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EMBANKMENT SLOPE FAILURE PROBLEMS IN DISTRICT 12

Prepared by

State Department of Highways and Public Transportation

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SUMMARY OF
EMBANKMENT SLOPE FAILURE PROBLEMS IN DISTRICT 12

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Frank Y. Wadlington

Numerous embankment slope failures occurring in District 12 have prompted a need for reviewing the current design criteria, and remedial actions taken, as a means of solving the problem of embankment failures. The recent embankment failure locations are illustrated in attached Exhibit A along with each location's original design details, construction dates and restorative measures. Approximately sixty major structures are currently in the design stage in District 12, requiring reliable and feasible design criteria for their embankments.

As a result of this need, District 12 has been gathering information and doing research to determine the cause for long-term embankment slope failures. On February 17, 1983, a meeting was held in our District Office to bring together engineers from the Department, FHWA and State Universities to discuss slope failures and possible remedies. An account of this meeting is attached as Exhibit B, and summarized as follows.

The embankment design criteria for District 12 has been to limit embankment height to 20 feet, side slope maximum ratios to a 3:1, and borrow material to a maximum liquid limit of 65. Even with these restrictions, sliding has occured in 16 ft. high embankments with liquid limits of less than 55, and with 3:1 side slope ratios. The State's current policy on embankment safety factors is not specific, but recommends a range between 1.5 and 2.0.

One of the first items discussed was the most common cause for embankment failure, the material's loss of shear strength from excessive wetness. Preventative measures were suggested by Mr. Andy Munoz, Jr. of the FHWA in Fort Worth for this type of failure and are as follows:

1) Placement of a granular blanket on the embankment slope to help drain the water and to give additional support to the overburden.
2) Stabilizing the embankment slope with lime/fly ash.
3) Reinforcing the slope with fabric, etc.

In addition, current construction practices could be refined to produce more stable embankments by adopting the following measures:

1) Provide adequate drainage of surface water at both the head and toe of the slope to prevent saturation of the embankment slope.
2) Specify embankment material, preferably granular material.
3) Flatten the slope angle.
4) Lime stabilize the slope.
5) Use of fabric materials and reinforced earth slopes.

Selection of these measures, singly or in combination, should be considered for their cost-effectiveness and should depend on individual project conditions. According to Mr. Munoz, the correlation of safety factor to field performance is a function of the accuracy of the soils data input and methods used. The critical height of the embankment should be a function of the sub-soil stability and not a function of slope. This, in practice, will require more engineering work for each embankment.

As a means of providing embankment stability as well as a remedial measure for failing slopes, the use of a plastic grid developed by Gulf Oil Corporation was presented by Mr. Clinton Bond of Beaumont. Used as a corrective device, the grid appeared to be an economical way to increase embankment stability. Mowers had no effect on the plastic grid and although the demonstration site had a surface erosion problem, it was due to vegetation inadequacies and not the use of the material.

Dr. Stephen Wright, Associate Professor of Civil Engineering at the University of Texas, divided embankment failures into two separate categories referred to as short-term stability and long-term stability. Most foundation or sub-soil failures will be short-term failures due to low cohesive soil strengths. The majority of slides in District 12, however, concern long-term stability where the soil has had adequate opportunity for water to flow through it. The time required for such an occurrence is dependent upon the availability of water, drainage, and the nature of the soil. Dr. Wright summarized by explaining that in a long-term stability problem, the stability analysis will show that the friction angle is dominate and the cohesion value is reduced to zero. Since the long-term strength of the material is dependent upon the angle of internal friction of the soil, Dr. Wright concluded that the factor of safety is influenced primarily by the embankment slope.

Mr. Bob Hauck introduced the subject of slope protection and discussed various slope protection treatments used by the Houston Urban Office. Surface erosion continued to occur with the use of plastic fabric, and where a nylon mesh fabric was tested, routine mowing destroyed the fabric material. Lime treatment of soil has worked well, but the lime acted adversely against the vegetation used for cover. Mr. Hauck concluded that the most successful slope protection is the placement of concrete riprap at a slope of 3½ to 4:1. Riprap, however, is primarily a slope erosion protection technique and does little as far as stability is concerned.

As a further means of resolving the prevalent embankment problems in District 12, a research study was initiated through an interagency contract between the University of Texas Center for Transportation Research and District 12 of the Texas SDHPT. A report dated November 1, 1983, was prepared by Dr. Wright and is included as Exhibit C. This research was conducted on tests of fill material sampled from two embankment slide locations in Harris County. Classification, compaction and strength testing, and slope stability analyses were performed on the selected material.

Stability calculations revealed that the slopes were stable as all factors of safety exceeded 1.25. To gain insight into why such high factors of safety were obtained for slopes which failed, shear strengths were back-calculated using the available knowledge of the slope and slide geometrics at the two sites. When these calculations were compared with the actual measured effective stress cohesion values, the back-calculated values were almost an order of magnitude smaller, while the effective stress friction angles revealed close similarities. This discrepancy was not believed due to laboratory testing errors, and at the conclusion of the study was still unresolved. However, it was evident that the relatively high effective stress cohesion values derived from the laboratory tests do not apply to the field. Strong evidence from slope failure sites indicates that there is a negligible effective stress cohesion component of shear strength in the field. Regardless of future research, Dr. Wright recommends immediate changes in design practice for earth embankments constructed of highly plastic clays in District 12 and suggests that embankment slopes not exceed 4:1 as an interim solution.

In conclusion, District 12 does have a substantial problem with long-term embankment failures. Various methods have been presented to increase embankment stability and to prevent recurring slides. Some have been tested and found inadequate. Others, tested too recently, have not had the opportunity to be considered for their long-term effects.

There was a recommendation that each project's design be based on existing subsurface conditions and specific characteristics of the embankment fill material. However, this may prove unfeasible and uneconomical for District 12 because evidence in the majority of embankment failures in the past 20 years in the Houston area indicated that it was not the problem of the subsurface conditions, but was the steepness of the slopes.

On most of our embankment construction projects, the location of the source where the embankment material will originate from is not known, so material properties are hard to determine for design purposes. Using more stringent material specifications to compensate for the inability to design embankments based on obtainable fill material properties is not feasible due to the unavailability of this material within the District.

Also to be considered are the results of the calculations and material testing conducted by Dr. Wright at existing embankment failure locations. Regardless of what actually caused these failures, Dr. Wright believes future slope failures can be significantly reduced by using 4:1 side slopes.

There are many questions that arise while attempting to solve the embankment problem for District 12; however, an immediate answer is essential for current design projects. The one fact that is very evident is that we are experiencing slope failures in embankments with 3:1 slope ratios even though soil analysis indicated safety factors greater than 1.25. Until more research can be completed to provide conclusive evidence of the cause of these failures, District 12 recommends the use of the following criteria as an interim measure on all design stage projects to reduce the possiblity of long-term failures of these future embankment slopes:

1) Embankment heights are to be designed within a range of 16 to 20 feet if possible.
2) End and side slopes are to have a maximum slope ratio of 4:1.
3) Slope stability analysis is to be performed on all embankments over 16 feet in height.
4) Proposed bridge on roadway embankments with heights of 20 feet or more are to be submitted to the District Engineer or District Design Engineer for approval.
5) Minor variations from the above criteria are to be submitted to the District Designing Engineer for approval.

EXHIBIT A

Embankment Slope Failure Locations
In District 12

EMBANKMENT SLIDE LOCATIONS

1. IH 10 at Sjolander Road---------------NE and NW Corners
2. IH 10 at Gulf Plant-------------------NW, NE, and SE Corners
3. IH 10 at Garth Road-------------------NW and SE Corners
4. US 90A at Clinton Drive---------------SW and SE Corners
5. I 610 at IH 10-----------------------From I 610 SB to I 10 EB
6. I 610 at US 59-----------------------SE Corner
7. I 610 at M. L. King-------------------NE Corner
8. I 610 at Scott Street-----------------NE Corner
9. I 45 at FM 2351----------------------NE and SW Corners
10. I 45 at College Street----------------NE and SE Corners
11. I 610 at SH 225----------------------SE Corner
12. SH 225 at Scarborough-----------------SE Corner
13. SH 225 at Shell Overpass--------------SE Corner
14. SH 225 at SH 146---------------------Upper level - NW and SW Corners
15. SH 225 at SH 146---------------------Lower level - SE Corner
16. I 45 at West Road--------------------NE Corner (Repaired by Contract)
17. US 59 at FM 525----------------------NE Corner
18. I 45 at Gulf Bank--------------------NE Corner
19. I 45 at MPRR in Spring----------------Slides between main lanes North and South of tracks
20. I 45 at SH 146 (Texas City "Y")----------------------SE Quadrant

(2)

21. SH 146 at I 45
    (Texas City "Y")----------------------South Side

22. I 610 at Richmond Street--------------NW Corner

23. US 59 at Shepherd Street--------------SE Corner

24. IH 10 at Crosby-Lynchburg Rd.---------North Side

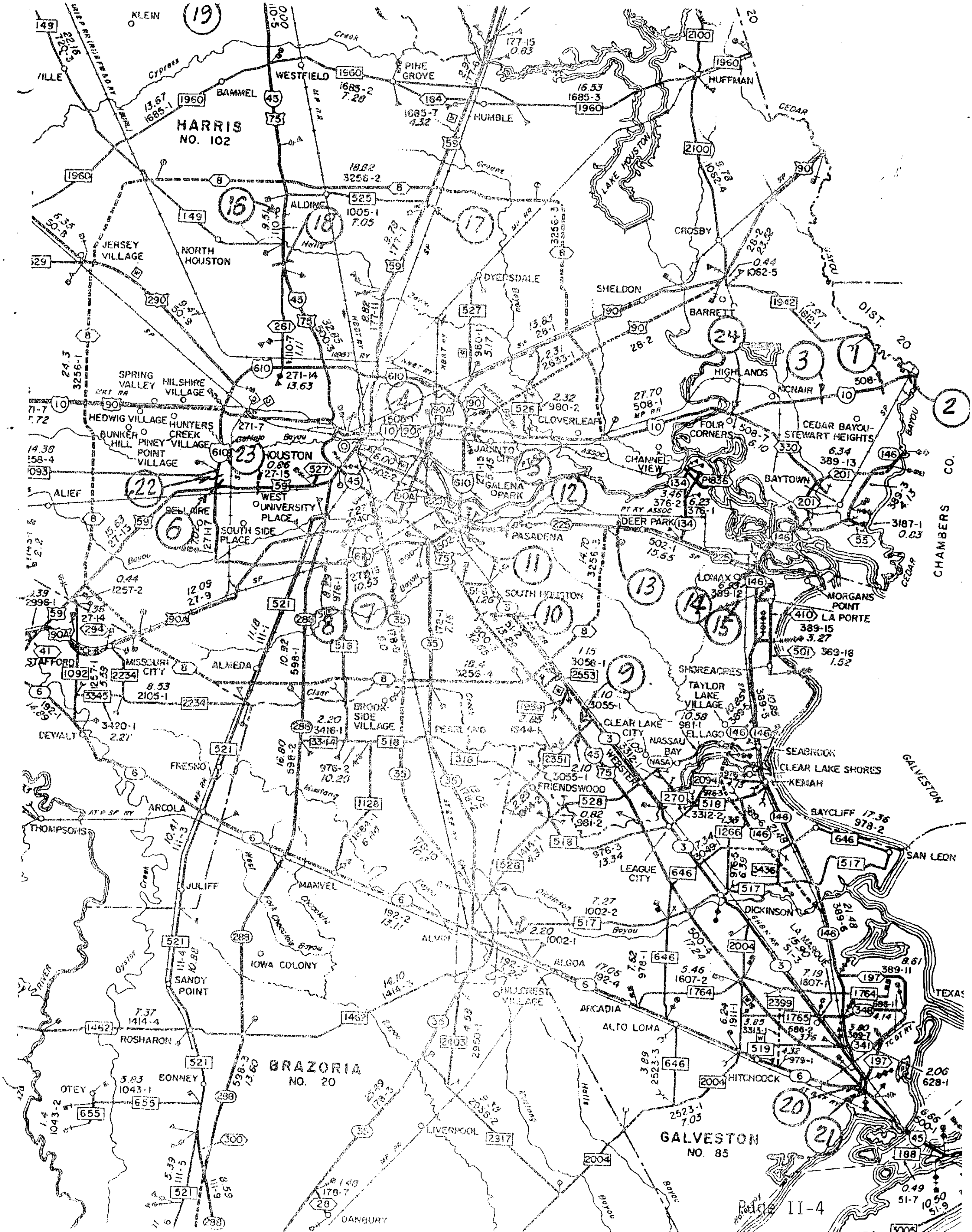
HARRIS
NO. 102
BRAZORIA
NO. 20
GALVESTON
NO. 85
CHAMBERS CO.
DIST. 20
KLEIN
WESTFIELD
PINE GROVE
HUMBLE
HUFFMAN
BAMMEL
LAKE HOUSTON
ALDINE
CROSBY
JERSEY VILLAGE
NORTH HOUSTON
DYERSDALE
SHELDON
BARRETT
HIGHLANDS
MCNAIR
SPRING VALLEY
HILSHIRE VILLAGE
HEDWIG VILLAGE
HUNTERS CREEK VILLAGE
BUNKER HILL
PINEY POINT VILLAGE
HOUSTON
CLOVERLEAF
FOUR CORNERS
CEDAR BAYOU-STEWART HEIGHTS
CHANNELVIEW
BAYTOWN
JACINTO CITY
GALENA PARK
ALIEF
BELLAIRE
WEST UNIVERSITY PLACE
SOUTH SIDE PLACE
PASADENA
DEER PARK
LOMAX
MORGANS POINT
LA PORTE
SOUTH HOUSTON
STAFFORD
MISSOURI CITY
ALMEDA
SHOREACRES
TAYLOR LAKE VILLAGE
EL LAGO
CLEAR LAKE CITY
NASSAU BAY
WEBSTER
SEABROOK
CLEAR LAKE SHORES
KEMAH
BROOKSIDE VILLAGE
PEARLAND
DEWALT
FRESNO
FRIENDSWOOD
BAYCLIFF
SAN LEON
ARCOLA
THOMPSONS
LEAGUE CITY
DICKINSON
LA MARQUE
JULIFF
MANVEL
ALVIN
IOWA COLONY
SANDY POINT
ALGOA
HILLCREST VILLAGE
ARCADIA
ALTO LOMA
TEXAS CITY
ROSHARON
HITCHCOCK
BONNEY
OTEY
LIVERPOOL
DANBURY

EXHIBIT B

Report on February 17, 1983 Meeting on
Roadway Embankments and Bridges

MEETING ON ROADWAY EMBANKMENTS AND BRIDGES

Place - District 12 Office - Houston

Date - February 17, 1983

Time - 9:00 a.m.

A. Welcome: Omer F. Poorman, District Engineer.

B. Discussion of Embankment Heights and Bridge Lengths.

C. Discussion of Embankment Slope Protection and Slide Repair Techniques.

D. Discussion of Bridge Piling Lengths.

E. Discussion of Rough Pavement at Bridge Ends.

F. People in Attendance.

G. Appendix A: Locations of Embankment Slope Failures in District 12.

H. Appendix B: Slope Stability Analysis of Embankments.

Section A

Mr. Poorman opened the meeting on embankment failures by welcoming all those in attendance and by having everyone introduce themselves. He then introduced the problem District 12 is having with embankment slides by giving various examples of failures and the associated cost for repair. Mr. Poorman then turned the meeting over to Mr. Wadlington.

Mr. Wadlington began his presentation by showing slides depicting various embankment slope failures within District 12. Not all the embankment slope failures in the District are major failures, but there are several extreme cases. One such example is the embankment slope failure located at IH 610 and Westpark. This failure has been repaired several times by various means and is at this time under contract for further repairs.

After presenting the slides, Mr. Wadlington discussed the present design criteria for District 12 concerning embankments and what the purpose is for this meeting. A synopsis of embankments was distributed to the attendees showing various embankments in District 12 (Appendix A). The present embankment criteria in District 12 is to limit the embankment height to 20' and the maximum side slope at 3:1. However, Mr. Wadlington brought up the fact that embankment failures have occured on 16' high embankments. He also stated that we have limited our borrow material for embankments to a liquid limit of 65, but we have some embankments with liquid limits of 55 that are sliding.

District 12 is in the process of designing approximately 60 major structures to be built in the near future, so a reliable and feasible design criteria for embankments is needed. A maximum side slope of 4:1 has been considered and where such a stipulation is impossible due to limited right of way, the use of retaining walls is required. After his presentation, Mr. Wadlington called on Mr. Michael Ho, Laboratory Engineer for District 12.

Mr. Ho discussed the lab analysis of four embankments within our District (Appendix B). Mike based talk on embankment stability, primarily on the factor of safety. The factor of safety of these four embankments ranged from an unstable 1.00 to a stable 1.75. Mr. Wadlington stated that with a factor of safety of 1.75 and higher, we are not having failures.* Mike agreed and stated that somewhere between 1.50 to 2.00 would be some kind of minimum factor we could choose for safety. The State's current policy on embankment safety factors is not specific, but recommends between 1.5 to 2.0. Mr. Billy Rogers stated that it might be more feasible to increase the minimum factor of safety to insure adequate stability.

The next speaker was Mr. Andy Munoz of the FHWA in Fort Worth. Andy's discussion centered on the types of failures that have occurred and the possible reasons for these failures.

The most common embankment failure Andy observed concerned the embankment material getting so wet that it lost its shear strength and failed (see Figure 1A). Andy presented a working example of how the moisture content of a soil affects the shear strength of that soil

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*Dr. Wright's research project has investigated locations where embankments failed with safety factors as high as 3.5.

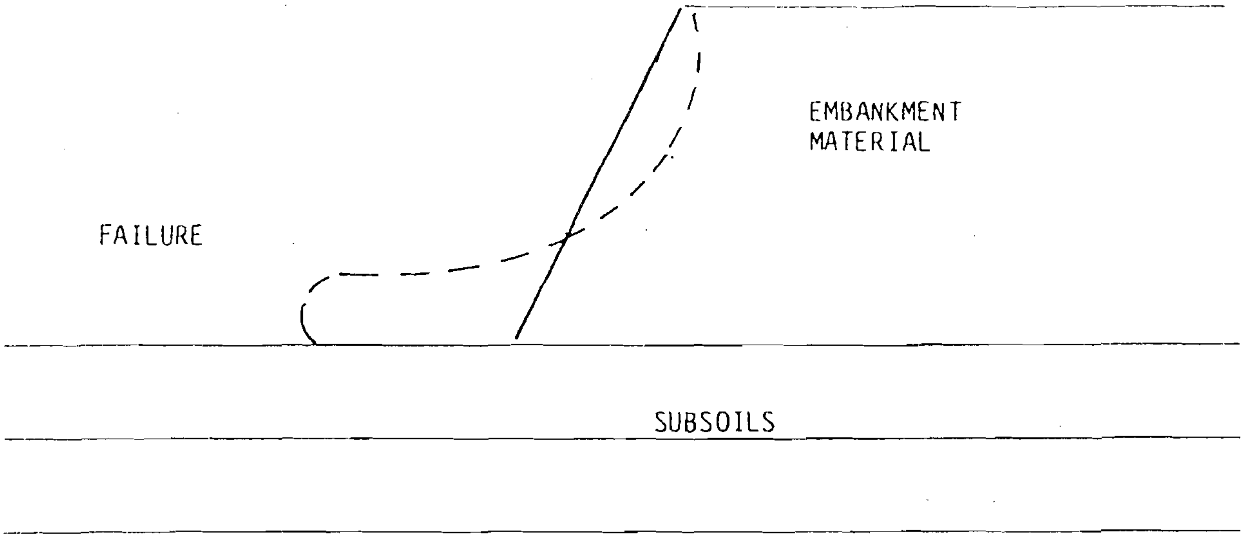

FIGURE 1A

EXAMPLE:

PI = LL - PL

PI	= 53	PLASTICITY INDEX
LL	= 75	LIQUID LIMIT
PL	= 22	PLASTIC LIMIT
MC	= 43	MOISTURE CONTENT

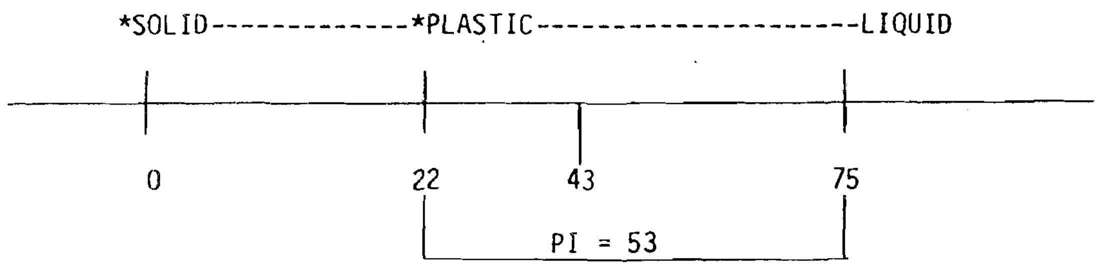

FIGURE 1B

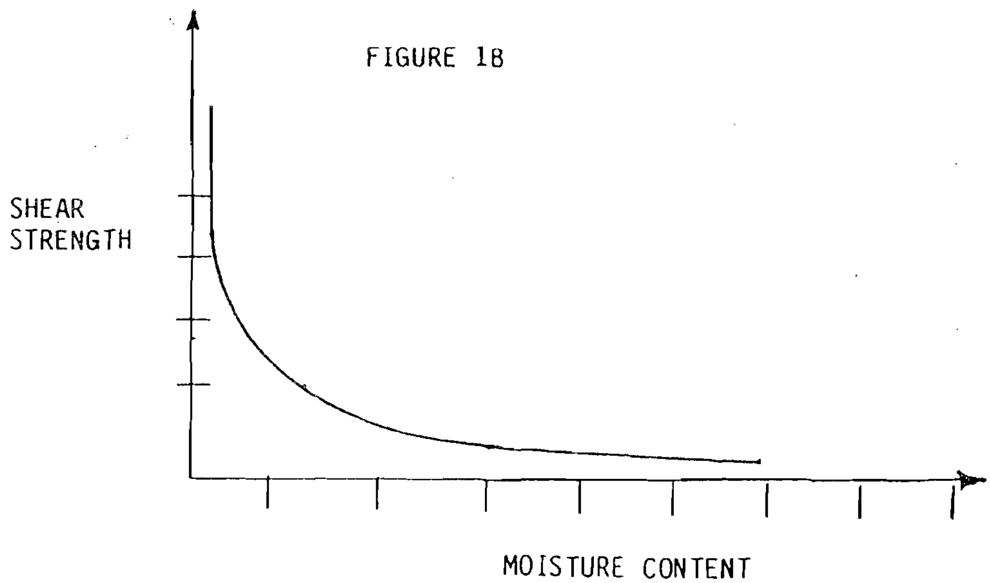

FIGURE 1C

(see Figure 1B). The graph in Figure 1C shows that the shear strength of a soil is proportional to its moisture content.

Another type of failure Andy mentioned pertained to the stability and strength of the sub-soil. As in the previous cases, an increase in the moisture content of the sub-soil will cause the shear strength to decline. The weight of the overburden will become too great for the weakened sub-soil to support that it will actually start "flowing" outward. This causes the embankment to settle or slip (Figure 2). Also, low cohesive soil strength will cause the sub-soil to fail.

Andy suggested possibly using one or more of the following measures in preventing sub-soil failures:

1. Providing adequate drainage of surface and subsurface water to prevent an influx of water into the sub-soil.
2. Putting in some type of berm to increase the resistance for lateral movement (See Figure 3).
3. Placement of a "key" into the sub-soil that will act as a sort of "dam" to prevent the material from flowing (Figure 3).

To prevent and/or correct slope failures within the embankment the following methods are suggested:

1. Placement of a granular blanket on the embankment slope to help drain the water and to give additional strength in supporting the overburden.
2. Stabilizing the embankment slope with lime/fly ash.
3. Reinforcing the embankment slope with fabric, etc.

Concerning construction of the embankment, Andy suggested several preventative measures that may be used to avoid failures. These measures are as follows:

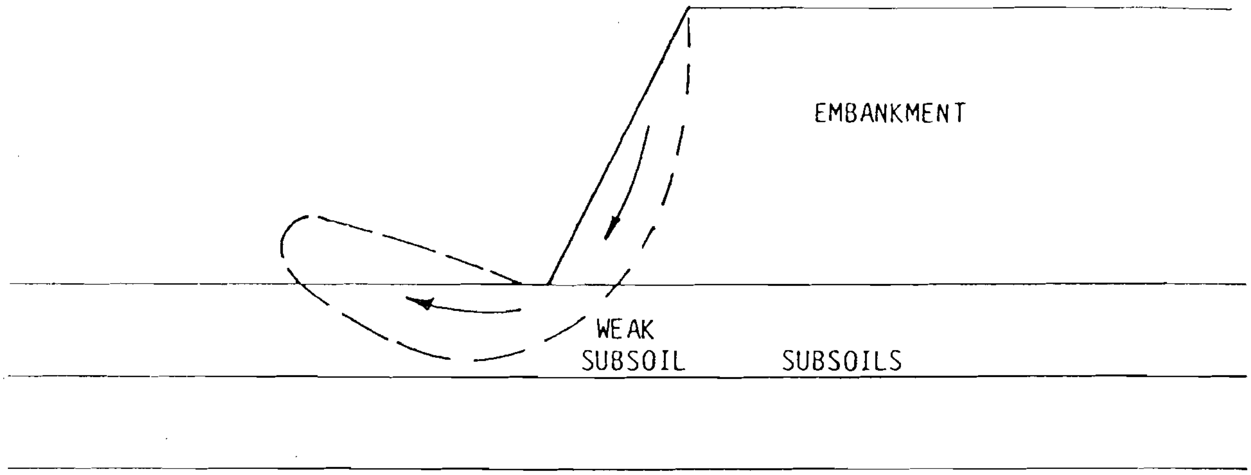

FIGURE 2

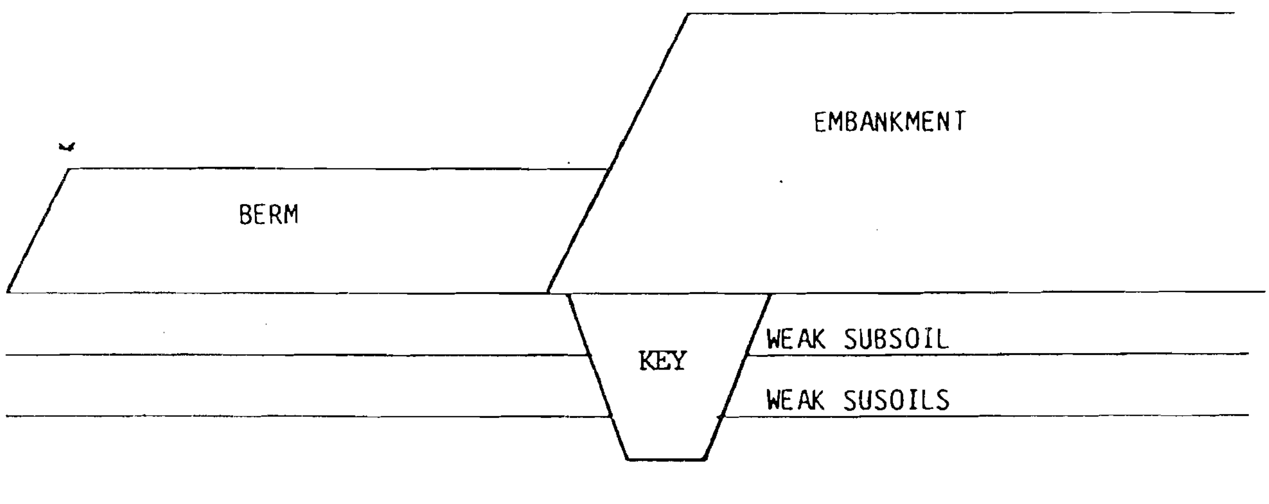

FIGURE 3

1. Provide adequate drainage of surface water at both the head and toe of the slope to prevent saturation of the embankment slope. Proper drainage will prevent water from entering the slope and to drain the water that does get in there.

2. Specify embankment material, preferably granular material. This would increase the stability of the embankment because the soil properties are known.

3. Fatten the slope. This decreases the slope angle and increases the S. F.

4. Lime stabilize the slope. This "cements" the material in a way, preventing an influx of water and creates a bond that holds the material together.

5. Use of fabric materials and reinforced earth slopes. Both of these allow for drainage and provide frictional surfaces that aid in preventing movement.

Any one of these measures would reduce the possibility of embankment slope failure and when two or more are used in combination, the possibility of failure is further decreased. The selection of a cost-effective method should depend on specific project conditions. This will require more engineering work on each embankment.

Andy stated that our problem is that of stability within the embankment. This again brings up the question concerning the factor of safety. Andy said that in most cases, a safety factor of 1.35 is adequate and is used by most State Highway Agencies in their slope stability analysis. The correlation of safety factor to field performance is a function of the accuracy of the soils data input and the method used. Andy also said that so far as determining the critical height of an embankment, it should be a function of sub-soil stability and not a function of slope.

In response to the discussion to this point, Dr. Steve Wright, Professor at the University of Texas at Austin said we should look at the embankment failures as two separate problems or categories. The two categories he referred to as short-term stability and long-term stability and that they were distinctively two different problems.

The short-term stability problem pertains to the stability of the slope immediately after it is built or rather the condition immediately after construction.

Because there has not been a substantial amount of time to allow an appreciable flow of water into or out of the soil, the strength of the soil at this stage is not dominated by the frictional angle, but rather it is dependent upon the cohesive strength. So most of our early strength is derived or expressed in terms of cohesion.

Steve stated that the Texas-Triaxial Test used by the Highway Department is essentially a valid and appreciable tool for getting the early strength for the short-term stability condition. He also said that when you have a material you have characterized in strength, primarily in terms of cohesion, you will find when doing stability calculations that the factor of safety is dominated by slope height rather than slope angle. In others words, your factor of safety for subsurface failures is going to depend primarily on slope height.

Most foundation or sub-soil failures are going to be short-term failures due to low cohesive soil strengths. To minimize these short-term failures, we have determined that limiting the embankment height will limit the amount of overburden load being placed on the sub-soil. This is why short waiting periods between embankment lifts are specified for embankments built on soft soils.

The long-term stability condition is in reference to where the soil has had adequate opportunity for water to flow either into or out of the soil. The time necessary for such an occurrence is a function of the availability of water, drainage and the nature of the soil. Steve stated that in a long-term stability problem, the stability analysis will show that the frictional angle (PHI) is dominate and the cohesion term is small or zero (0). So your strength is going to be characterized primarily in terms of an angle of internal friction. At present, the Highway Department has no test procedures for measuring the strength applicable to this stability condition but can provide reasonable estimates using the Triazial Test. However, knowing what water conditions you are going to have is very critical to long-term stability.

When doing stability calculations of the embankment slope for long-term conditions, we find that the factor of safety is influenced primarily by the slope angle. The long-term stability condition may or may not be more critical than the short-term because it depends on whether the soil effective stress increases or decreases with time. If the embankment slope can not be changed, you will probably have to build retaining walls to improve stability. Retaining Walls is a <u>very expensive</u> approach to correcting this type of slope problem and other more economical and effective methods are available.

As evident in the majority of embankment failures in the Houston area, it is not the problem of slope height so much as it is the steepness of the slopes. In conclusion, Steve stated that perhaps limiting the maximum side slope to 4:1 would reduce the number of long-term failures.

Section B

After Steve Wright's conclusion, Mr. Wadlington introduced Mr. Bill Hauck of the Houston Urban Office, who talked about slope protection. Mr. Hauck discussed some of the problems experienced by District 12 and the Urban Office pertaining to slope protection and the various treatments for protection.

Various slope protection treatments used by District 12 and the Urban Office are as follows (See Appendix C):

1. On some slopes where the topsoil has slipped or eroded, the Maintenance Section removed all the failed soil, placed new material in 10-foot wide strips to the original slope line. Then they drove 10-foot piles at 4 to 8 feet spacings for stabilization. For placement of vegetation on the slopes, straw mulch or fertilizer-seeding mixtures were used.

2. Another slope protection treatment used was the placement of a plastic fabric. As before, the failed soil was removed and new soil replaced, but in 18-inch layers with the plastic fabric. This aided in stabilizing the slope, but surface erosion occurred.

3. Some places where piles were previously driven experienced surface erosion still so a nylon mesh fabric was placed on the surface to aid vegetation in setting roots. This works okay, but mowers have torn the fabric up during routine mowing.

4. One treatment that has worked well with slope protection has been to remove the material, mix with lime and then replace the material and recompact it. This was ideal for short-term slope protection, but growing a vegetation cover on the surface is a problem. The lime in the soil acted against the vegetation. A solution to this has been to go back to the slope, scarify the surface 2 to 3 inches and place a good topsoil.

John Nixon asked what kind of success concerning slope stability we have had with lime and Mr. Hauck stated that there are embankments that were lime treated 3 or 4 years ago and are still holding.

Concerning slope protection, Mr. Hauck said that if we could control mud flows on the surface, San Augustine grass could probably work for surface protection on slopes of 4:1 or flatter. However, Mr. Hauck concluded that the only totally successful surface and slope protection is the placement of concrete riprap at a slope of 3½ to 4:1.

Mr. Dave Williamson observed that we are not going to hold these slopes and embankments with just a good vegetation cover. We must stabilize these slopes first and then provide a medium for the living plants to germinate and take hold. Once you do that, you probably will have a lot more successful block sodding than what you are getting now. He stated that two or three inches are needed, not two to three feet on the slope surfaces. Also that the Forest Service could possibly supply information slope/surface protection due to their experience in revegetation and reforestation of severe areas.

Mr. Wadlington asked for further comments concerning slope protection at which time Mr. Andy Munoz said he would like to comment on the methods of correction. Andy stated he felt that we would probably get good success with lime stabilization or reinforcing the soil, but would definitely not recommend pilings for any kind of slope protection. He discussed the principle that pilings depend upon the arching of the soil to strengthen it, but because of the nature of the material used in the embankments here, when this material becomes plastic, it just flows around the pilings. The material never localizes any kind of arching between the pilings, therefore making the pilings useless as far as embankment stability is concerned.

The use of retaining walls in conjunction with embankments in District 12 Construction was discussed by Mr. Wadlington. Mr. Wadlington said that the use of retaining walls has advantages in that we can use slopes of 4:1 or flatter and allow for vegetation to take root. We have had mudflows on slopes of 3:1, but the retaining walls will reduce this by allowing flatter slopes.

Concerning the construction of retaining walls, Mr. Wadlington discussed the types of backfill used. Cement stabilized backfill is currently being used for backfilling retaining walls. This greatly stabilizes the backfill. Sand backfill was used primarily, but the sand washed out from behind the retaining walls. Contractors like to use cement stabilized base because it is easier to handle and it is cheaper to use than granular materials in some cases.

Reinforced earth retaining walls are designed using metal straps placed in the backfill. These have been proven to add stability to the walls and to the embankment.

Mr. Wadlington brought up the current design of using concrete barrier walls in the center of roadways on the sides or the top of the embankment. These barrier walls aid drainage in that they carry the water from bridges and roadway embankments down to where the embankments are 10 feet high or lower where a drain is placed. Before, the water drained off the bridges and onto the embankments, running down the slopes and saturating the fill material. Now, the only water on the embankments is what actually falls on the slopes.

Andy Munoz said he would like to comment on some of the things that Steve Wright had talked about earlier. Andy said that there are computer programs and charts available now and can be used as design tools for stability problems. In response, Steve said that we can do calculations and use charts and that they are adequate. Computer programs just speed up design. In some cases, all design techniques can be used.

However, on long-term stability problems, we don't have the strength information to do the calculations. We have the techniques to do it, it's just obtaining the input that is difficult and in some cases, we can only estimate the input. That is why we should consider developing a laboratory testing analysis based on situations rather than estimates.

The problem now is not calculations, but about embankments. Fairly simple charts can be used in design, if the proper strength data was available. Our problem is knowing very little about the strength characteristics of the material. Mr. Wadlington commented that we don't know where the embankment material will be coming from so material information is hard to determine for design.

Mr. Ho asked Steve if these computer programs were available for our computers. Steve stated that a slope stability program was developed over 10 years ago, but it didn't work on our computer. We are now in the process of updating that program and converting it to be compatible with our computer, but it will not be ready for about 18 months. The present program in use is complex and needs to be updated and simplified.

Mr. Munoz added that a computer program based on Bishop's Modified Method is on-line in the Austin computer terminal.

Concerning the material available for embankments in this area and the scarcity of information for design, Steve suggests that a 4:1 slope would work better. At present, the 3:1 slope has many stability problems, but going to a 4:1 slope may not be the ultimate design solution. However, it will decrease the problems both in embankment stability and surface erosion.

Since riprap is primarily a slope protection technique and does little as far as stability is concerned, someone directed a question to Steve concerning the advantage of using rock riprap or lime stabilization. Steve responded that they help if you go deep enough with them so that the factor of safety is affected. Right now, you can use a computer program to determine what depth you need to go with lime stabilization to get the factor of safety up to an acceptable level. Lime decreases the PI in clay, thus decreasing the liquid and plastic limits of the soil.

There is a definite strength increase in lime stabilized soils if they are constructed well, but even this increase is hard to determine.

At this point the meeting was adjourned for lunch.

Section C

After lunch, Mr. Clinton Bond of District 20 in Beaumont showed some slides and discussed the use of a plastic grid for reinforcing embankments. A polyethylene plastic grid fabric developed by a division of Gulf Oil Corporation was used. Mr. Bond said that Gulf representatives had come to them and wished to demonstrate their product. A section of embankment that was experiencing failures on the average of once a year was used. Gulf engineers did the design on the embankment reinforcements after a core sample was sent to them by the State. Gulf engineers, along with State engineers, oversaw the placement of the material.

The embankment was reinforced in the following method:

1. A 1:1 slope was used.
2. The existing failed material was removed and reused. The material was compacted and the density and moisture content were closely watched. A compaction of 95 to 97% density was achieved. The plasticity index of the material was at 53 while its liquid limit was about 76.
3. The plastic grid was placed in about 9 layers. We started out with three layers of about 18 inches and the remainder in 2-ft. layers. The material was placed using about 8-inch lifts with each being compacted to about 95% density.

No anchoring was used for the material. There are various sizes and gauges available so use can be varied due to soil material and site properties.

Mr. Bond said the cost of the grid material for this job was paid for by Gulf because it was for demonstration purposes, but estimated it to be about $10,000. The cost of equipment and labor ran about $20,000. So, overall, this was an inexpensive technique to reinforce an embankment slope.

Mr. Wadlington commented that this method looked like it would be ideal for use on our present embankments of 3:1. Mr. Bond agreed and said that they had no problems with mowers affecting the plastic grid. Their only problem at this site has been surface erosion, but that was a problem with vegetation and not with the use of the material.

While still on the subject of embankments, Mr. Wadlington asked Andy Munoz if he could tell us some procedures we need to go through for design of embankment slopes.

Andy's response was that the reinforcement of the embankment slope in Beaumont was a good idea to consider for design. A typical approach of many State Highway Departments is to build test sections and observe the results.

We should make use of the data that was collected from the embankment sections in our present design. Assign a factor of safety of 1.0 to the embankment slopes that have failed and calculate back to get the soil strength at failure. By doing this, you can probably get a good idea on what types of soil strength to use for slope stability analysis of embankment slopes. Some of this can be correlated through tests such as the Texas Triaxial Test.

Mr. Wadlington agreed and added that the material in these embankment sections are pretty typical of the material we are using on jobs and the strengths would be applicable for design. Mr. Wadlington concluded that this meeting has established the fact that riprap is for surface erosion protection and not for embankment stability. Also, that we are leaning toward a 4:1 slope in District 12 because it would be much safer than a 3:1 slope. In areas where we are restricted by right of way and can not use a 4:1 slope, we may be able to use a 3:1 if we put in this new polyethylene grid fabric. That may also be cost effective. Also, the fabric is preferable over lime because the lime inhibits vegetation growth. It wouldn't be any problem to lay the fabric in 10 to 20-ft. wide layers, whatever is needed, on the outer edges of your embankment. In effect, this creates a reinforced embankment slope.

However, setting requirements on the plasticity limits and liquid limits would be difficult due to the wide range of material sources in the District and the cost would be too great to transport a borrow material a great distance.

Concerning embankment height, a statement was made by a State representative that the current standard of 20 feet being used by the District should still be in effect even with the 4:1 slope. Higher embankments may reduce the factor of safety, cause problems concerning right of way (using 4:1 slope) and the cost of constructing an embankment higher than 20 feet approaches that of a bridge. Andy Munoz does not agree with this statement and emphasis is made that embankment height is governed by the properties of the subsurface soils and embankment slope is governed by the properties of the fill soil. It is strongly recommended that each project be designed based on existing subsurface conditions and specific characteristics of the embankment fill material.

Section D

The next topic for discussion concerned bridge pile lengths. Mr. Wadlington stated that in construction projects around here the actual driving of the piles varies considerably from what our tests show the bearing should be.

In some cases, we put a K factor on the piles and they were driven to a certain elevation and stopped. Other cases occurred where we drove the pile all the way to grade trying to obtain a suitable bearing. Is this really cost effective? The cost of the test pile sometimes costs more than the savings. The piling is one of the most important things in a bridge and we would like to find out if saving a little on the pile lengths will cost us more in the future.

At present, we are not able to test that soil and determine the bearing accurately. For example, when driving piles, the first pile driven in a footing may not get the required bearing. The next four or five piles in the footing are driven but left about 2 feet above grade until all the piles are in place. We then drive each one to grade and obtain sufficient bearing. This technique consolidates the soil down there to where we get the bearing capacity needed. What we would like to have is more accurate tests in determining the bearing and pile lengths.

In some cases, a test pile is used, in others we run a Triaxial Test on the soil and determine the bearing capacity and pile length from that. The problem with this testing is that the pile length is designated by a length range. By using the maximum length, we may be wasting piling and money but, on the other end of the scale, we may not have strong enough piles so what is a suitable length?

In response to this, Bob Standford stated that safety was the first concern, then economy. If we have a test piling and it says to cut off 10 feet, we ought to do it. Our general policy, however, is to promote any economy we can get and alternate designs whether it is in the substructure, superstructure or anything like that.

Mr. Ho stated that we can not base the bearing capacity and pile lengths on one test pile because that pile may have been driven in the weakest or strongest place. Mr. Standford agreed and felt that sometimes 5 or more test piles should be used. He stated that sometimes we go as far as to require wave equation analysis and additional research and feels that this information should be used. In some cases, the borings indicate good bearing soils and this inforamtion should be utilized in determining where to put the piling.

Mr. Ho said that the current pile laying policy is that we determine the soil strength from a soil boring and arrive at a minimum penetration. So far we have had no problems with our pilings using this method. We haven't had a load test pile in the past 17 years. I feel that the redriving procedure works well, but I disagree with the idea of cutting off piles based on one test load pile data.

In response, Mr. Standford stated that they have had good response with the redriving procedure, but felt that a dynamic analysis should be derived to aid in the design.

Mr. W. V. Ward stated that other pile characteristics should be considered and one of them is the shape of the pile. Square piles, for the most part, are used all over the State. Some areas like to use steel H beams or tubular piles, but there are a lot of soil conditions where such pile types do not work well. One such case is driving in fat clay. Another example is driving a steel H pile in sand. Initially, you can drive the H pile out of sight and won't have any bearing, but if you let them set up for about a week, the pile will have high bearing strength.

Mr. Ward also stated that the use of tapered piles was a better shape to use in the clays around District 12 than square piles. Square piles actually destroy some of the strength of the pile and surrounding soil.

Concerning stream crossings, Mr. Wadlington stated that it has been our policy to go 20 feet below the ultimate flow line of the bayou or channel as a factor of safety even if bearing is obtained above this elevation. This method has proved to be very effective. We seldom have to go more than 20 feet below the stream flow line to obtain a good bearing.

It was concluded that current policies were adequate, but we need to fully utilize all the information given us from tests to achieve more economical designs.

Section E

The final topic of discussion concerned bridge ends. There exists a problem of having a bump where the pavement and bridge ends meet. The bumps are caused by different reasons. One such cause is that the concrete pavement is pushing against the bridge. Anchors are placed under the pavement to secure the slab into the embankment, but even the anchors are moving.

Mr. Lewis expressed the fact that 10 to 12 years ago a committee set a type of standard where you built a "wedge" close to the bridge. Then you lower the approach slab and cover it with asphalt so you could whittle it off or build it up as the need be. This does not work very well in Houston due to the traffic.

Mr. Wadlington stated that in places the roadway slab has raised up a couple of inches above the bridge slab at both ends. This makes it necessary to grind the pavement ends down because they are pushing against the abutments. These abutments were backfilled with cement stabilized base. If we had brought the cement stabilized base up and finished it flush with the bridge, it would be no problem to trim it or add to it. Bringing the cement stabilized base and covering it with hot mix would be a reasonable solution.

Older pavements that have the redwood expansion joints have fewer bumps at bridge ends, but that may also be caused by the embankment being equalized in its swelling.

Mr. Lewis said that another solution was to extend the continuous reinforced concrete pavement right on across the bridge. This would be feasible on short bridges, but on a long bridge you may have problems. Right now about five to six hundred foot bridges can be done adequately.

Concerning the causes of the bumps, Andy Munoz stated that this was a nationwide problem. The problem has been deemed to be caused by either settlement within the embankment itself or due to the sub-soil. This differential settlement due to sub-soil conditions between your abutment and embankment is aggravated with piling or drill shaft supported abutments. Waiting periods between embankment construction and pile driving can alleviate the problem. In many cases, the "bump" is caused by inadequate compaction of the embankment material next to the abutment. Use of select material and adequate compaction can solve the problem.

In response, Ed Suchicki said that it is not a settlement problem here, but rather it is the expansion of the CRCP on its running pavement. The pavement expands and shoves up against the bridge and has no place else to go but up. The bridges are stronger than the pavements so it is the pavement that buckles. We have tried to remedy this problem by putting a 4" expansion joint before or at the end of the approach slab and make it a pavement maintenance problem and not a bridge maintenance problem. Still, that 4" joint will eventually close and have to be maintained. There is no fool-proof solution to the expansion bumps at bridge ends--we can only ease them.

At this point, Mr. Wadlington decided that this was a good time to end the meeting. We have accomplished a great deal and appreciated the attention and comments of all those present.

Section F

People in Attendance

NAME	ORGANIZATION
Edward T. Addicks	SDHPT - D-8 - Austin
Harold Albers	SDHPT - D-9 - Austin
Dwight A. Allen	SDHPT - Houston - District 12
Tony Ball	FHWA - Austin
Ralph K. Banks	SDHPT - D-18M - Austin
Clinton B. Bond	SDHPT - Beaumont - District 20
William Boy	SDHPT - Houston Urban
Martin Brown	SDHPT - Houston - District 12
Billy F. Davis, Jr.	SDHPT - Houston - District 12
Hunter F. Garrison	SDHPT - Houston - District 12
Donald E. Harley	FHWA - Austin
Robert Hauck	SDHPT - Houston Urban
Frank Hebner	SDHPT - Houston - District 12
Michael Ho	SDHPT - Houston - District 12
John Inabinet	FHWA - Austin
George E. Kishi	SDHPT - Houston - District 12
R. L. Lewis	SDHPT - D-8 - Austin
Tim McNamara	SDHPT - Beaumont - District 20
Andy Munoz, Jr.	FHWA - Fort Worth

NAME	ORGANIZATION
Billy R. Neeley	SDHPT - D-9 - Austin
John F. Nixon	SDHPT - D10R - Austin
Cecil E. Norris	SDHPT - Beaumont - District 20
Omer F. Poorman	SDHPT - Houston - District 12
Billy R. Rogers	SDHPT - D-8 - Austin
Charlie Smith	SDHPT - Houston - District 12
Robert Standford	FHWA - Austin
Peter A. Stauffer	C.T.R. - U.T. - Austin
Ed Suchicki	SDHPT - Houston Urban
Ronald T. Templeton	SDHPT - Houston - District 12
Jon Underwood	SDHPT - D10R - Austin
Frank Wadlington	SDHPT - Houston - District 12
W. V. Ward	SDHPT - Houston Urban
Dan M. Williams	SDHPT - D-5 - Austin
Dave R. Williamson	FHWA - Fort Worth - Landscape Architect
Stephen G. Wright	C.T.R. - U.T. - Austin

Section G

APPENDIX A
(As Attachment Referenced in Meeting of February 17, 1983)

Locations of Embankment Slope Failures
In District 12

LOCATIONS OF EMBANKMENT FAILURES IN DISTRICT 12

HIGHWAY	EMBANKMENT LOCATION	SLIDE AND EMBANKMENT INFORMATION SHEET NO.
IH 10 E	Wade Road and Missouri Pacific RR Overpass	3, 13
IH 10 E	Thompson Road	3, 14
IH 10 E	Sjolander	3
IH 10 E	Garth Road	3
IH 10 E	Lynchburg - Crosby Road	3
IH 10 E	Gulf Plant Road and Southern Pacific RR Overpass	4, 15
IH 10 E	Penn City Road	4
IH 45 N	West Road	5
IH 45 N	Quitman	5
IH 45 N	Crosstimbers	5
IH 45 S	College Avenue	5
IH 45 S	FM 2351	5
US 59 S	Greenbriar and Shepherd Drive	6
US 59 S	Bellaire Boulevard	6
US 59 N	FM 525	6
IH 610 W	Westpark	7, 16
IH 610 W	US 59 South	7
IH 610 S	Crestmont	7
IH 610	Martin Luther King Boulevard	8

HIGHWAY	EMBANKMENT LOCATION	SLIDE AND EMBANKMENT INFORMATION SHEET NO.
IH 610	Broad Street	8
IH 610	Telephone Road	8
IH 610	Woodridge	8
IH 610	Kirby Drive	9
IH 610	South Main	9
IH610	Buffalo Speedway	9
IH 610	Fannin	10
IH 610	Gellhorn	10
IH 610	Wayside to Long	10
IH 610	Scott	10
SH 146	SH 225	11, 17
SH 225	Scarborough	11
SH 225	Shell Overpass	11
US 90	Clington Drive	11
Loop 201	Decker Drive Goose Creek	12
SH 225	Sims Bayou Bridge	12
NASA 1	Taylor Bayou Bridge	12

APPENDIX

LOCATIONS OF EMBANKMENT FAILURES IN DISTRICT 12

HIGHWAY	EMBANKMENT LOCATION	HEIGHT	SIDE SLOPE	TYPE OF EMBANKMENT MATERIAL
IH 10 E 1958:	Wade Road 508-1-20 (508-14)	29' All	3:1 Usual	Common Roadway Excav. & Fill (Clayey Soil)
	1. Failures occured frequently. Repaired by pushing dirt back and compacting with dozer. 2. 198- - 508-1-157: Construction of reinforced earth retaining walls at all quadrants. Embankment Slope 4:1.			
IH 10 E 1958:	Thompson Road 508-1-22 (508-16)	16½' East 16' West	3:1 Usual	Common Roadway Excav. & Fill (Clayey Soil)
	1. Failures occured frequently. Repaired by pushing dirt back and compacting with dozer. 2. 1980-508-1-157: Construction of reinforced earth retaining walls at all quadrants. Embankment slope 4:1.			
IH 10 E 1958:	Sjolander 508-1-22- (508-16)	18' All	3:1	Common Roadway Excav. & Fill (Clayey Soil)
	1. Failures repaired by pushing dirt back and compacting with dozer. Lime added.			
IH 10 E 1958:	Garth Road 508-1-22 (508-16)	16' All	3:1	Common Roadway Excav. & Fill (Clayey Soil)
	1. Failures repaired by pushing dirt back and compacting with dozer. Lime added.			
IH 10 E 1958:	Lynchburg-Crosby 508-1-22 (508-16)	18' All	3:1	Common Roadway Excav. & Fill (Clayey Soil)
	1. Failures repaired by pushing dirt back and compacting with dozer. Lime added.			

APPENDIX

LOCATIONS OF EMBANKMENT FAILURES IN DISTRICT 12

HIGHWAY	EMBANKMENT LOCATION	HEIGHT	SIDE SLOPE	TYPE OF EMBANKMENT MATERIAL
IH 10 E	So. Pac. RR Overpass (Gulf Pland Rd.)	30'± All	3:1	Common Roadway Excav. & Fill (Clayey Soil)
1958: 508-1-21 (508-14)				
	1. Failures have occured several times. Dirt was pushed back and compacted with dozer. Lime was added at one such occasion.			
IH 10 E	Penn City Rd.		3:1	Common Roadway Excav. & Fill (Clayey Soil)
1959: 508-4, 20				
	1. Failures repaired by pushing and recompacting embankments.			

APPENDIX

LOCATIONS OF EMBANKMENT FAILURES IN DISTRICT 12

HIGHWAY	EMBANKMENT LOCATION	HEIGHT	SIDE SLOPE	TYPE OF EMBANKMENT MATERIAL
IH 45 N 1963: 500-72	West Road	20' All	2½:1	Common Roadway Excav. & Fill (Clayey Soil)
	1. Repaired by placing 5" riprap (CONC)(CLA)			
IH 45 N 500-53: 500-57	Quitman	20' All	2:1	Common Roadway Material (Clayey Soil)
	1. Drove sheet piling w/cement stabilized material (compacted) added.			
IH 45 N 500-53	Crosstimbers	20' All	2:1 MAX 4:1 MIN.	Common Roadway Material (Clayey Soil)
	1. 1967 - 500-91: Placed 5" riprap (CONC)(CLA) with toewall; 3.5:1 min. slope, 2:1 max. 2. 1979 - 500-119 (508-3-268): Placed sheet piling, w/reinforced concrete retaining wall, in conjunction with existing riprap (built for construction of U-turns).			
IH 45 S	College	20' All	3:1	Common Roadway Material (Clayey Soil)
	1. 1977 - Pushed dirt back and compacted with dozer. 2. 1982 - Repaired by same method.			
IH 45 S	FM 2351	20'	1.4:1 West 2:1 East	5" Riprap CLB (RR9) & Common Roadway Embank.

APPENDIX

LOCATIONS OF EMBANKMENT FAILURES IN DISTRICT 12

HIGHWAY	EMBANKMENT LOCATION	HEIGHT	SIDE SLOPE	TYPE OF EMBANKMENT MATERIAL
US 59 S	Greenbriar	16' West 18' East	$2\frac{1}{2}$:1 $2\frac{1}{2}$:1	Common Roadway Excav. & Fill
	Shepherd	18' All	$2\frac{1}{2}$:1	(Clayey Soil)

1961: 27-58 (Both)

1. Pushed dirt back and compacted with dozer.
2. Drove wood pilings and compacted fill.
3. 1969- 27-80 (27-13-55)
   A. Placed pilings in areas without existing piles; placed compacted granular material fill, 6" topsoil and 1" block sodding @ 2.6:1 slope.

   B. Placed different fill materials and compacted: TYB, CL3 @ 2:1, TYC, CL3 3:1; both covered by 4" TYC, CL2 and block sodding # 3:1. Constructed retaining wall with drilled shaft footings, installed perforated pipe drains.

HIGHWAY	EMBANKMENT LOCATION	HEIGHT	SIDE SLOPE	TYPE OF EMBANKMENT MATERIAL
US 59 S	Bellaire	20' All	$2\frac{1}{2}$:1	Clayey Soil

1964: 27-66

1. 1980 - Southwest side 27-134
   Removed and recompacted fill; replaced partially with TYB, CL3 fill. Placed 5" riprap (CONC)(CLA) at varying slopes (2:1 Max., 4:1 Min.)

HIGHWAY	EMBANKMENT LOCATION	HEIGHT	SIDE SLOPE	TYPE OF EMBANKMENT MATERIAL
US 59 N	FM 525	18'	3:1	Common Roadway Excav. & Fill (Clayey Soil)

1959; 177-27, 28

1. Pushed back dirt and compacted with dozer.

APPENDIX

LOCATIONS OF EMBANKMENT FAILURES IN DISTRICT 12

HIGHWAY	EMBANKMENT LOCATION	HEIGHT	SIDE SLOPE	TYPE OF EMBANKMENT MATERIAL
IH 610 1962: 271-53	Westpark	24'°	Embank: 2:1 Topsoil: 3:1 Max.: 2:1	Select Embank. Material W/block Sod (Clayey Soil)

1. First failures were repaired by pushing dirt back & compacing with dozer.
2. Drove sheet and /or timber piling & pushed & compacted embankment.
3. 1980 - 27-134 (271-17-56); cut tops of piling at excavation line, placed & compacted material - excav, bor, and embank (DENS CONT)(TYB,CL3). Placed 5" riprap (CONC)(CLA) at 2:1 slope with toewall.
4. 1981 - Failure of riprap embankment.
   1982 - 271-17-71 - Plans initially designed for installation of soil anchors, riprap, and retaining walls. Before work began, failure occurred again. Sheet piling was driven to temporarily support embankments. Plans are undergoing field change to use double-wall retaining walls instead.

HIGHWAY	EMBANKMENT LOCATION	HEIGHT	SIDE SLOPE	TYPE OF EMBANKMENT MATERIAL
IH 610 1962: 271-53	US 59 S	20°	3:1	Select Embank. Material with Block Sod

1. Pushed back and compacted dirt with dozer.
2. 1980 - 27-134 (271-17-56); placed 5" riprap (CONC)(CLA) with toewall at min. slope of 3.5:1. 50 ft. of straw mulch seeding is placed on each end of riprap at same slope.

HIGHWAY	EMBANKMENT LOCATION	HEIGHT	SIDE SLOPE	TYPE OF EMBANKMENT MATERIAL
IH 610 1967: 271-110	Crestmont	15'	3:1	Common Roadway Excav. & Fill (Clayey Soil)

1. Push and compacted embankment with dozer.
2. 1980 - 27-134 (271-14-44) on NE and SE quadrants removed existing embankment and replaced with lime stabilized embankment material and with soil retention and broadcast seeding.

APPENDIX

LOCATIONS OF EMBANKMENT FAILURES IN DISTRICT 12

HIGHWAY	EMBANKMENT LOCATION	HEIGHT	SIDE SLOPE	TYPE OF EMBANKMENT MATERIAL
IH 610	M. L. King 1967: 271-110 (South Park Blvd)	18'	3:1	Common Roadway Excav. & Fill (Clayey Soil)

1. Pushed and compacted embankment w/dozer.
2. 1978 - 271-156: NE quadrant - excavated embankment, installed filter fabric & placed material - excav, bor and embank (DENS CONT)(TYB CL3) - and compacted accordingly.
3. 1980 - 27-134: NW quadrant - replaced embankment material w/excav, bor, & embank (DENS CONT)(TYB CL3). Placed 5" riprap (CONC)(CLA) at 3.5:1 slope.

HIGHWAY	EMBANKMENT LOCATION	HEIGHT	SIDE SLOPE	TYPE OF EMBANKMENT MATERIAL
IH 610	Broad 1971: 271-123	18'	3:1 Max.	Compacted Embankment Density Control

1. Pushed and compacted embankments.
2. 1980 - 27-134: SW quadrant - Placed excav, bor, & embank (DENS CONT)(TYB CL3) 5" riprap (CONC)(CLA) with toewall at 3.5:1 min. slope.

HIGHWAY	EMBANKMENT LOCATION	HEIGHT	SIDE SLOPE	TYPE OF EMBANKMENT MATERIAL
IH 610	Telephone 1971: 271-123	19'	NW, NE, SW 3:1 SE - 2:1	Compacted Embank.-Common (DENS CONT) 5" Riprap (CONC) (CLA) w/retain. Wall.

1. Pushed & compacted embankment.
2. 1980 - 27-134: NW quadrant - placed excav, bor & embank (DENS CONT) (TYB CL3) 5" riprap (CONC)(CLA) at 3.5:1 with toewall.

HIGHWAY	EMBANKMENT LOCATION	HEIGHT	SIDE SLOPE	TYPE OF EMBANKMENT MATERIAL
IH 610	Woodridge 1971: 271-123	20'	NE, SE 3:1 NW - 2.8:1 SW - 2:1	Compacted Embank.-Common (DENS CONT) 5: riprap (CONC)(CLA)

1. Pushed and compacted embankment.
2. 1980 - 27-134: NW quadrant between Telephone and Woodridge - Recompact embankment & place 5" riprap (CONC)(CLA) w/toewall at 3.5:1 slope.

APPENDIX

LOCATIONS OF EMBANKMENT FAILURES IN DISTRICT 12

HIGHWAY	EMBANKMENT LOCATION	HEIGHT	SIDE SLOPE	TYPE OF EMBANKMENT MATERIAL
IH 610 1969: 271-120	Kirby	20'	3:1	Compacted Embank.-Common (DENS CONT)

1. Pushed and compacted embankments.
2. 1978 - 271-156: NE & SE quadrants. Excavated unsuitable embank. material. Drive treated timber piling & covered them w/excav, bor, and embank (DENS CONT)(TYB CL3). Material at 3:1 slope.
3. 1980 - 27-134: NW & SW quadrant. Excav, bor & embank (DENS)(TYB CL3) - 5" riprap (CONC)(CLA) w/toewall at 3.5:1 slope.

HIGHWAY	EMBANKMENT LOCATION	HEIGHT	SIDE SLOPE	TYPE OF EMBANKMENT MATERIAL
IH 610 1969: 271-120	South Main	20"	3:1	Compacted Embankment (DENS CONT) (COMMON)

1. Pushed and recompacted embankment.
2. 1978 - 271-156: NW & SW quadrants - Excavated unsuitable embank., drove treated timber piling & placed embankment of excav, bor, & embank (DENS CONT)(TYB CL3) at 3:1 slope.
3. 1978 - 271-150: NE & SE quadrants between S. Main & Buffalo Speedway - removed unsuitable embank. replacing it w/excav, bor, & embank (DENS CONT) (TYB CL3) at 3:1 slope & drive treated timber pilings.

HIGHWAY	EMBANKMENT LOCATION	HEIGHT	SIDE SLOPE	TYPE OF EMBANKMENT MATERIAL
IH 610 1969: 271-120	Buffalo	20'	3:1	Compacted Embankment (DENS CONT) (COMMON)

1. Pushed and recompacted embankment.
2. 1978 - 271-156: NE & SE quadrants - excav, bor & embank (DENS CONT)(TYB CL3) at 3:1. Drove treated timber piling.
3. 1978 - 271-150: NW & SW quadrants between S. Main & Buffalo Speedway - removed unsuitable embankment replacing w/excav., bor, and embank (DENS CONT) (TYB CL3) at 3:1. Drive treated timber piling.

APPENDIX

LOCATIONS OF EMBANKMENT FAILURES IN DISTRICT 12

HIGHWAY	EMBANKMENT LOCATION	HEIGHT	SIDE SLOPE	TYPE OF EMBANKMENT MATERIAL
IH 610 1969: 271-120	Fannin	20'	3:1	Compacted Embank.-Common (DENS CONT)
	1. Pushed and recompacted embankments. 2. 1978 - 271-156: NW, SW, and NE quadrants - Removed unsuitable embankment replacing w/excav, bor, and embank (DENS CONT)(TYB CL3) at 3:1 slope. Drove treated timber piling. 3. 1980 - 27-134: Southeast quadrant - Removed unsuitable embankment replacing w/excav, bor, & embank (DENS CONT)(TYB CL3). Place 5" riprap (CONC)(CLA) w/toewall at 3.5:1 slope.			
IH 610 1964: 271-61	Gellhorn	18'	3:1	Common Roadway Excav. & Fill (Clayey Soils)
	1. Pushed and recompacted embankments.			
IH 610 1964: 271-61	Wayside to Long	20'	3:1	Common Roadway Excav. & Fill (Clayey Soil)
	1. Pushed and recompacted embankments. Lime added for stabilization.			
IH 610 1966: 271-84	Scott	20' East 16' West	3:1	Common Roadway Excav. & Fill (Clayey Soil)
	1. NE quadrant - pushed and recompacted embankment. Failed again.			

APPENDIX

LOCATIONS OF EMBANKMENT FAILURES IN DISTRICT 12

HIGHWAY	EMBANKMENT LOCATION	HEIGHT	SIDE SLOPE	TYPE OF EMBANKMENT MATERIAL
SH 146	SH 225 1962: 389-26 Three Level Interchange	20' to 251± (cut & fill)	2½:1 Max 3:1 Usual	Common Roadway Excav. & Fill

1. Failures have occured on both cut and fill slopes, repairs were made by pushing and recompacting embankments with a dozer. Lime was added for stabilization. Failure has occured since.

HIGHWAY	EMBANKMENT LOCATION	HEIGHT	SIDE SLOPE	TYPE OF EMBANKMENT MATERIAL
SH 225	Scarborough 1966: 502-11	18'±	3:1	Common Roadway Excav. & Fill w/block sodding 2' select embankments on top of common.

1. Pushed and recompacted embankments.

HIGHWAY	EMBANKMENT LOCATION	HEIGHT	SIDE SLOPE	TYPE OF EMBANKMENT MATERIAL
SH 225	Shell Overpass (S.P. RR Overpass) 1963: 502-7	30'±	SW, SE 3:1	Common Roadway Excav. & Fill. 2' select borrow on top of common.
			NW, NE 2:1	Common rdwy material 5" CLB conc. riprap.

1. Failure in SE quadrant. Pushed & recompacted embankments with dozer.

HIGHWAY	EMBANKMENT LOCATION	HEIGHT	SIDE SLOPE	TYPE OF EMBANKMENT MATERIAL
US 90	Clinton Dr.	20'	3:1	Common Roadway Excav. & Fill

1. Pushed and recompacted embankments.

APPENDIX

LOCATIONS OF EMBANKMENT FAILURES IN DISTRICT 12

HIGHWAY	EMBANKMENT LOCATION	HEIGHT	SIDE SLOPE	TYPE OF EMBANKMENT MATERIAL
Loop 201	Decker Dr. Goose Creek	FAILED DURING CONSTRUCTION REDESIGNED - SEE BELOW.		
	Project C389-13-11; Completed 6 - 1980. Original plans required an embankment w/ height of 36'± & slope 3:1. Material is Borr (TYB CL3) with a 2 ft. lift of Borr (TYC CL2). On top - asph. mulch seeding. 1. Repair through field change. Embankment was placed on marshy soil adjacent to Goose Creek. Existing soil was of low grade & could not support the weight of the embankment. The existing soil was pushed out into Goose Creek causing the embankment to drop 7 to 8 feet at centerline of future freeway. To prevent added maintenance, redesign of this section required the removal of all embankment material for both the mainlanes & frontage road & extending the bridge structures on both roadways.			
SH 225	Sims Bayou Br. 1964: 502-8 (embankment) 1966: 502-9 (bridge)	20'±	3:1	Common Roadway Excav. & Fill.
	1. Embankment was under construction (502-8) when failure occurred. Field Change No. 10 called for removing 75' of embankment from the east approach of the Sims Bayou Bridge; reducing roadway length 75' & increasing length of bridge the same.			
NASA 1 (FM 528)	Taylor Bayou Br. 1965: 981-3	15	2:1	Common Roadway Excav. & Fill
	1. Embankment failed under construction. Length of embankment was shortened, lengthening the bridge. Side slopes were reduced to 3:1.			

SLOPE STABILITY QUESTIONNAIRE

1. Type of failure (Circle A, B and/or C).

   A. Slope failure - failure within the slope.
   B. Foundation failure - bearing failure below fill.
   C. Water induced failure of cut slope.

2. Height of embankment at failure, in feet.
   29'± quadrants

3. Slope of embankment at failure.
   3:1 usual

4. Type of material in embankment in failure.
   Common roadway excavation & fill.

5. What is the liquid limit and plasticity index?
   PI = 51% max.

6. What is the percent soil binder?
   N/A

7. Was the embankment placed by density contro or ordinary compaction? If density control, what was the percent density and moisture content at placement?
   Controlled density method not less than 100% proctor density for all embankments.

8. What was the moisture content at failure?
   N/A

9. Did high rainfall immediately precede failure?
   N/A

10. Does water pond at the toe of slope?

11. How old was embankment at time of failure?
    Completed 1959

12. Location information:

    County: Harris Highway: IH 10

    Intersection or Station: Wade Rd & MoPac RR Quadrant: All

    Original: 508-14 (508-1-20)
    Reinforced Earth Walls: 508-1-157

SLOPE STABILITY QUESTIONNAIRE

1. Type of failure (Circle A, B and/or C).

   A. Slope failure - failure within the slope.
   B. Foundation failure - bearing failure below fill.
   C. Water induced failure of cut slope.

2. Height of embankment at failure, in feet.
   16' ft. East, 16 ft. West

3. Slope of embankment at failure.
   3:1

4. Type of material in embankment in failure.
   Common roadway excav. & fill

5. What is the liquid limit and plasticity index?
   P.I. = 51% (max)

6. What is the percent soil binder?

7. **Was the embankment placed by density contro or ordinary compaction? If density control, what was the percent density and moisture content at placement?**
   Density Control compacted not less than 100 % proctor density.

8. **What was the moisture content at failure?**
   N/A

9. **Did high rainfall immediately precede failure?**
   N/A

10. **Does water pond at the toe of slope?**

11. **How old was embankment at time of failure?**
    Completed in Spring 1959

12. **Location information:**

    County: Harris **Highway:** IH 10

    Intersection or Station: Thompson Road **Quadrant:** All

    Original: Plan set 508-15 (508-1-26)
    Reinforced Earth Walls: Plans 508-1-157

SLOPE STABILITY QUESTIONNAIRE

1. Type of failure (Circle A, B and/or C).

   A. Slope failure - failure within the slope.
   B. Foundation failure - bearing failure below fill.
   C. Water induced failure of cut slope.

2. Height of embankment at failure, in feet.
   30'±

3. Slope of embankment at failure.
   3:1 Usual

4. Type of material in embankment in failure.
   Common road excavation & Borrow.

5. What is the liquid limit and plasticity index?
   PI-51% Max; usual 24% - obtained from two material sources.

6. What is the percent soil binder?
   N/A

7. Was the embankment placed by density contro or ordinary compaction? If density control, what was the percent density and moisture content at placement?
   Density Control - All embankments shall be compacted not less than 100% proctor.

8. What was the moisture content at failure?
   N/A

9. Did high rainfall immediately precede failure?
   N/A

10. Does water pond at the toe of slope?
    N/A

11. How old was embankment at time of failure?
    Completed 1959

12. Location information:

    County: Harris Highway: IH 10

    Intersection or Station: Gulf Plant Road and S. P. RR Overpass Quadrant: All

    Original: Plan set 508-14, Control 508-1-21
    Reinforced Earth Walls:

SLOPE STABILITY QUESTIONNAIRE

1. Type of failure (Circle A, B and/or C).

   A. Slope failure - failure within the slope.
   B. Foundation failure - bearing failure below fill.
   C. Water induced failure of cut slope.

2. Height of embankment at failure, in feet.
   24'±

3. Slope of embankment at failure.
   Embankment - 2:1
   Top Soil - 3:1 usual, 2:1 max.
4. Type of material in embankment in failure.
   Select embankment material (Clayey soil)
   Block sodding
5. What is the liquid limit and plasticity index?
   N/A

6. What is the percent soil binder?
   N/A

7. Was the embankment placed by density contro or ordinary compaction? If density control, what was the percent density and moisture content at placement? Density control.
   Embankment - 100% proctor; topsoil - 90% proctor.

8. What was the moisture content at failure?
   N/A

9. Did high rainfall immediately precede failure?
   N/A

10. Does water pond at the toe of slope?
    Yes

11. How old was embankment at time of failure?
    Constructed 1960 to 1962

12. Location information:

    County: Harris Highway: IH 610

    Intersection or Station: Westpark Quadrant: SW

    Original: 271-17-1,2 (271-52, 271-53)
    Reinforced Earth Walls:
    Repairs: 27-13-97
    271-17-71 (Field change for double walls)

SLOPE STABILITY QUESTIONNAIRE

1. Type of failure (Circle A, B and/or C).

   A. Slope failure - failure within the slope.
   B. Foundation failure - bearing failure below fill.
   C. Water induced failure of cut slope.

2. Height of embankment at failure, in feet.
   Heights vary. Some are cuts, others fills 20' to 25' max.

3. Slope of embankment at failure.
   Max 2.5:1
   Usual 3:1
4. Type of material in embankment in failure.
   Common road excavation & fill.

5. What is the liquid limit and plasticity index?
   N/A

6. What is the percent soil binder?
   N/A

7. Was the embankment placed by density contro or ordinary compaction? If density control, what was the percent density and moisture content at placement?
   Controlled Density - Not less than 100% proctor density on all embankments.
8. What was the moisture content at failure?
   N/A

9. Did high rainfall immediately precede failure?
   N/A

10. Does water pond at the toe of slope?

11. How old was embankment at time of failure?
    1949 to 1953

12. Location information:

    County: Harris Highway: SH 146

    Intersection or Station: SH 225 Quadrant: ________

    Original:
    Reinforced Earth Walls:
    Three level interchange
    Controls: 389-12-6, 7; 502-1-11
    Plan set: 389-26

VARIOUS EMBANKMENT COSTS

FOR MAINTENANCE AND REHABILITATION

DATE	PROJECT DESCRIPTION	PROJECT COST
1969	Slide repairs on US 59 (Southwest Freeway) for Entire Year (Estimated)	$ 200,000
	Repairs elsewhere in Harris County	$ 75,000 to 100,000
1974	Maintenance cost for slide repairs for District 12	$ 30,371
1975	Maintenance cost for slide repairs for District 12 (Estimated)	$ 52,400
1977	Embankment rehabilitation on IH 610 between South Main and Buffalo Speedway	$ 122,000
1979	IH 45 @ Crosstimbers - Construction of retaining walls for U-turns (cost of U-turn included)	$ 221,464
1979	Estimated cost for embankment rehabilitation on IH 45 @ West Road	$ 525,000
1980	Reinforced earth retaining walls on IH 10 @ Wade Road and Thompson Road	$ 3,204,496
1980	Embankment rehabilitation cost; planset 27-134 US 59 @ Bellaire; US 59 @ IH 612 S; IH 610 S @ Westpark, Kirby Dr., Fannin, M. L. King Blvd, Crestmont, Broad, Telephone and Woodridge.	$ 1,307,853.50

Section H

APPENDIX B
(As Attachment Referenced in Meeting of February 17, 1983)
Slope Stability Analysis of Embankments

Slope Stability of Highway Embankments

Scope:

The scope of this report is to analyze slope stability of highway embankments in District 12 in order to develop a maximum height of embankment to be built in District 12 as our general rule of thumb in engineering practice.

Location of Existing Embankment:

Several existing embankments in District 12 area were selected for the analyses. Locations of the embankments are as follows:

1. Montgomery County
   IH 45 at FM 830 underpass
   Control: 675-8

2. Harris County
   IH 10 at Wirt Road overpass
   Control: 271-7

3. Harris County
   IH 45 at Friendswood Extension underpass
   Control: 500-3

4. Harris County
   IH 10 at Wade Road overpass
   Control: 508-1

Methodology:

Each embankment is analyzed by two methods, Method (A) computer program based on the method of slices analysis for embankment foundation stability[1] and Method (B) Taylor slope stability method[2], with the following assumptions:

1. Embankment material properties:
   a. wet density of 120 pcf.
   b. cohesion of 500 psf.
   c. angle of internal friction is 0°.

2. Surcharge load of 300 psf. (no surcharge load for Taylor method).

Summary:

The results of the analyses by Method (A) are shown in Table 1. Based on the results of Method (A), the following conclusions can be made:

1. Embankments with factor of safety equal or greater than 1.5 are performing satisfactory.

2. Embankment with factor of safety approximately 1.0 has slope stability problem.

Embankment Location	FM 830 Underpass	Wirt Road Overpass	Friendswood Extension Underpass	Wade Road Overpass
Embankment Height	20.5	20.5	20	29
Side Slope	3:1	3:1	3:1	3:1
Factor of Safety	1.71	1.72	1.75	0.90
Remarks	No failure since completion	No failure since completion	No failure since completion	Failure

Table 1. Slope stability analysis by Method (A) Conventional Slices Method[1].

The results of the analyses by Method (B) are shown in Table 2. Based on the results of Method (B), the following conclusions can be made:

1. Embankments with factor of safety equal to or greater than 2.0 are performing satisfactory.

2. Embankment with factor of safety approximately 1.0 has slope stability problem.

Embankment Location	FM 830 Underpass	Wirt Road Overpass	Friendswood Extension Underpass	Wade Rd. Overpass
Embankment Height	20.5	20.5	20	29
Side Slope	3:1	3:1	3:1	3:1
Factor of Safety	2.03	2.03	2.08	1.11
Remarks	No Failure since completion	No Failure since completion	No failure since completion	Failure

Table 2. Slope Stability Analysis by Method (B) Taylor Slope Stability Method[2].

Conclusion:

Method (B) does not consider surcharge load on the embankment.

The factors of safety of the analyses made by Method (A) and (B) are almost the same, with a 0.3 difference. Since Method (A) considers surcharge load on the embankment, we would prefer to use the results of Method (A).

This District is generally covered by loamy and clayey soils with poor drainage. The bearing capacity of these soils does not show a significant difference from various locations. With a 48" annual rainfall in this District, and unavailability of sandy material (Type A, $LL < 45$, and $4 < PI < 15$), we believe the past rule of thumb of this District of limiting embankment height to 20' based on our previous experience is well justified from the viewpoint of the geotechnical engineer.

References:

1. Electronic Computer program for Stability Analysis of Slopes and Embankment Foundations, U.S. Department of Commerce, Bureau of Public Roads.

2. Terzaghi, "Theoretical Soil Mechanics", 1943, pp. 144-181.

EXHIBIT C

REPORT ON RESEARCH AND INTERIM RECOMMENDATIONS FOR EMBANKMENT SLOPE DESIGN IN DISTRICT 12

by Dr. Stephen G. Wright

TECHNICAL MEMORANDUM

COOPERATIVE RESEARCH PROGRAM • TEXAS STATE DEPARTMENT OF HIGHWAYS AND PUBLIC TRANSPORTATION

CENTER FOR TRANSPORTATION RESEARCH
THE UNIVERSITY OF TEXAS AT AUSTIN
AUSTIN, TEXAS 78712

TO: Frank Y. Wadlington
District 12 Design Engineer

FROM: Stephen G. Wright

SUBJECT: Report on Research and Interim Recommendations for Embankment Slope Design in District 12

Study No: IAC (82-83) 2187

Area No:

Date: November 1, 1983

INTRODUCTION

In February of 1983 personnel from the Center for Transportation Research visited a number of sites in Texas SDHPT District 12 and Houston Urban to observe slides, which had occurred in earth embankments. The problems with earth slopes were discussed with personnel of the Texas SDHPT and it was determined that a major problem existed with earth embankments constructed of highly plastic clays. In the case of such embankments, slides typically occurred a number of years after construction and the failures would be described as "long-term" failures in conventional geotechnical engineering terminology. It was evident that some modification to existing design practice was needed and that before designs could be improved further knowledge was needed of the long-term ("drained") shear strength properties of typical soils where stability problems occurred. In response to these needs, a study was initiated by The University of Texas Center for Transportation Research through an Interagency contract with Texas SDHPT District 12. The results of the study are the subject of this Technical Memorandum.

cc: Jon Underwood, D-10R
Harold Albers, D-9
Bob Mikulin, D-8
Omer Poorman, District 12

cc: Research Engineer, File D-10

2.510 ERNEST COCKRELL, JR., HALL

PHONE (512) 471-1414

SITE SELECTION AND SAMPLING

Two sites were selected for detailed study including laboratory testing. The first site was at Interstate 610 and Scott Street; the second site was at the intersection of State Highways 146 and 225. Both were sites of embankments where slides had occurred and at the time of this study the slides had not been repaired. In fact, at the SH 146 and SH 225 sites there were four slides, three in embankments and one in a cut slope. A slide in the embankment in the southwest quadrant of the site was chosen for this study; the other slides at the SH 146 and SH 225 site were not examined in detail.

Samples of fill material were taken from the two sites selected and brought to The University of Texas for further testing. The samples were all disturbed, bag samples. Care was taken to avoid taking samples of what appeared to be a surface plating of topsoil. Visual inspection of the samples from each site showed that there were apparently two distinct types of soil at each site based on color. There was a grey clay, referred to subsequently as "grey" clay, and a reddish-brown-to-brown clay, referred to subsequently as "red" clay. Atterberg limits were performed on both the red and grey clays from both sites and the results are summarized in the attached Table 1. Referring to this table it can be seen that the red clays from the two sites are very similar in terms of their Atterberg Limits (Liquid limit approximately 70; Plasticity index approximately 50). The grey clays from the two sites are also similar (Liquid limit approximately 55; Plasticity index approximately 37).

Based on the Atterberg limits, the red clay was judged to be potentially the worst of the two clays with respect to drained shear strength and slope stability. Accordingly, the emphasis in additional testing was placed on the red clay. In addition, because of the close similarities in the Atterberg limits of the red clays from the two sites, clay from only one of the sites was used for the bulk of the testing. The red clay from the Interstate 610 and Scott Street slide was chosen for most of the additional testing.

COMPACTION AND STRENGTH TESTING

Compaction tests were performed using both the ASTM D-698 ("Standard Proctor") and Texas SDHPT Test Method Tex-113-E compactive efforts. The actual compactive efforts used for the red and grey clays with the Texas method were 4 and 5 ft. lbs/cu. in., respectively. Compaction curves for the two compactive efforts are shown in Figures 1 and 2 for the red and grey clays, respectively. Based on the compaction data a dry unit weight of approximately 95.5 (plus and minus 1.0) lbs/cu ft was adopted for compaction of additional specimens of the red clay to be used in triaxial strength testing. This density (95.5 pcf) corresponds to approximately 95 percent of the ASTM D-698 maximum dry density and nearly 100 percent of the maximum dry density obtained using the compactive effort determined for the Texas method. Although the original placement density of the soils in the embankments studied is not known, the selected density of 95.5 pcf is believed to represent a reasonable minimum value for a properly constructed fill.

Triaxial specimens, 1.5 inch in diameter by approximately 3.0 inches in height, were compacted in the laboratory using a compactive effort which would produce the desired final density. Specimens were then placed in a triaxial cell and saturated by standard back-pressure saturation techniques. Following saturation, the specimens were brought to a desired final effective consolidation pressure, which ranged from 1 to 20 psi depending on the individual test and specimen. During saturation and adjustment to the desired final effective consolidation pressure, the specimens generally either swelled or exhibited no significant volume change, depending on the pressures applied; no significant volume decrease was observed for any of the specimens tested. Specimens were sheared using both consolidated-undrained ($C\overline{U}$,$\overline{R}$) and consolidated-drained (CD,S) test procedures. From the results of the triaxial shear tests effective stress shear strength envelopes were determined. These envelopes are summarized in terms of effective stress cohesion ($\bar{c}$) and friction angle ($\bar{\phi}$) values in Table 2. An "average" envelope, as well as "upper-bound" and "lower-bound" envelopes are summarized in this table. Relatively little scatter was observed in the shear strength data and the upper- and lower-bound envelopes represent extremes in the scatter.

SLOPE STABILITY ANALYSES

Several series of slope stability analyses were performed for the long-term stability condition using the effective stress ("drained") shear strength parameters summarized in Table 2. Analyses were performed for conditions believed to be representative of those at the time of the slides at each of the two sites selected for study. Parameters employed in the analyses are summarized in Table 3. The slope angles shown are estimates based on actual measurements of slope angles taken at the sites immediately adjacent to the slide areas.

The first series of slope stability analyses was performed using the three shear strength envelopes (upper-bound, average and lower-bound) summarized in Table 2 and assuming zero pore water pressures. The results of the first series of analyses for the two sites are summarized in Table 4, where the factors of safety are shown for the various shear strength envelopes and sites considered. The second series of slope stability analyses was performed using only the "average" shear strength envelopes, but assuming pore water pressures were equal to 9, 20, 40 and 60 percent of the overburden pressures (i.e. r_u - 0.0, 0.2, 0.4 and 0.6) everywhere within the slope. Results of the second series of stability calculations are summarized in Table 5. The results of all of the stability calculations summarized in Tables 4 and 5 indicate that the slopes are stable, i.e. all factors of safety shown exceed unity. The results of computations with pore water pressure ratios, r_u, of 0.4 and 0.6 represent what are believed to be unreasonably high pore water pressures and ones which are believed to be improbable for the sites examined. Discounting results of the analyses with r_u equal to 0.4 and 0.6, the analyses indicate that the factor of safety at the two sites should have been at least 1.8 and very likely as great as 2.0 or more.

DISCUSSION

To gain insight into why such high factors of safety were calculated for slopes which actually failed, shear strengths were back-calculated using the available knowledge of the slope and slide geometries at the two sites. Strengths were back-calculated using pore water pressure ratios, r_u, of 0, 0.2 and 0.4, although only the lower values (0 and 0.2) are believed likely. The back-calculated strengths, expressed as effective stress values of cohesion ($\bar{c}$) and friction angle ($\bar{o}$) are summarized in Table 6. Comparisons of the values of shear strength parameters shown in Table 6 with the measured values shown previously in Table 2 shows significant differences between measured and back-calculated cohesion values. Measured cohesion values ranged from approximately 140 to 330 psf while back-calculated values were almost an order of magnitude smaller, ranging from only 10 to 16 psf. In contrast, measured and back-calculated effective stress friction angles showed good agreement and could be considered virtually identical in view of the uncertainty involved in assuming values of pore water pressures to back-calculate shear strengths.

The discrepency between measured and back-calculated effective stress cohesion values is not believed to be due to errors in the laboratory tests. While piston friction in the triaxial cell and the strength of the filter paper and rubber membrane surrounding the specimens could contribute to an apparent cohesion, these factors are not believed to have contributed to the measured cohesion reported herein: the triaxial cells used for the testing were designed to produce only relatively low piston friction, and corrections to account for the strength of the filter paper and rubber membrane were examined and found to be very small. If loading rates in the shear tests were too fast, a high cohesion value might also be observed; however, loading rates in the tests performed are believed to have been substantially slower than required. Thus, loading rate does not appear to explain the relatively high cohesion values which were measured.

At the present time the reason, or reasons, for the discrepancy between measured and back-calculated effective stress cohesion values is unresolved. However, it is clear that the relatively high effective stress cohesion values derived from the laboratory tests do not apply to the field. Strong evidence from the two sites studied, as well as evidence from other sites examined less thoroughly, indicate that there is a negligible effective stress cohesion component of shear strength in the field. Possibly progressive failure with reduced, "residual" shear strengths being developed with time or some other mechanism, which is related to time and not reproduced in laboratory tests, causes the loss of "cohesion." Certainly there is no fundamental reason why an effective stress cohesion component of strength should exist in compacted clays of the type examined.

SUMMARY AND RECOMMENDATIONS

Significantly more basic and applied research is needed and strongly encouraged to answer some of the questions which have been raised by the studies described above. However, regardless of results of future research it is apparent that immediate changes in design practice for earth embankments constructed of highly plastic clays are warranted for District 12. Field evidence from slopes as flat as 3 (horizontal)-to-1 (vertical) indicates that a number of such slopes are not stable and suggests that an effective stress friction angle of approximately 20 degrees and zero cohesion are the maximum values that may be counted on for stability. Future testing could lead to even further reductions from these stated strength values ($\bar{o}$ = 20 degrees, $\bar{c}$ - 0).

Based on the results of work completed to date an interim recommendation is made that an embankment side slope not exceeding 4 (horizontal)-to-1 (vertical) be adopted for embankments constructed of highly plastic clays in SDHPT District 12. Specific Atterberg limits for which this recommendation should apply cannot be clearly estabilished with the limited data presently available. However, the recommendations probably apply to soils with liquid limits as low as 50 and perhaps even lower. A substantially more select material, such as sand, may be required before higher shear strengths and steeper embankment slopes can be adopted for design.

TABLE 1

Summary of Atterberg Limits on Soils from Two Selected Embankment Slope Failures in SDHPT District 12

Site	Visual Description of Soil	Liquid Limit	Plastic Limit	Plasticity Index
Scott St. and Interstate 610	"Grey" clay	54	15	39
Scott St. and Interstate	"Red" clay	71	20	52
State Highways 225 and 146	"Grey" clay	56	20	36
State Highways 225 and 146	"Red" clay	70	21	49

TABLE 2

Summary of Shear Strength Parameters ($\bar{c}$, $\bar{\phi}$)
for Effective Stress Failure Envelopes -
"Red" Clay from Interstate 610 and Scott Street Slide

Envelope	Cohesion, $\bar{c}$ (psf)	Friction Angle, $\bar{\phi}$ (degrees)
Upper-Bound	330	20.9
Average	240	21.4
Lower-Bound	140	21.7

TABLE 3

Summary of Parameters Used for "Long-Term" Slope Stability Analyses

<u>Interstate 610 and Scott Street</u>

Slope height, H = 20 feet

Slope angle, B = 22.5 degrees

Unit weight, Y = 121 pcf*

<u>State Highways 146 and 225</u>

Slope height, H = 15 feet

Slope angle, B = 18 degrees

Unit weight, Y - 121 pcf*

---

*Based on a water content of 30 percent, 100 percent saturation and an assumed specific gravity of solids of 2.70.

TABLE 4

Summary of Long-Term Slope Stability Calculations with Various Shear Strength Envelopes

Site	Strength Envelope	Factor of Safety
Interstate 610 and Scott Street	Upper-Bound	2.4
Interstate 610 and Scott Street	Average	2.1
Interstate 610 and Scott Street	Lower-Bound	2.0
State Highways 146 and 225	Upper-Bound	3.5
State Highways 146 and 225	Average	2.9
State Highways 146 and 225	Lower-Bound	2.3

Note: Zero water pressure assumed for all analyses.

TABLE 5

Summary of Long-Term Slope Stability Calculations
with Various Assumed Pore Water Pressures

Site	r_u*	Factor of Safety
Interstate 610 and Scott Street	0	2.1
Interstate 610 and Scott Street	.2	1.8
Interstate 610 and Scott Street	.4	1.6
Interstate 610 and Scott Street	.6	1.3
State Highways 146 and 225	0	2.9
State Highways 146 and 225	.2	2.6
State Highways 146 and 225	.4	2.3
State Highways 146 and 225	.6	2.0

$$*r_u = \frac{\text{Water Pressure}}{\text{Total Vertical Pressure}}$$

Note: All values based on "average" shear strength envelope.

TABLE 6

Summary of Back-Calculated
Shear Strength Parameters

BACK-CALCULATED STRENGTH VALUES

Site	r_u	Cohesion, $\bar{c}$ (psf)	Friction Angle, $\bar{\phi}$ (degrees)
Interstate 610 and Scott Street	0.0	10	18.1
Interstate 610 and Scott Street	0.2	11	22.7
Interstate 610 and Scott Street	0.4	12	30.2
State Highways 146 and 225	0.0	14	14.8
State Highways 146 and 225	0.2	15	18.6
State Highways 146 and 225	0.4	16	24.7

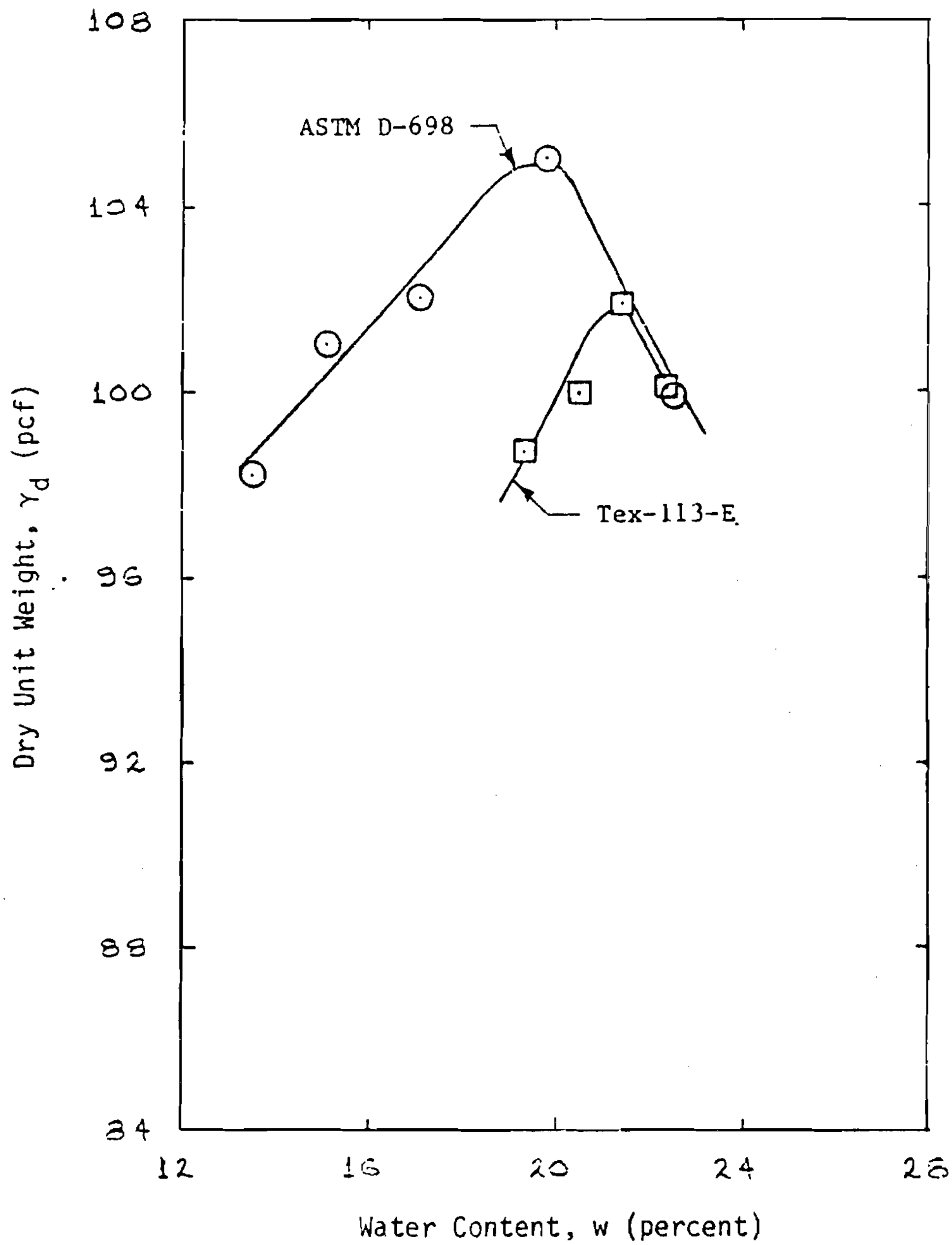

Figure 2 - Compaction Moisture-Density Curves From Grey Clay at Interstate 610 and Scott Street Slide

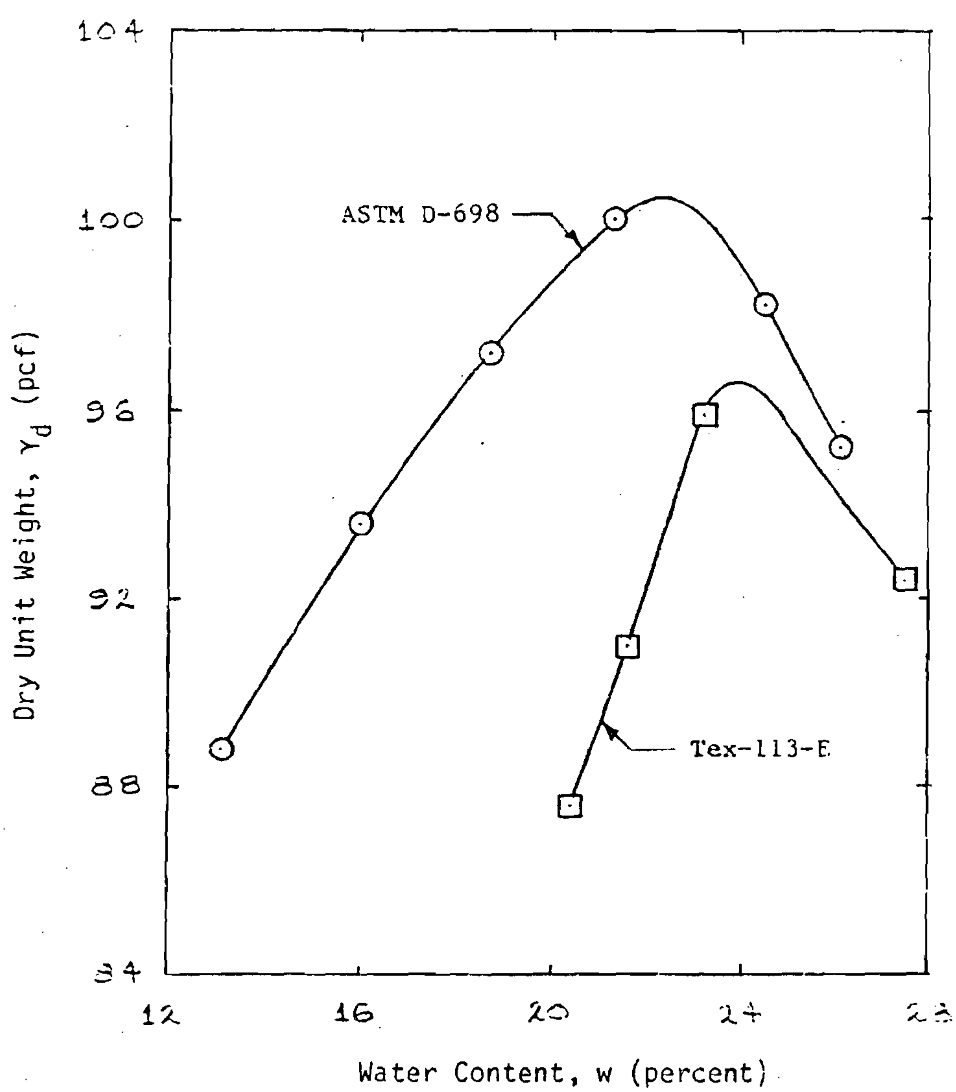

Figure 1 - Compaction Moisture-Density Curves
From Red Clay at Interstate 610
and Scott Street Slide