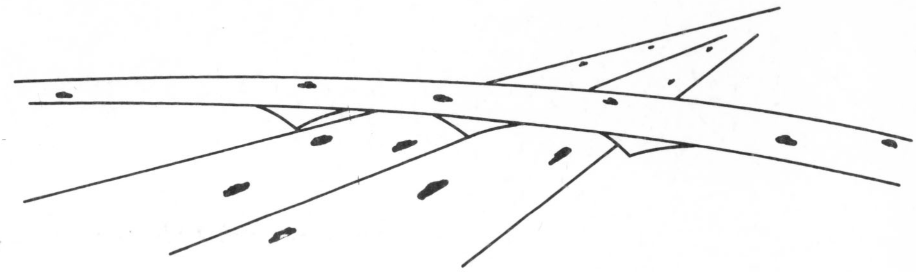

DEPARTMENTAL RESEARCH

Report Number 39 – 1

A FIELD SURVEY AND EXPLORATORY EXCAVATION OF TERMINAL ANCHORAGE FAILURES ON JOINTED CONCRETE PAVEMENT

by

B. F. McCullough

Technical Report No. 1
Research Project 1-8-63-39

TEXAS HIGHWAY DEPARTMENT

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By
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Technical Report No. 39-1
for
Terminal Anchorage Installations
Research Project 1-8-63-39

Conducted by

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A B S T R A C T

During the latter part of the 1950's, the Texas Highway Department constructed numerous terminal anchorage systems at the terminals of concrete pavements adjacent to structures to restrain pavement growth. By 1963, twenty-two anchor units on jointed concrete pavements had experienced failure in the Houston area.

A total of 152 anchor units were inspected and pertinent data collected concerning each; in addition, an excavation was performed adjacent to three separate units to determine the primary rationale of failure. The failures were attributed to an inadequacy in the method of transferring the pavement growth forces to the soil mass. Strength tests of the soil layers and visual observations indicated the soil had sheared along the horizontal plane at the lower extremities of the anchor lugs.

In addition, it was found that the anchor system's age, the distance between lugs and the header wall, concrete coarse aggregate type, constructed embankments, and the absence of transverse cracks in the anchor slab were secondary factors that could be associated with failure.

CONCLUSIONS AND RECOMMENDATIONS

On the basis of a study of 152 anchor units and an excavation of three in-service anchor units in Harris County, the following conclusions are warranted:

(1) A terminal anchor system such as used by the Texas Highway Department is a feasible method for preventing pavement growth forces from damaging an overpass or bridge structure if the anchor system is properly designed and constructed. The feasibility is manifested by satisfactory performance in certain cases where siliceous river gravel was used as a coarse aggregate in concrete pavement and a record of 100 per cent satisfactory performance where oyster shell was used as a coarse aggregate.

(2) The failures experienced on the terminal anchorage systems in Harris County can be attributed to a deficiency in transferring the pavement growth forces from the anchorage system to the soil mass. The excavations revealed a soil shear failure along the horizontal plane at the bottom of the lug members.

(3) There were no cases where failures could be attributed to the concrete structural members of the anchor slab or lug extensions.

(4) The presence of transverse cracks in the anchor slab is not a sign of alarm, but indicative that the anchor system is performing satisfactorily. Crack width measurements revealed that the cracks were only volume change cracks and the magnitude was minor which indicates that the cracks should not be sealed.

(5) Stress calculations on the basis of the soil strength data for the excavated sections indicates the pavement growth forces are probably in excess of 46,000 and less than 67,000 pounds per foot of pavement width.

On the basis of the observations made during this study, the following recommendations should be considered during any design analysis of a terminal anchorage system:

(1) The extreme ends of the lugs should be placed at different depths in order that the resistance may be developed in more than one soil layer. The deepest lug should be placed the greatest distance from the structure.

(2) Consideration should be given to reducing the depth of the lugs or providing for good soil density at and slightly below the bottom of lugs.

(3) The present distance of 15 feet between lugs should possibly be increased to a practical maximum in those instances where soil strength is very low.

(4) The distance between the extreme lug toward the structure and the header wall should be a minimum of 60 feet for soils similar to those in the Houston area for a two lug system.

I. Introduction

Background

In the late 1950's, numerous jointed concrete pavements on the Texas highway system were experiencing an alarming amount of pavement growth, especially along the coastal area. As a result of concrete pavement growth, internal forces are built up in the slab producing an outward push toward the free ends that closes the expansion joint at the bridge ends, ruptures the abutment walls, and applies an undesirable amount of pressure on the bridge or structure. In an effort to check this pavement growth problem, the Houston District constructed the first terminal anchorage system in Texas in March 1959. The satisfactory performance obtained with these initial installations consequently resulted in terminal anchorages being installed at a number of structures throughout the state.

Design

Messrs. Shelby and Ledbetter in their treatis on terminal anchorages enumerated the basic concepts and assumptions employed in designing the terminal anchorage system presently being used by the Texas Highway Department.[1] Basically the anchorage system for jointed concrete pavement consists of two anchor lugs, three feet deep and two feet wide at each pavement terminal. Figure 1.1 shows the details of the anchor slab. As can be seen, the terminal anchorages are heavily reinforced to provide a stiff and rigid resistance member. The design concept of the anchorage system is to transfer the pavement growth forces to the soil mass through the passive and shear resistance of the subsoil. In design, it was felt that the critical elements were the bearing area of the lugs and the shear plane along the bottom of the lugs, as well as along the face of a Coulomb Wedge.

Referring again to Figure 1.1, the nomenclature of various components of the anchorage system may be enumerated at this point. The slab placed on top of the base or on top of the subsoil is defined as the anchor slab. The members extending vertically into the ground are defined as lugs, with the one nearest the structure being considered as the front lug, the other as the back lug.

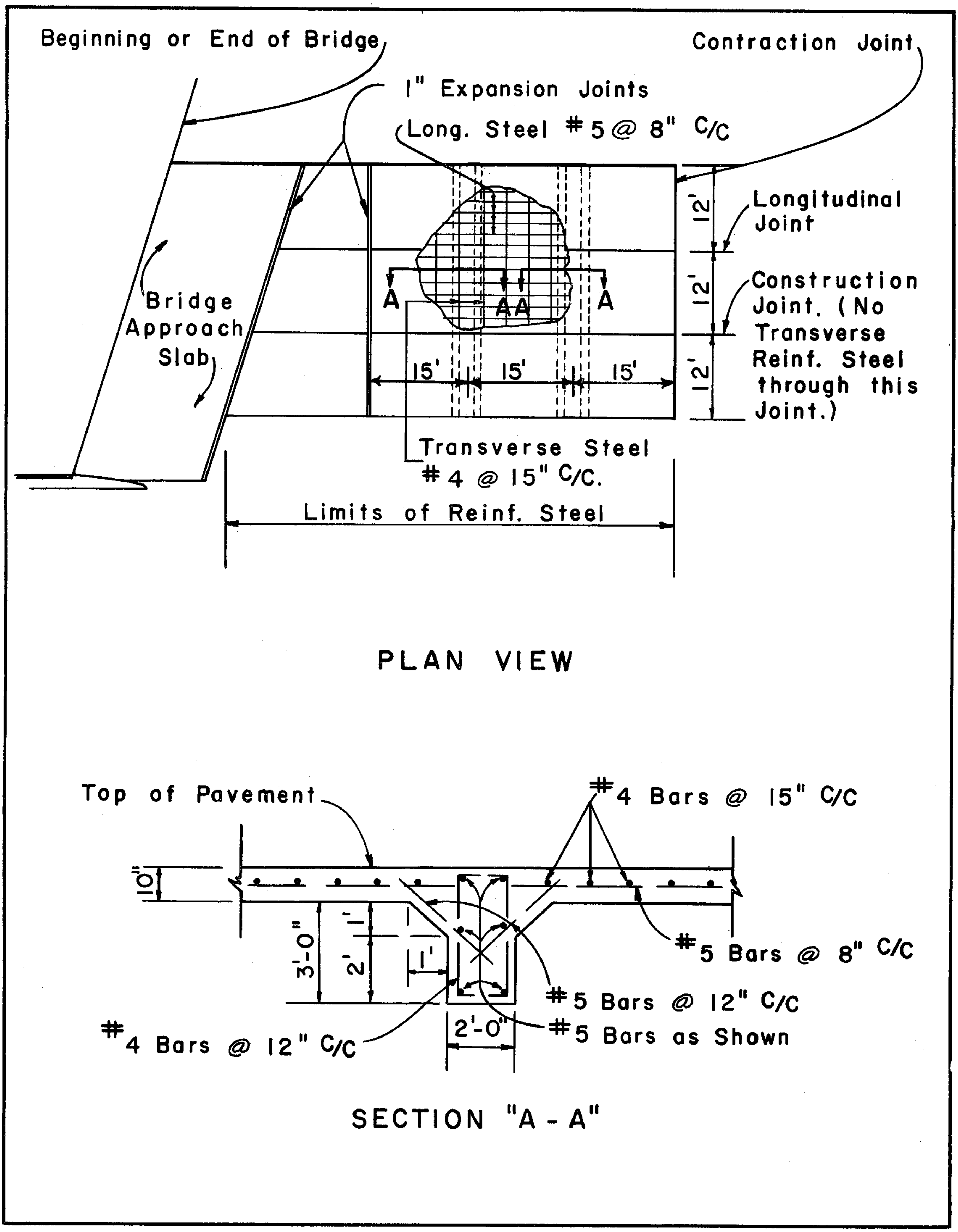


FIGURE 1.1 TYPICAL LUG DESIGN-
JOINTED PAVEMENT

Failures

During January 1963, several cases of terminal anchorage failure were reported in the Houston area. A preliminary survey indicated a number of the terminal anchorage systems had experienced cracking in the anchor slab, closing of the joints between the anchor slab and the bridge approach slab, and faulting of the abutment walls. Figures 1.2 through 1.5 show several of the irregularities that were associated with the general failure conditions during the preliminary inspection.

Figure 1.2 portrays transverse cracks in a typical anchorage slab. The magnified width is due to an unsuccessful attempt to seal the crack by state maintenance forces. The crack widths were so small that the sealing material failed to penetrate into the crack, hence it was spread over the pavement surface by traffic.

Figure 1.3 portrays a typical expansion joint at the end of the anchor slab. Considering the 3/4 inch core fill material placed in the joint, this joint is closed for all practical purposes.

Figures 1.4 and 1.5 depict the general nature of the cracking and faulting experienced in the abutment wall at several of the structures. The differential faulting ranged from 1/8 inch to approximately two inches at various locations. The cracking of the abutment walls was a graphic demonstration of the expansion forces involved. This faulting is different from abutment wall cracking normally associated with soil expansion in that there is a definite cleavage plane that can be associated with the pavement growth.

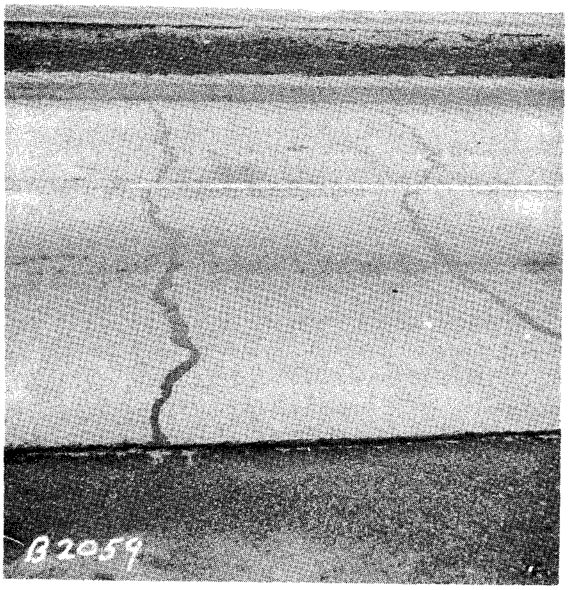


FIGURE 1.2 - Transverse cracks in the anchor slab. An attempt was made to seal cracks.

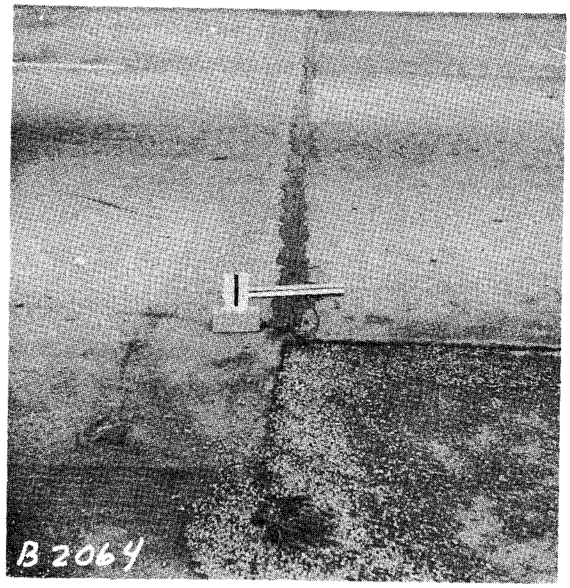


FIGURE 1.3 - Closed expansion joint between pavement and bridge approach slab. Joint was $1\frac{1}{2}$ inches wide at time of construction.

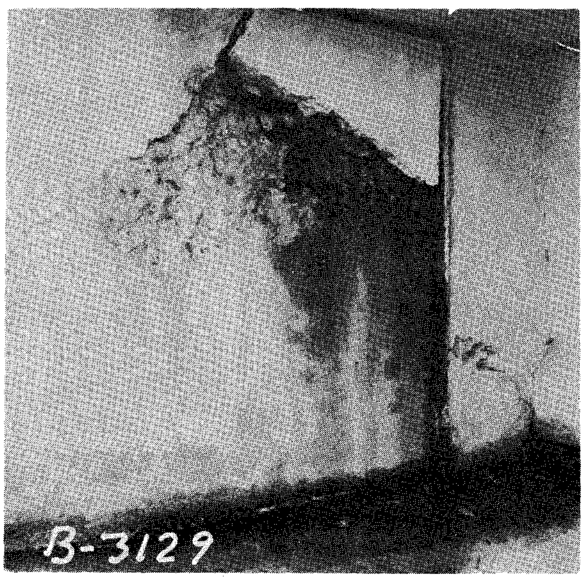


FIGURE 1.4 - Fracture and faulting of bridge abutment wall due to pavement growth.

FIGURE 1.5 - Edge view of abutment wall faulting due to pavement growth.

II. Plan of Investigaton

The objective of this investigation was to determine the extent of the anchorage failures, and to perform the field observations and operations necessary to obtain data for use in re-evaluating the terminal anchorage design. In order to attain the objective, the investigation was divided into two phases. The first phase of the study was a data survey of all terminal anchorage systems in the Houston area to determine the extent of the failures, and to record all pertinent parameters that might influence or result in a failure. The second phase of the investigation consisted of excavating adjacent to several in-service terminal anchorage slabs to determine if the failures were due to a structural failure of the anchorage system or to a failure in the soil mass.

Data Survey

The purpose of this phase of the investigation was to collect and classify all data that might be considered relevant to the failures. For the purpose of classification, terminal anchorage failure was defined as a condition where the system had experienced: (1) complete closure of all expansion joints, and (2) faulting of the abutment wall. As the study progressed, the need for an intermediate or third classification became evident. The third classification was designated as questionable which has the same criteria as the failure classification with the exception that the conditions could not positively be attributed to pavement growth due to a small degree of joint width.

A field survey was conducted to classify each anchorage system as to failure or non-failure. All pertinent observations such as cracking pattern of the anchor slab expansion joint width, and faulting or cracking of the abutment wall were recorded. Figure A.1 in Appendix A is a sample copy of the form used in this study.

In addition to the field survey, all design and construction records were examined in detail as well as interviewing the construction personnel on a particular project in order to obtain as much information as possible pertaining to each location. This data consisted of physical features, concrete properties, concrete placement conditions, subbase material, and any other useful information. Figure A.2 in Appendix A is a sample copy of this record sheet.

Field Excavation

An excavation adjacent to the anchor slab was deemed essential in order that the failure type might be defined before making any attempt to redesign the terminal anchorage system. The scope of this phase of the investigation encompassed the following:

(1) To examine the underlying conditions of the anchor system by use of a slit trench in order to determine if the failure was a result of a concrete structural weakness and/or soil conditions.

(2) On the basis of the data survey, three locations were selected for excavation. Two of the locations were in a failure condition and represented two basic types of soil - one being in a clay fill and the other in a sand fill. The third site was a satisfactorily performing anchor system that was to be used for control purposes.

(3) To obtain sufficient soil tests so that the soil characteristics could be defined.

III. Data Survey

During the months of May and June, 1963, a total of 152 individual anchorage units were inspected, classified as to performance, and the pertinent data as to design and construction was collected as outlined in the previous chapter. Of the 152 anchor units inspected, a total of 22 were classified as failure and 20 were classified as questionable.

For the purpose of this report, an anchor unit is defined as one anchor slab as shown in Figure 1.1. For every structure there were two anchor units in most cases, one at each end of the structure. Since this was not a preplanned experiment by the nature of the investigation, the data for each unit was categorized in an attempt to isolate possible factors contributing to failure. A factorial grouping of the data proved to be very beneficial even though the factorial blocks are incomplete due to concentrations of data with equal conditions.

The factors found to be pertinent in this investigation were the pavement-structure components, the anchor slab age, the distance of lug to abutment wall, the approach grade, and the presence of cracks in the anchor slab.

Pavement Structure Components

The coarse aggregate type, the joint spacing, the base type, and stabilization were the pavement-structure components considered during this phase of the investigation. Pavement thickness, which is also a pavement structure component will be discussed later. Table 3.1 shows the number of units of each classification - failure, questionable, and satisfactory - in terms of the pavement structure components represented in each factorial block.

The first pertinent point in Table 3.1 is that all failures occurred where siliceous river gravel was used as the coarse aggregate, and none occurred where oyster shell was used as the coarse aggregate. This variation in performance may be attributed to two possible factors, these being (1) the difference in the "thermal coefficient of expansion and contraction" of the concrete produced by the two types of aggregate and (2) the use of shell concrete as a base with an asphaltic concrete wearing course. Studies conducted in Texas have shown that concrete produced with oyster shell coarse aggregate has about one-half the thermal coefficient of concrete produced with siliceous river gravels.[2] Therefore,

TABLE 3.1
FACTORIAL ARRANGEMENT OF PAVEMENT STRUCTURE INFORMATION FROM DATA SURVEY OF ANCHORAGE SYSTEMS

Base Type	Stabilization	Coarse Aggregate Type: SILICEOUS RIVER GRAVEL		OYSTER SHELL	
	Joint Spacing (ft)	61'-6"	15'	61'-6"	15'
Shell W/Sand Admix	None		28S 4Q		26S
	Cement				
Shell	None		22S 4Q 12F		
	Cement				
Iron Ore	None	4S 4F	4S 12Q 6F		
	Cement	26S			

NOTE:

(1) The number in the factorial block is the number of anchorage system units that comply to the conditions of block.

(2) An anchor slab is considered as a unit, i.e., there would be two units at a structure, one on each side.

(3) S - anchorage system performing satisfactorily.

F - anchorage system had experienced sufficient movement to close the expansion joints, to crack the abutment wall, and to show a fault or cleft at the crack in the abutment wall.

Q - same as the failure criteria with the exception that the conditions could not be positively attributed to pavement growth due to joint conditions.

with all other conditions being equal, i.e., temperature change, pavement thickness, etc. the shell concrete pavement will produce about one-half the force on the anchor system that would be obtained with conventional pavement. It may be conjectured that an asphalt concrete wearing surface would reduce joint infiltration and the resulting pavement growth, but this point is questionable since many maintenance men are of the opinion that an asphalt overlay does not completely impede pavement growth on conventional concrete pavement.

The use of siliceous river gravel as a reason for failure should be looked upon as a secondary factor since the original design was conceived to prevent the growth of either type of pavement. By the progress of rational analysis, it may be surmised that the maximum allowable force on the present anchorage system is between the force developed by conventional pavement and that developed by oyster shell concrete pavement.

Another observation from Table 3.1 is that failures occur with both the 15 foot joint spacings and the 61.5 foot joint spacing. In considering the effect of the various subbases, it may be noted that failure occurs with every type of subbase with the exception of the cement stabilized iron ore gravel. The distribution of failures in the various non-stabilized subbases eliminates the possibility that failures occur because of various subbase frictions. The absence of failures on the cement stabilized subbase lends credence to the possibility that the high friction value on cement stabilized bases reduces movement since pavement cores have shown a high degree of adhesion between the two types, but the validity of this postulate is questioned for reasons enumerated later.

<u>Pavement Age</u>

In this phase of the study, all anchor units at the terminals of shell concrete pavement were deleted from the analysis. This deletion was made on the basis that the shell concrete units represent the oldest pavement with anchor systems since all are five years or older, and the presence of non-failed sections tend to bias the data. Table 3.2 shows the distribution of the total number of anchor units with a given age in terms of the various performance classifications.

Considering the units with the 15 foot joint spacing, no failures were experienced prior to an age of four years, although, four units were questionable at an age of 2½ years.

TABLE 3.2
FACTORIAL ARRANGEMENT OF DATA ON THE
AGE OF ANCHORAGE SYSTEMS

AGE OF ANCHORAGE SYSTEM	15 FOOT JOINT SPACING				61'-6" JOINT SPACING			
	TOTAL NUMBER UNITS	SATISFACTORILY PERFORMING UNITS	QUESTIONABLE UNITS	FAILURE UNITS	TOTAL NUMBER UNITS	SATISFACTORILY PERFORMING UNITS	QUESTIONABLE UNITS	FAILURE UNITS
1					16	16		
1½								
2					10	10		
2½	8	4	4					
3	2	2			4			4
3½	14	14						
4	18	10	4	4				
4½	16	8		8				
5	32	16	12	4				
5½	2			2				

NOTE:

(1) Anchorage systems with shell concrete pavements were deleted from this factorial.

From this, it may be surmised that a period of approximately $2\frac{1}{2}$ years is required before the full force of pavement growth is transmitted to the structure.

At this point, it is appropriate to define pavement growth. Pavement growth is defined as a progressive outward movement of the pavement end due to a combination of joint infiltration and the pavement's expansive forces. During the winter season, joints open and deleterious material filters into the open joint. When the concrete expands during the summer season, the joints cannot fully close, hence, an internal compressive force is built up within the slab that results in an outward push of the pavement ends.

Next considering the terminal anchorages on pavements with 61.5 foot joint spacing, it is obvious that most of the observations made previously apply here also. An earlier observation that there were fewer failures with a 61.5 foot joint spacing may be due to the fact that the majority of the pavements with this joint spacing are less than two years old. There is also a good possibility that the absence of failures with pavements on cement stabilized subbases may also be explained by this age factor, since all of the pavement observed with cement stabilized subbases are less than two years old.

Pavement Thickness and Soil Shear Resistance Available

This phase of the study is concerned with the effect of pavement thickness and the available soil shear resistance on the failures. Since the soil strength data was not available for each unit, the minimum distance from the header wall to the first anchor lug was taken as a gage of the shear resistance. Table 3.3 presents the performance classifications of the anchor units in terms of the distance from first lug to header wall and pavement thickness. The data used in this phase of the study is the same as used in the previous analysis with the exception that only the anchor units on pavements with 15 foot joints are used since a better age distribution is available.

One fact evident from a study of the table is that questionable performance occurs with various pavement thicknesses, i.e., 10, 11, and 12 inches. Theoretically, the magnitude of the expanding force developed by pavement growth will be directly proportional to the pavement thickness, but is evident in this case that sufficient force is developed with the

TABLE 3.3
FACTORIAL ARRANGEMENT OF DATA FROM PAVEMENTS WITH
15' c-c JOINT SPACING IN TERMS OF PAVEMENT THICKNESS AND
DISTANCE OF LUG FROM STRUCTURE

Distance From Abutment to First Lug in Feet / Pvt. Thickness-in.	9"	10"	11"	12"
35'		8F		
49'			2F 8Q	
60'		4F 4Q 22S		16S 4Q
70'			4F 4Q 4S	
75'			12S	

NOTE:

(1) Anchorage systems with shell concrete pavements deleted from this study. Concrete pavements with 61'-6" joint spacing also deleted.

(2) The number in the factorial block is the number of anchorage system units that comply to the conditions of block.

(3) An anchor slab is considered as a unit, i.e. there would be two units at a structure, one on each end.

(4) S - anchorage system performing satisfactorily.

F - anchorage system had experienced sufficient movement to to close the expansion joints, to crack the abutment wall, and to show a fault or cleft at the crack in the abutment wall.

Q - same as the failure criteria with the exception that the conditions could not be positively attributed to pavement growth due to joint conditions.

thinnest pavement to fail the anchor system.

From the standpoint of developing adequate shear resistance at the bottom of the lug, it may be seen in Table 3.3 that no failures occurred with a distance of 75 feet or greater. In addition, the table shows that any anchor unit with a minimum distance of less than 60 feet was in a failure or questionable condition. It may be surmised from the data that for soils in the Houston area, a minimum of 60 to 75 feet is required between the header or abutment wall to the first anchor lug.

Roadway Grade

In this phase, the data used in the previous study is analyzed from the standpoint of the roadway grade approaching the structure (see Table 3.4). A positive grade is defined as a condition where the pavement would be pushing upward toward the structure, and a negative grade is where the pavement is pushing downward toward the structure. None of the units in this study were on negative grade.

The data showed all failures occurred on a grade of 3 per cent or more, whereas the units on zero per cent grade had a relatively good performance record. This association of failure with a positive grade is counter to that expected in that a preliminary rational analysis indicates the downward component of the pavement weight parallel to the roadway grade would tend to offset the expansive forces. This observation can be attributed to, in all probability, the fact that all the positive grades are on overpass structures. Since Harris County has a relatively flat landscape, embankments must be constructed for all overpass structures. Hence, the lug in a constructed embankment may not have the available resistance that, relatively speaking, is available to a lug in a natural soil (as is common with a level grade).

Transverse Cracks

The presence of transverse cracks similar to those shown in Figure 1.2 were a point of contention among several of the design engineers. A number of engineers on casual inspection logically associated the transverse cracks with structural failure of the anchor system.

TABLE 3.4

FACTORIAL ARRANGEMENT OF DATA FROM PAVEMENTS WITH 15 FOOT JOINT SPACING IN TERMS OF GRADE

% GRADE	NUMBER OF UNITS	PERFORMANCE RATING		
		Satisfactorily	Questionable	Failures
0	26	22	4	
1				
2	4		4	
3	48	26	12	10
3.5	10	2		8
5	4	4		

NOTE:

(1) Anchorage systems with shell concrete pavements deleted from this study. Concrete pavements with 61'-6" joint spacing also deleted.

(2) The number in the factorial block is the number of anchorage system units that comply to the conditions of block.

(3) An anchor slab is considered as a unit, i.e. there would be two units at a structure, one on each end.

(4) S - anchorage system performing satisfactorily.

F - anchorage system had experienced sufficient movement to close the expansion joints, to crack the abutment wall, and to show a fault or cleft at the crack in the abutment wall.

Q - same as the failure criteria with the exception that the conditions could not be positively attributed to pavement growth due to joint conditions.

During the data survey, the cracks in the anchor slabs were recorded on the data sheets, and this data is presented in Table 3.5 in terms of performance. The data shows that 94 per cent of the satisfactorily performing anchor systems had cracks in the anchor slab. Whereas, only 56 per cent of the anchor systems that were in a failure condition had cracks in the slab. The four anchor units (6 per cent) without transverse cracks that were presently performing satisfactorily were less than two years old, hence were not old enough for the pavement growth forces to be transmitted to the structure. The anchor units with cracks, but experiencing failure, probably performed satisfactorily for a short period of time. Therefore, it may be concluded that the presence of transverse cracks in the anchor slab is one positive sign that the anchor system is performing satisfactorily.

The cracks are simply volume change cracks that occur due to contractive stresses, since the anchor slab is a short section of continuous pavement, if the lugs are providing proper retainment.[3] In all probability, the anchor systems that are in a failure condition and do not have volume change cracks never performed in accordance with the design intent. The lugs did not prevent expansive and contractive movement of the anchor slab due to volume changes since experience shows the stresses were of sufficient magnitude for tensile cracking to occur.

TABLE 3.5

EFFECT OF TRANSVERSE VOLUME CHANGE CRACKS ON PERFORMANCE

	TOTAL	NUMBER WITH CRACKS IN ANCHOR SLAB	PER CENT (%)	NUMBER WITH NO CRACKS IN ANCHOR SLAB	PER CENT (%)
FAILURES	18	10	56	8	44
QUESTIONABLE	20	13	65	7	35
SATISFACTORY PERFORMANCE	64	60	94	4	6

NOTE:

(1) The number in the factorial block is the number of anchorage system units that comply to the conditions of block.

(2) An anchor slab is considered as a unit, i.e. there would be two units at a structure, one on each end.

(3) S - anchorage system performing satisfactorily.

F - anchorage system had experienced sufficient movement to close the expansion joints, to crack the abutment wall, and to show a fault or cleft at the crack in the abutment wall.

Q - same as the failure criteria with the exception that the conditions could not be positively attributed to pavement growth due to joint conditions.

(4) 15 foot jointed pavement only was considered in this table.

(5) 16 units were taken from this study because they were overlayed.

IV. Exploratory Excavation of the Anchorage Systems

After evaluating the data collected in connection with the field survey and the office investigation, three sites were selected for the purpose of making exploratory excavations. All three sites were in the general proximity of Houston as shown on Figure 4.1. Two of these sites were in a failure condition and represented two basic soil types - one being in a sand fill (marked Site #1 on Figure 4.1) and the other in a clay fill (Site #2). The third excavation site was an anchor system that was performing satisfactorily and was used for control purposes (Site #3).

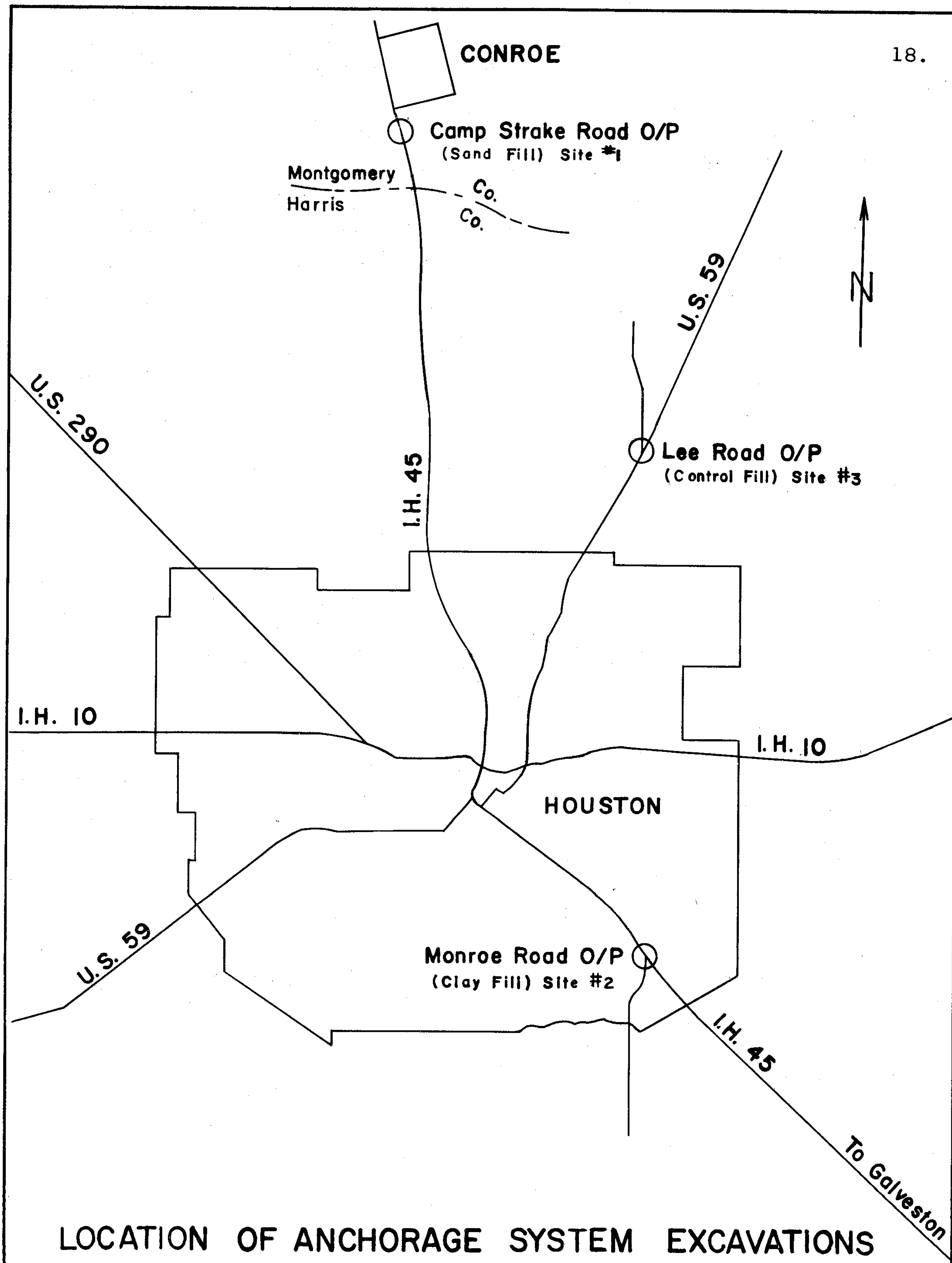
Excavation Procedure

State maintenance forces were used for all excavation work. The excavation work was on divided highways, therefore, the lane adjacent to the excavation trench was blocked off to provide maximum safety and working area (Figure 4.2). Sufficient equipment and men were available so that all excavation and soil testing for each site could be accomplished during the period of one day. The asphalt shoulder was removed to the limits of the excavation by use of a jack hammer (Figure 4.3). A backhoe digger was then used to excavate the underlying material to the depth desired (see Figure 4.4). Figure 4.5 shows the trench immediately after the completion of work by the backhoe digger.

The trench is approximately five feet deep which is one foot greater than the deepest extremity of the lug, and the excavation was carried along the entire length of the anchor slab - approximately 45 feet. About one inch of material was left along the face of the cut adjacent to the lugs during the backhoe excavation, in order that this material could be excavated by hand to obtain a smooth face for observation purposes. During the entire excavation operation, periodic stops were made for the purpose of obtaining soil samples and making visual observations.

Excavation of Anchor Sites Experiencing Failure

Two different sites experiencing failure were excavated on successive days. These locations represented an extreme



LOCATION OF ANCHORAGE SYSTEM EXCAVATIONS

FIGURE 4.1

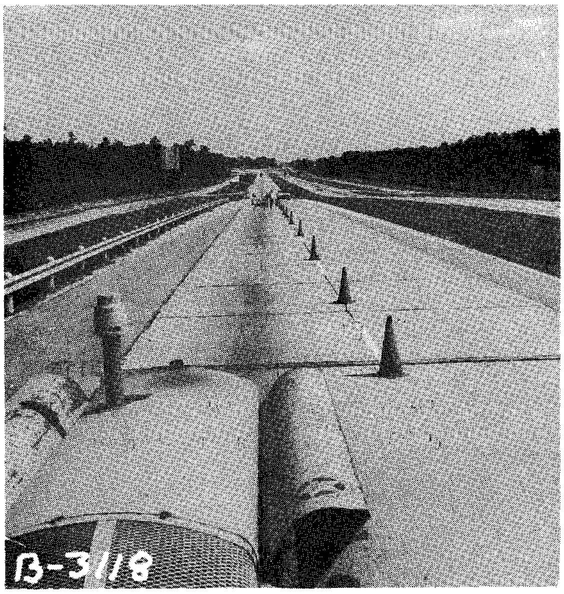


FIGURE 4.2 - General view of an excavation site looking from the structure toward the approach. The generator is sitting on the anchor slab.

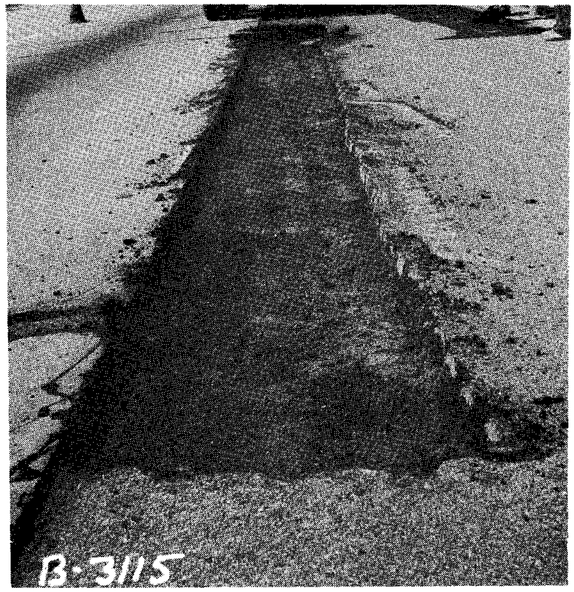


FIGURE 4.3 - Removal of the asphalt shoulder material prior to excavation of the underlying materials.

FIGURE 4.4 - Back-hoe digger used in removing the underlying material.

FIGURE 4.5 - Observation trench after completion of the excavation and prior to hand removal of the face adjacent to the lugs.

in soil conditions as enumerated previously.

Sand Fill (Site #1). A summary of observations pertaining to age, construction, excavation, etc. is presented in Table 4.1. The removal of the first layers exposed a large void behind each of the lugs which presented some initial concern (Figure 4.6). The voids were approximately one foot high and two feet long and extended across the entire width of the pavement. Further excavation revealed that the voids were created by the upper level of iron ore base percolating into the vertical void developed behind the lug as the lug moved toward the structure. Measurements of the void indicated the anchor system had moved approximately 3½ inches toward the structure. (This verified earlier observations made on the basis of measuring expansion joints.) The vertical seam was found to be of equal width along its entire height. This observation indicates that very little or no bending of the lug occurred during the expansive action.

The excavation revealed that both the anchor slab and the lug were structurally intact as may be seen in Figure 4.7. This observation precluded the possibility of an inadequate concrete structure design as a possible cause of failure. A side view of the anchor slab (Figure 4.8) eliminated apprehension that the transverse cracks in the anchor slab were detrimental to the structure. The cracks were the result of volume change stresses and were typical of those found in continuously reinforced concrete pavement. The cracks became progressively narrower from top to bottom and the maximum range was from 0.024 to 0.008 of an inch.

From a visual standpoint, there appeared to be excessive amount of moisture and in some areas free water was present in the soil stratas. In addition, it was evident that the lower layers were extremely loose relative to the upper layer. This observation was especially true of the layer in which the bottom of the lug was located. This layer could be removed by hand with relatively little effort. A definite slippage plane could not be established at any location along the face of the excavation.

Figure 4.9 is an edge view of the lug showing the thickness and a description of the layers encountered as

SUMMARY OF OBSERVATIONS PERTAINING TO ANCHORAGE EXCAVATIONS

OBSERVATION / SITE OF EXCAVATION	CAMP STRAKE ROAD O/P ON IH 45 MONTGOMERY COUNTY	MONROE STREET O/P ON IH 45 HARRIS COUNTY	LEE ROAD O/P ON US 59 HARRIS COUNTY
1. Basic soil type	Sandy soil with some sandy clay	Expansive clay	Silty clay and clay
2. P.I. range for soil	5 - 10	40 - 50	25 - 35
3. Date of excavation	May 27, 1963	May 28, 1963	September 16, 1963
4. Date anchorage installed	January 1959	March 1959	April 1960
5. Age of anchorage	Four years, four months	Four years, two months	Three years, five months
6. Soil moisture conditions during excavation	Moisture near optimum; free water present	In excess of optimum	Dry
7. Weather conditions during excavation	Sunny, 90° F. (+)	Sunny, 90° F. (+)	Cloudy and showers, temperature 70-85° F.
8. Approach pavement type	10" concrete with 15' c-c joint spacing	10" concrete with 15' c-c joint spacing	10" concrete with 15' c-c joint spacing
9. Subbase type	Iron ore	Sand shell	Sand shell
10. Construction joint at top of lug	No, intergral placement	Yes, but good bond	No, intergral placement
11. Movement experienced	In excess of 3 1/2 inches	Approximately 3 inches	Less than 1/2 inch
12. Structural failure of concrete	No	No	No
13. Evidence of lug bending or tilting	No.	No.	No
14. Distance from header wall to first lug.	70 feet	35 feet	60 feet

Table 4.1

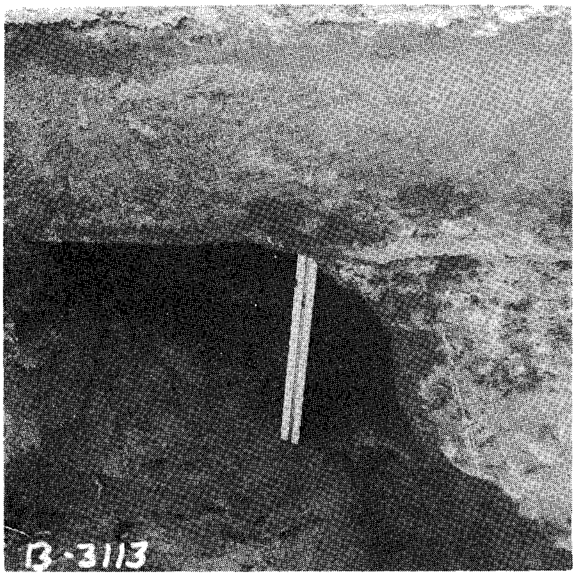


FIGURE 4.6 - Void behind anchor lug due to movement of the anchor system. The anchor slab is at the top of the picture and the lug is to the right. Camp Strake Road Overpass.

FIGURE 4.7 - Edge view of lug and anchor slab looking into the trench. Camp Strake Road Overpass.

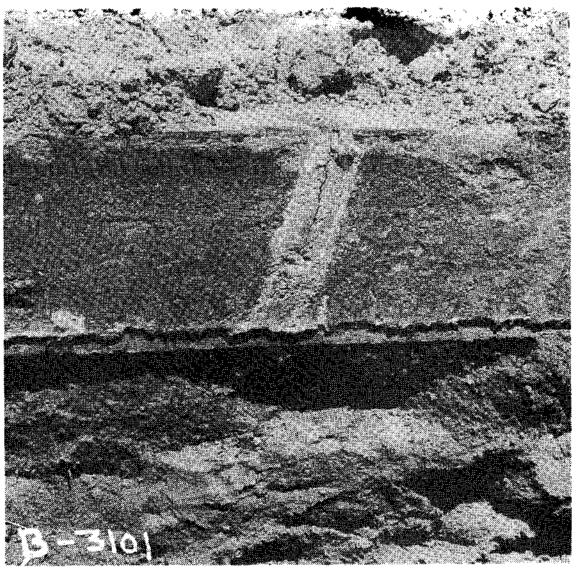
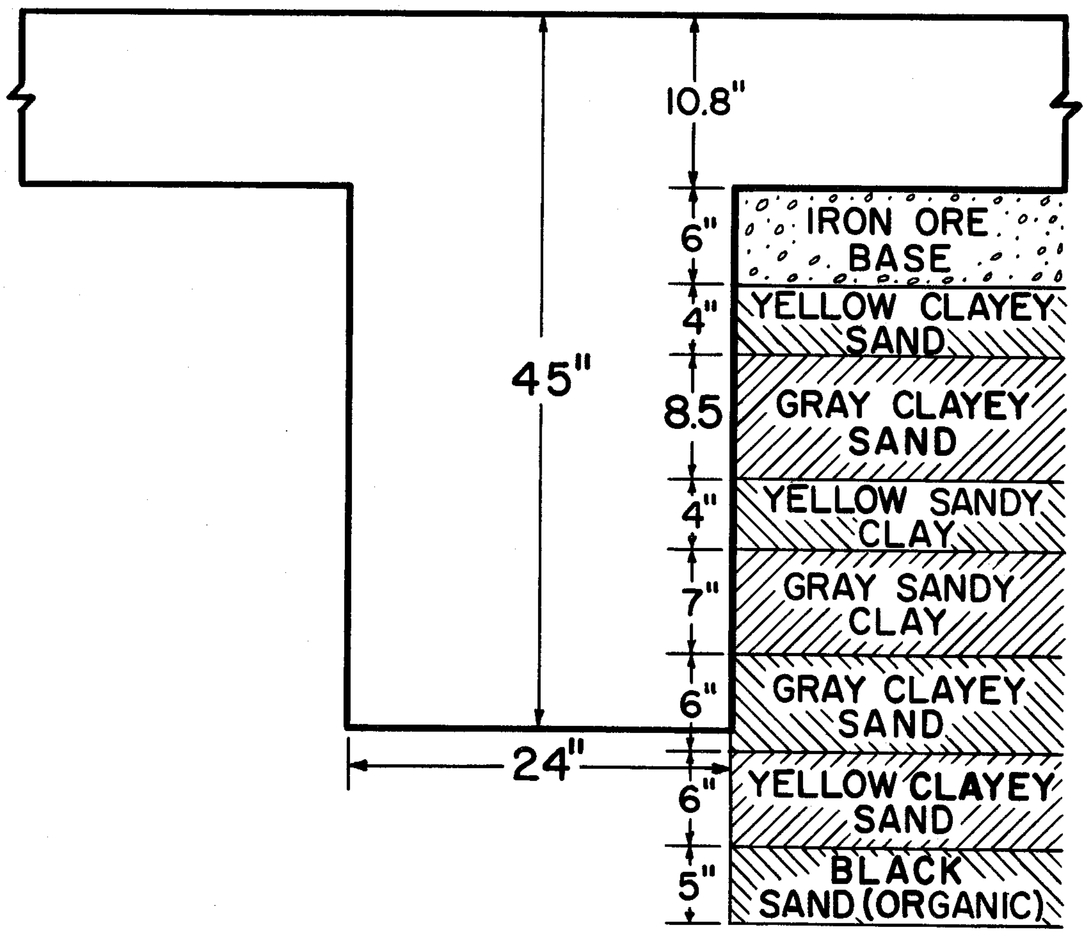


FIGURE 4.8 - Edge view of a transverse crack in the anchor slab. Camp Strake Road Overpass.



EDGE VIEW OF LUG SHOWING THICKNESS AND DESCRIPTION OF THE SOIL LAYERS ENCOUNTERED IN SAND FILL AT CAMP STRAKE ROAD O/P

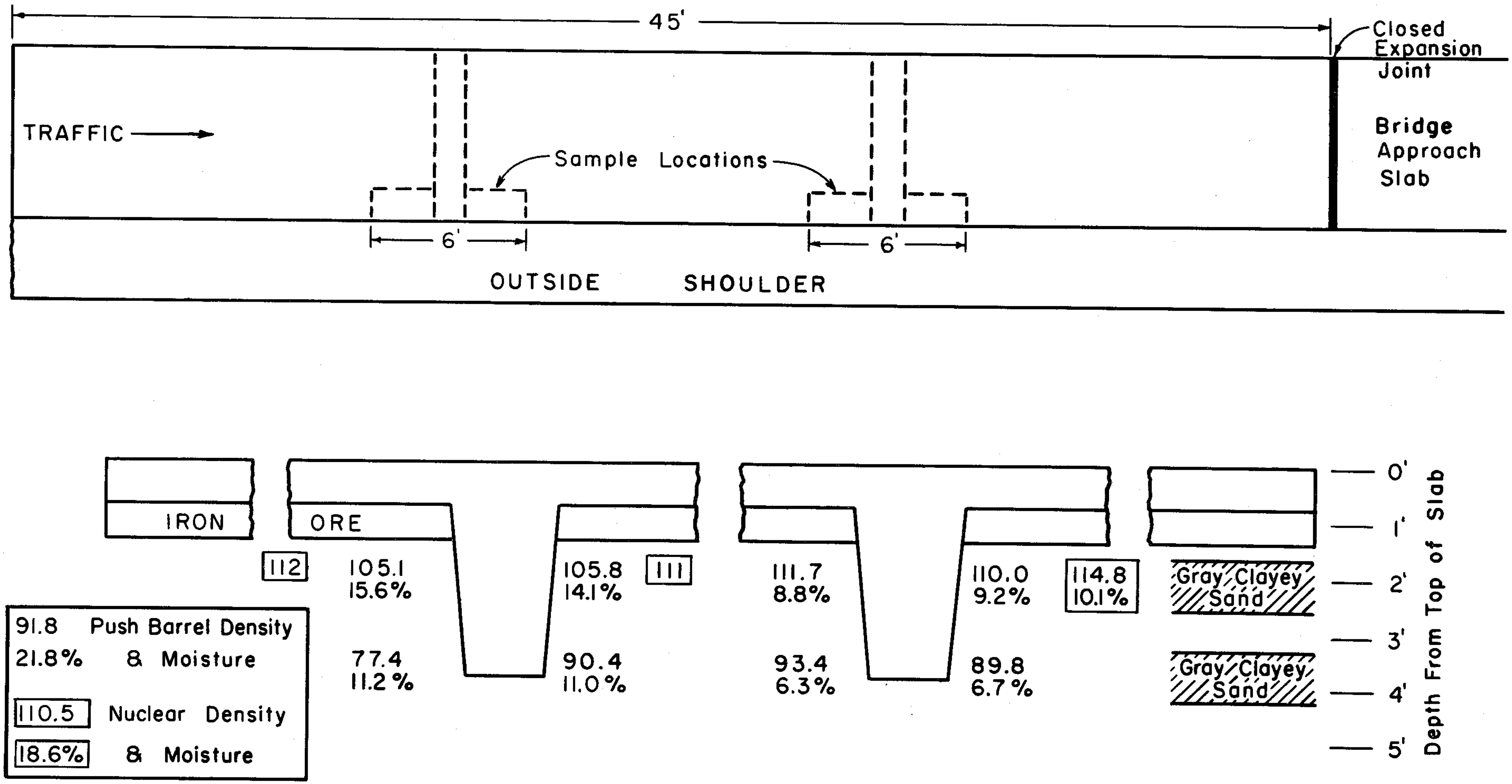
Fig. 4.9

well as the physical dimensions of the anchor system. Figure 4.10 portrays the dry density and moisture contents of the soil at various locations along the face of the excavation that were obtained by both the nuclear method and push-barrel sampling. In the case of both lugs, the density on the up-side and down-side of the lug were approximately equal, although the densities nearer the structure were greater relative to those fartherest away from the structure. The converse is true of moisture content, in that the upper side is less than the lower side. For any given layer, the wet densities are approximately equal which conjects the possibility of the soil moving as a block.

Considering the densities from a vertical standpoint, there is a sharp reduction in density progressing downward from the top of the slab. A slight reduction in density would be expected as a result of normal construction operations, but not to the degree experienced here. As a result of the low densities at the bottom of the lug, which is an area of high stress concentration, a shear failure occurred along this plane in all probability. The relative looseness of this layer would obscure any definite slippage plane.

The mechanical analysis and soil constants for each layer may be found in Appendix B. The triaxial test results of the layer at the bottom of the lug may also be found in Table B.2 of Appendix B. These triaxial tests were run at the optimum moisture and density since the specimens could not be molded at the field density. Hence, the cohesion value of one psi and the angle of internal friction of 40° is considerably higher than field values.

<u>Clay Fill (Site #2)</u>. This excavation was conducted in a manner similar to the preceeding, and the general observations may be found in Column 3 of Table 4.1. As was the case with the earlier excavation, the removal operation disclosed that there was an excessive amount of moisture present in each of the layers and various amounts of free water. The median on this project was a raised, free draining, sand-shell material without an impermeable membrane on top. Hence, the median and the sand shell subbase served as an aqueduct for bringing water into the area of the lug and distributing it across the width of the slab.



COMPARATIVE MOISTURE AND DRY DENSITY VALUES NEAR ANCHOR LUGS CAMP STRAKE ROAD O/P

Fig. 4.10

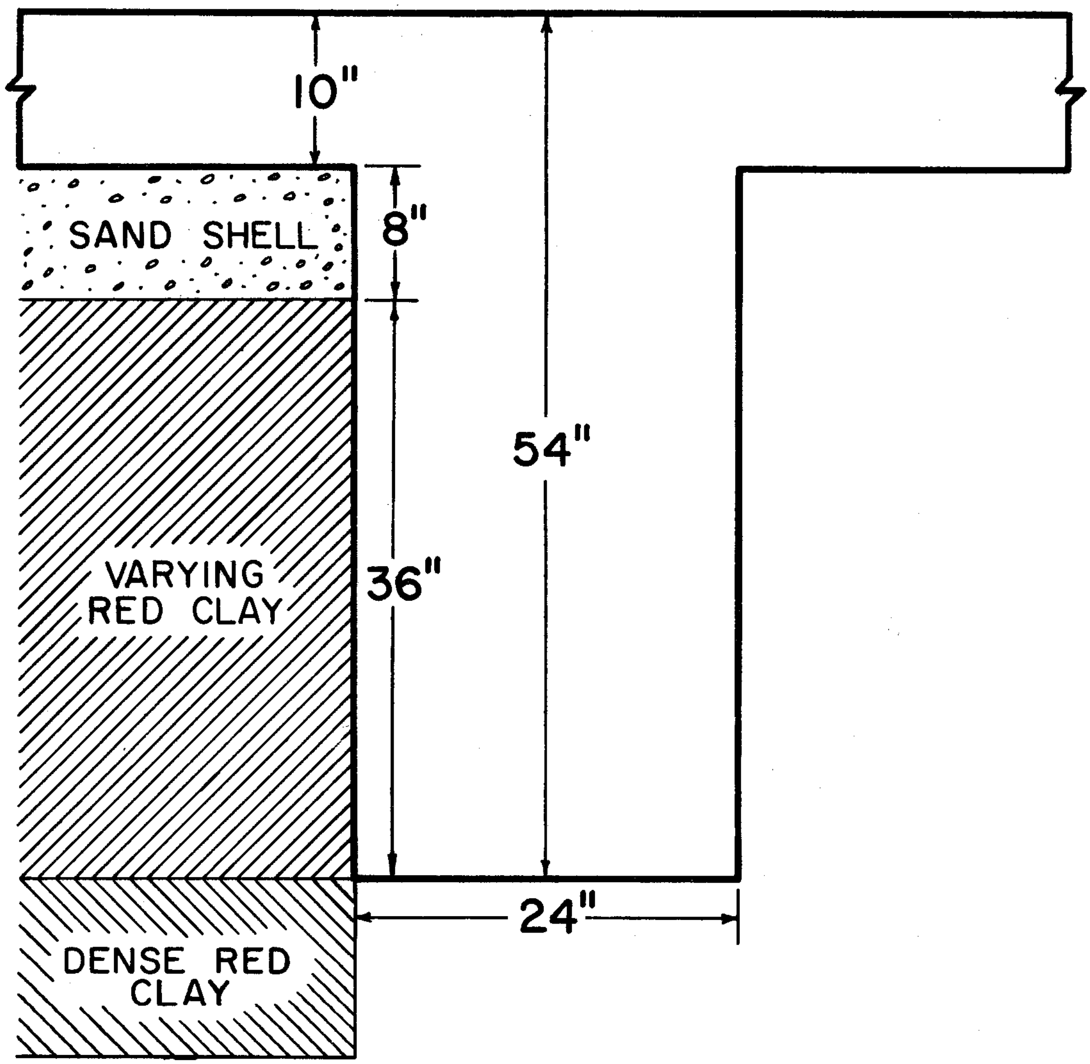
Measurements indicated the anchor slab had moved approximately $2\frac{1}{2}$ inches toward the structure, but there was not a definite void behind the lug as was the case previously. The highly expansive clay had expanded into the void although the boundaries could be detected. The anchor slab was structurally intact, and the lugs did not show any indication of excessive bending. The bottom of the lugs coincided exactly with the interface of two distinct layers as may be seen in Figure 4.11 which shows the thickness and description of layers encountered during the excavation.

The results of the density and moisture test are shown on Figure 12. Although the measurements obtained with the nuclear device have not proven to be completely reliable for clay materials, they may be used as a relative measure. In contrast to the previous excavation, there appears to be some consolidation of material on the side towards the structure since the densities are higher on the up-side of the lug. The results of the unconfined compressive tests shows that the maximum strength is less than 14 psi and in most cases less than eight psi for the range of moisture contents present (see Figure B.1 in Appendix B). This gives shear values of four to seven psi which are relatively low.

The field observations along with soil tests evince the possibility that the anchor slab and lugs slid as a block along the interface between the varying red clay and the dense red clay. Although a clear shear plane could not be readily discerned, the slickened face of the dense red clay lends credence to this deduction.

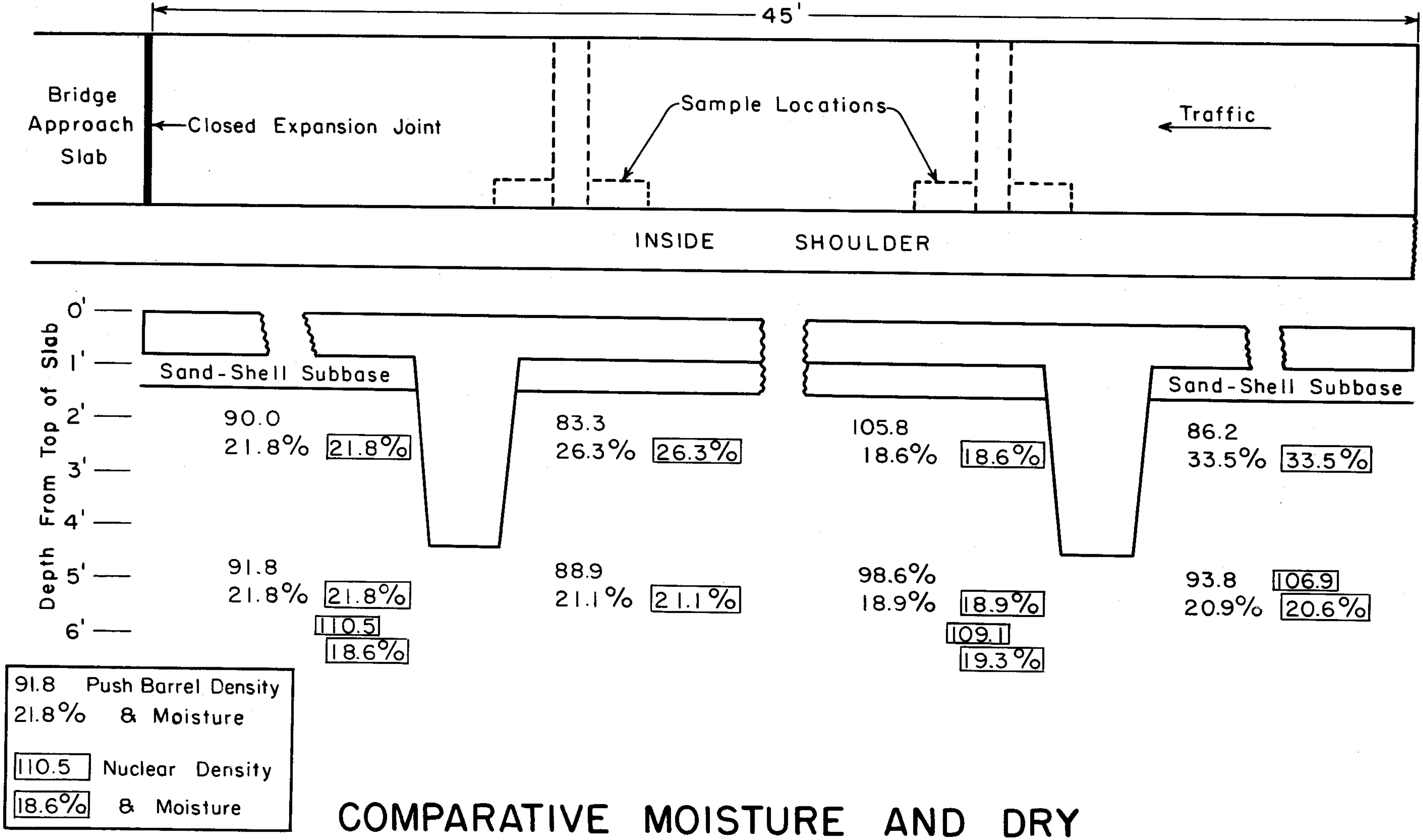
Excavation of Control Site (Site #3)

The general observations made in connection with the excavation of the control site are outlined in Table 4.1. Observations in the vicinity of the lug during excavation indicated that only a minute longitudinal movement had occurred at this location. The face of the lugs were against a tan-gray clay, and the bottom of each lug was in a tough silty clay (see Figure 4.13). The silty clay material was hard and quite difficult to dig even with the mechanical equipment. This was borne out by the density test which showed this material to have a considerably higher density than found in connection with the previous excavations (see Figure 4.14). It was readily apparent that



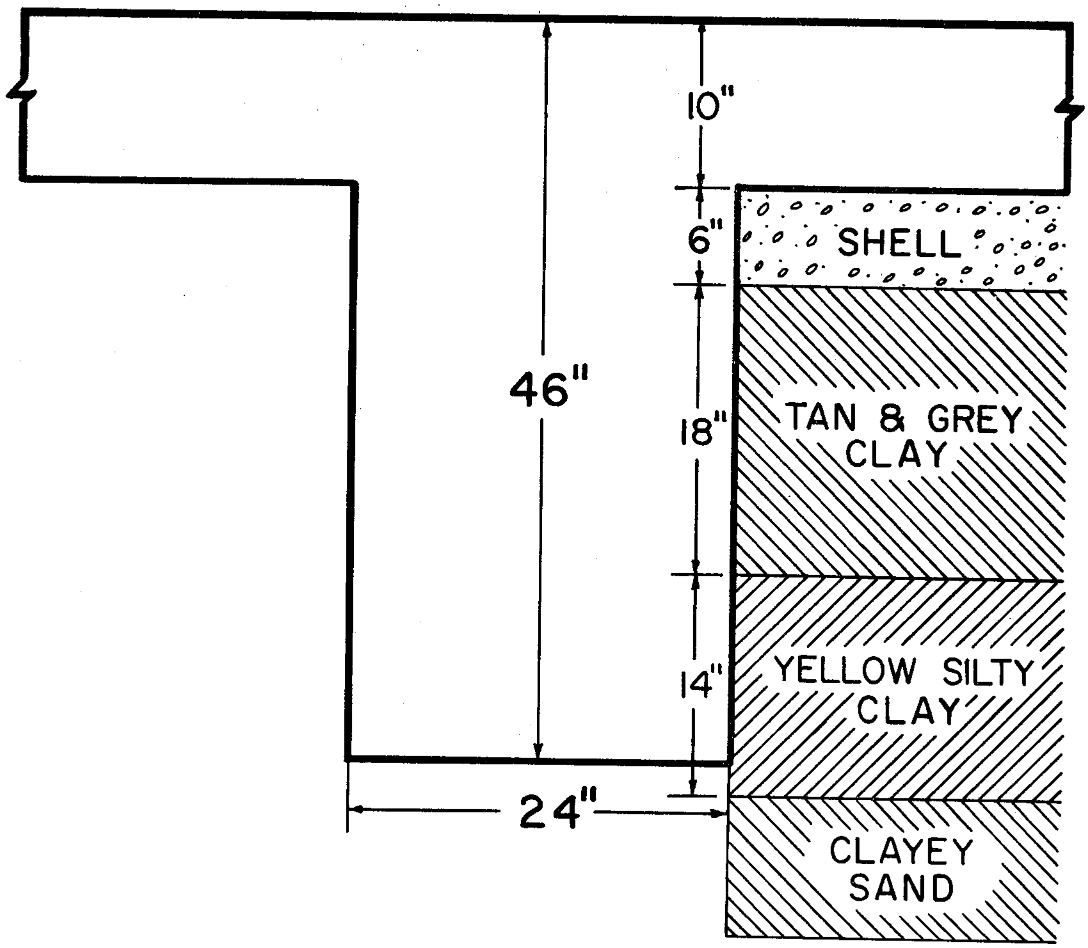
EDGE VIEW OF LUG SHOWING THICKNESS AND DESCRIPTION OF THE SOIL LAYERS ENCOUNTERED IN CLAY FILL AT MONROE ROAD O/P

Fig. 4.11



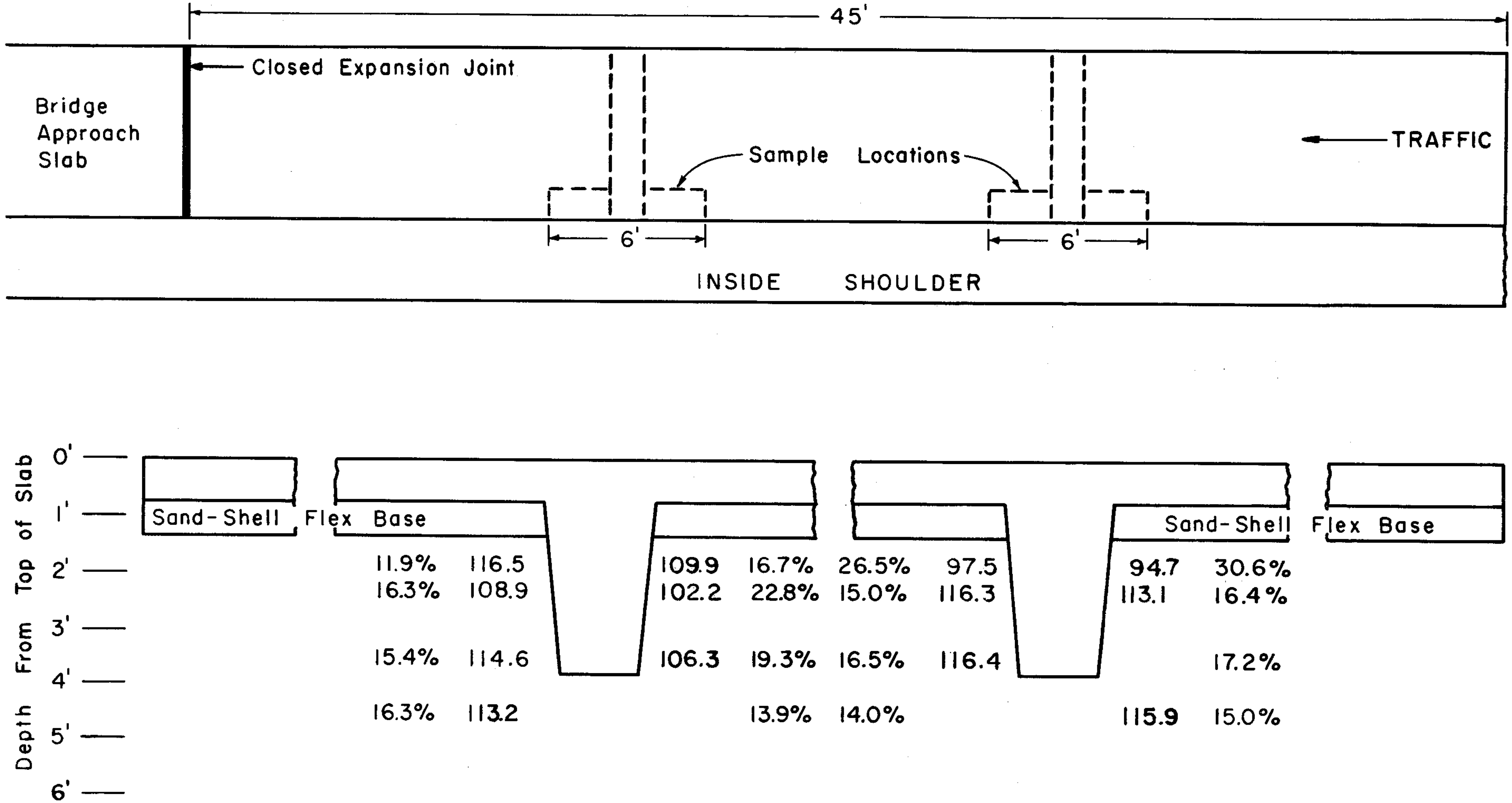
COMPARATIVE MOISTURE AND DRY DENSITY VALUES NEAR ANCHOR LUGS MONROE ROAD O/P

Fig. 4.12



EDGE VIEW OF LUG SHOWING THICKNESS AND DESCRIPTION OF THE SOIL LAYERS ENCOUNTERED IN CONTROL EXCAVATION LEE ROAD O/P

Fig. 4.13



COMPARATIVE MOISTURE AND DRY DENSITY VALUES NEAR ANCHOR LUGS LEE ROAD O/P

FIGURE 4.14

116.5 Push Barrel Density
11.9 % & Moisture

the material in this embankment was considerably dryer than that removed from the other two locations. This was borne out by the moisture test as shown in Figure 4.14; with the exception of two high readings, the moisture contents were less than 17%. The higher densities and lower moisture-contents resulted in higher shear values. Laboratory tests showed the soil shear strength to be in excess of six psi.

The densities on the side of the lug toward the structure were slightly greater for each layer than on the down-side of the lug. This slight differential indicates a small amount of consolidation has occurred, but as indicated the movement is small. A certain amount of deformation is required in order to develop the strength potential of the soil.

V. Discussion of Results

During the preceeding chapter, the observations made in connection with the data survey and the excavation operations were presented. The next step is to fuse these observations so that the primary cause of failure may be defined. After this step, the secondary factors associated with the failure may be discussed. In addition to the failure rationale, the forces on the anchorage system may be roughly estimated.

Primary Rationale of Failure

The excavation of the in-service anchor systems revealed that the design of the concrete structural members was adequate, and the failure may be attributed to the mode of transferring the expansive forces into the soil mass. During the early development of the anchor system, the horizontal shear plane at the bottom of the lugs was recognized as a critical part of the design and the excavation indicated a soil shear failure occurred along this horizontal plane. This statement is based on observations during excavation as well as an analysis of the data. The failure at site #1 was unquestionable due to a soil shear at the bottom of the lugs, but this deduction at site #2 is questionable on first analysis. There is a considerable increase in soil density on the bearing face of the lug in comparison to that behind the back face. Comparing the density data from site #2 with site #3, which is performing satisfactorily, it is noted that the same relative decrease in soil density is experienced between the front and back of the lug. Since the anchor slab at site #2 moved approximately three inches, whereas, the anchor slab at site #3 moved less than ½ of an inch, the differential densities are not due to a bearing failure. This analysis along with visual observations lends evidence to a soil-shear failure at site #2.

An inadvertant decision was made in placing the bottom of the lugs at the same depth in that a critical part of the design was made dependant upon one layer of soil. The data survey found the majority of the failures to be in embankments or fills. The selected magnitude for extending the lug into the embankment was in essence a factor that compounded the primary cause of failure. In normal construction operations, the density as well as the quality of material improves towards the top of the embankment. Hence, the depth of the lug placed the critical part of the design in a relatively weaker layer.

The absence of any large magnitude of rotating or bending of the structural members showed the design of the lug members was completely adequate from a rigidity standpoint. In no case could the present policy of permitting construction joints at the top of the lug be associated with failure or poor performance as was the case in other studies.[4]

Secondary Rationale of Failure

Time is a factor influencing failure in that it generally takes about three years for visual evidences of the failure to be manifested, i.e. ruptured abutment walls, closed joints, etc. Using Moyer's data as to pavement growth this would be the approximate time required to close an expansion joint 1½ inches wide.[5] Although a three year period is required for the deterioration to manifest itself, the absence of cracks in the slabs experiencing failure indicates the deficiency was present during the first year. If the anchor slab is operating as a fixed member as intended in design, volume change cracks will occur in the slab during the first year due to temperature variations.

The use of siliceous coarse aggregates appears to be a secondary factor since all the failures occur where this aggregate type was used. The absence of failures where oyster shell was used as the coarse aggregate (due to a lower thermal coefficient) demonstrates the feasibility of using terminal anchorages to prevent pavement growth. In other words, a satisfactorily performing anchor system can be developed.

Another secondary factor that may be associated with the soil failure is the need for an adequate distance between the last lug and the header wall to develop the necessary resistance in shear forces. For the soils in the Harris County area, it was found that a minimum of 60 feet is required for this distance in order to obtain satisfactory performance of the anchorage system.

Estimation of Pavement Growth Forces

Analytical calculations of the failure stresses involved in the terminal anchorage system would require an elaborate mathematical model that simulates the interaction between the anchor slab system and the soil mass such as proposed by Matlock and Reese.[6] This type of approach is beyond the scope of this paper, but a rough estimate of forces involved would be of benefit for future designs. By making the following assumptions an estimate of the forces can be made:

(1) The anchor slab system acts as a fixed unit, i.e. the internal compressive movements (stress-strain) due to the pavement growth forces are not accounted for.

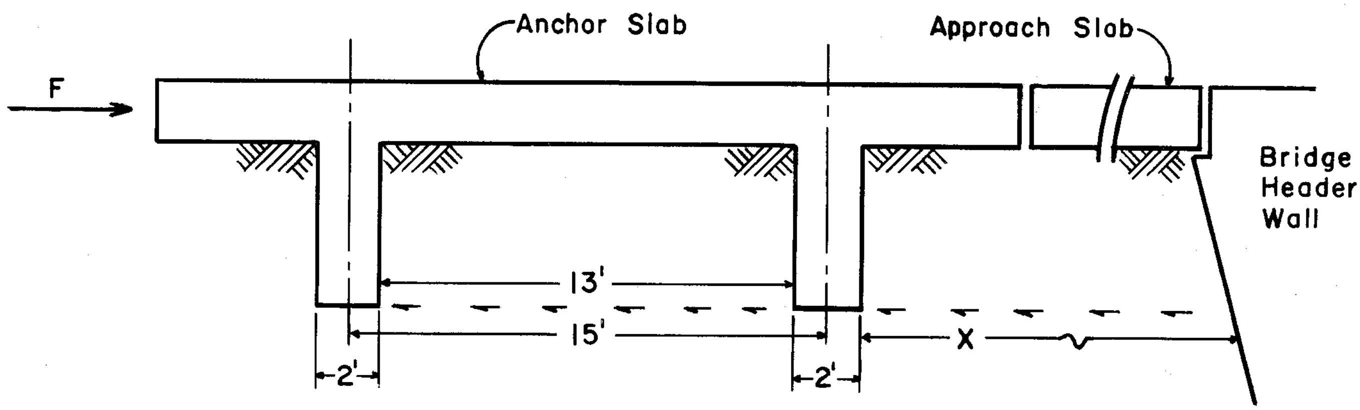
(2) After the pavement growth forces develop, the failure period is of a short duration.

(3) An average distribution of the shear stresses along the horizontal plane at the bottom of the lug is used in lieu of the more probable parabolic distribution with the maximum stress adjacent to the back lug.

Using the soil strength characteristics as outlined in Appendix B, the forces per foot of pavement width present on the three excavated systems were estimated as shown in Table 5.1. The first fact evident from the table is that the Lee Road Overpass which was a satisfactorily performing anchorage system was capable of developing a resistance force of 67,000 pounds per foot of width, whereas the two anchor systems experiencing failure did not exceed 46,000 pounds per foot width. The soil strength characteristics in the Monroe Street Overpass were adequate relative to the Lee Road Overpass, but due to the short distance between the lug and the header wall, the developed resistance forces were insufficient. In contrast, the distance on the Camp Strake Road Overpass was great enough, but the soil strength was completely inadequate.

TABLE 5.1

DETERMINATION OF FORCES DEVELOPED IN ANCHOR SYSTEMS WITH FAILURE CONDITION



Location	Shear or Cohesion (psi)	Effective Resistance (psi)	X (ft.)	15 + X (ft.)	(15+X)(144) (in^2/ft.)	F (Available Resistance) #/ft. of Width
Camp Strake	1 at 40° ∢	3.6	75	90	12,960	46,267
Monroe	6	6.0	35	50	7,200	43,200
Lee Road	6.2	6.2	60	75	10,800	66,960

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6. Matlock, Hudson, and Reese, L. C. Generalized Solutions for Laterally Loaded Piles. Journal of Soil Mechanics and Foundations Division, Proceedings, A.S.C.E., 86:SM5, October 1960.

7. Volume 1, Texas Highway Department, Manual of Testing Procedures.

A P P E N D I X   A

TERMINAL CONDITION DATA

FOR: IH-10 - Harris County Structure 1-1 (Thompson Rd. Overpass)

DATE: 6-25-63

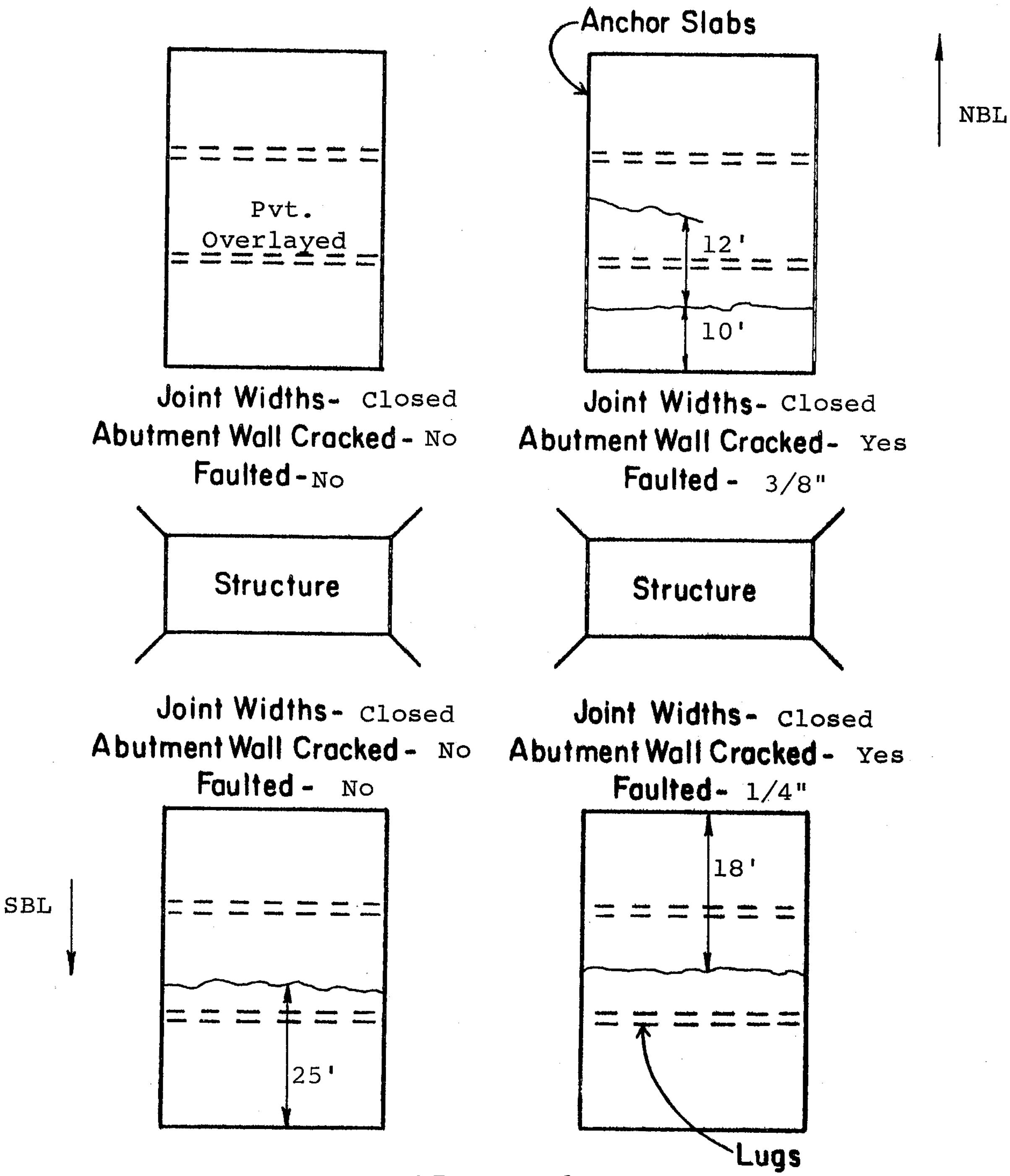


FIGURE A.1

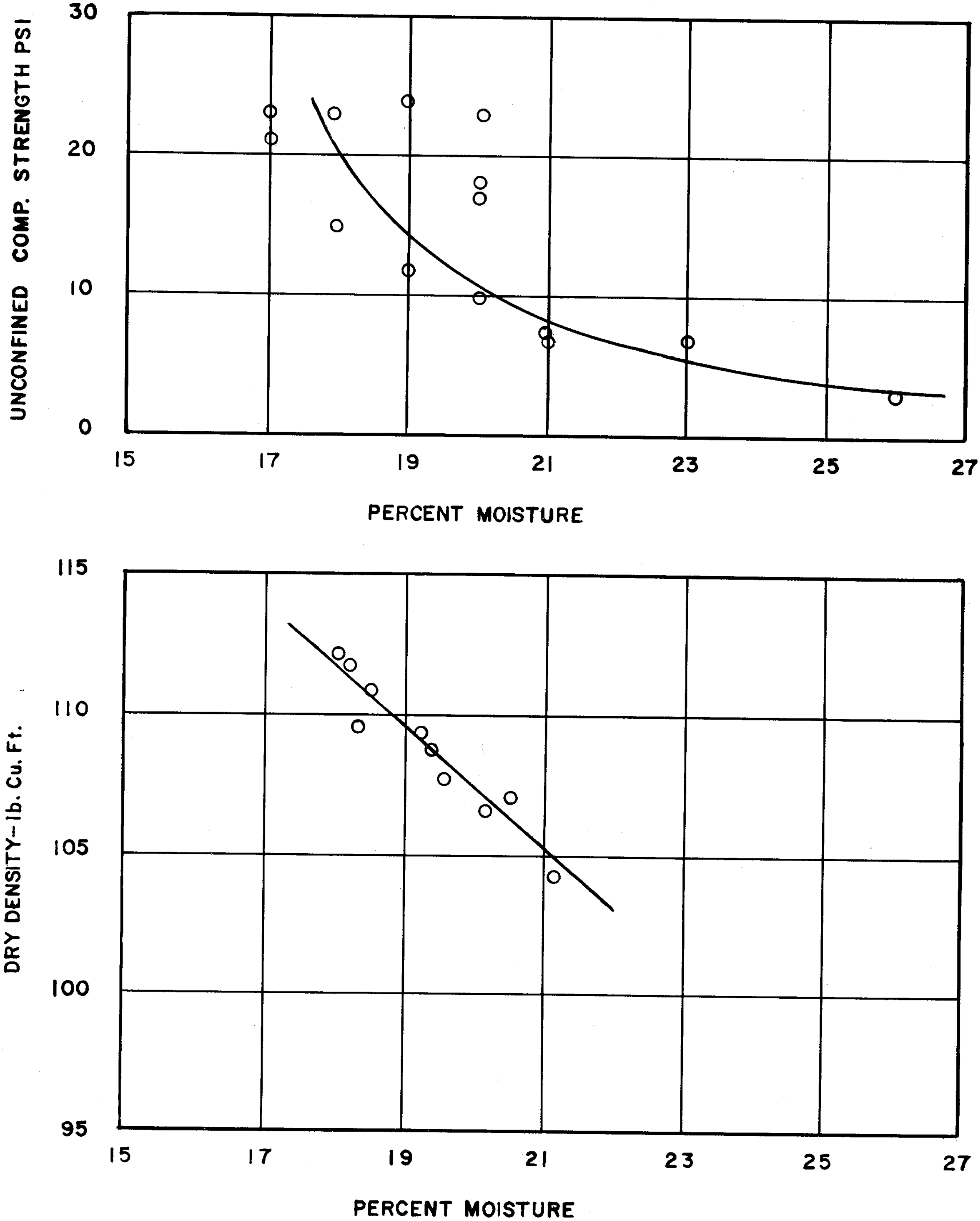
NOTES:

1. Structure in NBL is a failure.
2. Structure in SBL is questionable because joints are closed but no faulting or cracking has occurred.

	REFERENCE NUMBER						
	1-1	1-2	1-3	1-4	1-5	1-6	1-7
A. Physical Features 1. Const. Jt. @ top of lug	No	No	No	No	No	No	No
2. Pavement Thickness	12"	12"	12"	12"	12"	12"	12"
3. Grade	+3.5%	None	+3.5%	+3.5%	+3.5%	+3.5%	None
B. Concrete Properties 1. Coarse Aggregate	siliceous river gravel	siliceous river gravel	siliceous river gravel	siliceous river gravel	siliceous river gravel	siliceous river gravel	siliceous river gravel
2. Cement Factor	5 sack	5 sack	5 sack	5 sack	5 sack	5 sack	5 sack
C. Placement Conditions 1. Time of construction	3-59	4-59	6-59	1-60	3-59	3-59	4-59
2. Temperature							
D. Subbase Material 1. Type	4" Flex.Base	4" Flex.Base	4" Flex.Base	4" Flex.Base	4" Flex.Base	4" Flex.Base	4" Flex.Base
2. Stabilized	No	No	No	No	No	No	No
E. Unusual Conditions 1. Skew	22° 33'	35°	3° 41'	3° 41'	None	10°	27°
2. Fill Material							
3. Pavement	15' CMJ	15' CMJ	15' CMJ	15' CMJ	15' CMJ	15' CMJ	15' CMJ
4. Distance From first lug to header wall	60'	60'	60'	60'	60'	60'	60'
5. No. of lugs	2	2	2	2	2	2	2

FIGURE A.2

A P P E N D X   B



UNCONFINED COMPRESSIVE STRENGTHS AND DRY DENSITIES FOR CLAY SOIL FROM THE MONROE ROAD OVERPASS.

FIGURE B.1

TABLE B.1
SOIL CONSTANTS AND MECHANICAL ANALYSIS
FOR SOIL LAYERS AT CAMP STRAKE OVERPASS

		1	2	3	4
		Gray Clayey Sand at Depth of 2 Ft.	Gray Clayey Sand at Depth of 4 Ft.	Yellow Clayey Sand	Black (Organic) Sand
Soil Constants	Liquid Limit	20	18	26	20
	Plastic Limit	15	14	16	14
	Plasticity Index	5	4	10	6
	Linear Shrinkage	3%	0%	0%	1.5%
	Shrinkage Limit	14.8	17.6	16.9	18.9
	Shrinkage Ratio	1.74	1.74	1.74	1.70
Mechanical Analysis	Clay	7.4%	4.4%	11.0%	-
	Silt	12.8%	8.7%	12.4%	6.8%
	Fine Sand	53.7%	67.8%	63.2%	87.8%
	Coarse Sand	25.4%	17.1%	12.7%	5.4%
	Coarse Aggregate	0.7%	2.0%	0.7%	-

Note:

Soils tested in accordance with Test Methods Tex 100-E, 101-E, 103-E, 104-E, 105-E, 106-E, 107-E, and 110-E of the Texas Highway Department's Manual of Testing Procedures.

TABLE B.2

TRIAXIAL TESTS FOR MATERIAL FROM CAMP STRAKE OVERPASS

Location: Camp Strake Overpass

Material: Gray Clayey Sand from Depth of Four Feet

Moisture Density Data: Tests run in accordance with Test Method Tex 113-E.[7]

Optimum Wet Density = 120 pcf; Opt. Moisture = 14.2%

Triaxial Tests: The specimens were tested in accordance with Test Method Tex 117-E.[7]

The test specimens could not be molded at the field densities; therefore, approximate optimum conditions were used.

Specimens for Mohr's Diagram

% Moisture	Dry Density pcf	Lateral Pressure psi	Alt. Comp. psi
13.8	108.6	0	2.1
13.4	108.9	5	31.7
14.0	108.9	15	81.6

Strength Values: Cohesion of one psi at an angle of internal friction of 40°.