge End Backfill shall be placed to, but not exceed, the full width of the approach slab as shown in Section C - C.
8. Granular Bridge End Backfill material shall also be placed behind the abutment wing walls as shown in Section E - E.
9. Excavated material not used for backfill of abutments shall be used to build the spill cones around the ends of the wingwalls as directed by the Engineer.
10. The Drainage Fabric shall envelop the Granular Bridge End Backfill as shown on this sheet and shall meet the requirements of Section 831 of the Standard Specifications. All joints in the fabric shall be lapped a minimum of two feet. On slopes, the upper piece of fabric shall be lapped over the lower one (See Section B - B).
11. Polyethylene Sheeting shall be placed as detailed on this sheet and each layer shall be a minimum of 6 mils in thickness. Care shall be taken so as not to puncture the Polyethylene Sheeting during backfill operations.
12. All cost involved in furnishing and placing drainage fabric and polyethylene sheeting shall be included in the Bid price for Granular Bridge End Backfill.




SEC. D-D
(TYP.)



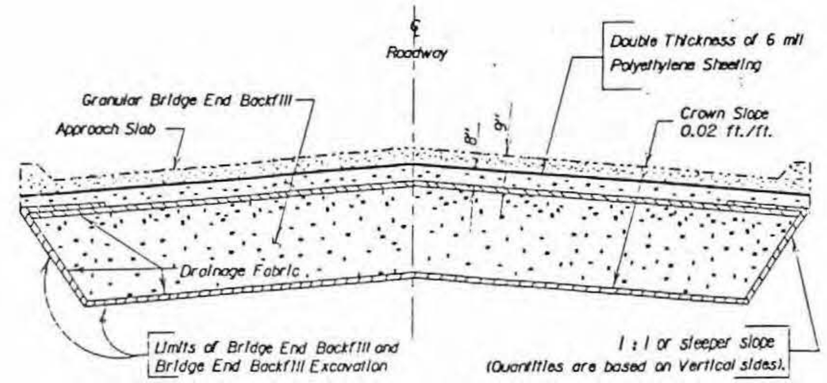

DETAIL "X"
(at ℄ Roadway)




SEC. B-B
(℄ Roadway)

## TABLE OF STATIONS AND ELEVATIONS

| POINT | | A | B | C | D | E | F | G | H | I |
|---|---|---|---|---|---|---|---|---|---|---|
| Begin Bridge | Station | 48+54.77 | 48+90.44 | 49+00.00 | 48+54.77 | 48+89.61 | 49+00.00 | 48+54.77 | 48+90.44 | 49+00.00 |
| | Elevation | 1219.93 | 1220.20 | 1215.42 | 1220.35 | 1220.61 | 1215.42 | 1219.93 | 1220.20 | 1215.42 |
| End Bridge | Station | 52+55.73 | 52+20.72 | 52+10.50 | 52+55.73 | 52+21.58 | 52+10.50 | 52+55.73 | 52+20.72 | 52+10.50 |
| | Elevation | 1225.15 | 1224.49 | 1219.38 | 1225.57 | 1224.92 | 1219.38 | 1225.15 | 1224.49 | 1219.38 |



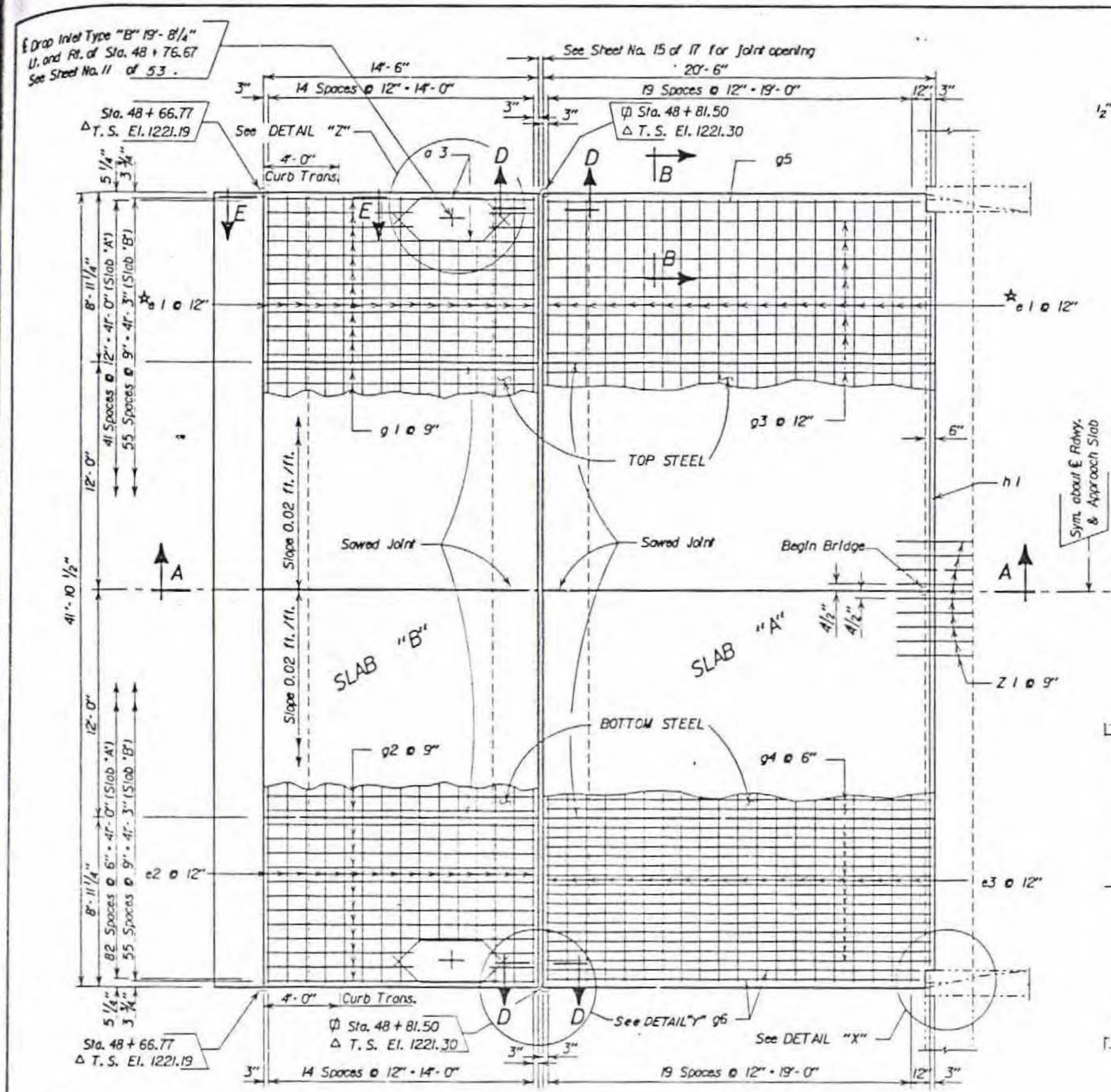

SEC. C-C

DETAILS OF BRIDGE END BACKFILL
FOR
307'-6" CONT. COMP. GIRDER BRIDGE
40'-0" ROADWAY 0° SKEW
OVER JAMES RIVER SEC. 24/25-T103N-R60W
STA. 49+01.50 TO 52+09.00 BRF0038(10)302
STR. NO. 18-180-100 HS20-44 (& ALT.)
DAVISON COUNTY
S. D. DEPT. OF TRANSPORTATION
JUNE 1987 13 OF 17

| DESIGNED BY | DRAWN BY | CHECKED BY | APPROVED |
|---|---|---|---|
| K.G./S.J. | LAK | K.G./S.J. | BRIDGE ENGINEER |

Rev. 12-23-87 KG

| FED.HWY. ADMIN.NO. | STATE OF | PROJECT | SHEET NO. | TOTAL SHEETS |
|---|---|---|---|---|
| 8 | S.D. | BRF0038(10)302 | 43 | 53 |



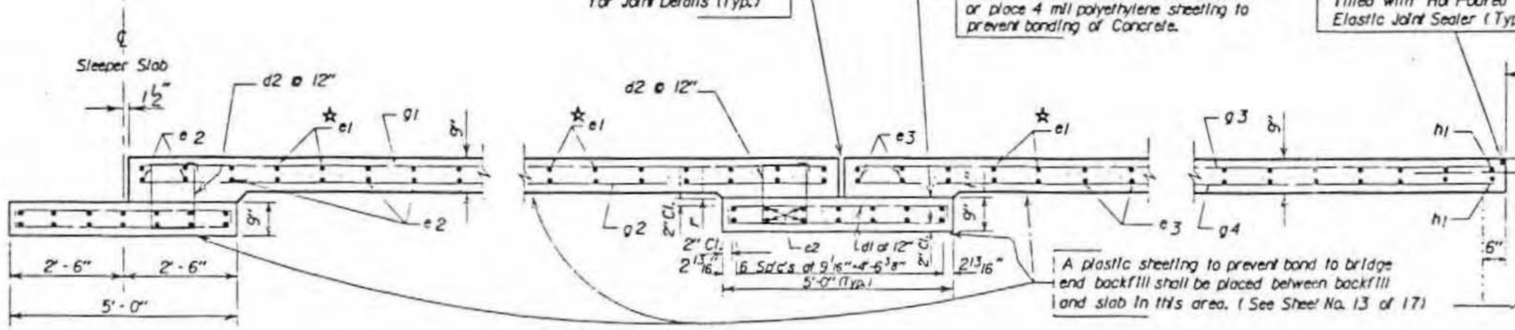

Δ NOTE: Elevation Top Slab back of Curb to this point.

℄ NOTE: Stations true only at 70° F.

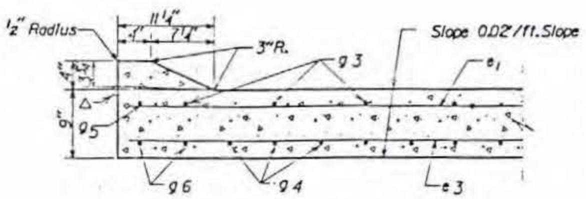
☆ NOTE: 12" Min. Lap on all e1 bars.

PLAN

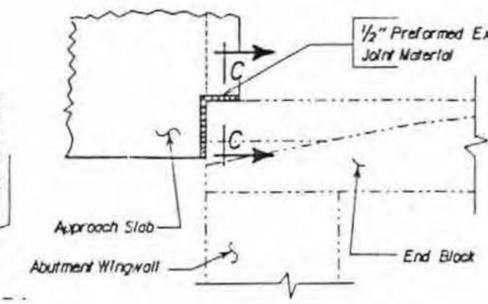
(Shown adjacent to Abut. No. 1, Abut. No. 4 will be similar by rotation except as shown.



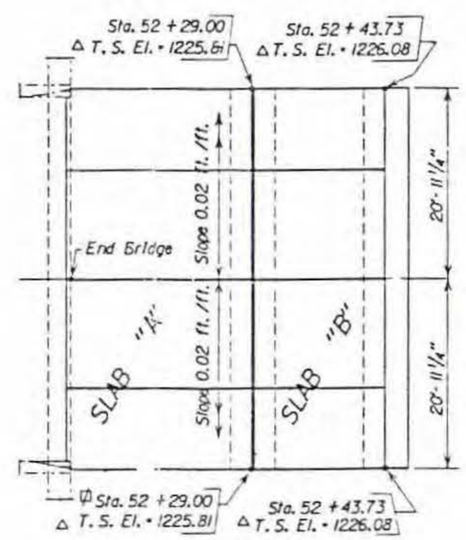

SEC. A - A




SEC. B - B

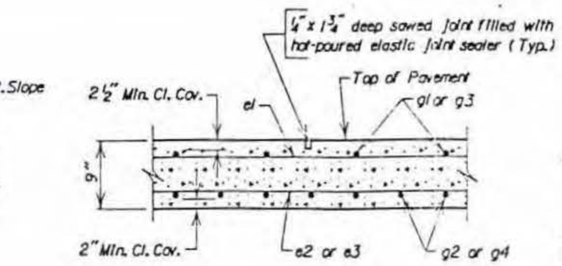



DETAIL "X"

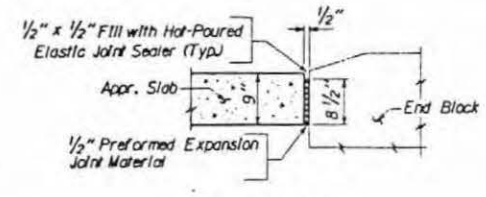



PLAN

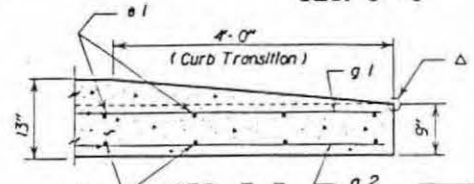
(At Abutment No. 4 )




SAWED LONGITUDINAL JOINT




SEC. C - C



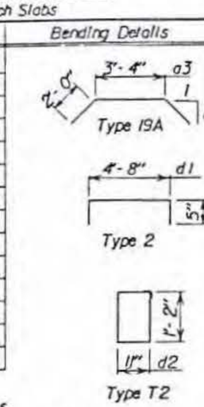

SEC. E- E

## REINFORCING SCHEDULE

For Two Approach Slabs

| | Mk. | No. | Size | Length | Type |
|---|---|---|---|---|---|
| * | a3 | 8 | 3 | 7'- 4" | 19A |
| | c2 | 56 | 5 | 41'- 6" | Str. |
| | d1 | 336 | 4 | 5'- 6" | 2 |
| * | d2 | 168 | 4 | 5'- 0" | T2 |
| * | e1 | 140 | 3 | 21'- 3" | Str. |
| | e2 | 30 | 4 | 41'- 6" | Str. |
| | e3 | 40 | 6 | 41'- 6" | Str. |
| * | g1 | 112 | 3 | 14'- 2" | Str. |
| | g2 | 112 | 7 | 14'- 2" | Str. |
| * | g3 | 80 | 3 | 20'- 2" | Str. |
| | g4 | 158 | 8 | 20'- 2" | Str. |
| * | g5 | 4 | 3 | 19'- 8" | Str. |
| | g6 | 8 | 8 | 19'- 8" | Str. |
| * | h1 | 4 | 6 | 39'- 8" | Str. |

Bending Details




NOTE –
All dimensions are out to out of bars.
$Z_1$ bars are listed and included in superstructure quantities
* Bars to be epoxy coated.

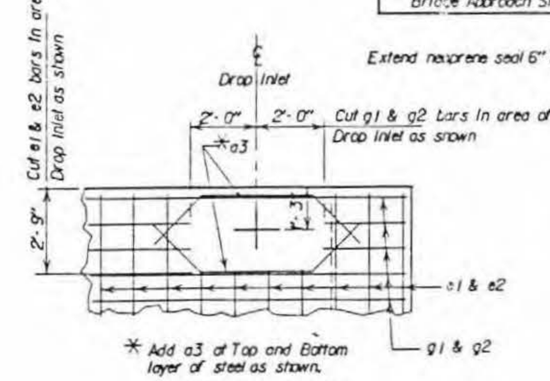
1. 84.5 Cu. Yds. Concrete in Approach Slabs.
2. 15496 Lbs. re-steel in Approach Slabs.
3. 2612 Lbs. Epoxy Coated Re-Steel in Approach Slabs
4. 23.3 Cu. Yds. Concrete in Sleeper Slabs
5. 3658 Lbs. Re-Steel in Sleeper Slabs
6. 561 Lbs. Epoxy Coated Re-Steel in Sleeper Slabs

Items 1 thru 6 are approximate quantities contained in the bid items above and are for information only. (The above Quantities are for 2 Approach Slabs).

## ESTIMATED QUANTITIES

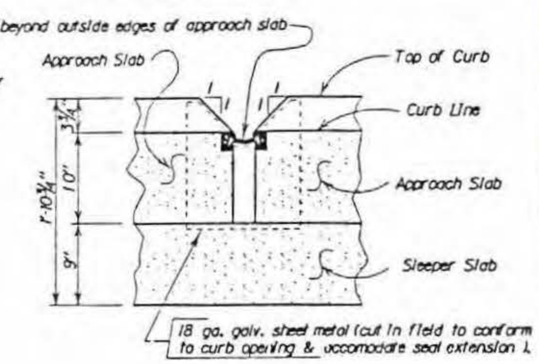
(For Two Approach Slabs)

| ITEM | UNIT | QUANTITY |
|---|---|---|
| Concrete Approach Slab Adjacent to Bridge | Sq. Yds. | 325.5 |
| Bridge Approach Sleeper Slab | Sq. Yds. | 93.1 |

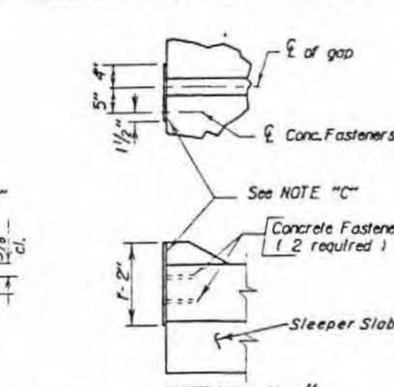



DETAIL "Z"

(Typical Plan for Steel Revision when Drop Inlet is used.)




SEC. D - D




DETAIL "Y"

NOTE "C"
18 gauge galv. sheet metal. Attach to one side of joint only, after slabs "A" and "B" are poured. Use fasteners that will not spall the concrete as approved by the Engineer.

DETAILS OF APPROACH SLABS ADJACENT TO BRIDGE

FOR

307'-6" CONT. COMP. GIRDER BRIDGE

40'-0" ROADWAY 0° SKEW

OVER JAMES RIVER SEC. 24/25-T103N-R60W

STA. 49+01.50 TO 52+09.00 BRF0038(10)302

STR. NO. 18-180-100 HS20-44 (& ALT.)

DAVISON COUNTY

S. D. DEPT. OF TRANSPORTATION

MAY 1986



| DESIGNED BY | DRAWN BY | CHECKED BY | APPROVED |
|---|---|---|---|
| K.G./S.J. | L.M. | K.G./S.J. | BRIDGE ENGINEER |

# APPENDIX D

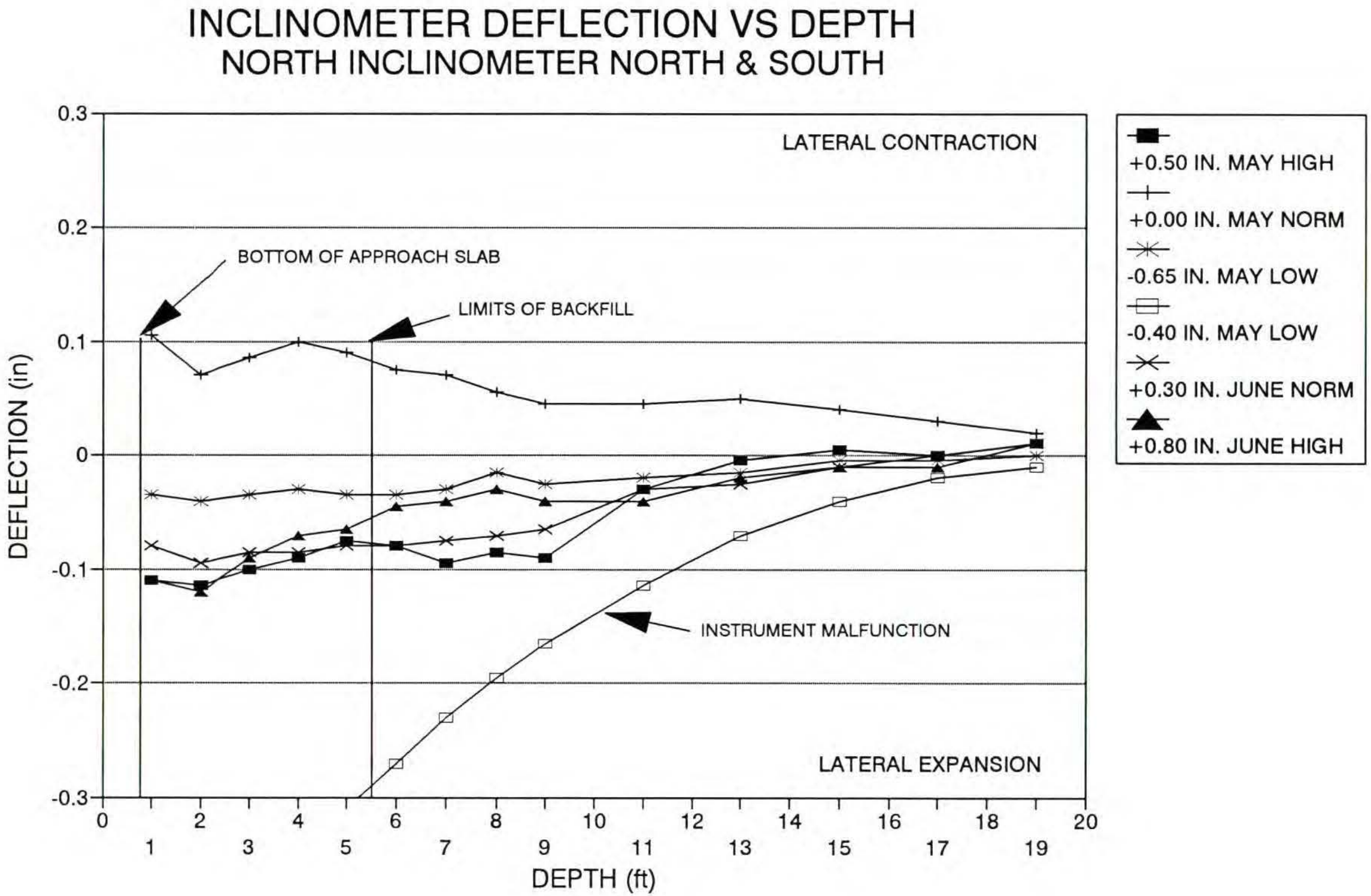
# MODEL TEST DATA

SDDOT Bridge End Backfill Study

South Dakota State University

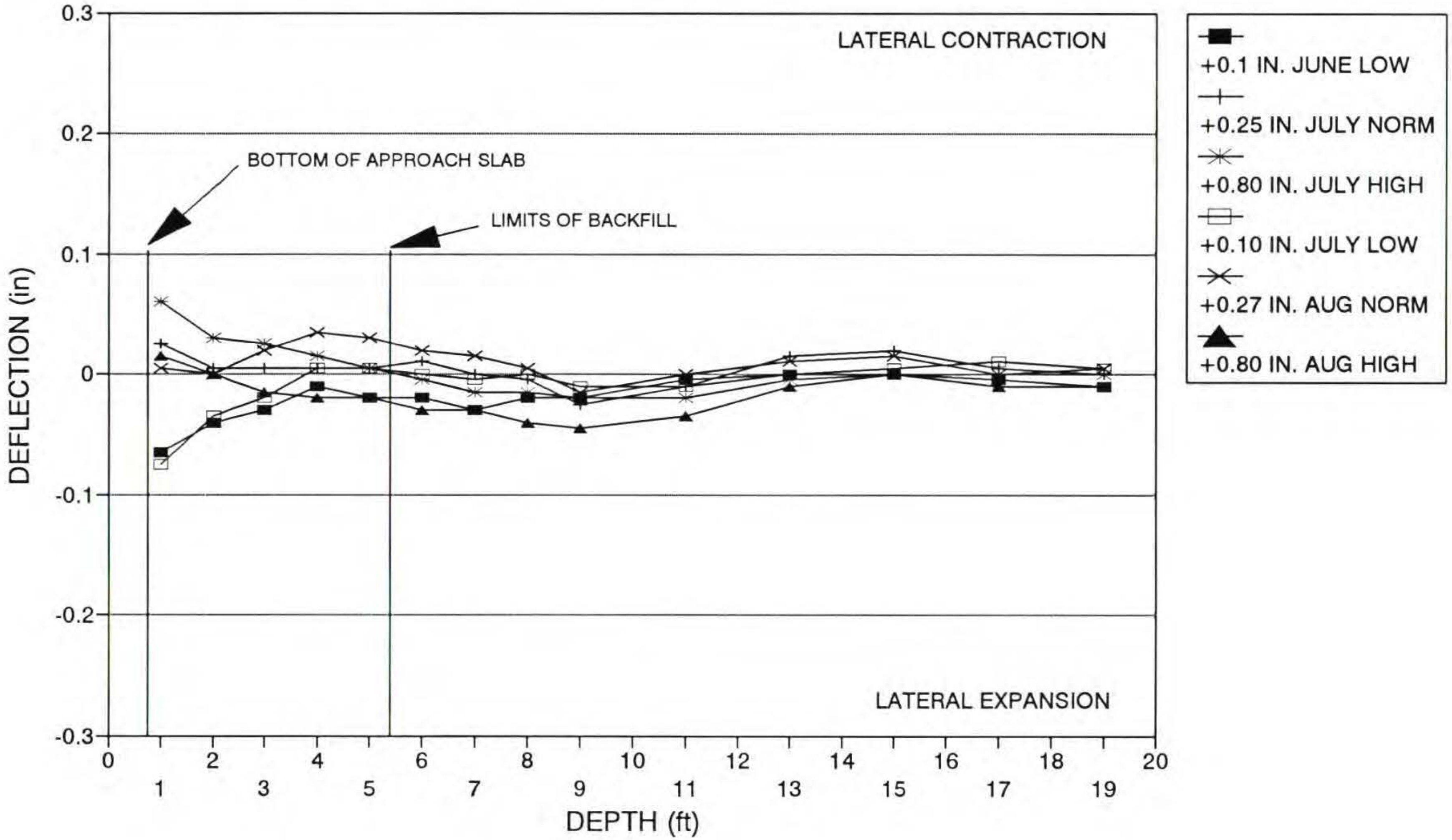
Abutment Movements and Rotations as Measured With Dial Indicators

| Date | Cycle | Abutment Movement (in) | Movements according to Dial Indicators | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | S. Top (in.) | S. Bottom (in.) | Rotation (deg) | N. Top (in.) | N. Bottom (in.) | Rotation (deg) |
| 5-20-92 | May norm to May high | 0.5 | 0.540 | 0.425 | 0.183 | 0.572 | 0.524 | 0.076 |
| 5-20-92 | May high to May norm | -0.5 | -0.549 | -0.435 | -0.181 | -0.622 | -0.568 | -0.086 |
| 5-21-92 | May norm to June norm | -0.4 | -0.185 | -0.147 | -0.060 | -0.242 | -0.251 | 0.014 |
| 5-26-92 | June norm to June max | 1.2 | 1.230 | 1.224 | 0.010 | 1.269 | err | err |
| 5-26-92 | June max to June low | -0.6 | -0.572 | -0.555 | -0.027 | -0.580 | -0.555 | -0.040 |
| 5-27-92 | June low to July norm | 0.13 | 0.182 | 0.176 | 0.010 | 0.185 | 0.174 | 0.018 |
| 5-27-92 | July norm to July max | 0.46 | 0.433 | 0.416 | 0.027 | 0.439 | 0.403 | 0.057 |
| 5-27-92 | July max to July min | -0.73 | -0.694 | -0.667 | -0.043 | -0.706 | -0.655 | -0.081 |
| 5-27-92 | July min to Aug norm | 0.17 | 0.145 | 0.145 | 0.000 | 0.152 | 0.146 | 0.010 |
| 5-28-92 | Aug norm to Aug high | 0.53 | 0.475 | 0.455 | 0.032 | 0.491 | 0.462 | 0.046 |
| 5-28-92 | Aug high to Aug low | -0.96 | -0.923 | -0.891 | -0.051 | -0.930 | -0.850 | -0.127 |
| 5-28-92 | Aug low to Sept norm | 0.29 | 0.273 | 0.271 | 0.003 | 0.276 | 0.226 | 0.080 |
| 6-2-92 | Sept norm to Sept high | 0.62 | 0.555 | 0.560 | -0.008 | 0.603 | 0.568 | 0.056 |
| 6-2-92 | Sept high to Sept low | -0.4 | -0.791 | -0.733 | -0.092 | -0.800 | -0.744 | -0.089 |
| 6-3-92 | Oct norm to Oct high | 0.77 | 0.753 | 0.739 | 0.022 | 0.672 | 0.631 | 0.065 |
| 6-3-92 | Oct high to Oct low | -1.02 | -0.757 | -0.784 | 0.043 | -0.846 | -0.816 | -0.048 |
| 6-4-92 | Oct low to Nov norm | -0.1 | -0.056 | -0.670 | 0.977 | -0.136 | 0.026 | -0.258 |
| 6-4-92 | Nov norm to Nov high | 0.7 | 0.552 | 0.595 | -0.068 | 0.649 | 0.565 | 0.134 |
| 6-4-92 | Nov high to Nov low | -0.81 | -0.675 | -0.692 | 0.027 | -0.728 | -0.701 | -0.043 |
| 6-5-92 | Nov low to Dec high | 0.91 | 0.750 | 0.766 | -0.025 | 0.838 | 0.800 | 0.060 |
| 6-8-92 | Dec high to Dec low | -0.9 | -0.810 | -0.816 | 0.010 | -0.856 | -0.780 | -0.121 |
| 6-8-92 | Dec low to Jan norm | 0.3 | 0.244 | 0.266 | -0.035 | 0.285 | 0.240 | 0.072 |
| 6-10-92 | Jan norm to Jan high | 0.74 | 0.666 | 0.554 | 0.178 | 0.675 | 0.542 | 0.212 |
| 6-10-92 | Jan high to Jan low | -1.04 | -0.936 | -0.856 | -0.127 | -1.004 | err | err |
| 6-10-92 | Jan low to Feb norm | 0.37 | 0.280 | 0.312 | -0.051 | 0.339 | 0.313 | 0.041 |
| 6-11-92 | Feb norm to Feb high | 0.93 | 0.929 | 0.905 | 0.038 | 0.940 | 0.886 | 0.086 |
| 6-11-92 | Feb high to Feb low | -1.1 | -1.089 | -1.054 | -0.056 | -1.095 | -1.034 | -0.097 |
| 6-11-92 | Feb low to March norm | 0.45 | 0.374 | 0.452 | -0.124 | 0.505 | 0.496 | 0.014 |
| 6-16-92 | Mar norm to Mar high | 1.05 | 1.027 | 0.996 | 0.049 | 1.029 | 0.975 | 0.086 |
| 6-18-92 | Mar. high to Mar. low | -1.2 | -0.945 | -1.105 | 0.255 | -1.148 | -1.055 | -0.148 |
| 6-18-92 | Mar. low to April norm | 0.58 | 0.314 | 0.496 | -0.290 | 0.513 | 0.487 | 0.041 |
| 6-18-92 | April norm to Apr high | 0.88 | 0.776 | 0.760 | 0.025 | 0.791 | 0.731 | 0.095 |
| 6-19-92 | April high to April low | 1.16 | -1.039 | -1.030 | -0.014 | -1.080 | -1.005 | -0.119 |

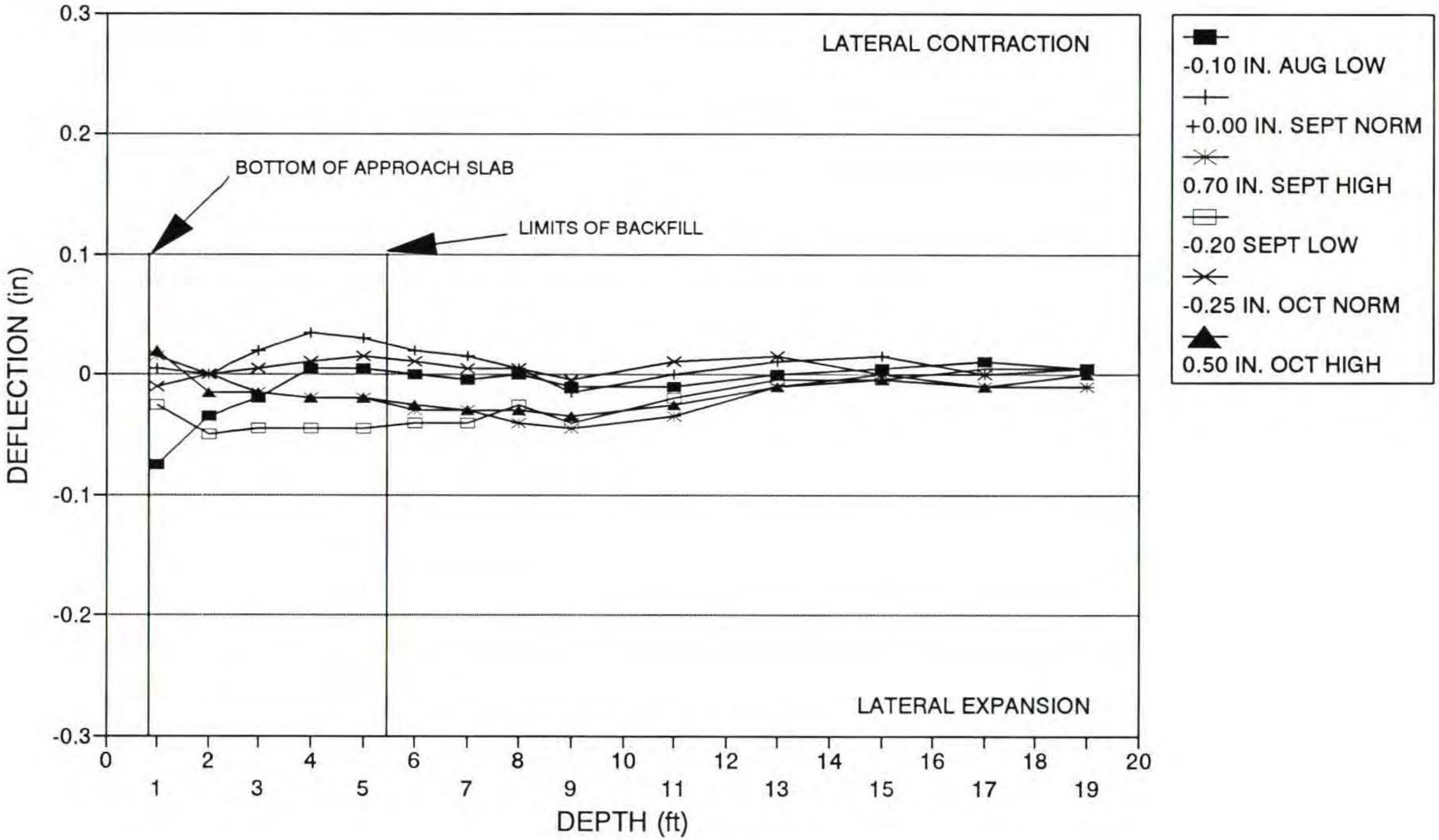


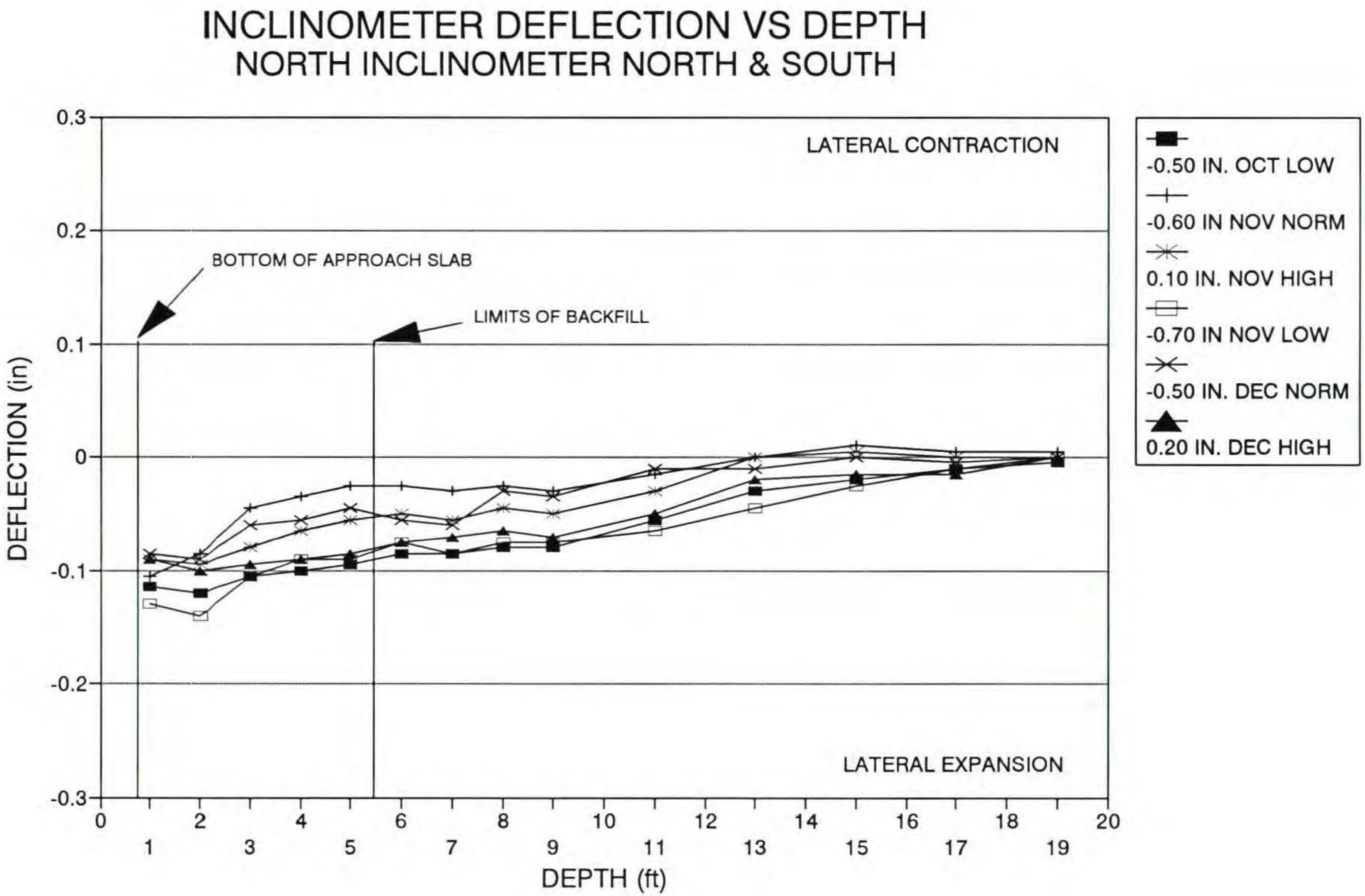
INCLINOMETER DEFLECTION VS DEPTH
NORTH INCLINOMETER NORTH & SOUTH
DEFLECTION (in)
DEPTH (ft)
LATERAL CONTRACTION
LATERAL EXPANSION
BOTTOM OF APPROACH SLAB
LIMITS OF BACKFILL
INSTRUMENT MALFUNCTION
+0.50 IN. MAY HIGH
+0.00 IN. MAY NORM
-0.65 IN. MAY LOW
-0.40 IN. MAY LOW
+0.30 IN. JUNE NORM
+0.80 IN. JUNE HIGH

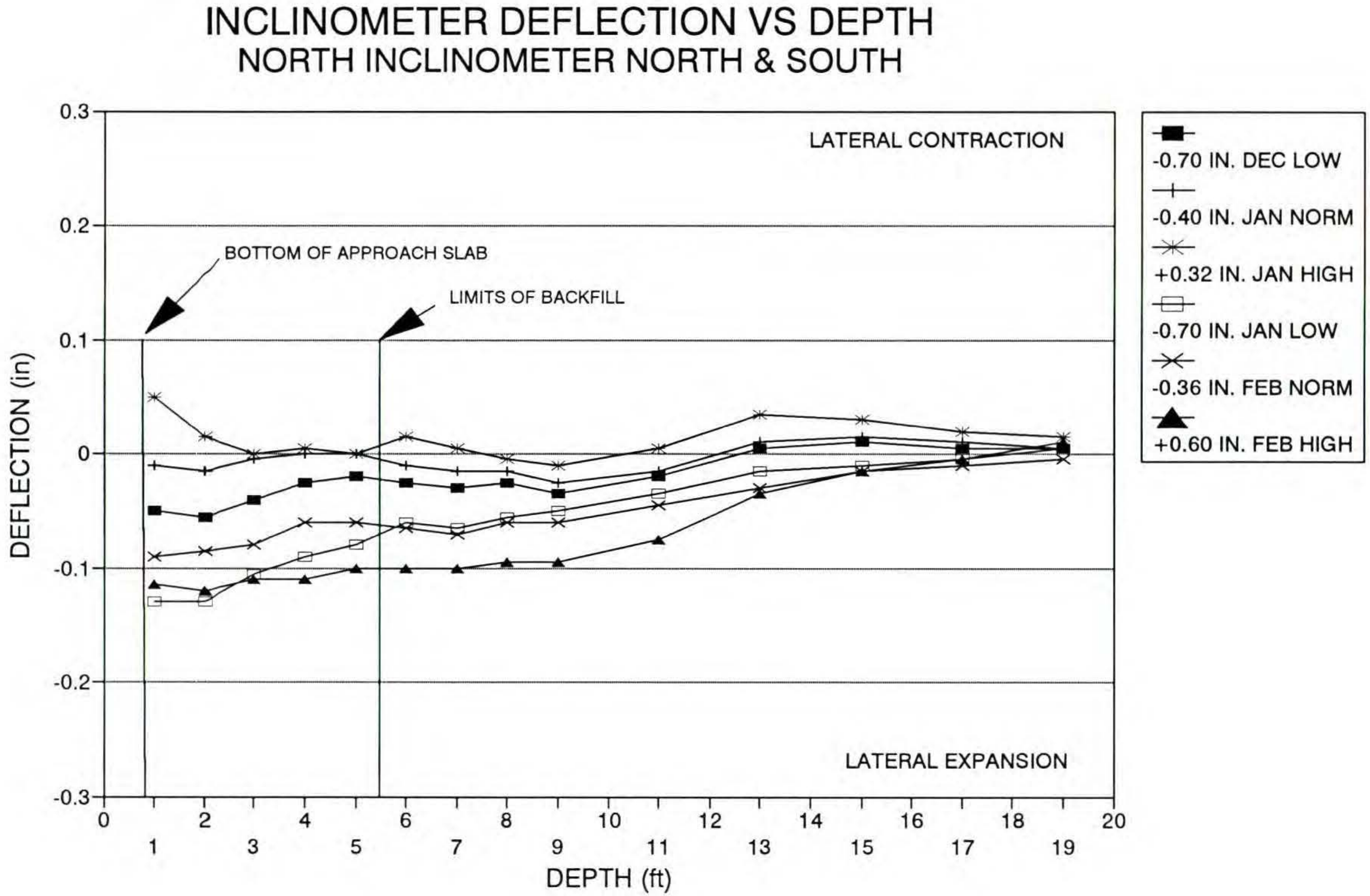
# INCLINOMETER DEFLECTION VS DEPTH
## NORTH INCLINOMETER NORTH & SOUTH



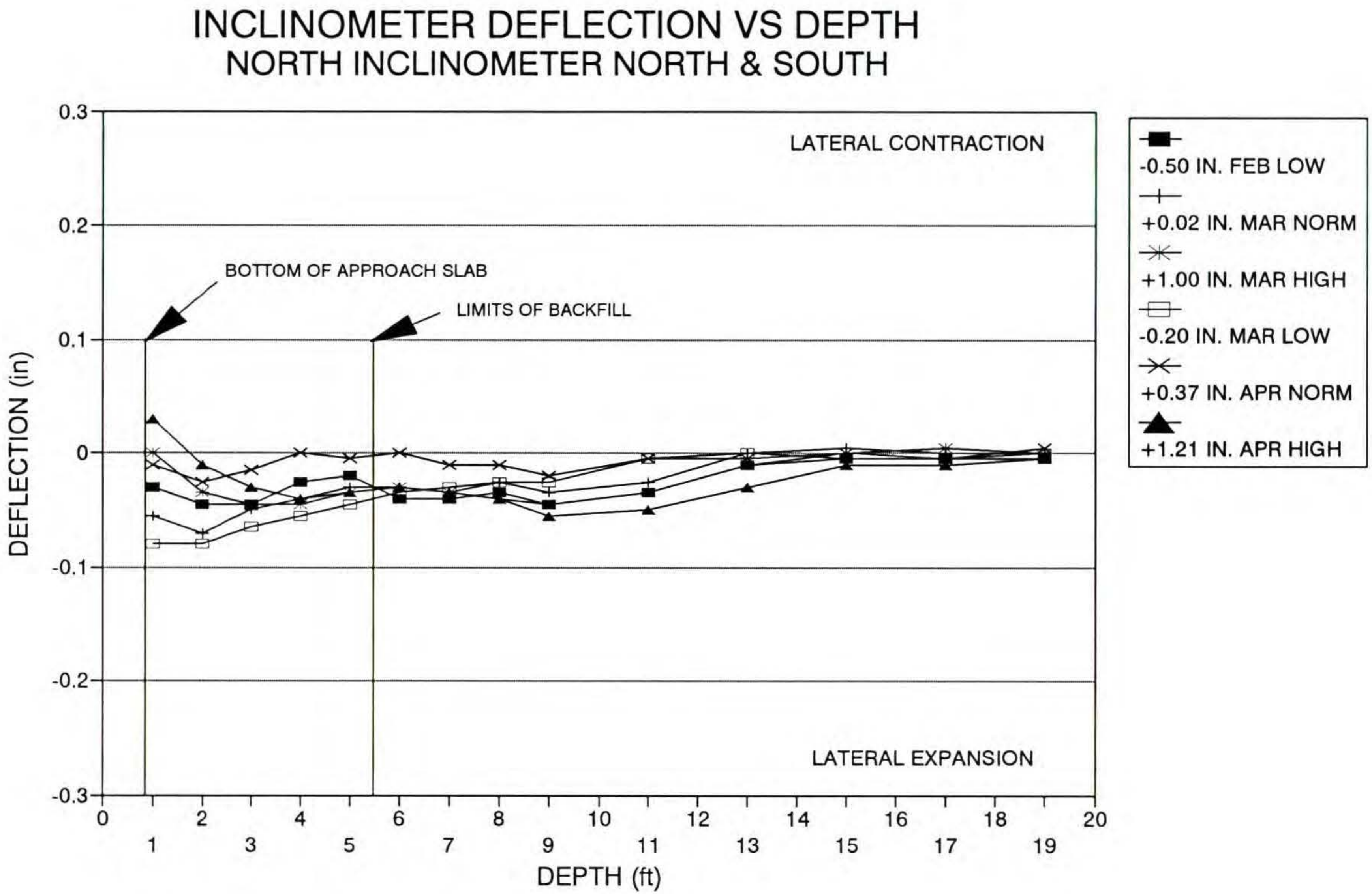
## INCLINOMETER DEFLECTION VS DEPTH
### NORTH INCLINOMETER NORTH & SOUTH

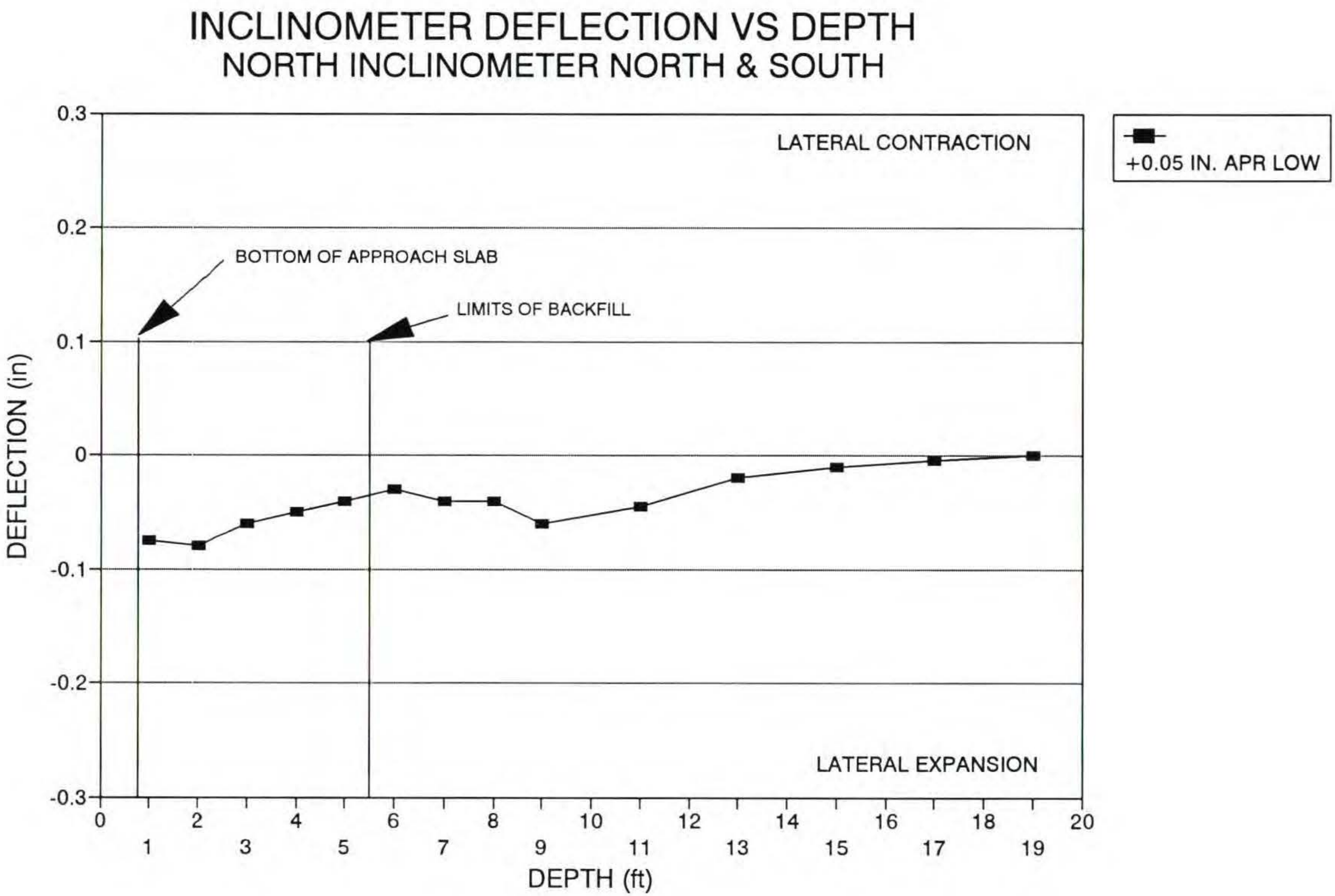


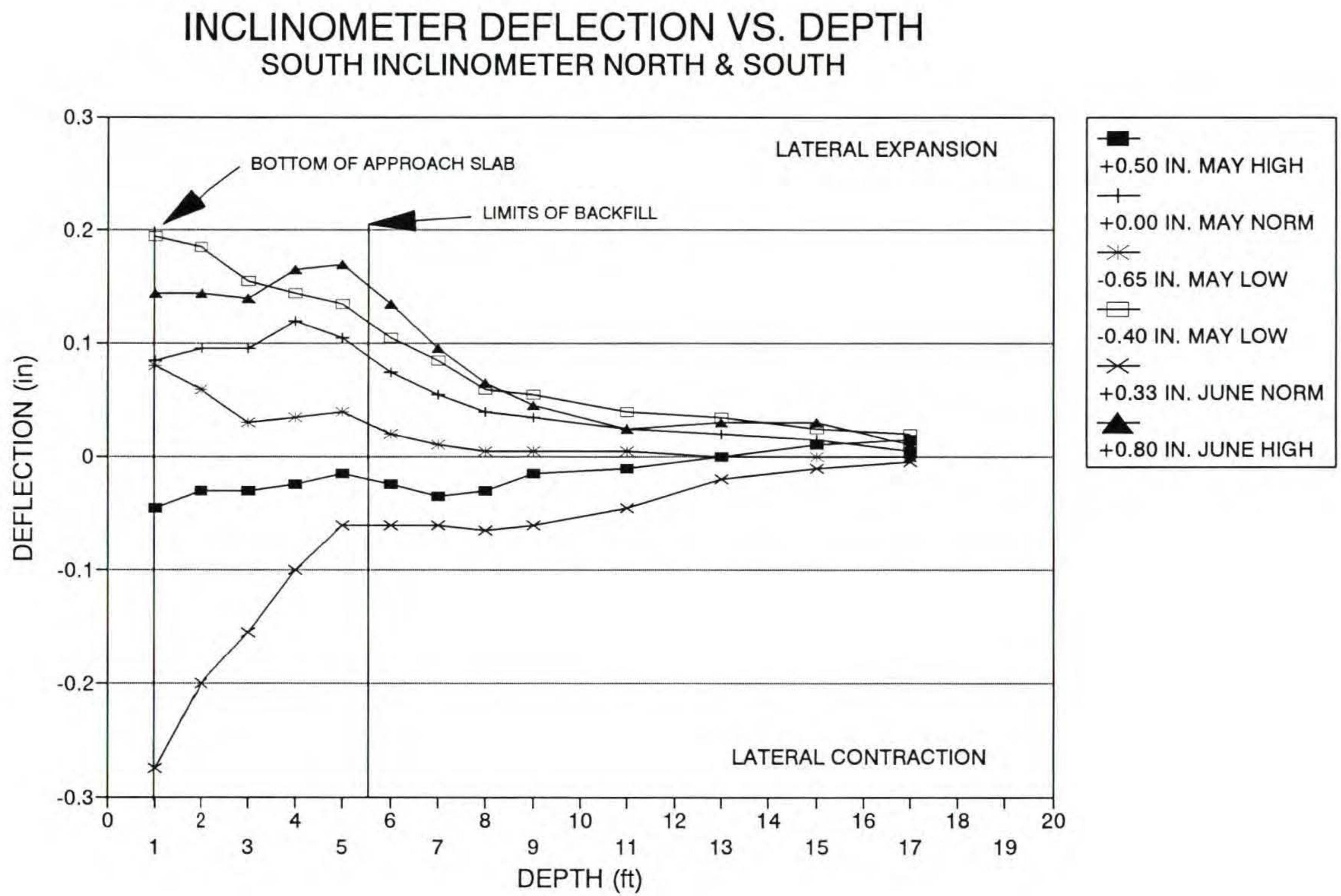

INCLINOMETER DEFLECTION VS DEPTH
NORTH INCLINOMETER NORTH & SOUTH
LATERAL CONTRACTION
LATERAL EXPANSION
BOTTOM OF APPROACH SLAB
LIMITS OF BACKFILL
DEFLECTION (in)
DEPTH (ft)
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
0
2
4
6
8
10
12
14
16
18
20
1
3
5
7
9
11
13
15
17
19
-0.50 IN. OCT LOW
-0.60 IN NOV NORM
0.10 IN. NOV HIGH
-0.70 IN NOV LOW
-0.50 IN. DEC NORM
0.20 IN. DEC HIGH



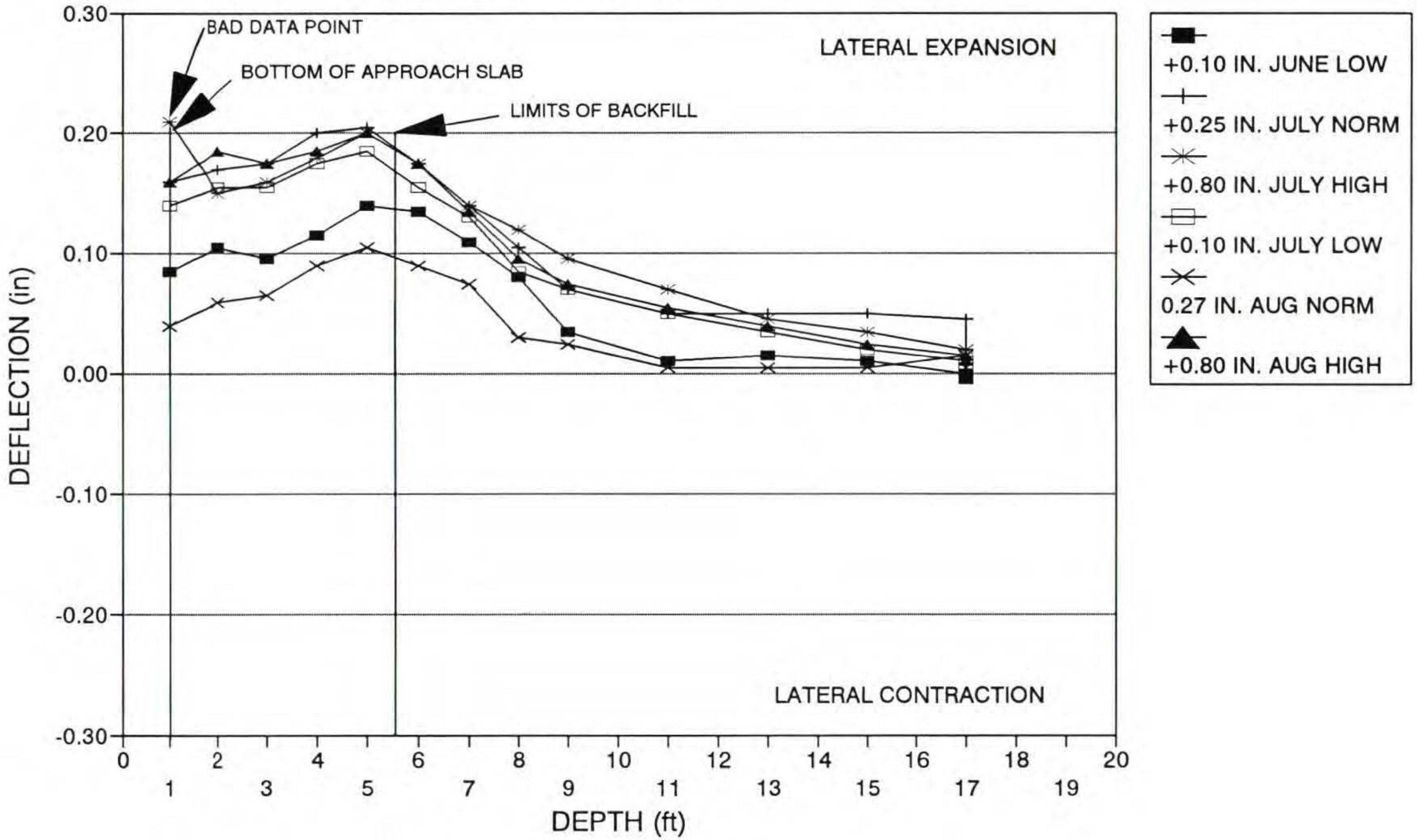
INCLINOMETER DEFLECTION VS DEPTH
NORTH INCLINOMETER NORTH & SOUTH
LATERAL CONTRACTION
LATERAL EXPANSION
BOTTOM OF APPROACH SLAB
LIMITS OF BACKFILL
DEFLECTION (in)
DEPTH (ft)
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
0 2 4 6 8 10 12 14 16 18 20
1 3 5 7 9 11 13 15 17 19
-0.70 IN. DEC LOW
-0.40 IN. JAN NORM
+0.32 IN. JAN HIGH
-0.70 IN. JAN LOW
-0.36 IN. FEB NORM
+0.60 IN. FEB HIGH

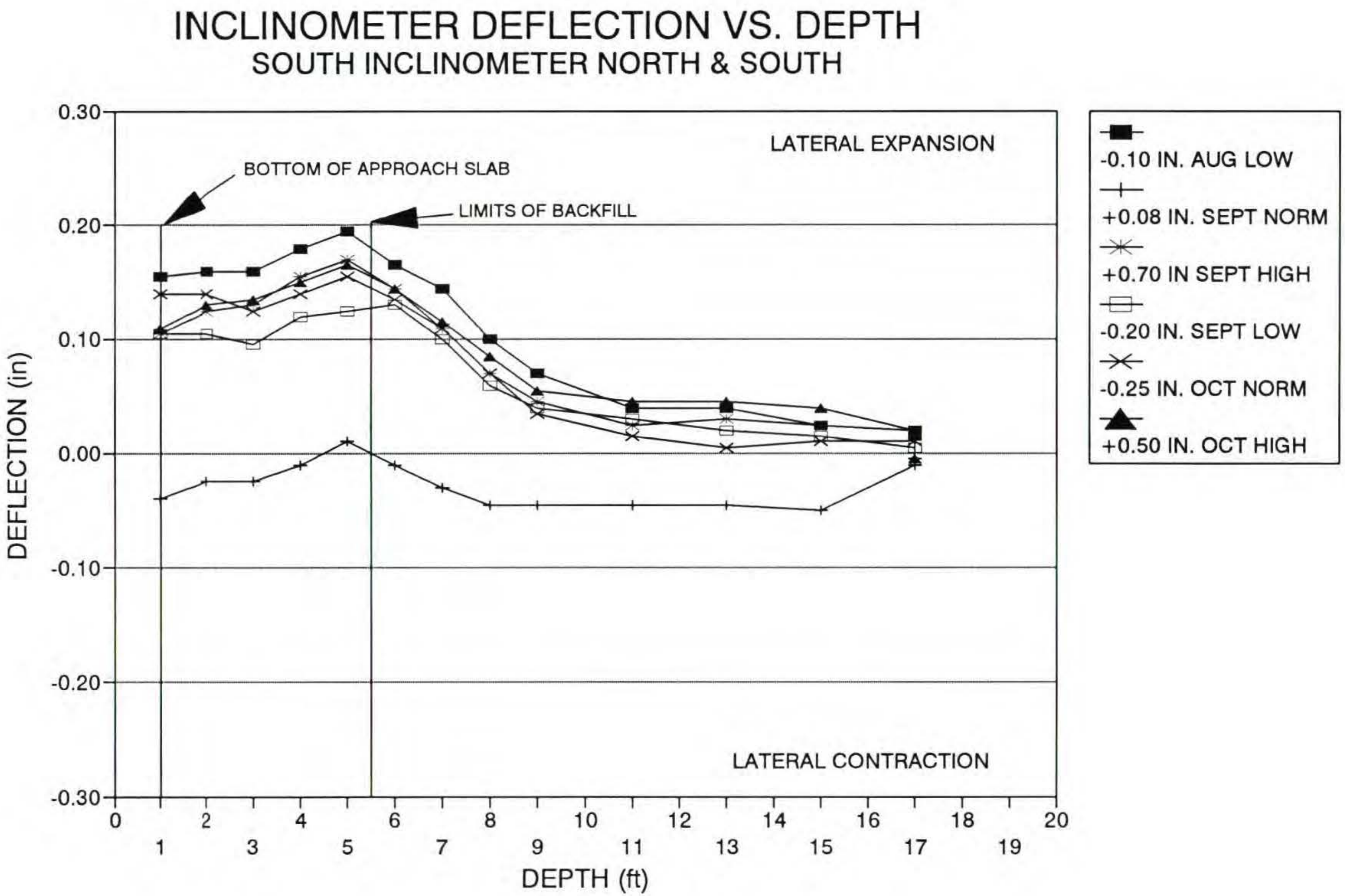

INCLINOMETER DEFLECTION VS DEPTH
NORTH INCLINOMETER NORTH & SOUTH
LATERAL CONTRACTION
BOTTOM OF APPROACH SLAB
LIMITS OF BACKFILL
LATERAL EXPANSION
DEFLECTION (in)
DEPTH (ft)
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
0 2 4 6 8 10 12 14 16 18 20
1 3 5 7 9 11 13 15 17 19
-0.50 IN. FEB LOW
+0.02 IN. MAR NORM
+1.00 IN. MAR HIGH
-0.20 IN. MAR LOW
+0.37 IN. APR NORM
+1.21 IN. APR HIGH


INCLINOMETER DEFLECTION VS DEPTH
NORTH INCLINOMETER NORTH & SOUTH
+0.05 IN. APR LOW
LATERAL CONTRACTION
BOTTOM OF APPROACH SLAB
LIMITS OF BACKFILL
LATERAL EXPANSION
DEFLECTION (in)
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
0 2 4 6 8 10 12 14 16 18 20
1 3 5 7 9 11 13 15 17 19
DEPTH (ft)

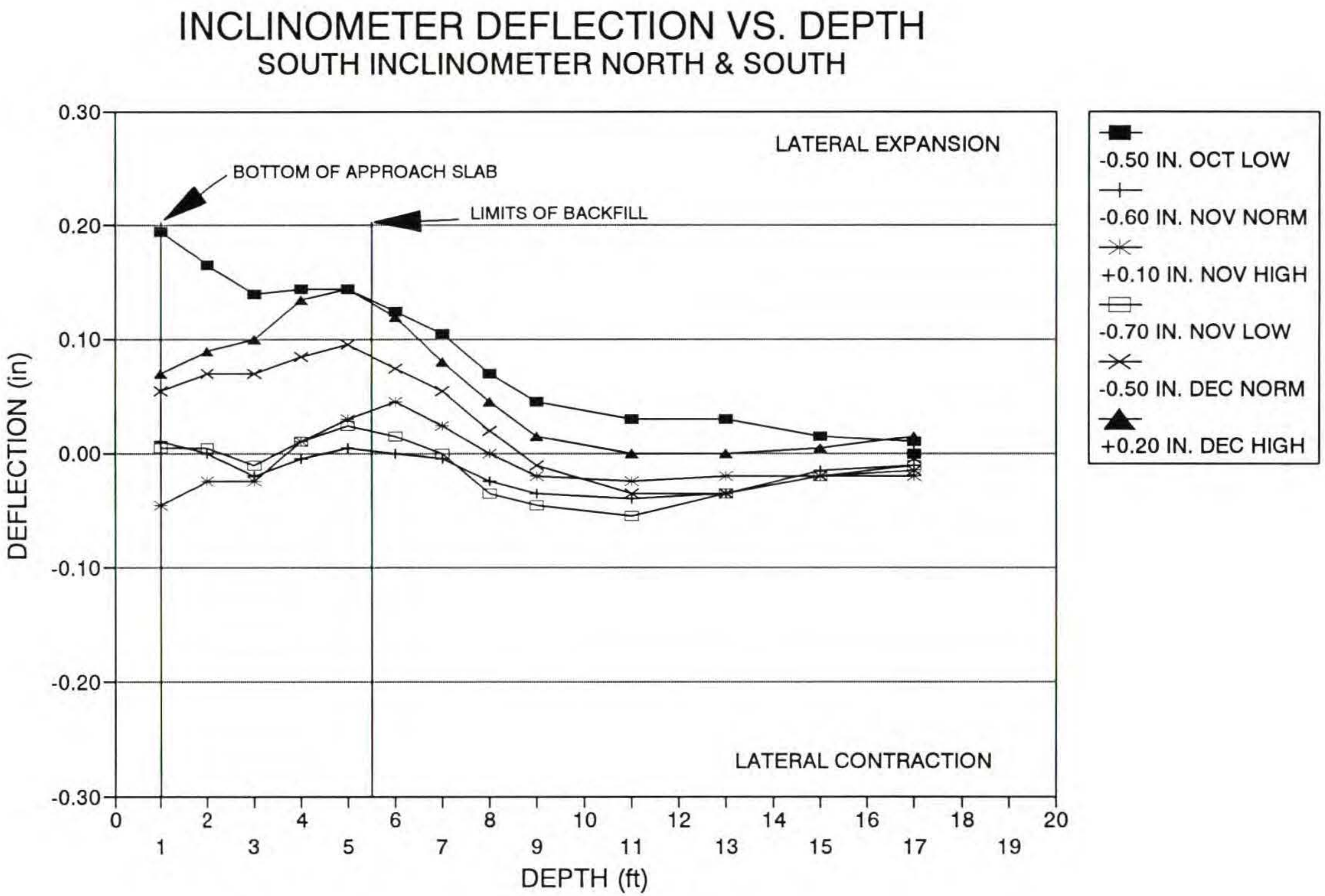

INCLINOMETER DEFLECTION VS. DEPTH
SOUTH INCLINOMETER NORTH & SOUTH
BOTTOM OF APPROACH SLAB
LIMITS OF BACKFILL
LATERAL EXPANSION
LATERAL CONTRACTION
DEFLECTION (in)
DEPTH (ft)
+0.50 IN. MAY HIGH
+0.00 IN. MAY NORM
-0.65 IN. MAY LOW
-0.40 IN. MAY LOW
+0.33 IN. JUNE NORM
+0.80 IN. JUNE HIGH

# INCLINOMETER DEFLECTION VS. DEPTH
## SOUTH INCLINOMETER NORTH & SOUTH

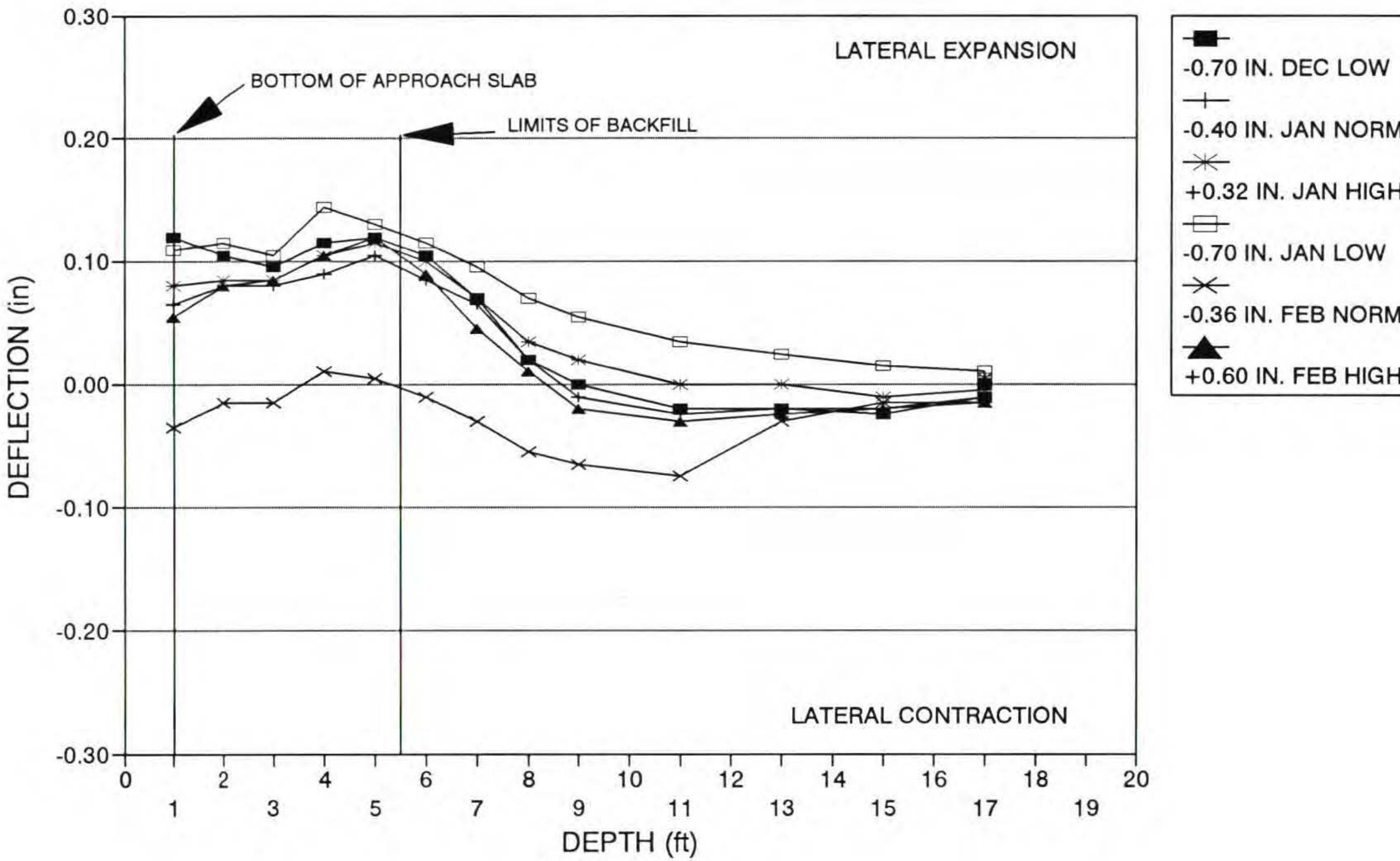


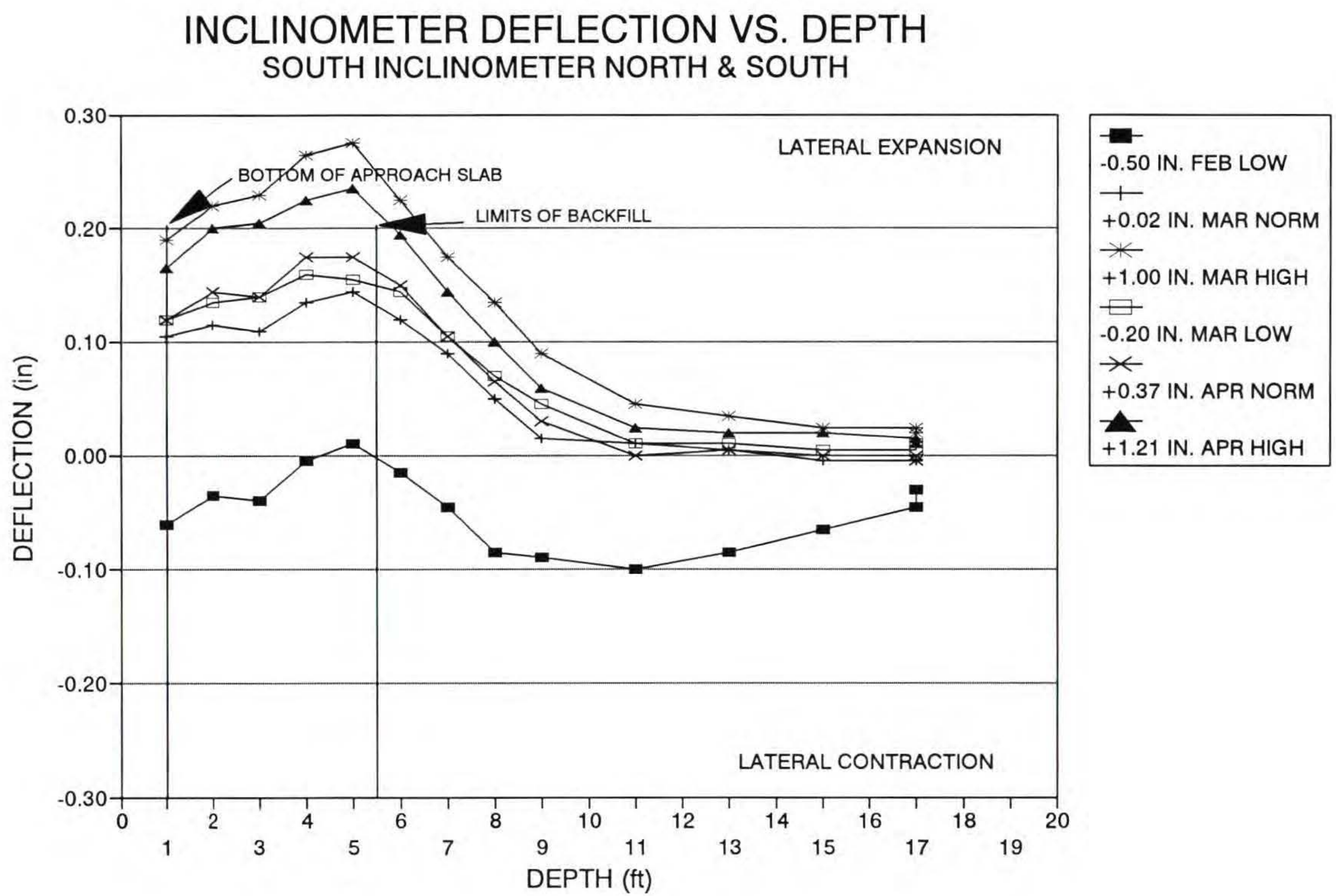


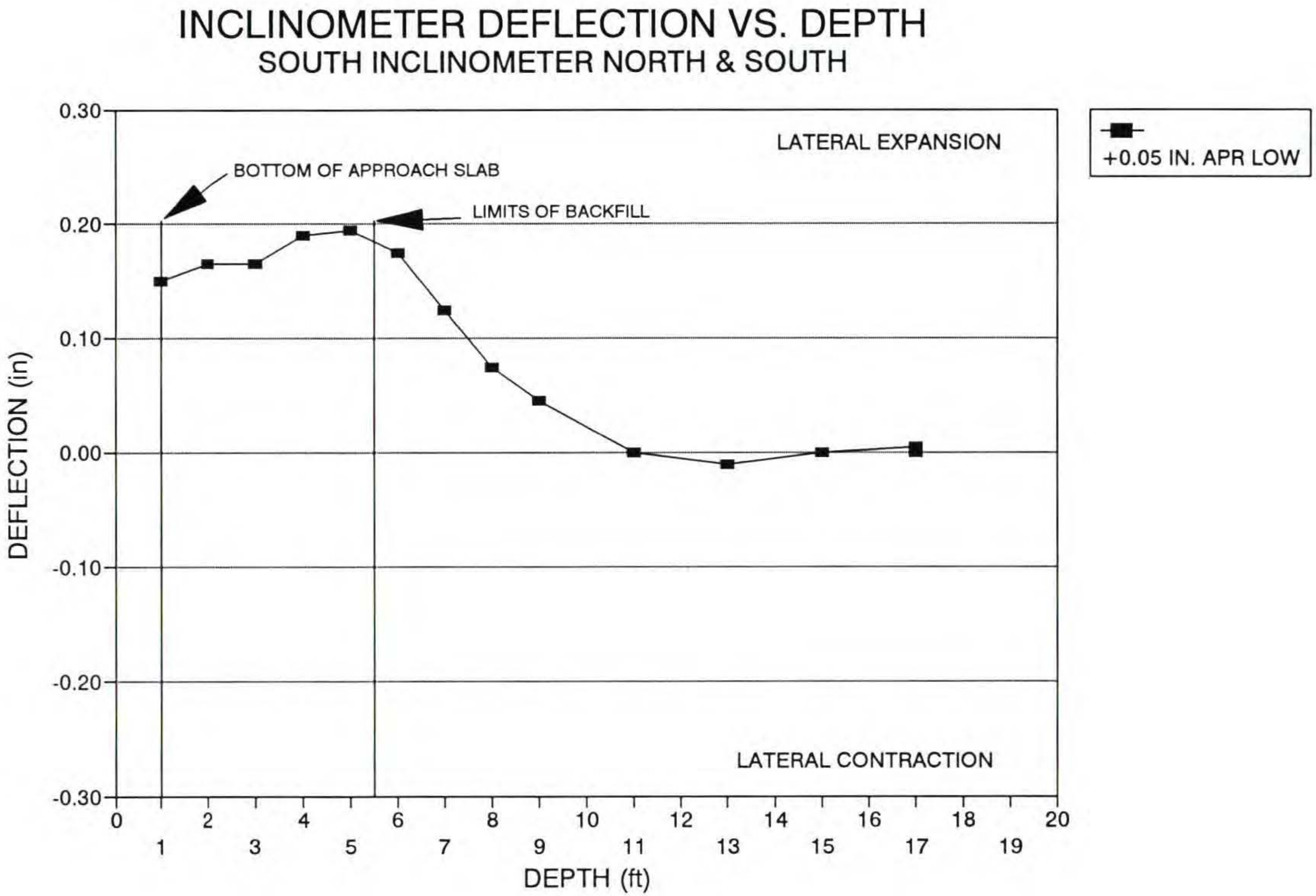
INCLINOMETER DEFLECTION VS. DEPTH
SOUTH INCLINOMETER NORTH & SOUTH
DEFLECTION (in)
DEPTH (ft)
LATERAL EXPANSION
LATERAL CONTRACTION
BOTTOM OF APPROACH SLAB
LIMITS OF BACKFILL
0.30
0.20
0.10
0.00
-0.10
-0.20
-0.30
0
2
4
6
8
10
12
14
16
18
20
1
3
5
7
9
11
13
15
17
19
-0.10 IN. AUG LOW
+0.08 IN. SEPT NORM
+0.70 IN SEPT HIGH
-0.20 IN. SEPT LOW
-0.25 IN. OCT NORM
+0.50 IN. OCT HIGH

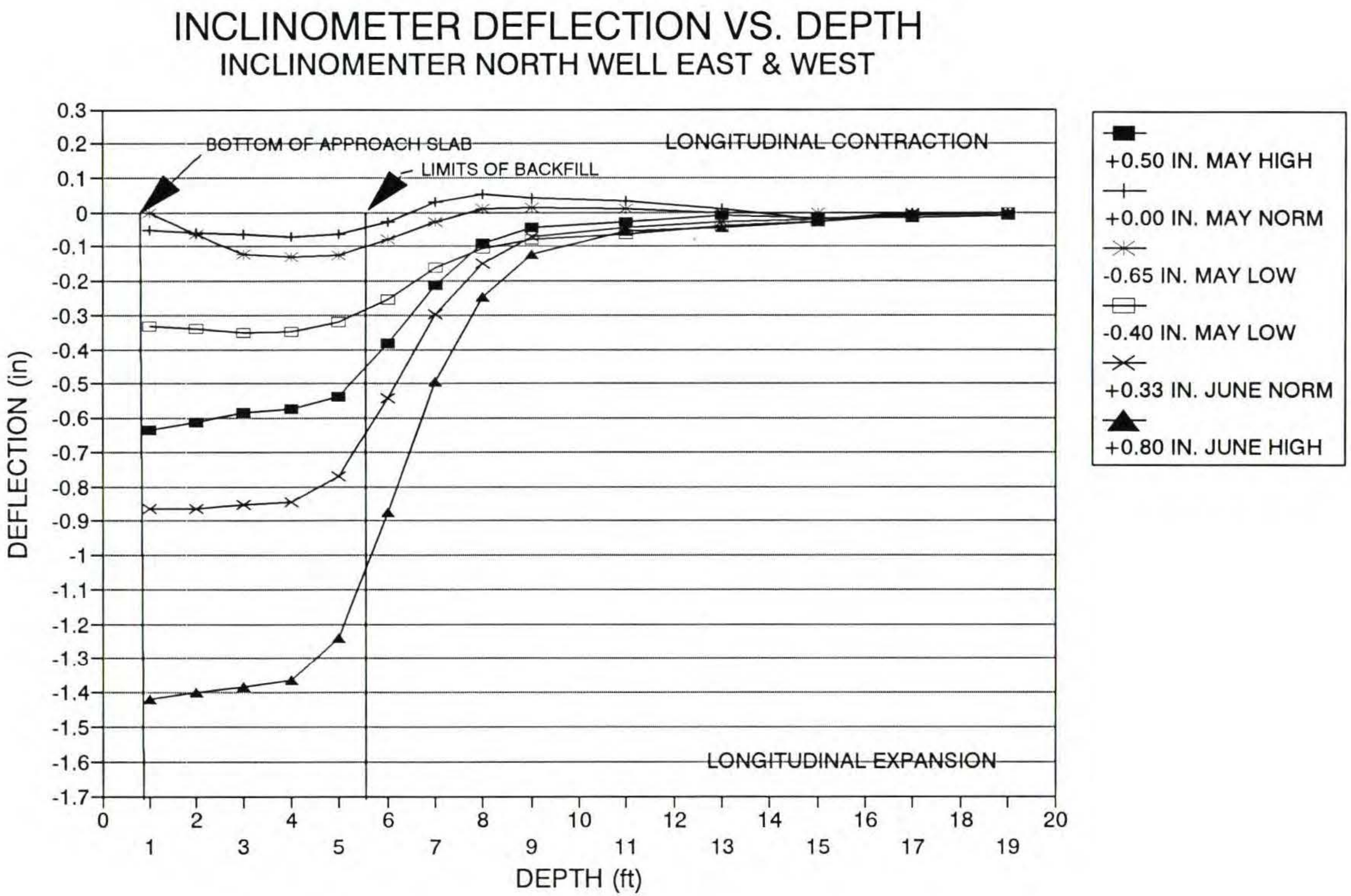

INCLINOMETER DEFLECTION VS. DEPTH
SOUTH INCLINOMETER NORTH & SOUTH
LATERAL EXPANSION
BOTTOM OF APPROACH SLAB
LIMITS OF BACKFILL
LATERAL CONTRACTION
DEFLECTION (in)
DEPTH (ft)
0.30
0.20
0.10
0.00
-0.10
-0.20
-0.30
0
2
4
6
8
10
12
14
16
18
20
1
3
5
7
9
11
13
15
17
19
-0.50 IN. OCT LOW
-0.60 IN. NOV NORM
+0.10 IN. NOV HIGH
-0.70 IN. NOV LOW
-0.50 IN. DEC NORM
+0.20 IN. DEC HIGH

# INCLINOMETER DEFLECTION VS. DEPTH
## SOUTH INCLINOMETER NORTH & SOUTH




INCLINOMETER DEFLECTION VS. DEPTH
SOUTH INCLINOMETER NORTH & SOUTH
LATERAL EXPANSION
BOTTOM OF APPROACH SLAB
LIMITS OF BACKFILL
LATERAL CONTRACTION
DEFLECTION (in)
0.30
0.20
0.10
0.00
-0.10
-0.20
-0.30
0
2
4
6
8
10
12
14
16
18
20
1
3
5
7
9
11
13
15
17
19
DEPTH (ft)
-0.50 IN. FEB LOW
+0.02 IN. MAR NORM
+1.00 IN. MAR HIGH
-0.20 IN. MAR LOW
+0.37 IN. APR NORM
+1.21 IN. APR HIGH

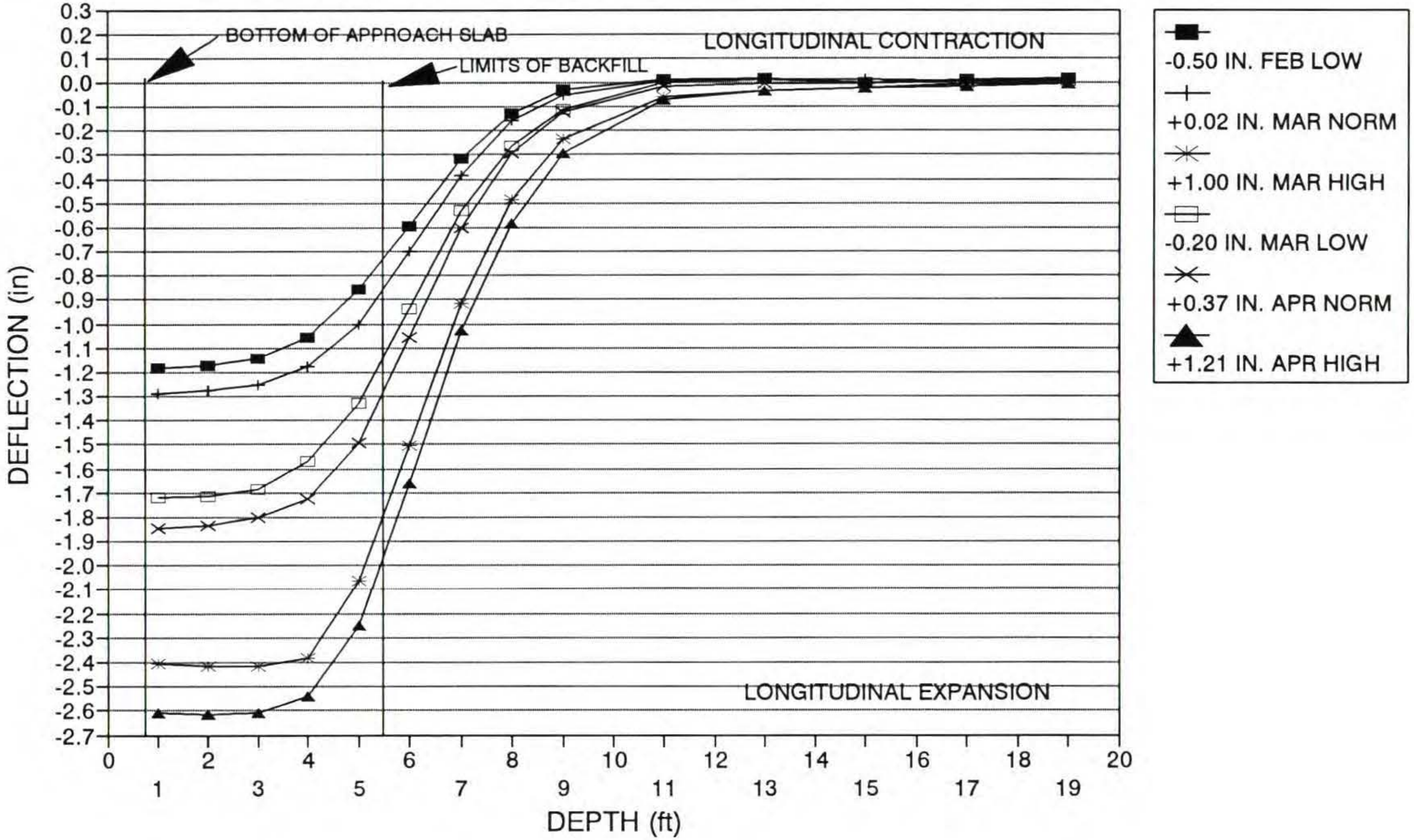

INCLINOMETER DEFLECTION VS. DEPTH
SOUTH INCLINOMETER NORTH & SOUTH
LATERAL EXPANSION
BOTTOM OF APPROACH SLAB
LIMITS OF BACKFILL
LATERAL CONTRACTION
+0.05 IN. APR LOW
DEFLECTION (in)
0.30
0.20
0.10
0.00
-0.10
-0.20
-0.30
0 2 4 6 8 10 12 14 16 18 20
1 3 5 7 9 11 13 15 17 19
DEPTH (ft)

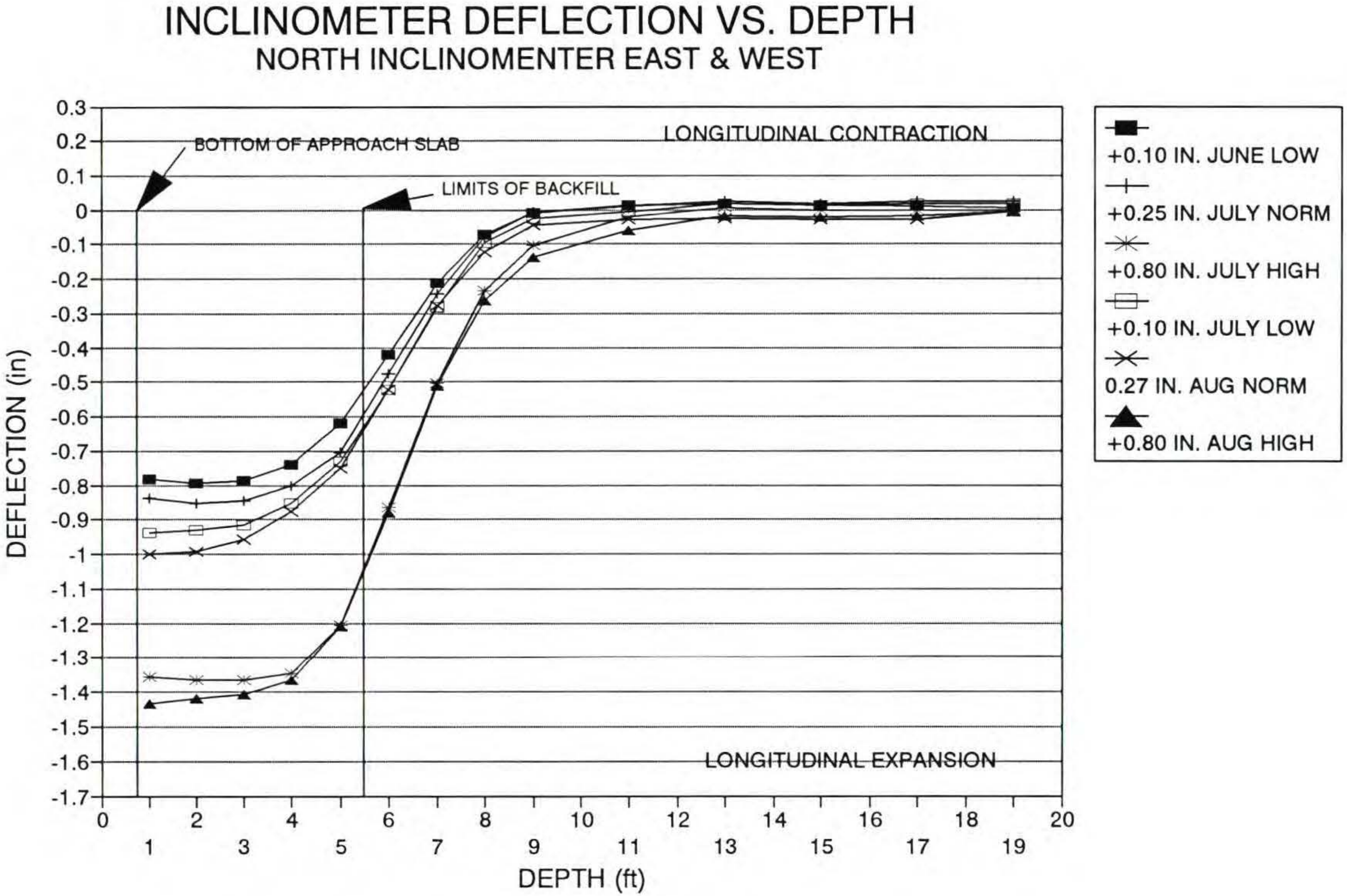


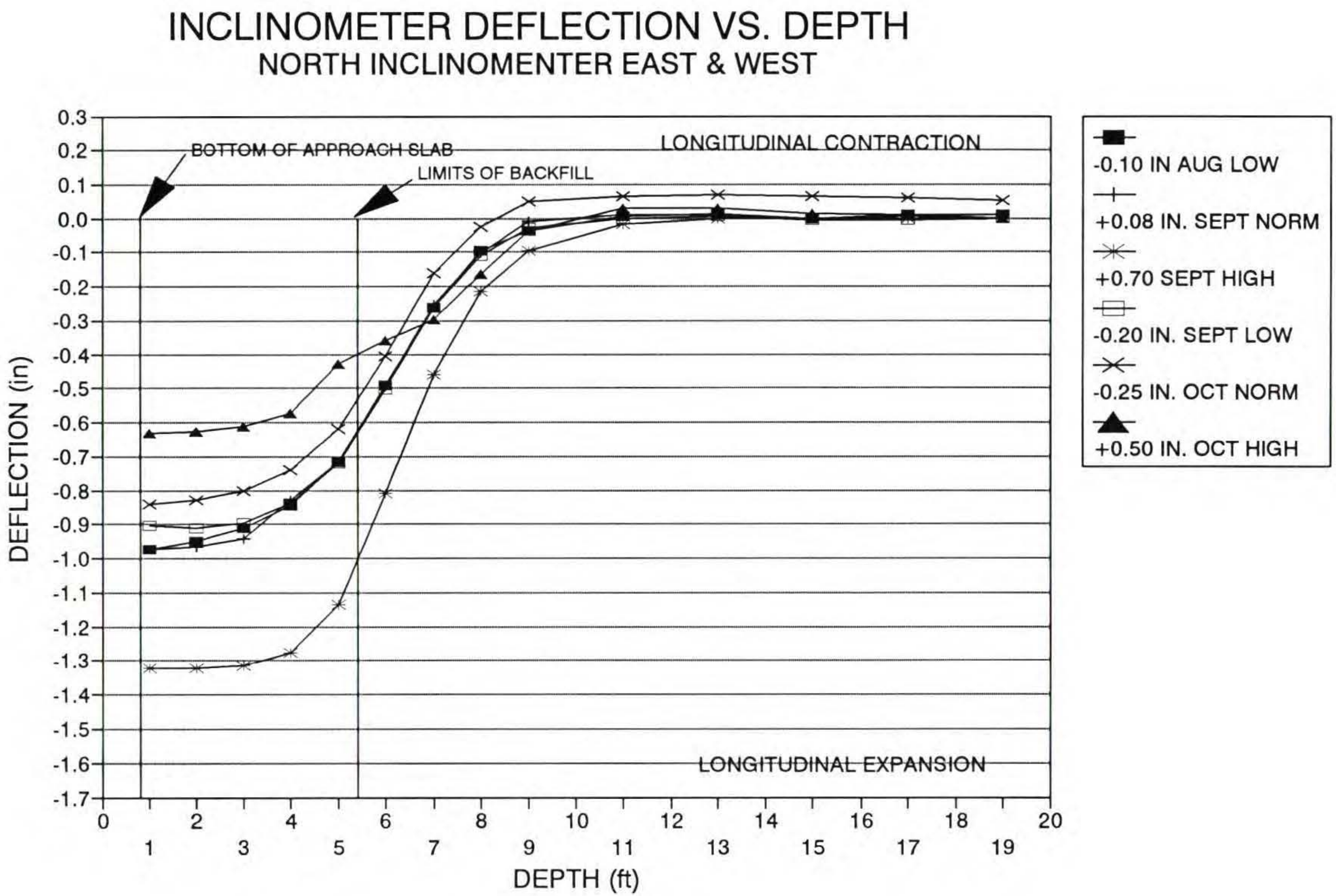
INCLINOMETER DEFLECTION VS. DEPTH
INCLINOMENTER NORTH WELL EAST & WEST
BOTTOM OF APPROACH SLAB
LIMITS OF BACKFILL
LONGITUDINAL CONTRACTION
LONGITUDINAL EXPANSION
DEFLECTION (in)
DEPTH (ft)
+0.50 IN. MAY HIGH
+0.00 IN. MAY NORM
-0.65 IN. MAY LOW
-0.40 IN. MAY LOW
+0.33 IN. JUNE NORM
+0.80 IN. JUNE HIGH

## INCLINOMETER DEFLECTION VS. DEPTH

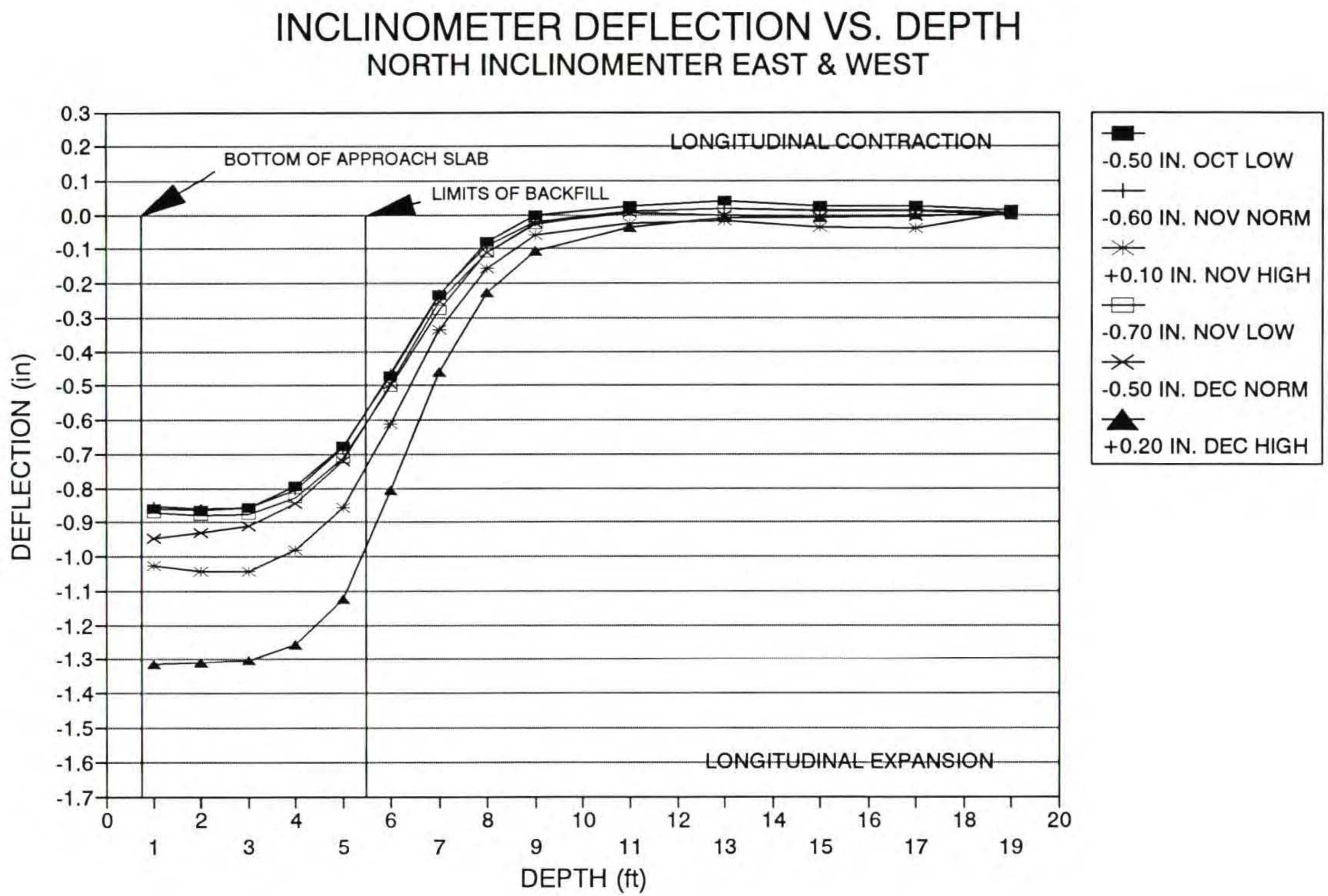
### NORTH INCLINOMENTER EAST & WEST



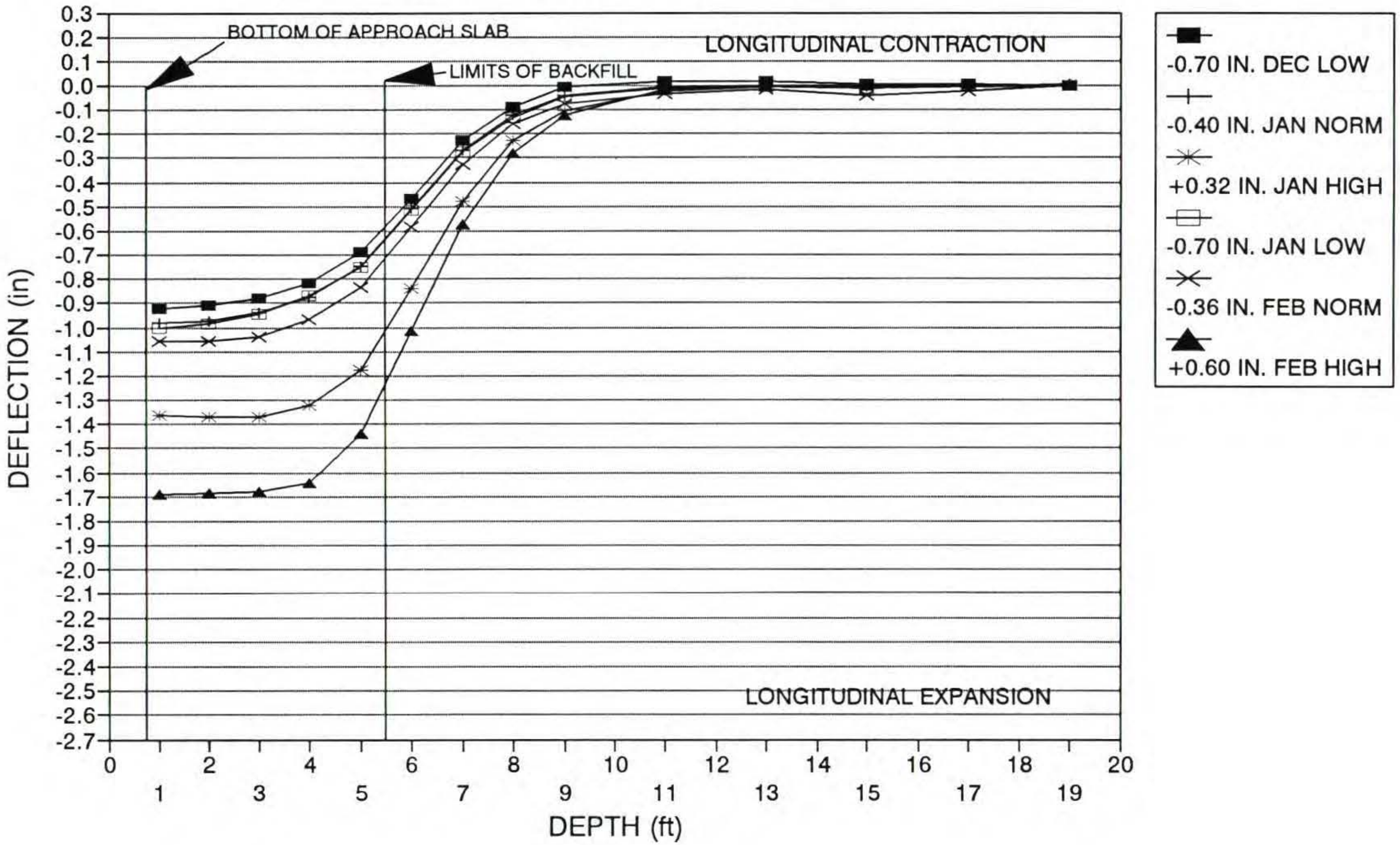

INCLINOMETER DEFLECTION VS. DEPTH
NORTH INCLINOMENTER EAST & WEST
BOTTOM OF APPROACH SLAB
LIMITS OF BACKFILL
LONGITUDINAL CONTRACTION
LONGITUDINAL EXPANSION
DEFLECTION (in)
DEPTH (ft)
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
-0.4
-0.5
-0.6
-0.7
-0.8
-0.9
-1
-1.1
-1.2
-1.3
-1.4
-1.5
-1.6
-1.7
0
2
4
6
8
10
12
14
16
18
20
1
3
5
7
9
11
13
15
17
19
+0.10 IN. JUNE LOW
+0.25 IN. JULY NORM
+0.80 IN. JULY HIGH
+0.10 IN. JULY LOW
0.27 IN. AUG NORM
+0.80 IN. AUG HIGH



INCLINOMETER DEFLECTION VS. DEPTH
NORTH INCLINOMENTER EAST & WEST
BOTTOM OF APPROACH SLAB
LIMITS OF BACKFILL
LONGITUDINAL CONTRACTION
LONGITUDINAL EXPANSION
DEFLECTION (in)
DEPTH (ft)
-0.10 IN AUG LOW
+0.08 IN. SEPT NORM
+0.70 SEPT HIGH
-0.20 IN. SEPT LOW
-0.25 IN. OCT NORM
+0.50 IN. OCT HIGH

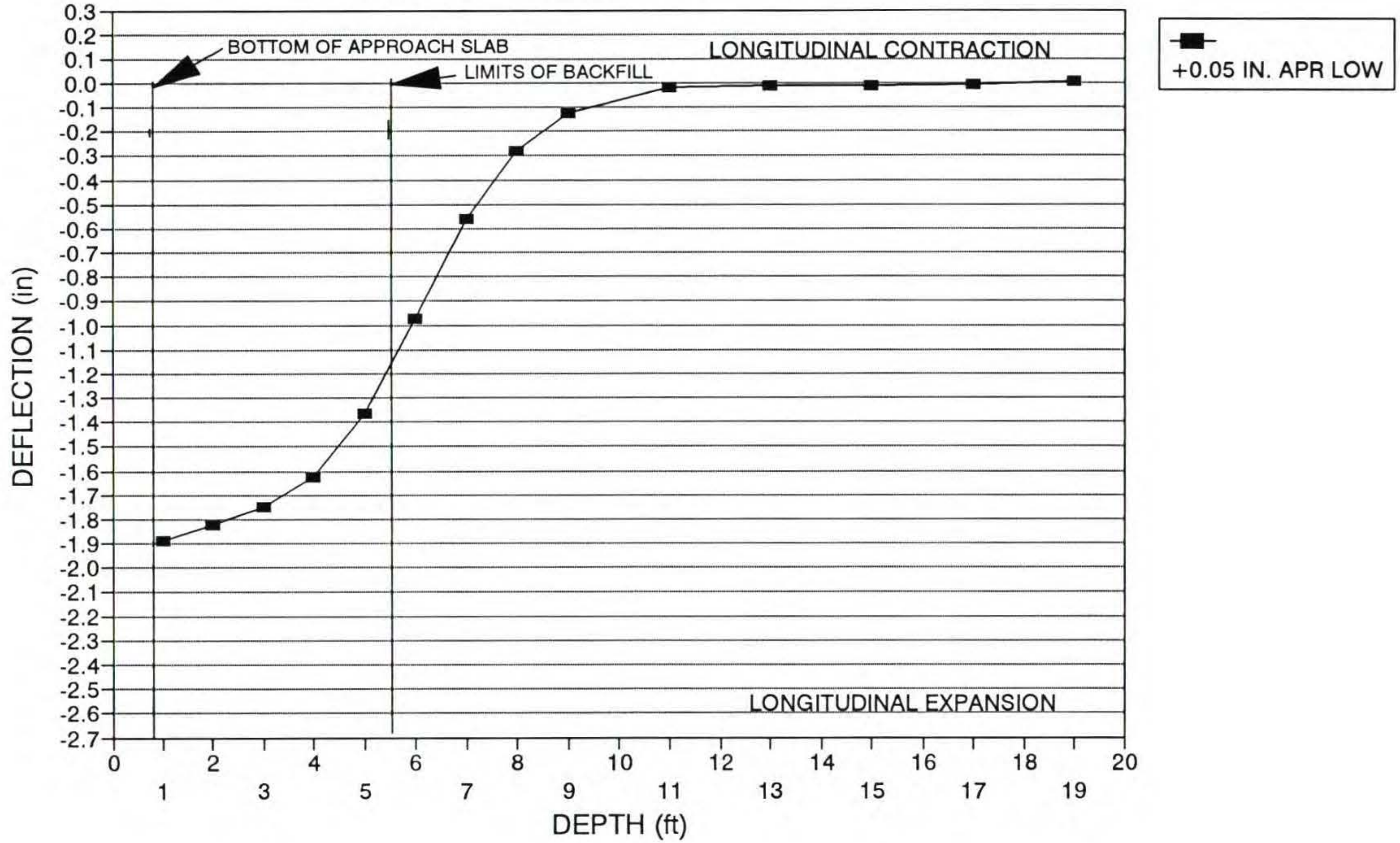

INCLINOMETER DEFLECTION VS. DEPTH
NORTH INCLINOMENTER EAST & WEST
LONGITUDINAL CONTRACTION
BOTTOM OF APPROACH SLAB
LIMITS OF BACKFILL
LONGITUDINAL EXPANSION
DEFLECTION (in)
DEPTH (ft)
0.3
0.2
0.1
0.0
-0.1
-0.2
-0.3
-0.4
-0.5
-0.6
-0.7
-0.8
-0.9
-1.0
-1.1
-1.2
-1.3
-1.4
-1.5
-1.6
-1.7
0
2
4
6
8
10
12
14
16
18
20
1
3
5
7
9
11
13
15
17
19
-0.50 IN. OCT LOW
-0.60 IN. NOV NORM
+0.10 IN. NOV HIGH
-0.70 IN. NOV LOW
-0.50 IN. DEC NORM
+0.20 IN. DEC HIGH

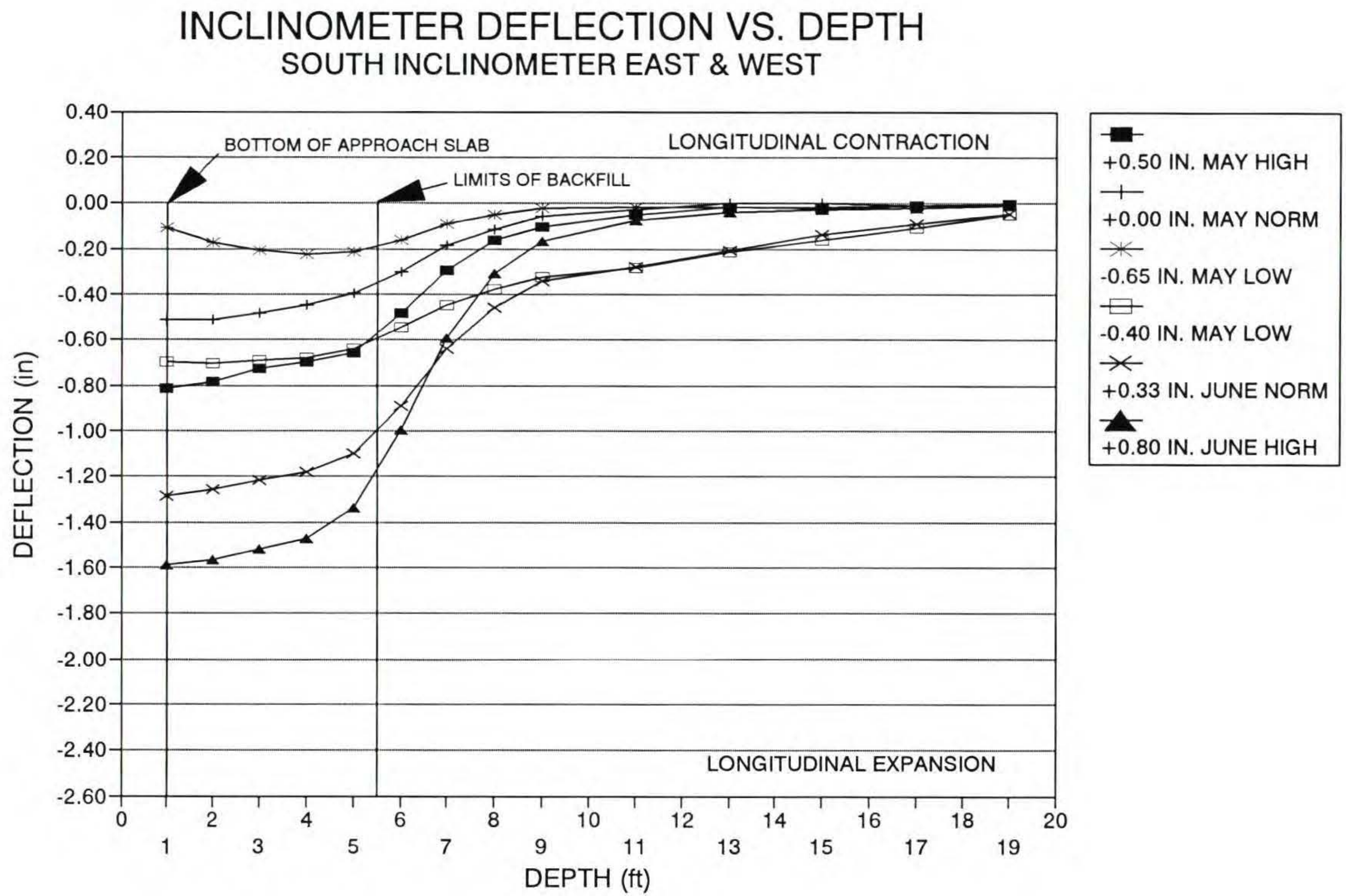
# INCLINOMETER DEFLECTION VS. DEPTH
## NORTH INCLINOMENTER EAST & WEST



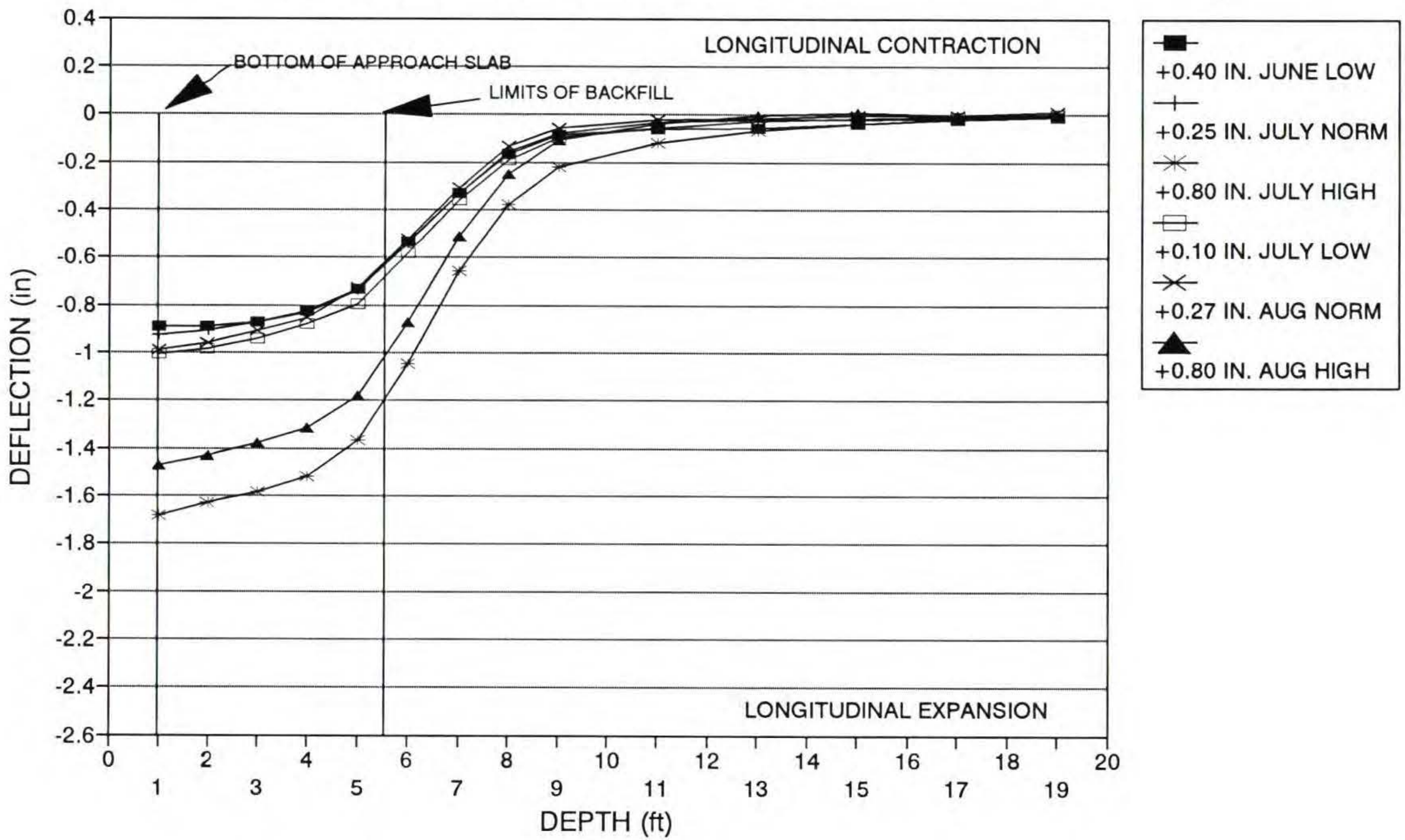
# INCLINOMETER DEFLECTION VS. DEPTH

## NORTH INCLINOMENTER EAST & WEST

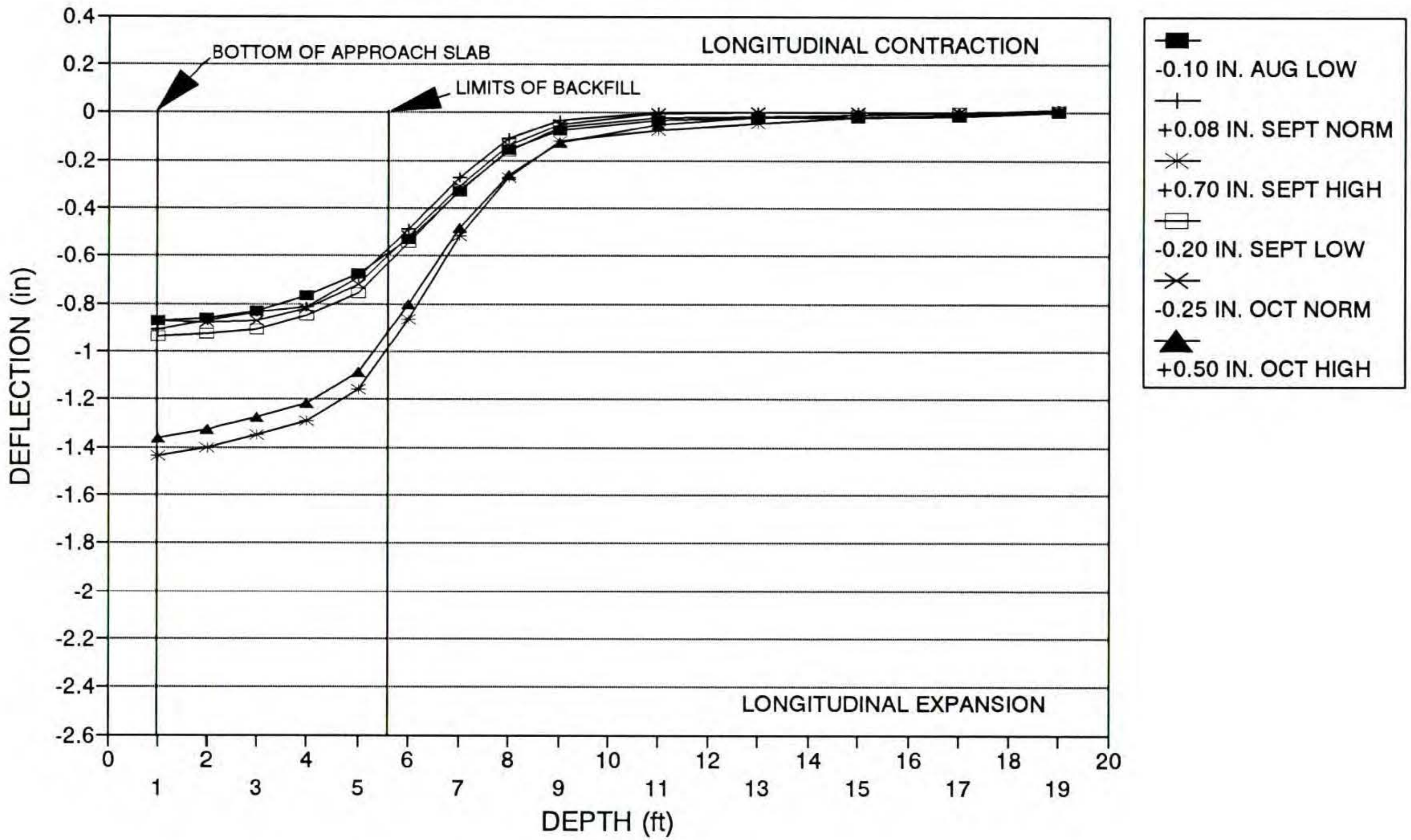



INCLINOMETER DEFLECTION VS. DEPTH
SOUTH INCLINOMETER EAST & WEST
BOTTOM OF APPROACH SLAB
LIMITS OF BACKFILL
LONGITUDINAL CONTRACTION
LONGITUDINAL EXPANSION
DEFLECTION (in)
DEPTH (ft)
0.40
0.20
0.00
-0.20
-0.40
-0.60
-0.80
-1.00
-1.20
-1.40
-1.60
-1.80
-2.00
-2.20
-2.40
-2.60
0
2
4
6
8
10
12
14
16
18
20
1
3
5
7
9
11
13
15
17
19
+0.50 IN. MAY HIGH
+0.00 IN. MAY NORM
-0.65 IN. MAY LOW
-0.40 IN. MAY LOW
+0.33 IN. JUNE NORM
+0.80 IN. JUNE HIGH

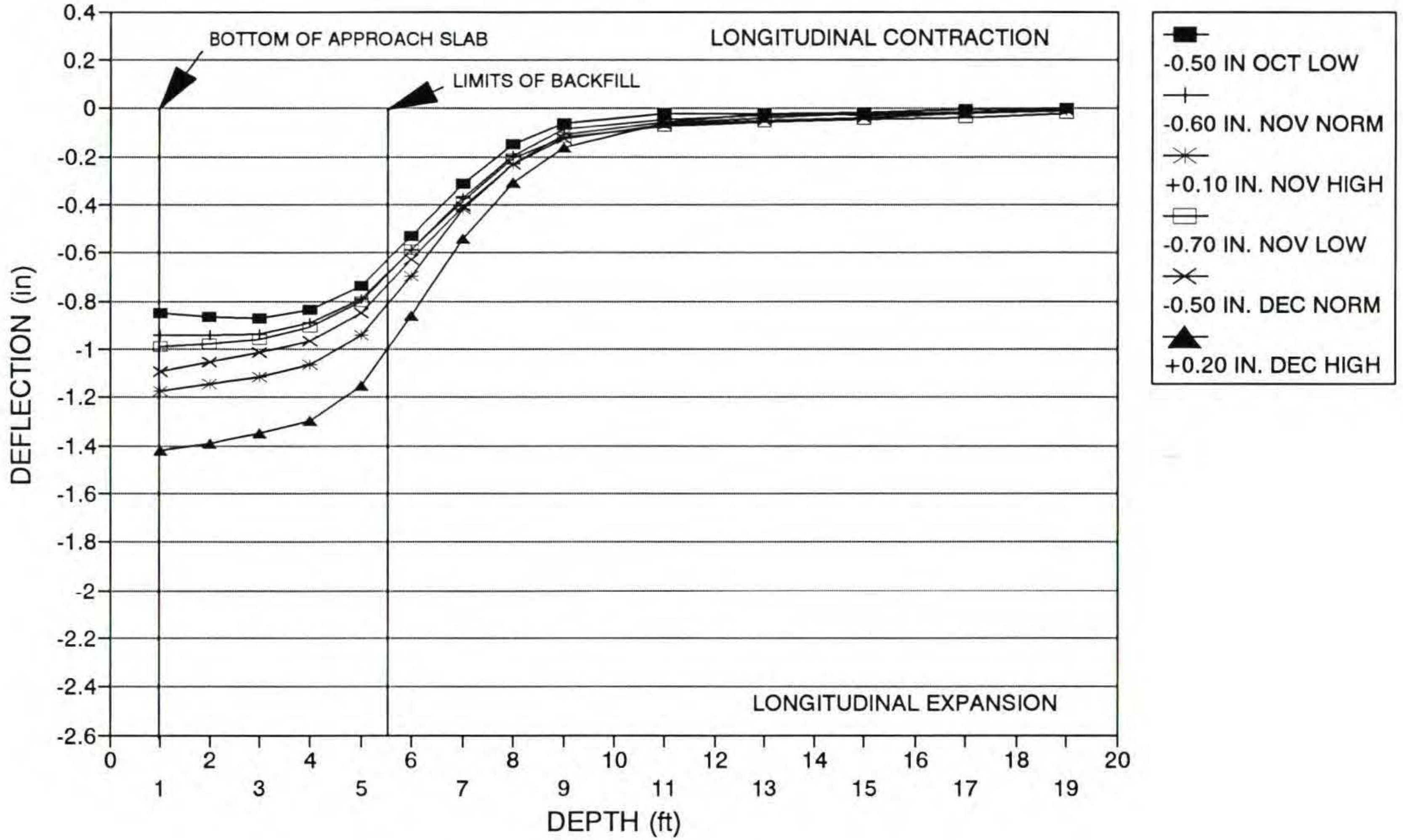
# INCLINOMETER DEFLECTION VS. DEPTH
## SOUTH INCLINOMETER EAST & WEST

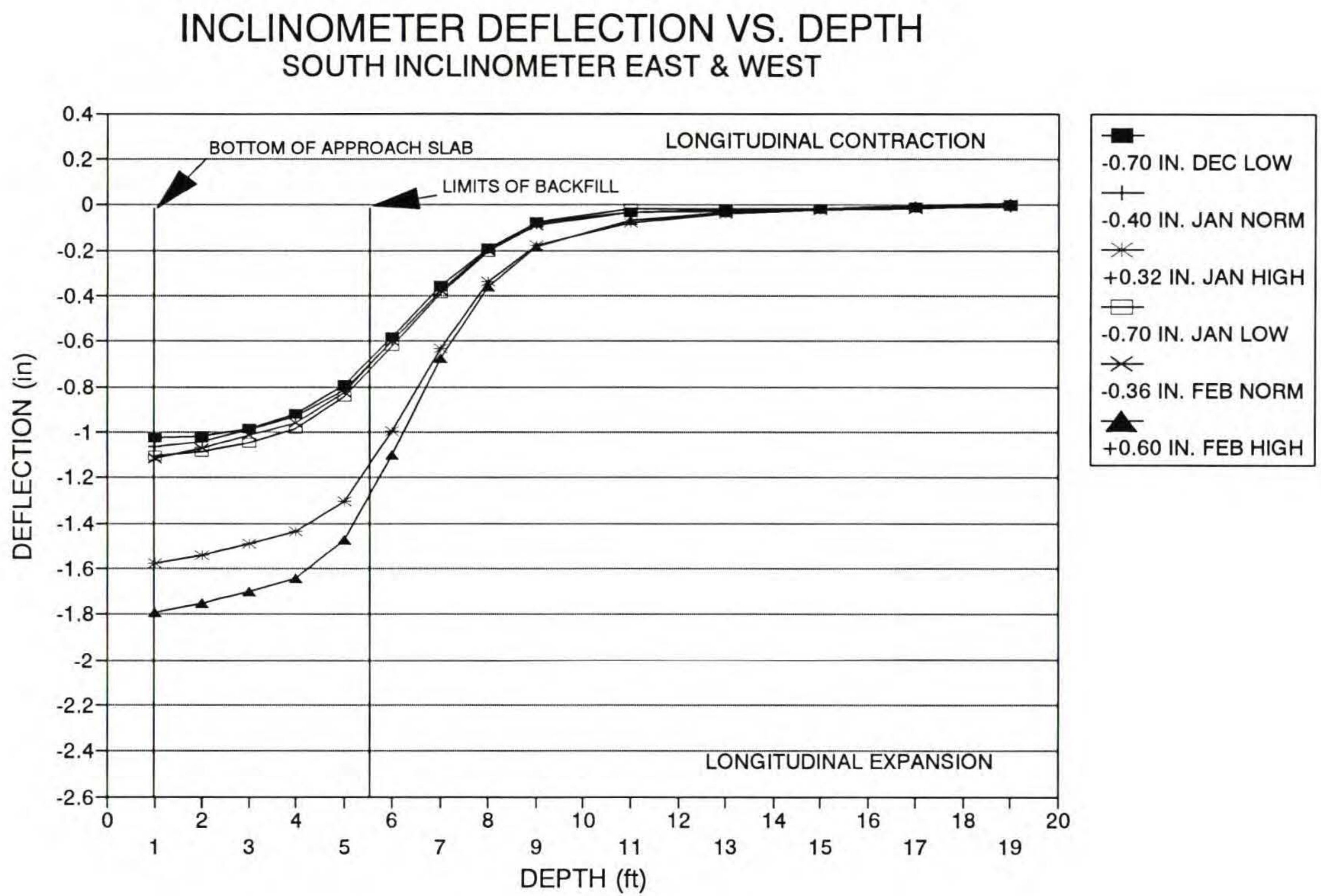


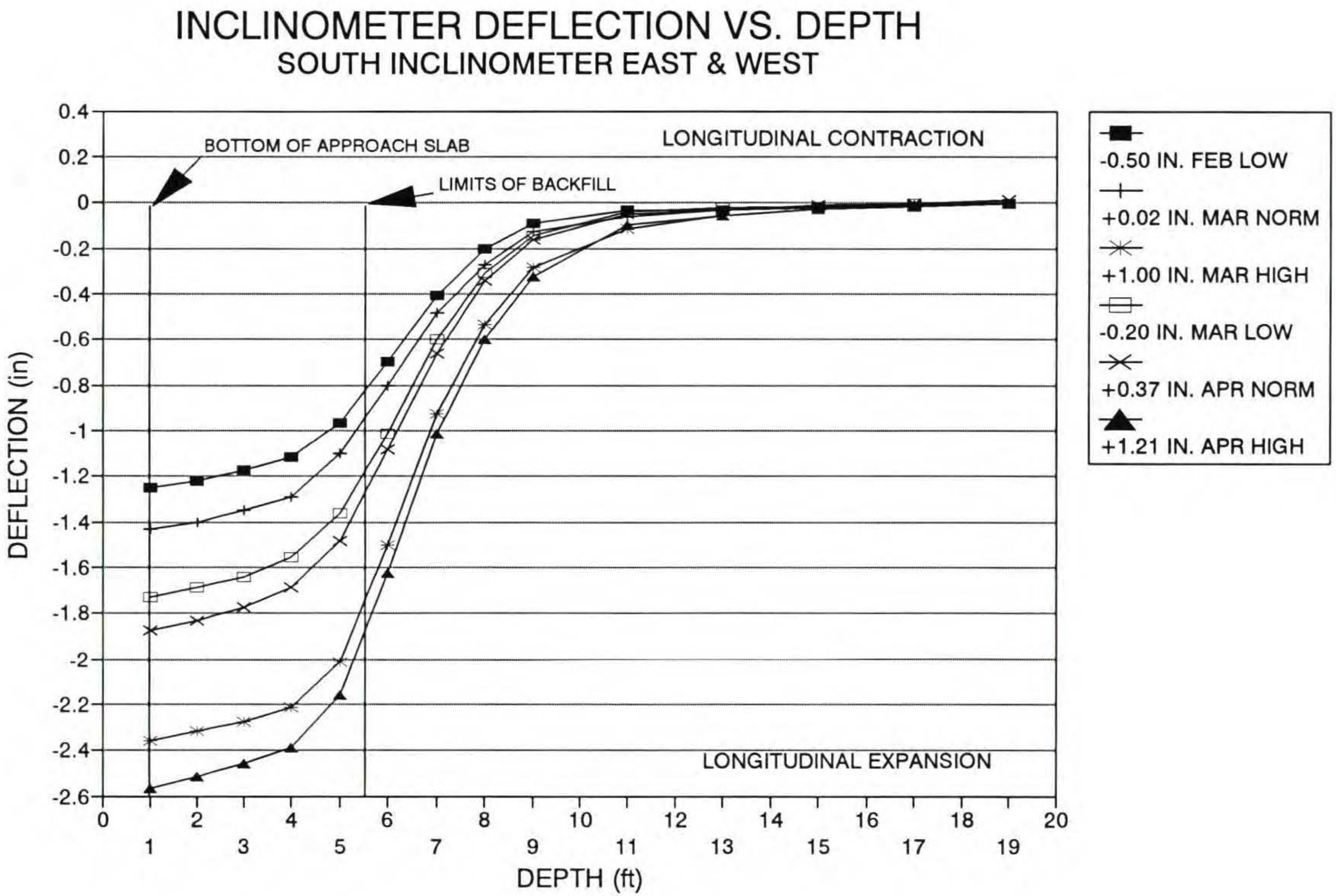
# INCLINOMETER DEFLECTION VS. DEPTH
## SOUTH INCLINOMETER EAST & WEST

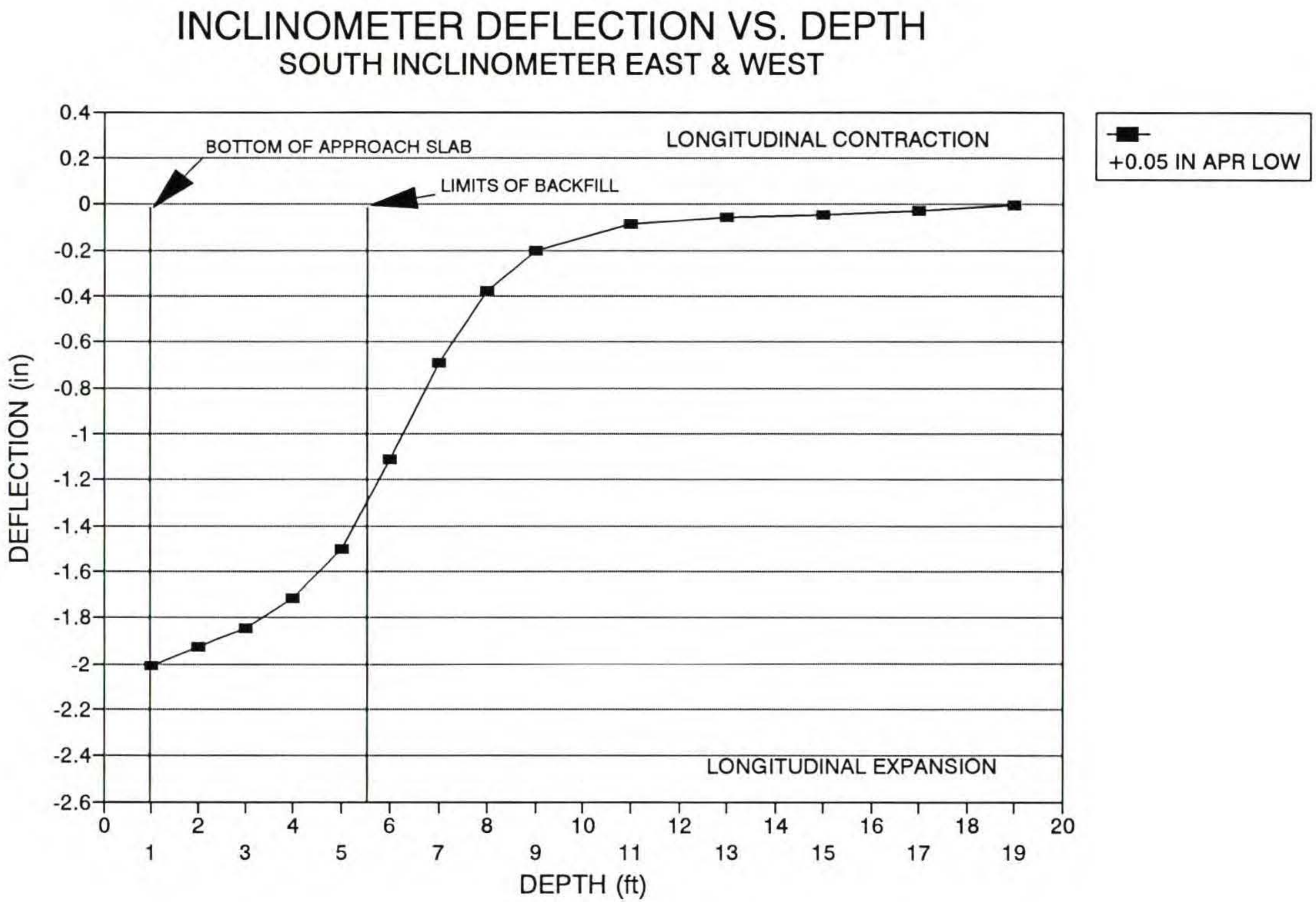


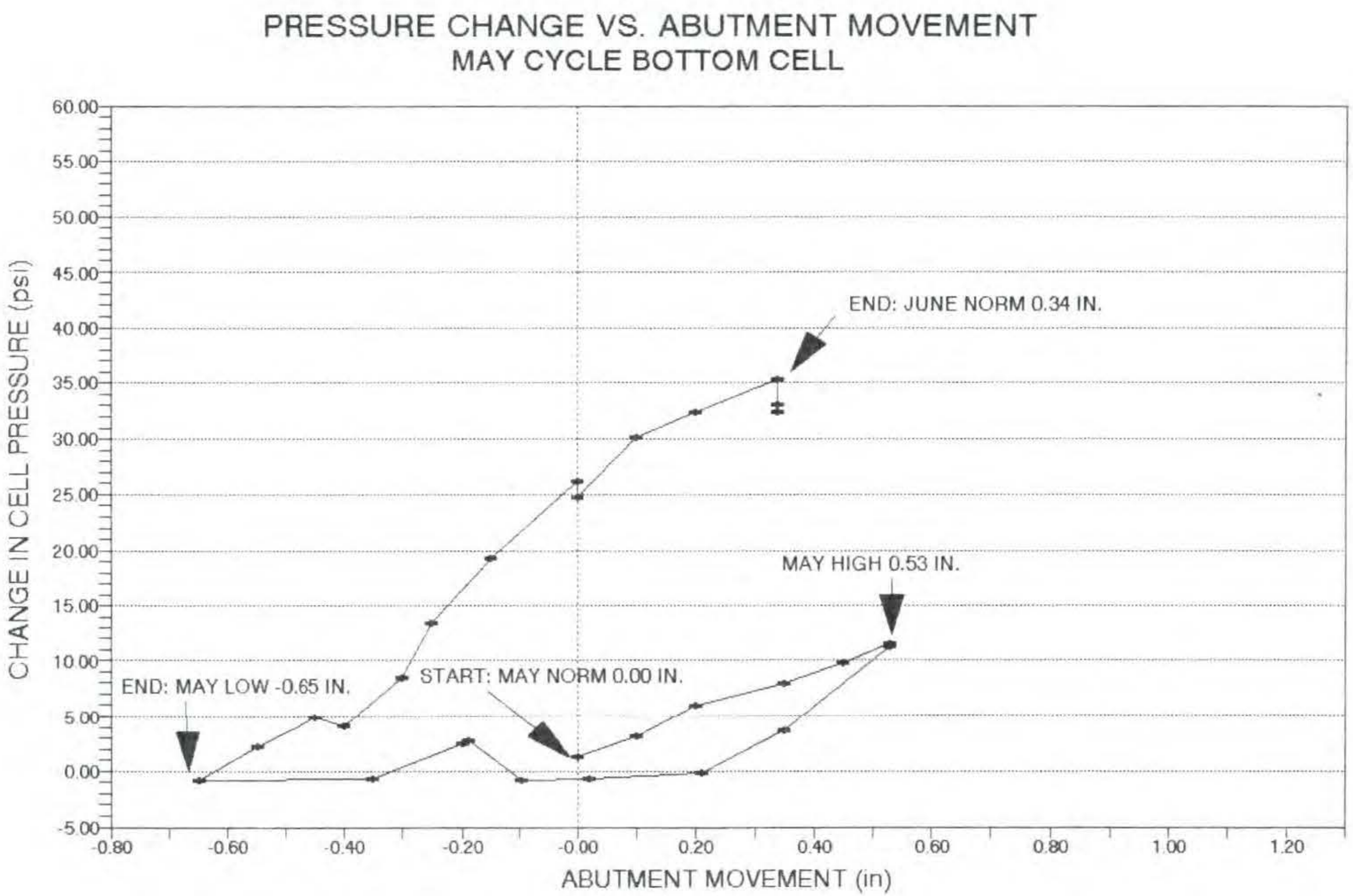
# INCLINOMETER DEFLECTION VS. DEPTH
## SOUTH INCLINOMETER EAST & WEST



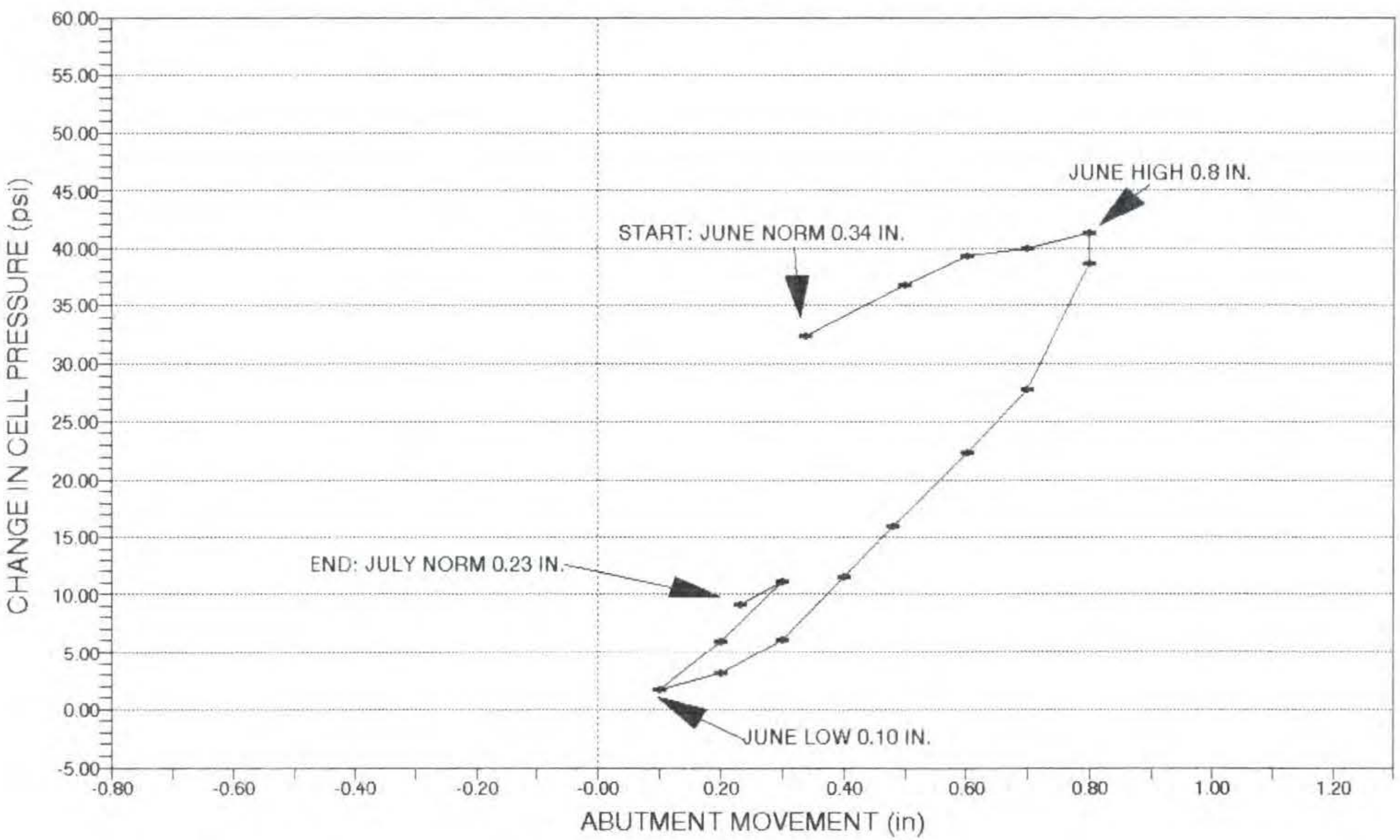

INCLINOMETER DEFLECTION VS. DEPTH
SOUTH INCLINOMETER EAST & WEST
BOTTOM OF APPROACH SLAB
LIMITS OF BACKFILL
LONGITUDINAL CONTRACTION
LONGITUDINAL EXPANSION
DEFLECTION (in)
DEPTH (ft)
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
-1.2
-1.4
-1.6
-1.8
-2
-2.2
-2.4
-2.6
0 2 4 6 8 10 12 14 16 18 20
1 3 5 7 9 11 13 15 17 19
-0.70 IN. DEC LOW
-0.40 IN. JAN NORM
+0.32 IN. JAN HIGH
-0.70 IN. JAN LOW
-0.36 IN. FEB NORM
+0.60 IN. FEB HIGH


INCLINOMETER DEFLECTION VS. DEPTH
SOUTH INCLINOMETER EAST & WEST
BOTTOM OF APPROACH SLAB
LIMITS OF BACKFILL
LONGITUDINAL CONTRACTION
LONGITUDINAL EXPANSION
DEFLECTION (in)
DEPTH (ft)
-0.50 IN. FEB LOW
+0.02 IN. MAR NORM
+1.00 IN. MAR HIGH
-0.20 IN. MAR LOW
+0.37 IN. APR NORM
+1.21 IN. APR HIGH

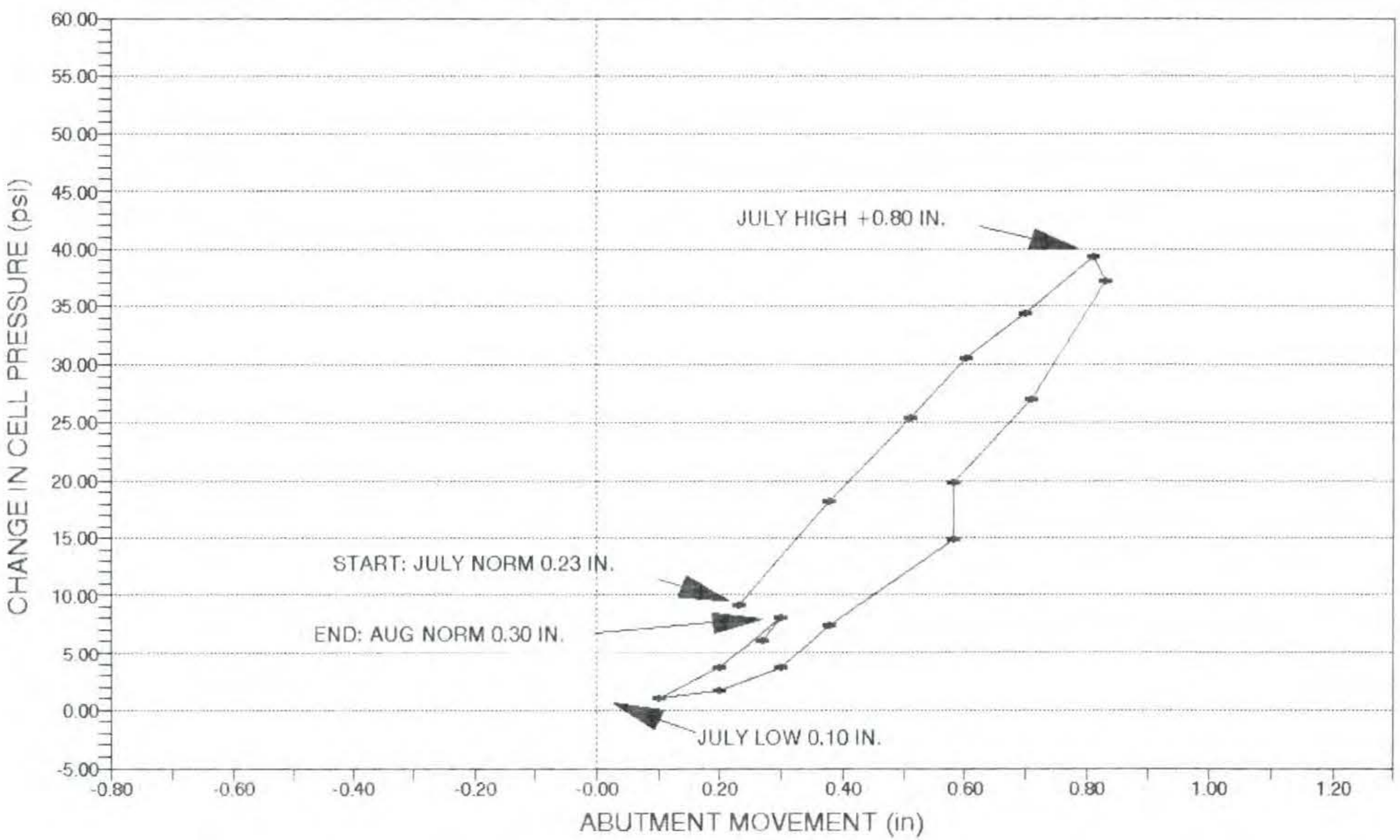

INCLINOMETER DEFLECTION VS. DEPTH
SOUTH INCLINOMETER EAST & WEST
BOTTOM OF APPROACH SLAB
LIMITS OF BACKFILL
LONGITUDINAL CONTRACTION
LONGITUDINAL EXPANSION
+0.05 IN APR LOW
DEFLECTION (in)
DEPTH (ft)
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
-1.2
-1.4
-1.6
-1.8
-2
-2.2
-2.4
-2.6
0
2
4
6
8
10
12
14
16
18
20
1
3
5
7
9
11
13
15
17
19


PRESSURE CHANGE VS. ABUTMENT MOVEMENT
MAY CYCLE BOTTOM CELL
CHANGE IN CELL PRESSURE (psi)
ABUTMENT MOVEMENT (in)
60.00
55.00
50.00
45.00
40.00
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
-5.00
-0.80
-0.60
-0.40
-0.20
-0.00
0.20
0.40
0.60
0.80
1.00
1.20
END: JUNE NORM 0.34 IN.
MAY HIGH 0.53 IN.
END: MAY LOW -0.65 IN.
START: MAY NORM 0.00 IN.

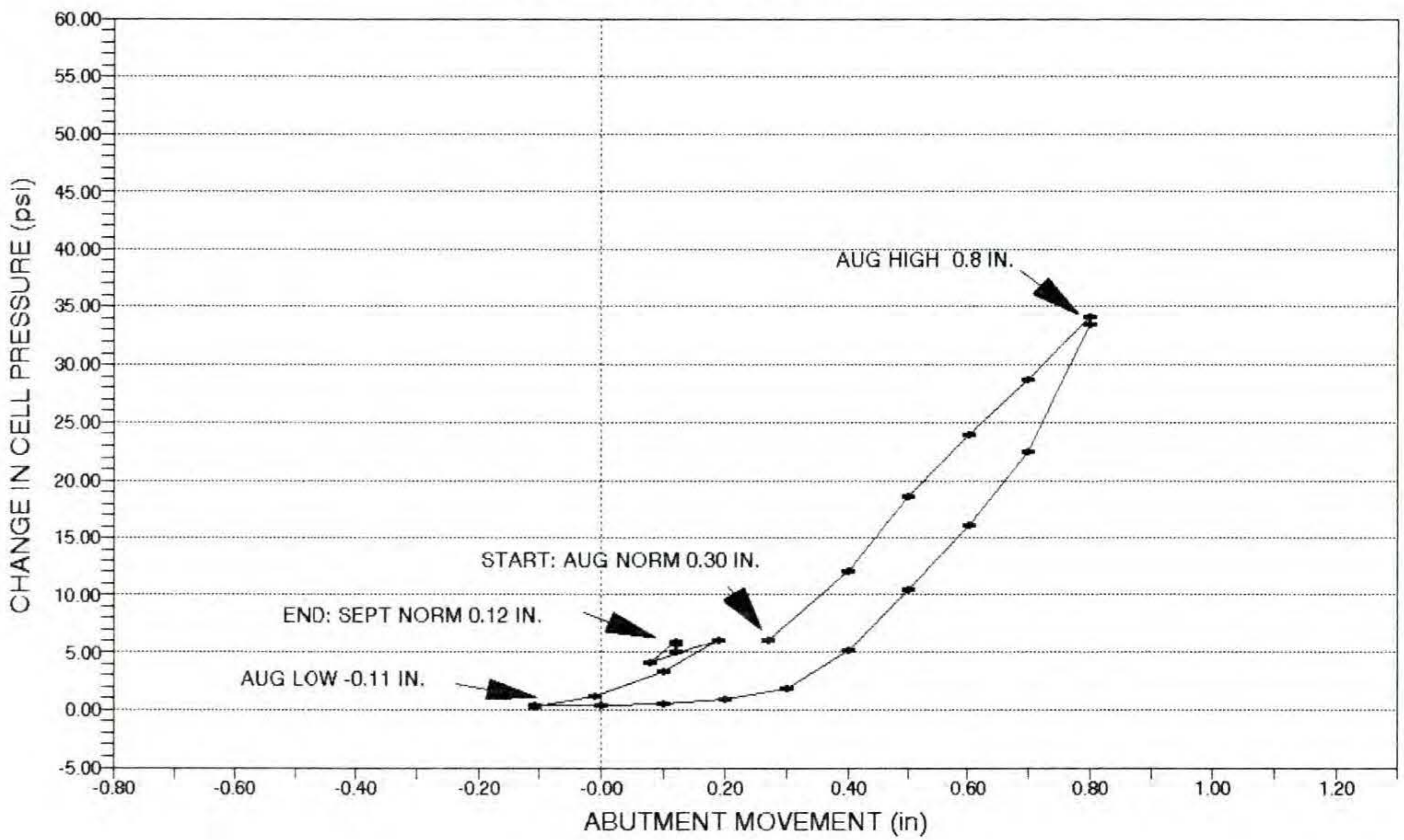
## PRESSURE CHANGE VS. ABUTMENT MOVEMENT
## JUNE CYCLE BOTTOM CELL

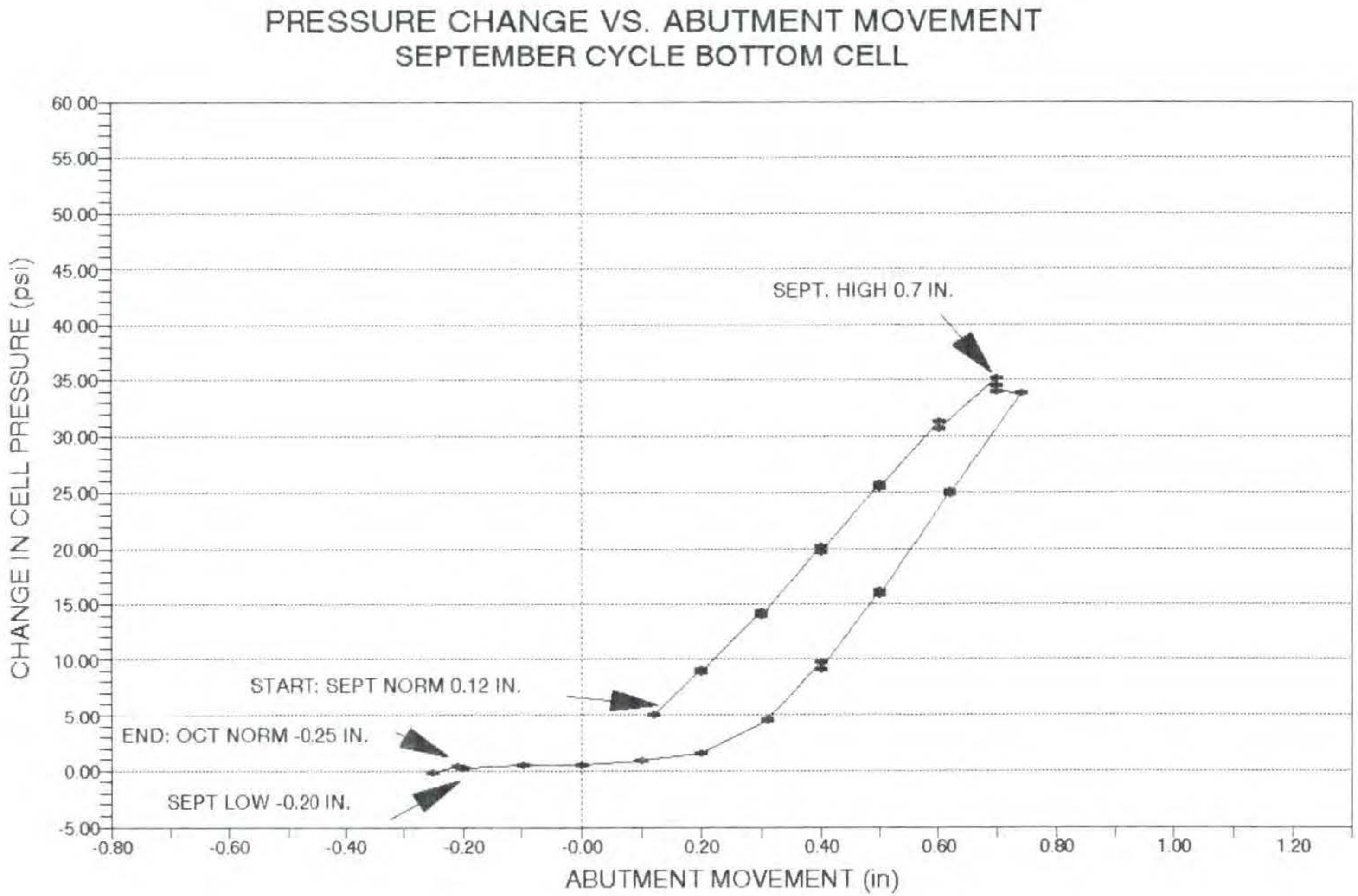


# PRESSURE CHANGE VS. ABUTMENT MOVEMENT
## JULY CYCLE BOTTOM CELL

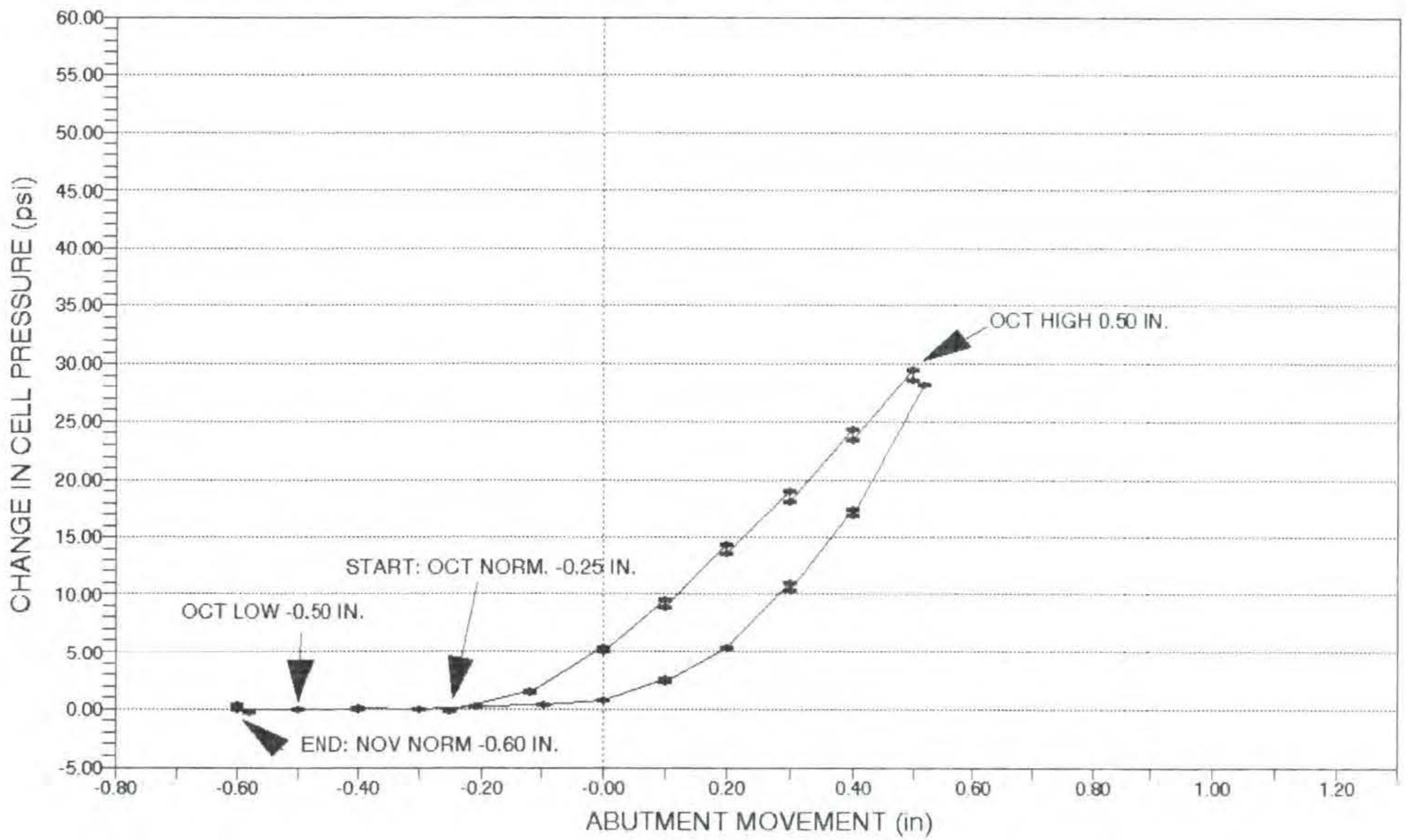


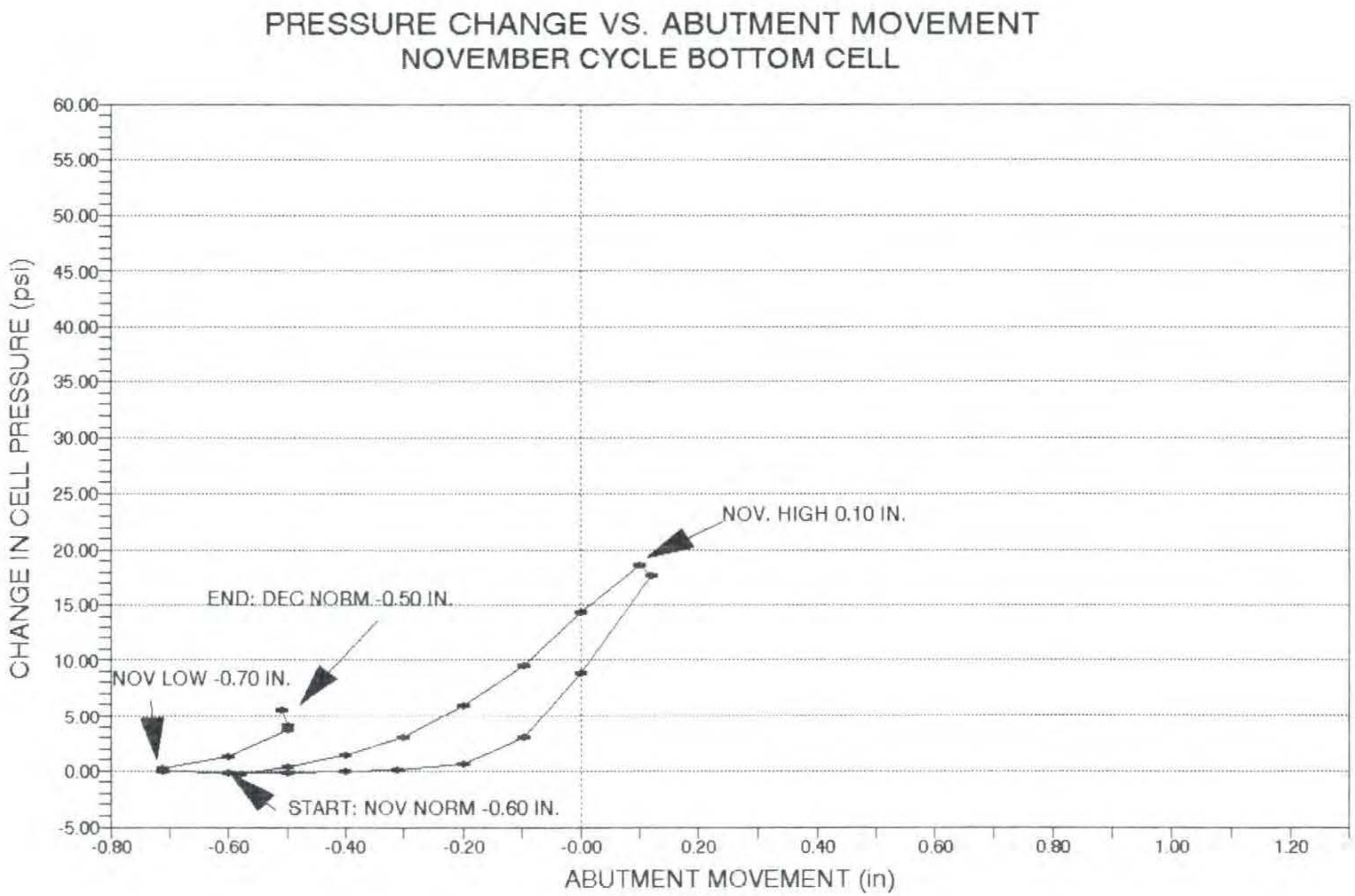
PRESSURE CHANGE VS. ABUTMENT MOVEMENT
AUG CYCLE BOTTOM CELL

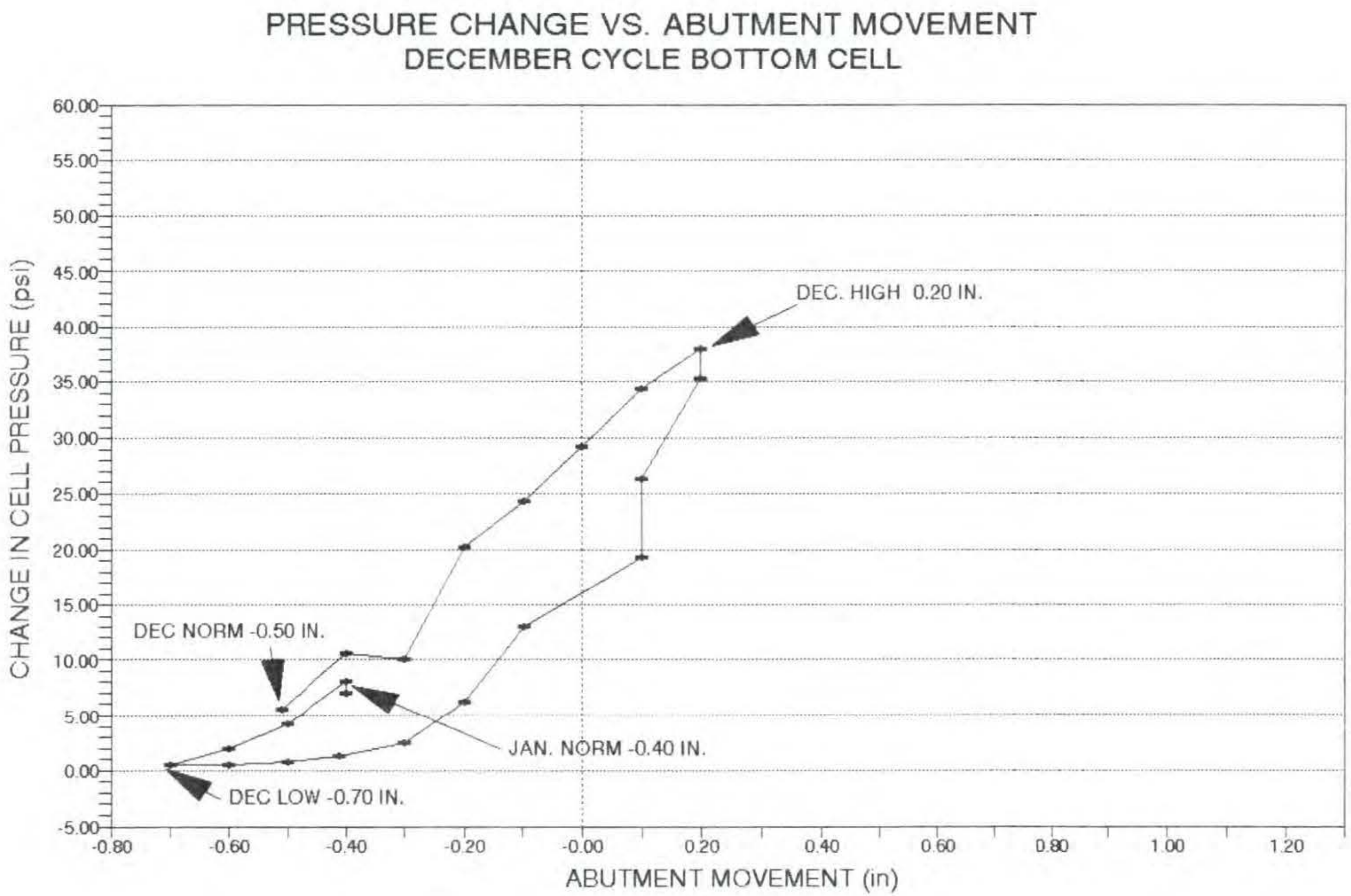



PRESSURE CHANGE VS. ABUTMENT MOVEMENT
SEPTEMBER CYCLE BOTTOM CELL
CHANGE IN CELL PRESSURE (psi)
ABUTMENT MOVEMENT (in)
SEPT. HIGH 0.7 IN.
START: SEPT NORM 0.12 IN.
END: OCT NORM -0.25 IN.
SEPT LOW -0.20 IN.
60.00
55.00
50.00
45.00
40.00
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
-5.00
-0.80
-0.60
-0.40
-0.20
-0.00
0.20
0.40
0.60
0.80
1.00
1.20

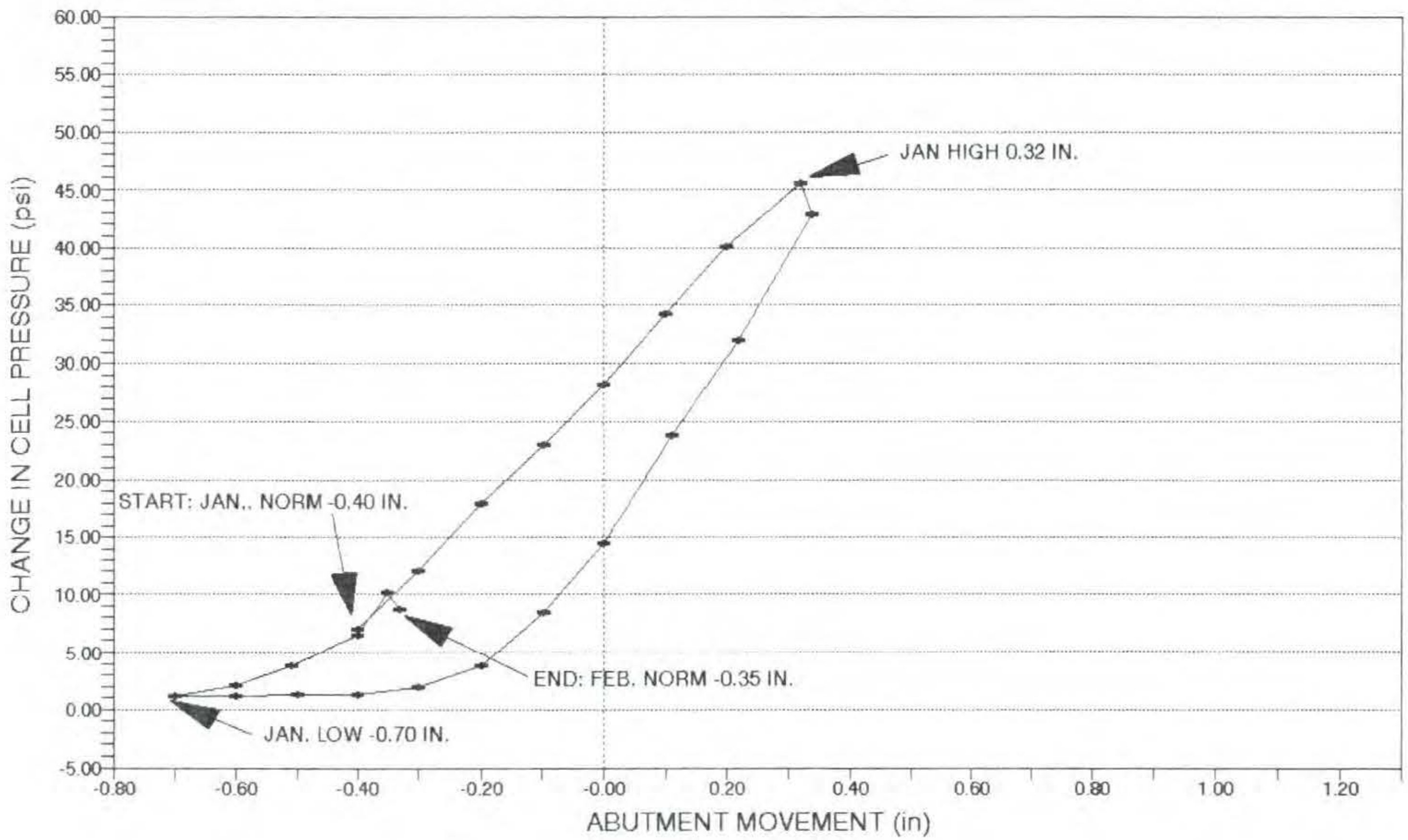
PRESSURE CHANGE VS. ABUTMENT MOVEMENT
OCTOBER CYCLE BOTTOM CELL

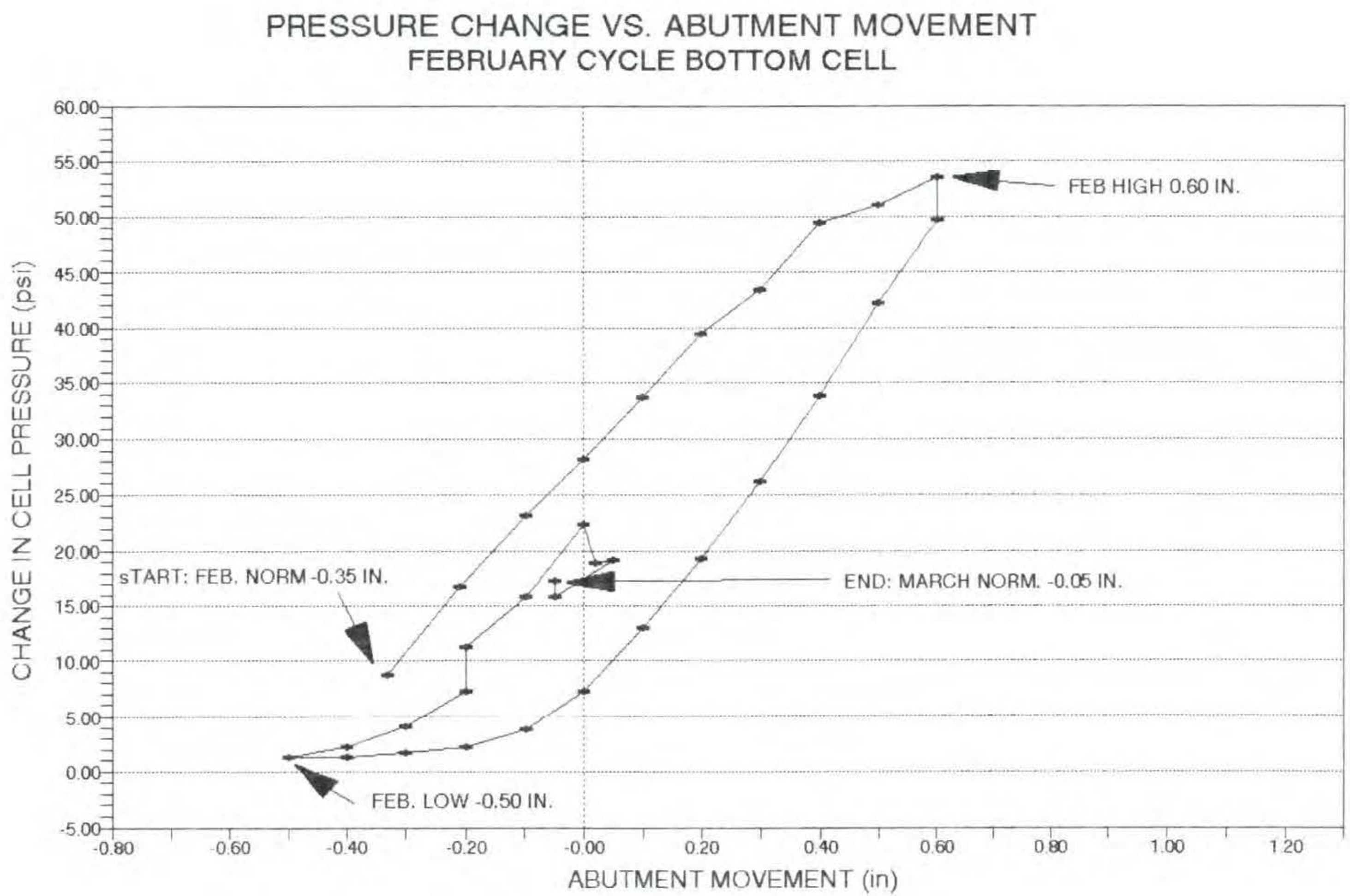



PRESSURE CHANGE VS. ABUTMENT MOVEMENT
NOVEMBER CYCLE BOTTOM CELL
CHANGE IN CELL PRESSURE (psi)
ABUTMENT MOVEMENT (in)
NOV. HIGH 0.10 IN.
END: DEC NORM -0.50 IN.
NOV LOW -0.70 IN.
START: NOV NORM -0.60 IN.
60.00
55.00
50.00
45.00
40.00
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
-5.00
-0.80
-0.60
-0.40
-0.20
-0.00
0.20
0.40
0.60
0.80
1.00
1.20

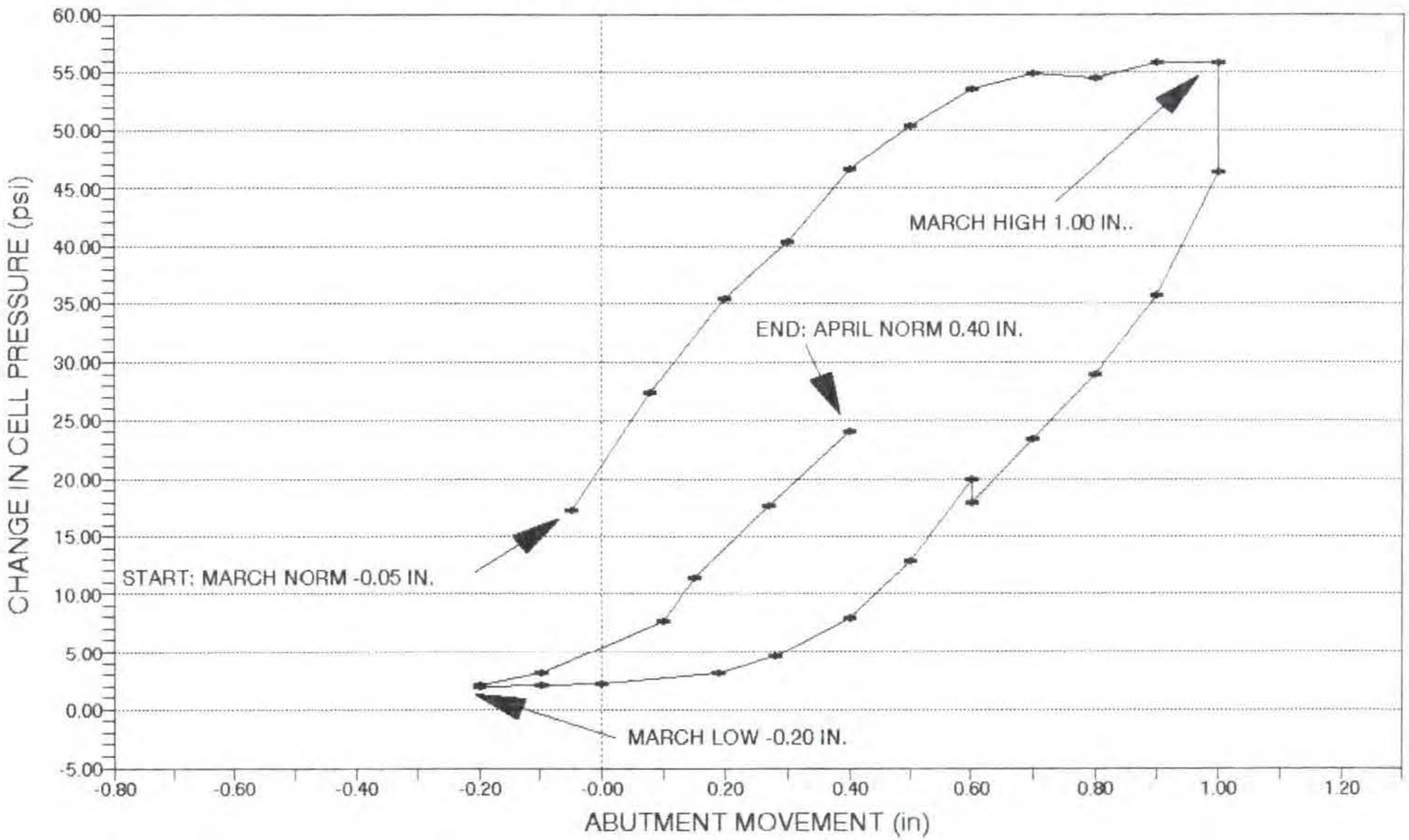

PRESSURE CHANGE VS. ABUTMENT MOVEMENT
DECEMBER CYCLE BOTTOM CELL
CHANGE IN CELL PRESSURE (psi)
ABUTMENT MOVEMENT (in)
DEC. HIGH 0.20 IN.
DEC NORM -0.50 IN.
JAN. NORM -0.40 IN.
DEC LOW -0.70 IN.
60.00
55.00
50.00
45.00
40.00
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
-5.00
-0.80
-0.60
-0.40
-0.20
-0.00
0.20
0.40
0.60
0.80
1.00
1.20

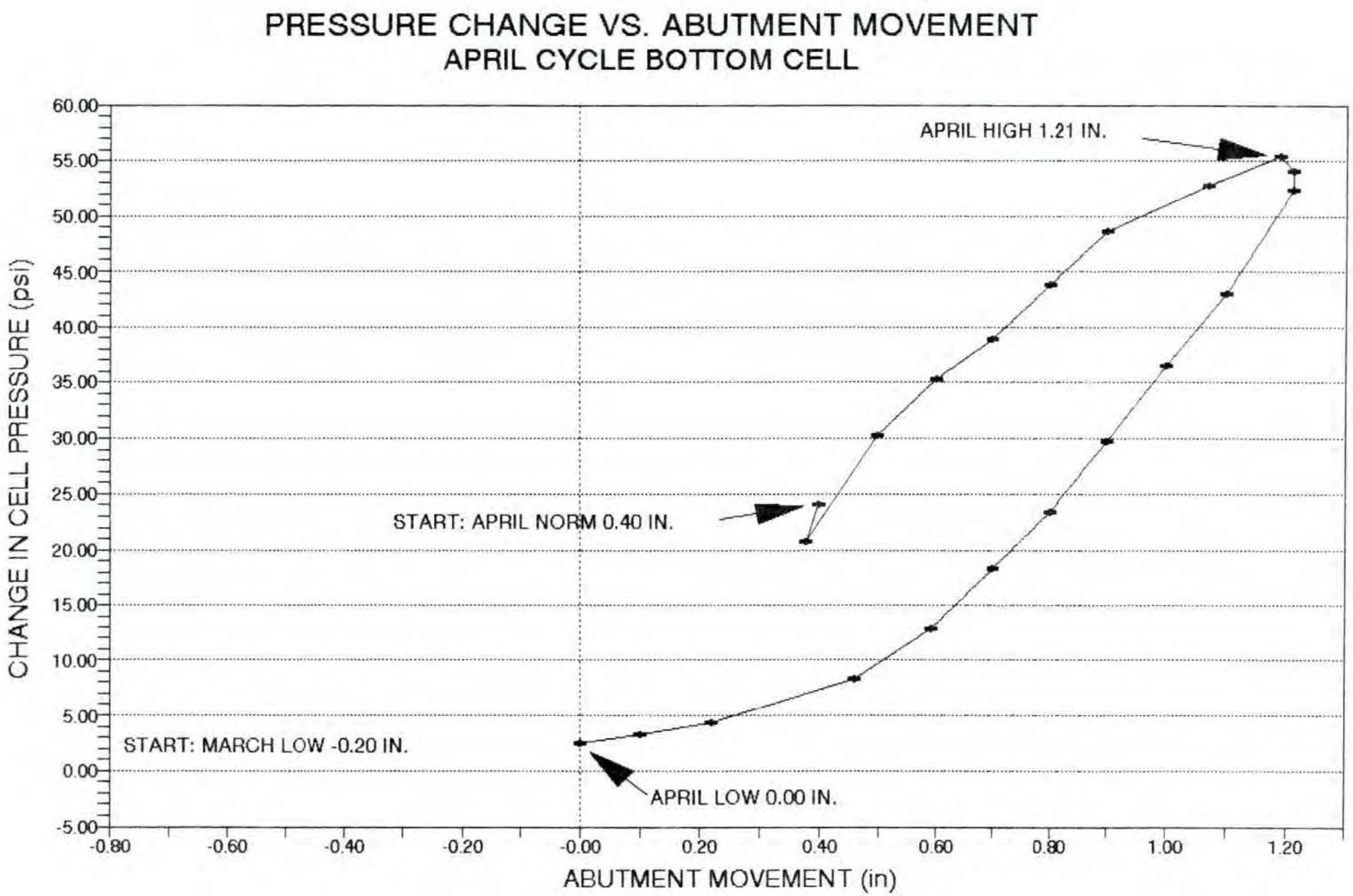
PRESSURE CHANGE VS. ABUTMENT MOVEMENT
JANUARY CYCLE BOTTOM CELL

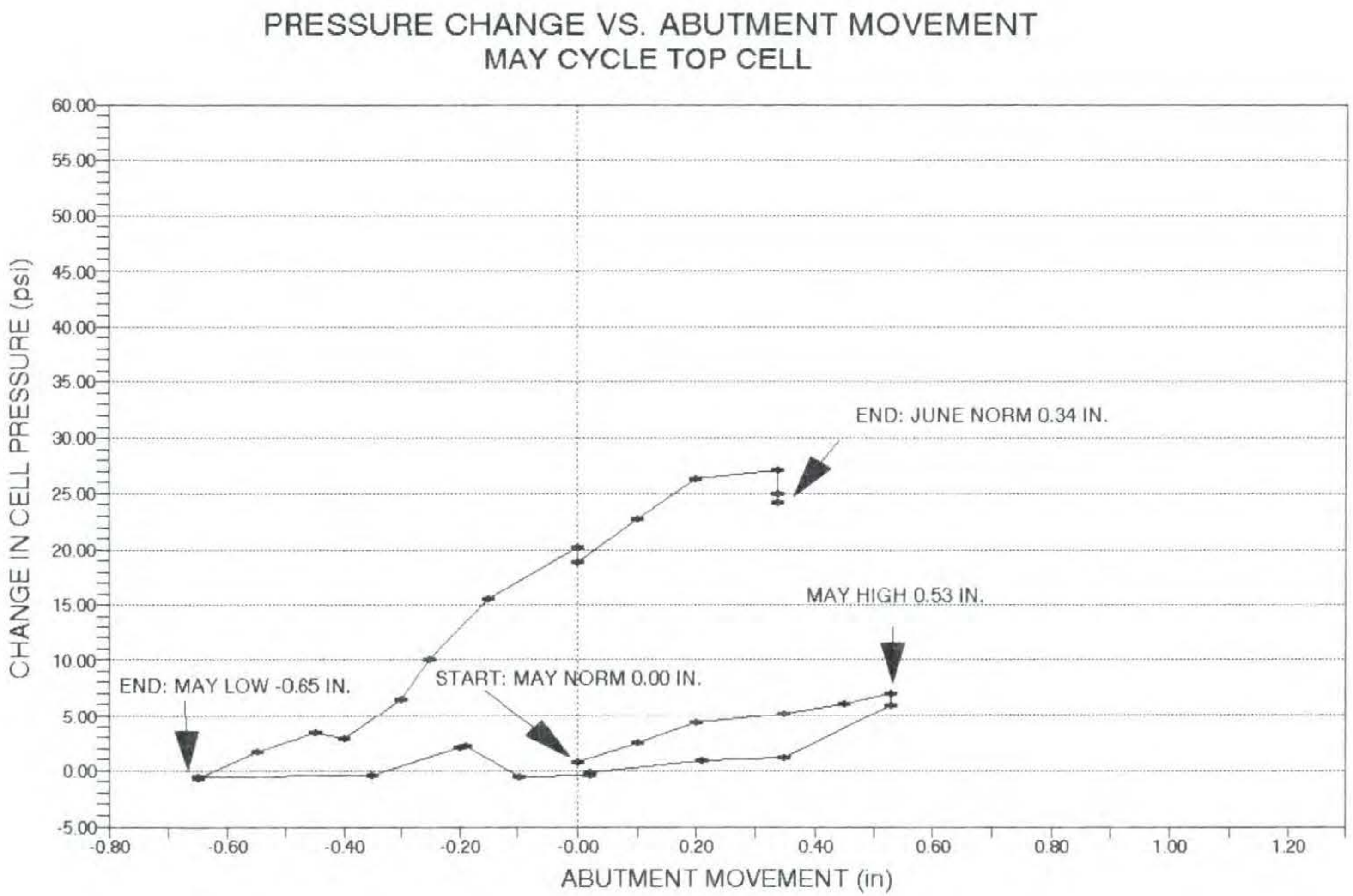



PRESSURE CHANGE VS. ABUTMENT MOVEMENT
FEBRUARY CYCLE BOTTOM CELL
CHANGE IN CELL PRESSURE (psi)
60.00
55.00
50.00
45.00
40.00
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
-5.00
-0.80
-0.60
-0.40
-0.20
-0.00
0.20
0.40
0.60
0.80
1.00
1.20
ABUTMENT MOVEMENT (in)
FEB HIGH 0.60 IN.
sTART: FEB. NORM -0.35 IN.
END: MARCH NORM. -0.05 IN.
FEB. LOW -0.50 IN.

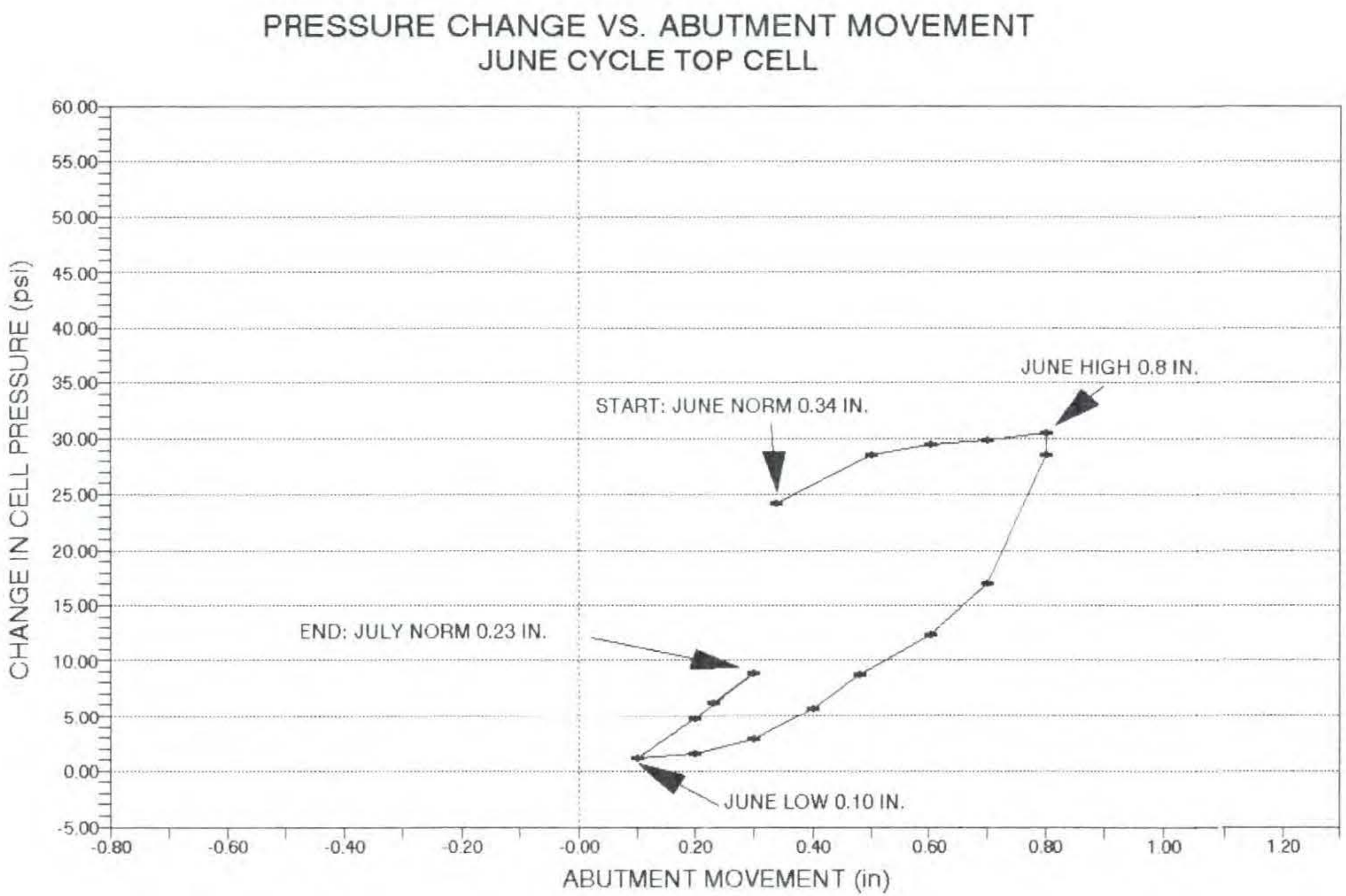
PRESSURE CHANGE VS. ABUTMENT MOVEMENT
MARCH CYCLE BOTTOM CELL

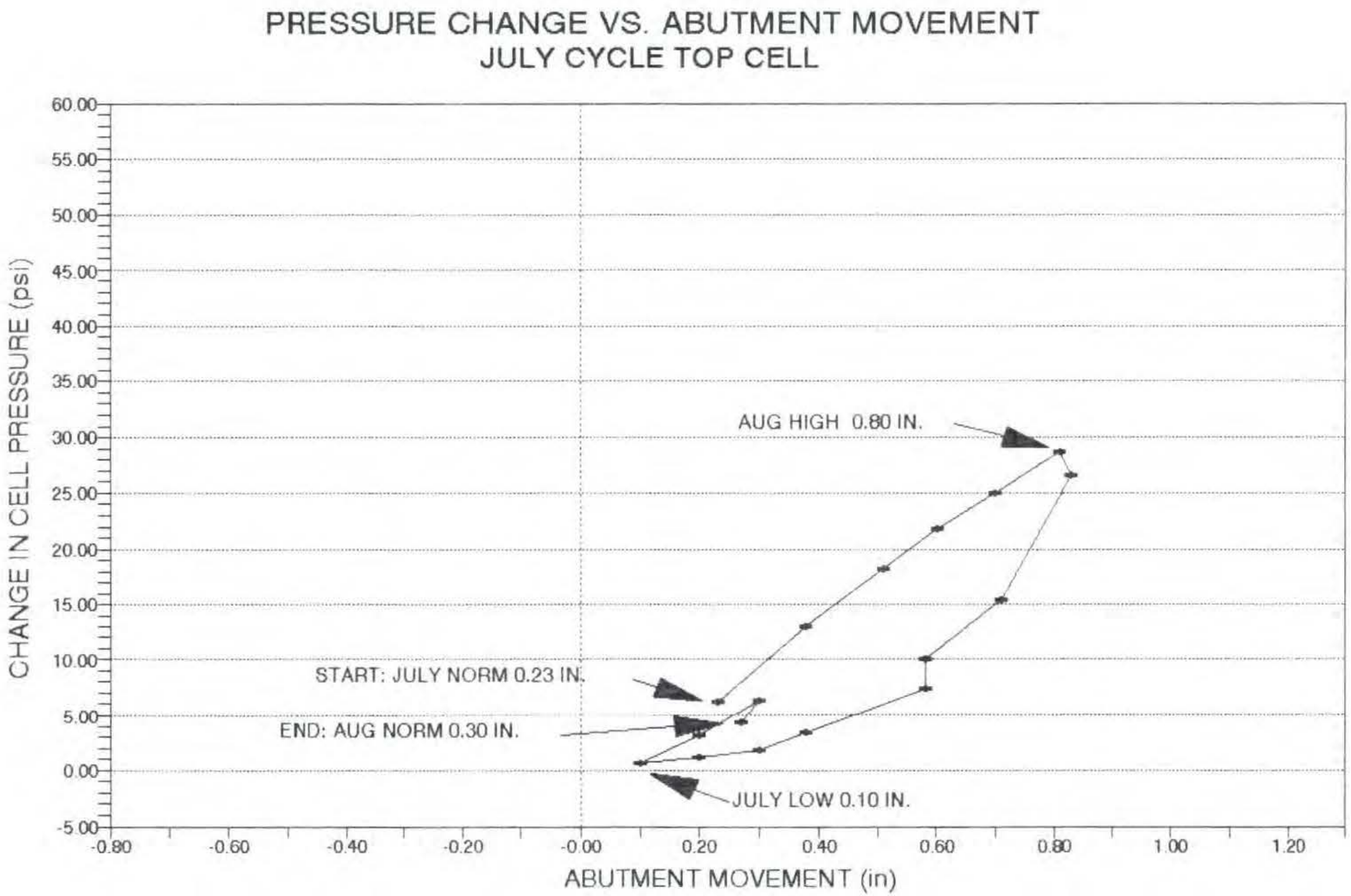


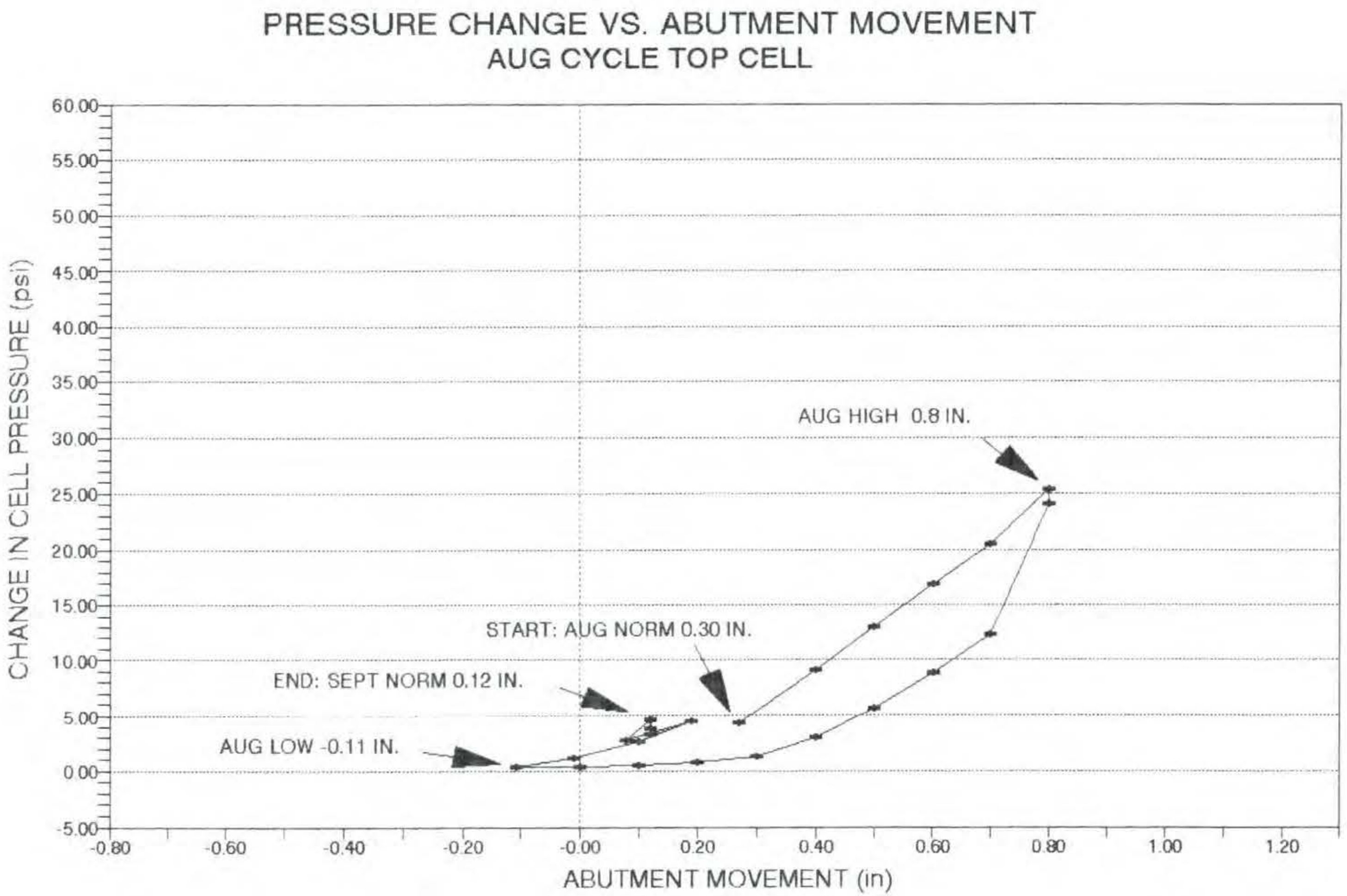

PRESSURE CHANGE VS. ABUTMENT MOVEMENT
APRIL CYCLE BOTTOM CELL
APRIL HIGH 1.21 IN.
START: APRIL NORM 0.40 IN.
START: MARCH LOW -0.20 IN.
APRIL LOW 0.00 IN.
CHANGE IN CELL PRESSURE (psi)
ABUTMENT MOVEMENT (in)
60.00
55.00
50.00
45.00
40.00
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
-5.00
-0.80
-0.60
-0.40
-0.20
-0.00
0.20
0.40
0.60
0.80
1.00
1.20

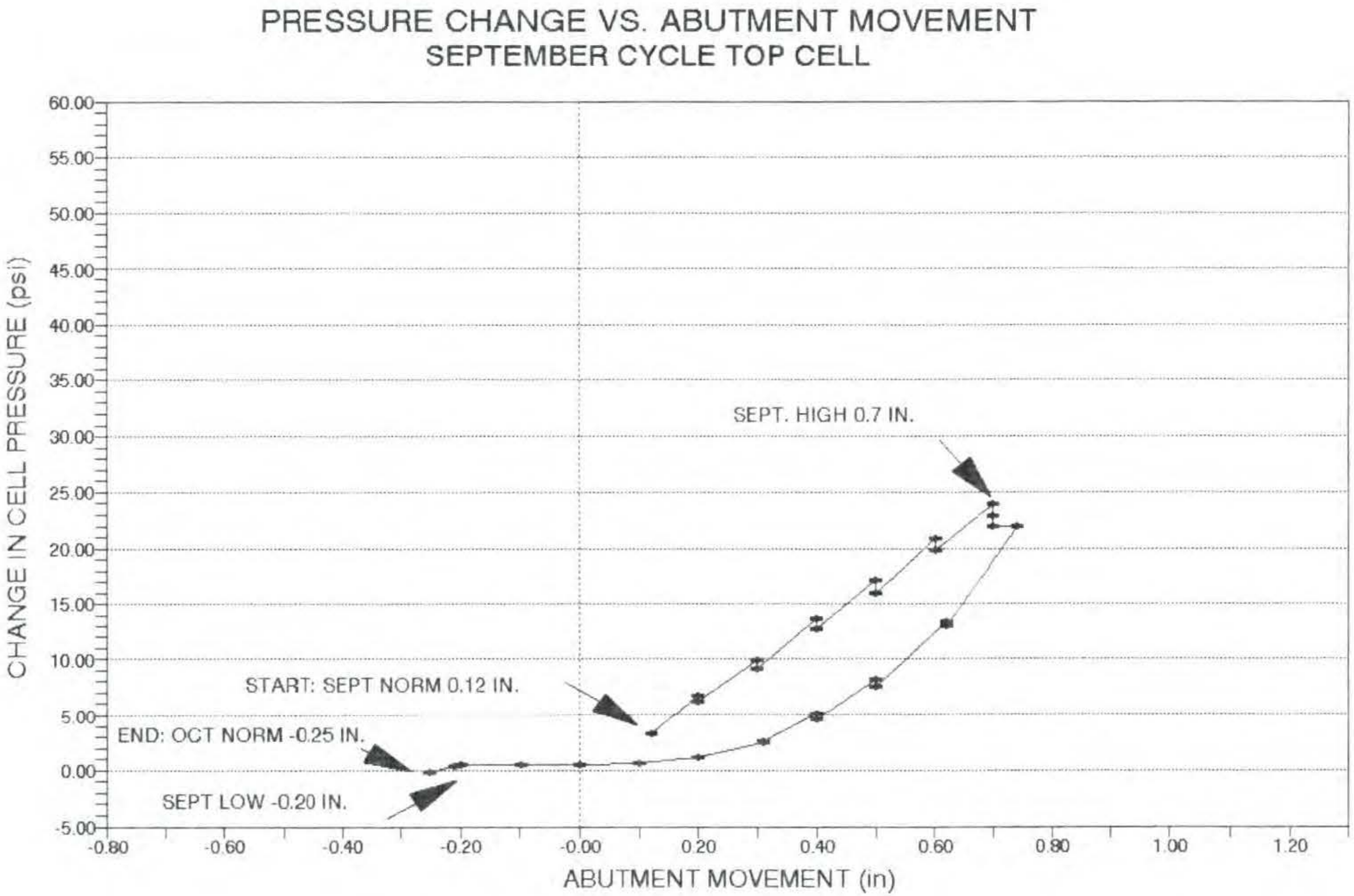


PRESSURE CHANGE VS. ABUTMENT MOVEMENT
MAY CYCLE TOP CELL
CHANGE IN CELL PRESSURE (psi)
ABUTMENT MOVEMENT (in)
END: JUNE NORM 0.34 IN.
MAY HIGH 0.53 IN.
START: MAY NORM 0.00 IN.
END: MAY LOW -0.65 IN.
60.00
55.00
50.00
45.00
40.00
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
-5.00
-0.80
-0.60
-0.40
-0.20
-0.00
0.20
0.40
0.60
0.80
1.00
1.20

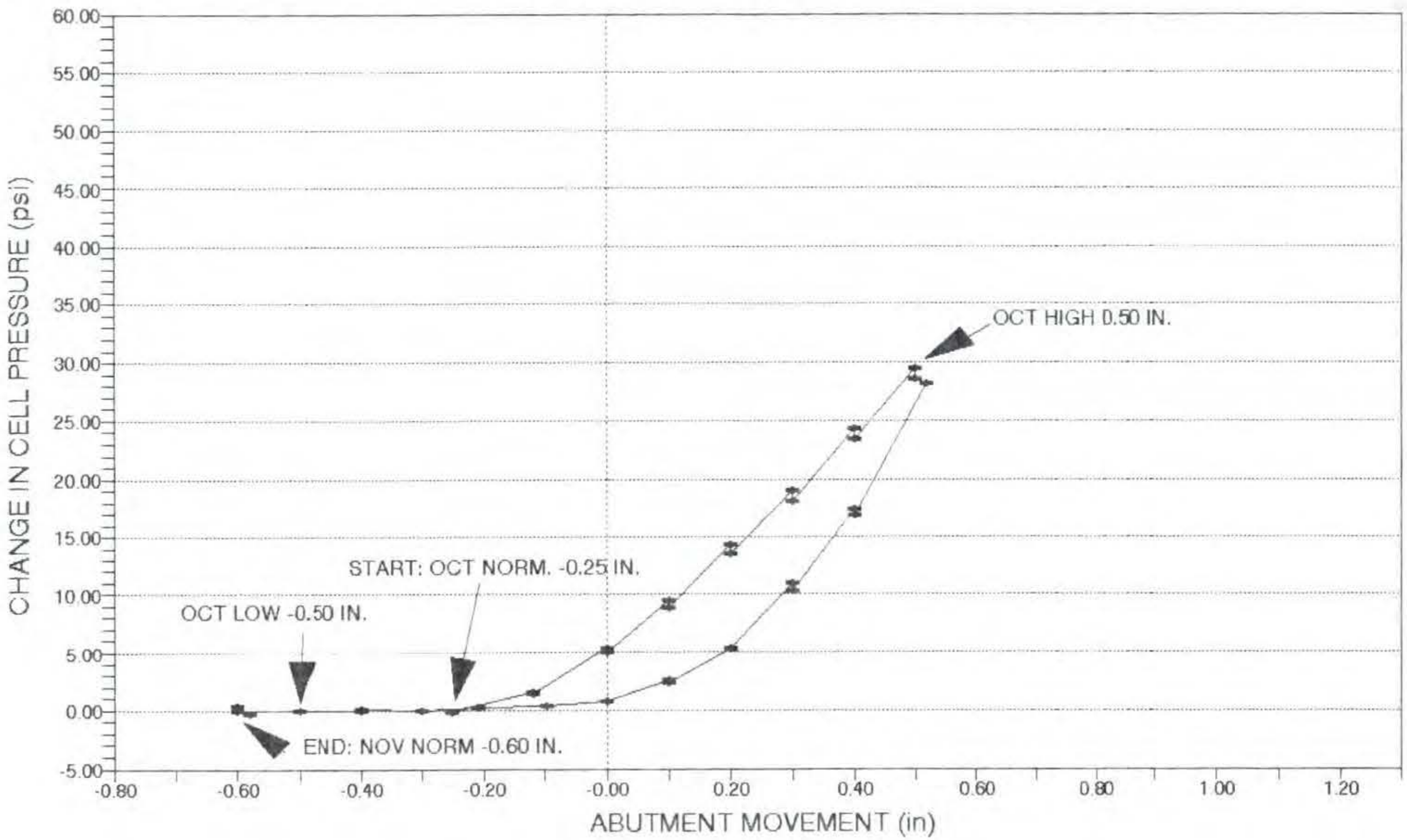

PRESSURE CHANGE VS. ABUTMENT MOVEMENT
JUNE CYCLE TOP CELL
CHANGE IN CELL PRESSURE (psi)
ABUTMENT MOVEMENT (in)
START: JUNE NORM 0.34 IN.
JUNE HIGH 0.8 IN.
END: JULY NORM 0.23 IN.
JUNE LOW 0.10 IN.
60.00
55.00
50.00
45.00
40.00
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
-5.00
-0.80
-0.60
-0.40
-0.20
-0.00
0.20
0.40
0.60
0.80
1.00
1.20


PRESSURE CHANGE VS. ABUTMENT MOVEMENT
JULY CYCLE TOP CELL
CHANGE IN CELL PRESSURE (psi)
ABUTMENT MOVEMENT (in)
AUG HIGH 0.80 IN.
START: JULY NORM 0.23 IN.
END: AUG NORM 0.30 IN.
JULY LOW 0.10 IN.

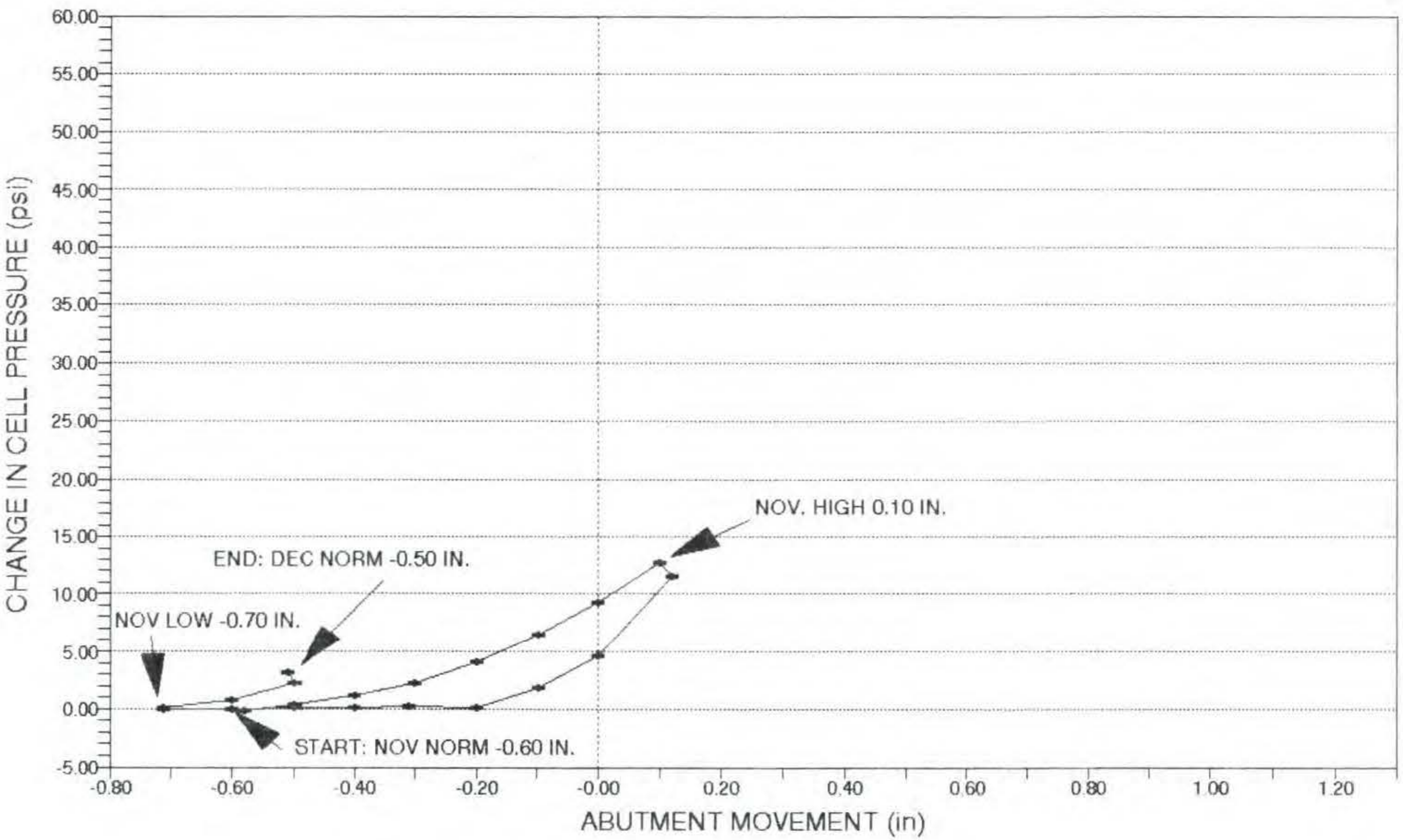

PRESSURE CHANGE VS. ABUTMENT MOVEMENT
AUG CYCLE TOP CELL
CHANGE IN CELL PRESSURE (psi)
60.00
55.00
50.00
45.00
40.00
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
-5.00
-0.80
-0.60
-0.40
-0.20
-0.00
0.20
0.40
0.60
0.80
1.00
1.20
ABUTMENT MOVEMENT (in)
AUG HIGH 0.8 IN.
START: AUG NORM 0.30 IN.
END: SEPT NORM 0.12 IN.
AUG LOW -0.11 IN.


PRESSURE CHANGE VS. ABUTMENT MOVEMENT
SEPTEMBER CYCLE TOP CELL
CHANGE IN CELL PRESSURE (psi)
ABUTMENT MOVEMENT (in)
SEPT. HIGH 0.7 IN.
START: SEPT NORM 0.12 IN.
END: OCT NORM -0.25 IN.
SEPT LOW -0.20 IN.
60.00
55.00
50.00
45.00
40.00
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
-5.00
-0.80
-0.60
-0.40
-0.20
-0.00
0.20
0.40
0.60
0.80
1.00
1.20

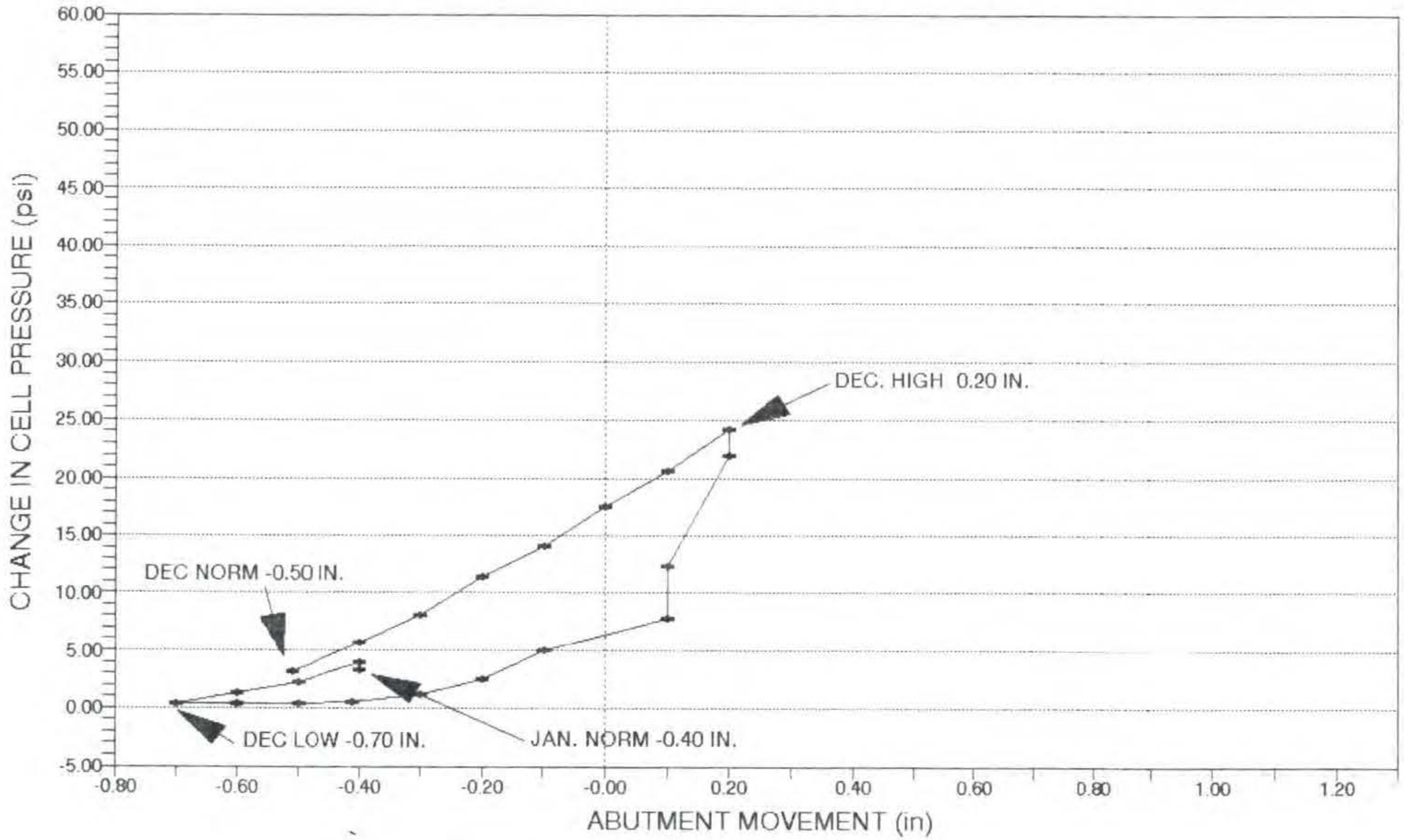
# PRESSURE CHANGE VS. ABUTMENT MOVEMENT
## OCTOBER CYCLE BOTTOM CELL

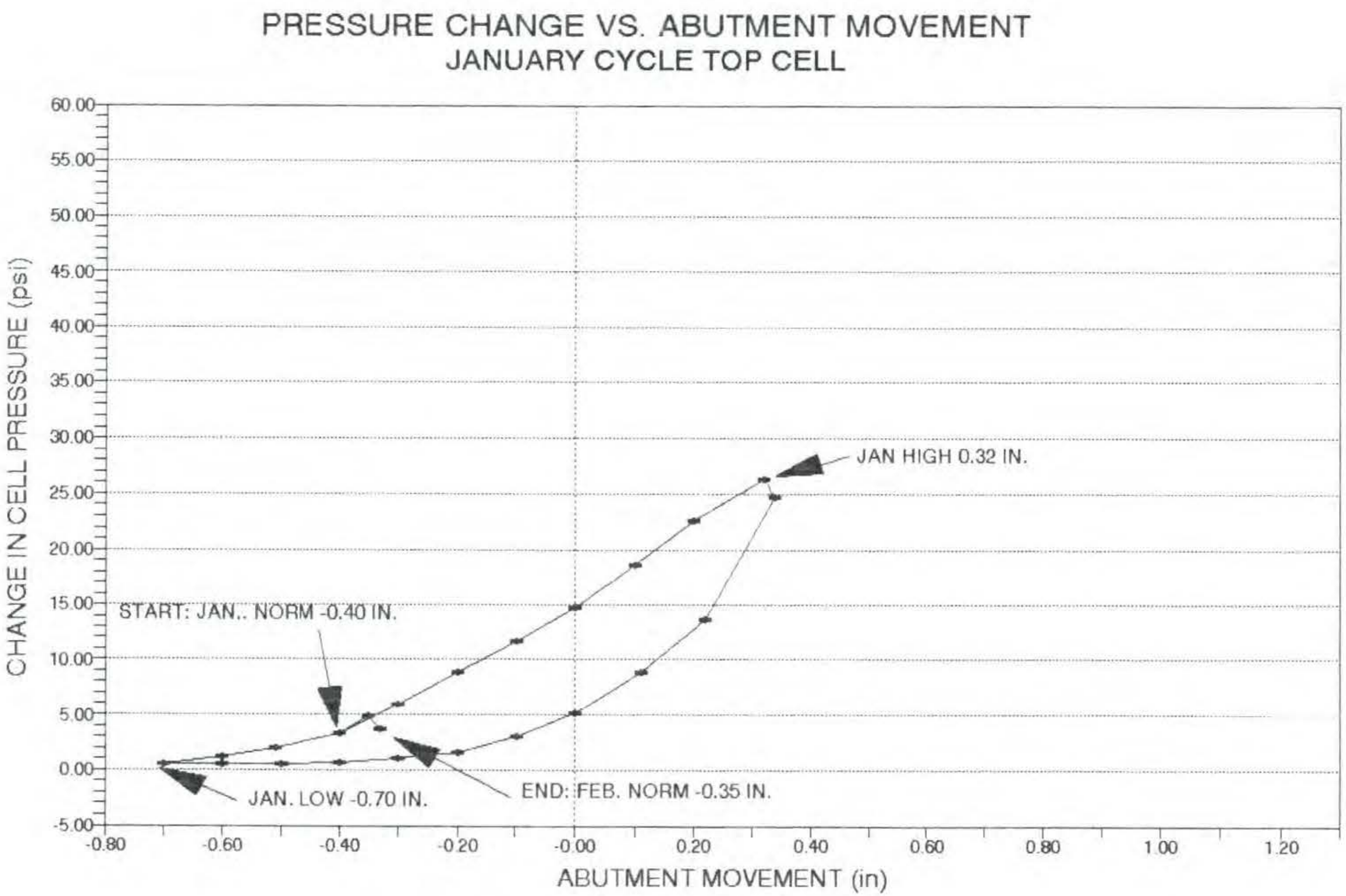


PRESSURE CHANGE VS. ABUTMENT MOVEMENT
NOVEMBER CYCLE TOP CELL

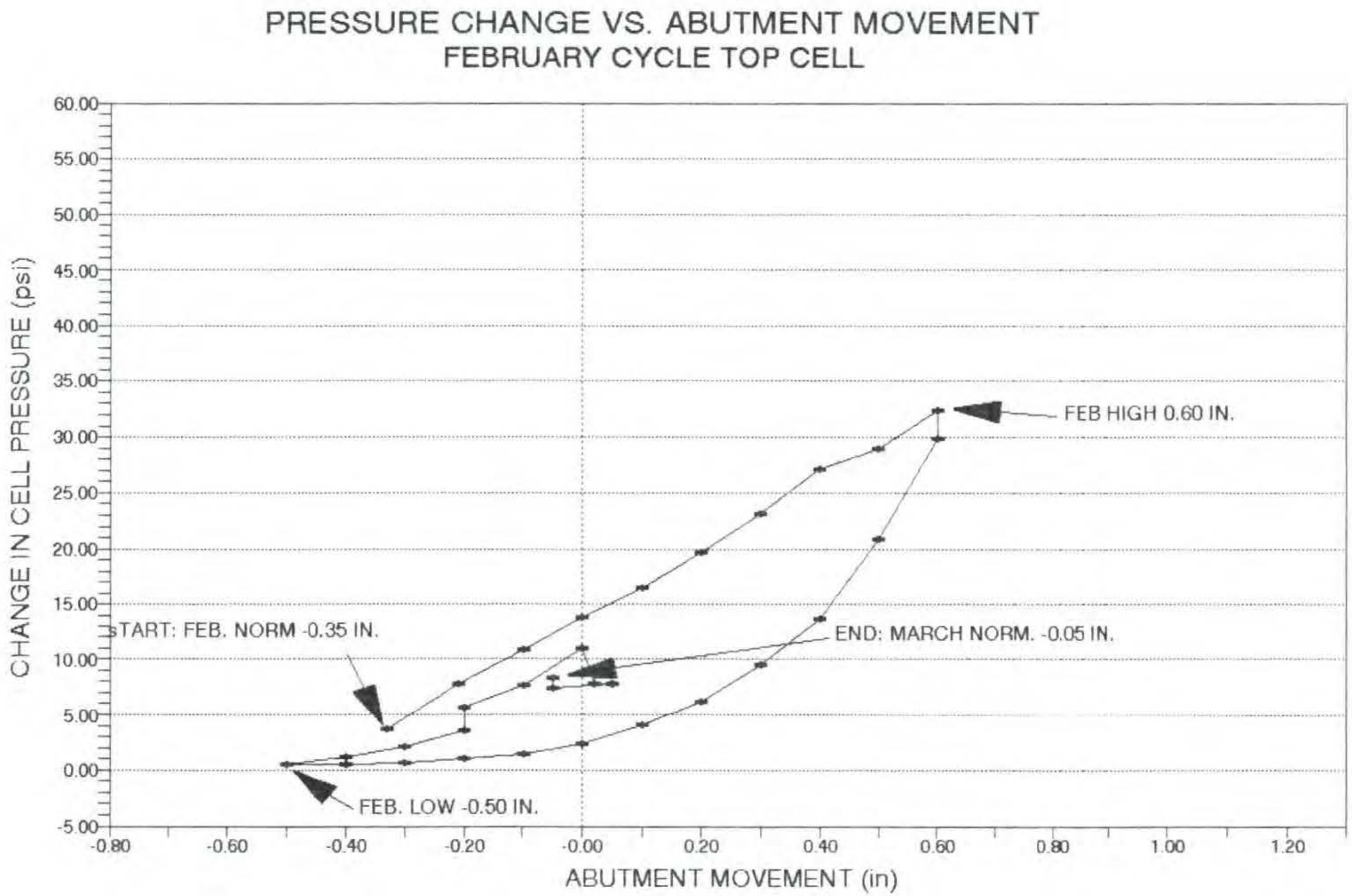


## PRESSURE CHANGE VS. ABUTMENT MOVEMENT
## DECEMBER CYCLE TOP CELL

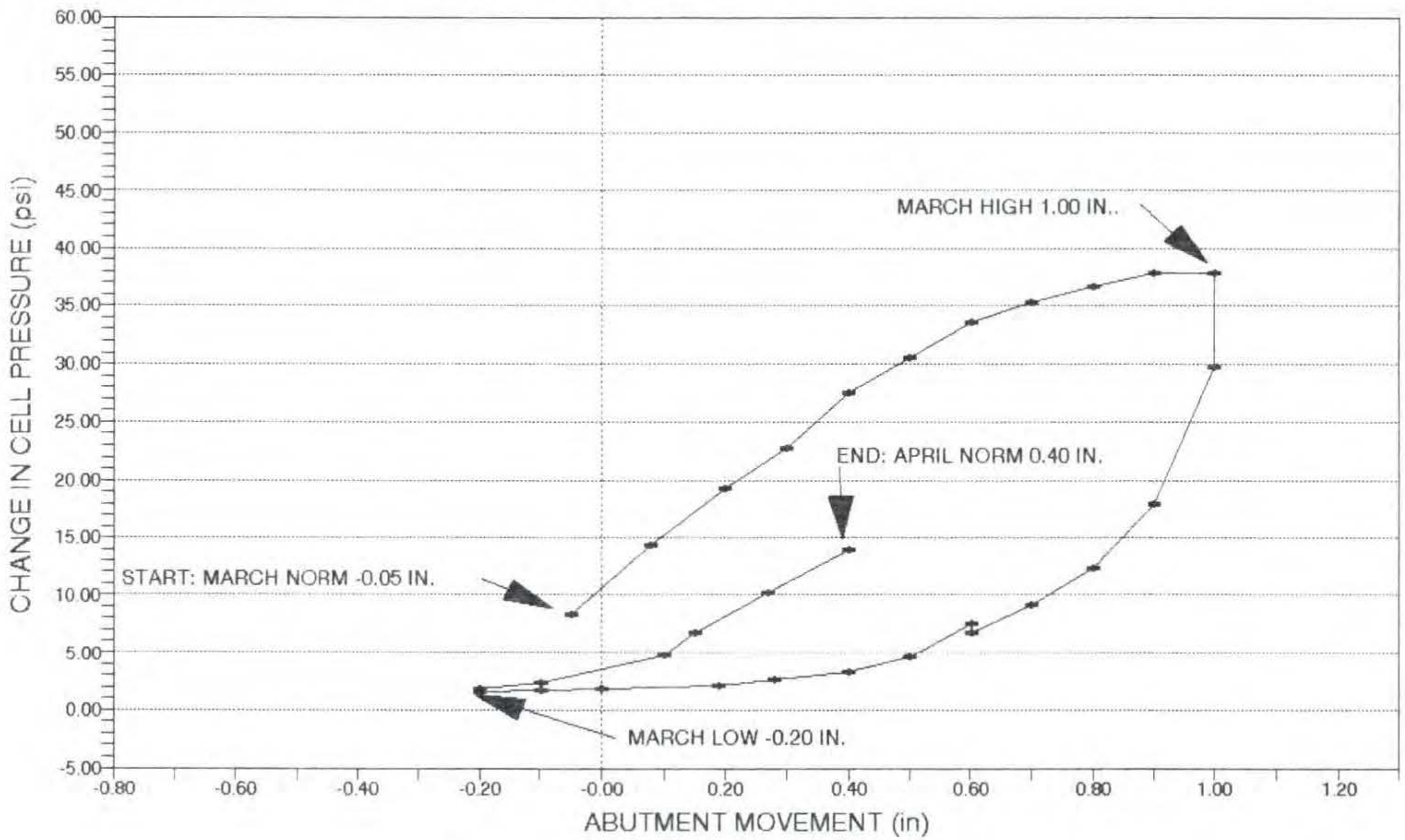




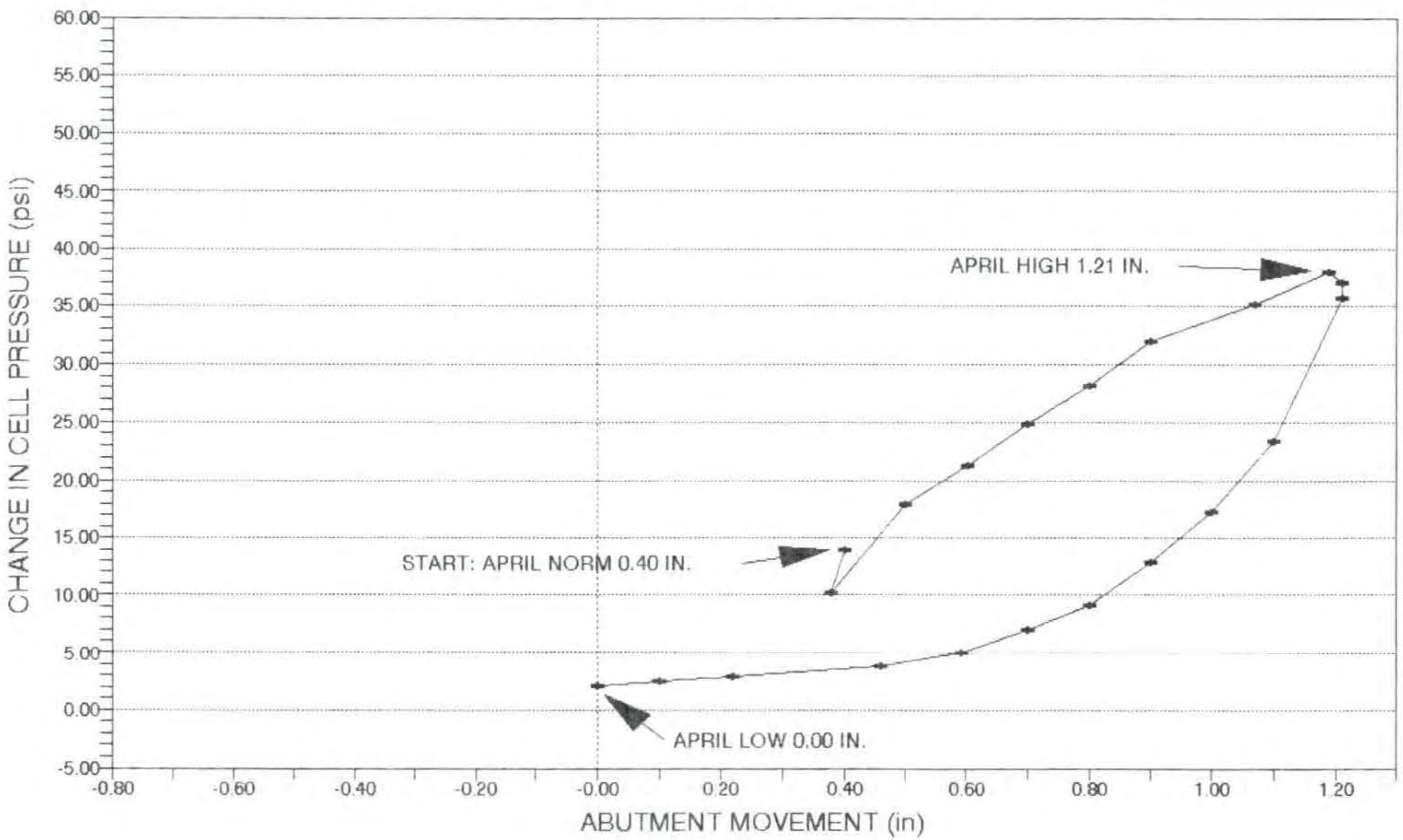
PRESSURE CHANGE VS. ABUTMENT MOVEMENT
JANUARY CYCLE TOP CELL
CHANGE IN CELL PRESSURE (psi)
ABUTMENT MOVEMENT (in)
JAN HIGH 0.32 IN.
START: JAN.. NORM -0.40 IN.
JAN. LOW -0.70 IN.
END: FEB. NORM -0.35 IN.
60.00
55.00
50.00
45.00
40.00
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
-5.00
-0.80
-0.60
-0.40
-0.20
-0.00
0.20
0.40
0.60
0.80
1.00
1.20

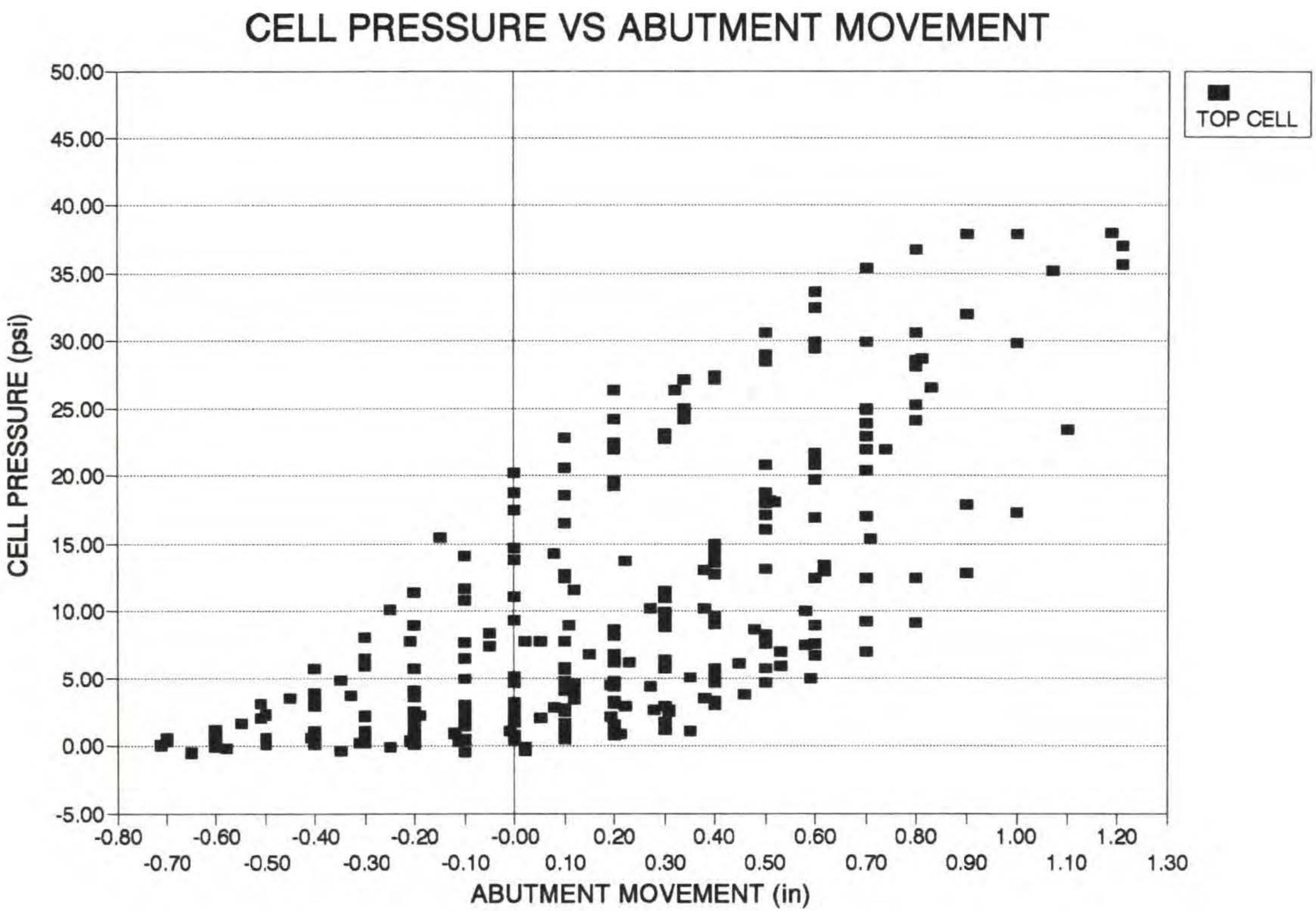

PRESSURE CHANGE VS. ABUTMENT MOVEMENT
FEBRUARY CYCLE TOP CELL
CHANGE IN CELL PRESSURE (psi)
ABUTMENT MOVEMENT (in)
START: FEB. NORM -0.35 IN.
END: MARCH NORM. -0.05 IN.
FEB HIGH 0.60 IN.
FEB. LOW -0.50 IN.
60.00
55.00
50.00
45.00
40.00
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
-5.00
-0.80
-0.60
-0.40
-0.20
-0.00
0.20
0.40
0.60
0.80
1.00
1.20

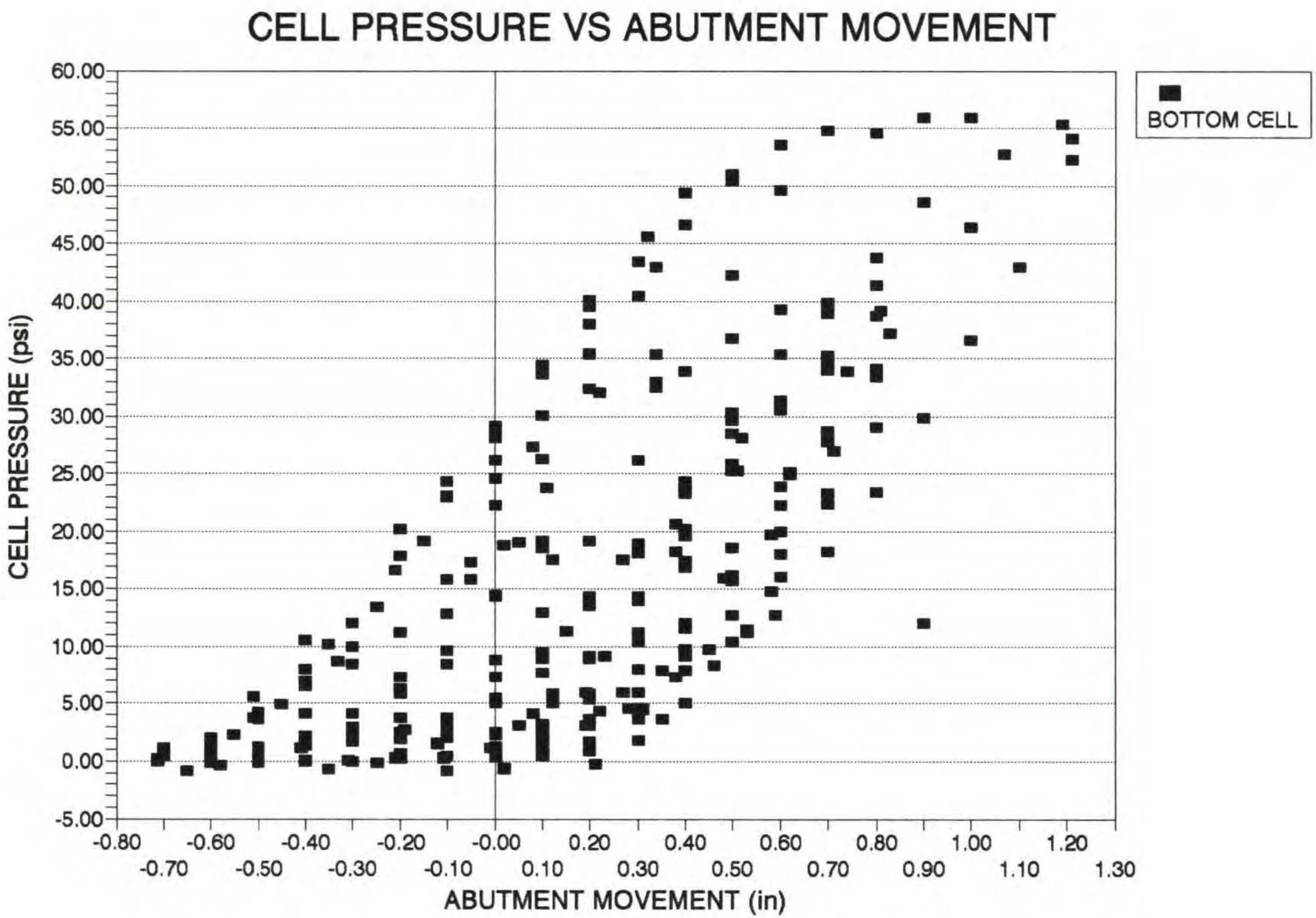
PRESSURE CHANGE VS. ABUTMENT MOVEMENT
MARCH CYCLE TOP CELL



# PRESSURE CHANGE VS. ABUTMENT MOVEMENT
## APRIL CYCLE TOP CELL




CELL PRESSURE VS ABUTMENT MOVEMENT
TOP CELL
CELL PRESSURE (psi)
ABUTMENT MOVEMENT (in)
50.00
45.00
40.00
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
-5.00
-0.80
-0.70
-0.60
-0.50
-0.40
-0.30
-0.20
-0.10
-0.00
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
0.90
1.00
1.10
1.20
1.30


CELL PRESSURE VS ABUTMENT MOVEMENT
CELL PRESSURE (psi)
ABUTMENT MOVEMENT (in)
BOTTOM CELL
60.00
55.00
50.00
45.00
40.00
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
-5.00
-0.80
-0.70
-0.60
-0.50
-0.40
-0.30
-0.20
-0.10
-0.00
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
0.90
1.00
1.10
1.20
1.30