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 S.H. 146 and S.H. 225 site 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 S.H. 146 and S.H. 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 adpoted 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 ($C\bar{U},\bar{R}$) 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 0, 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 ($\bar{\phi}$) 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 pressure 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 discrepance 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 ($\bar{\phi}$ = 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 established 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 610 | "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, $\beta$ = 22.5 degrees

Unit weight, $\gamma$ = 121 pcf*

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

Slope height, H = 15 feet

Slope angle, $\beta$ = 18 degrees

Unit weight, $\gamma$ = 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 |

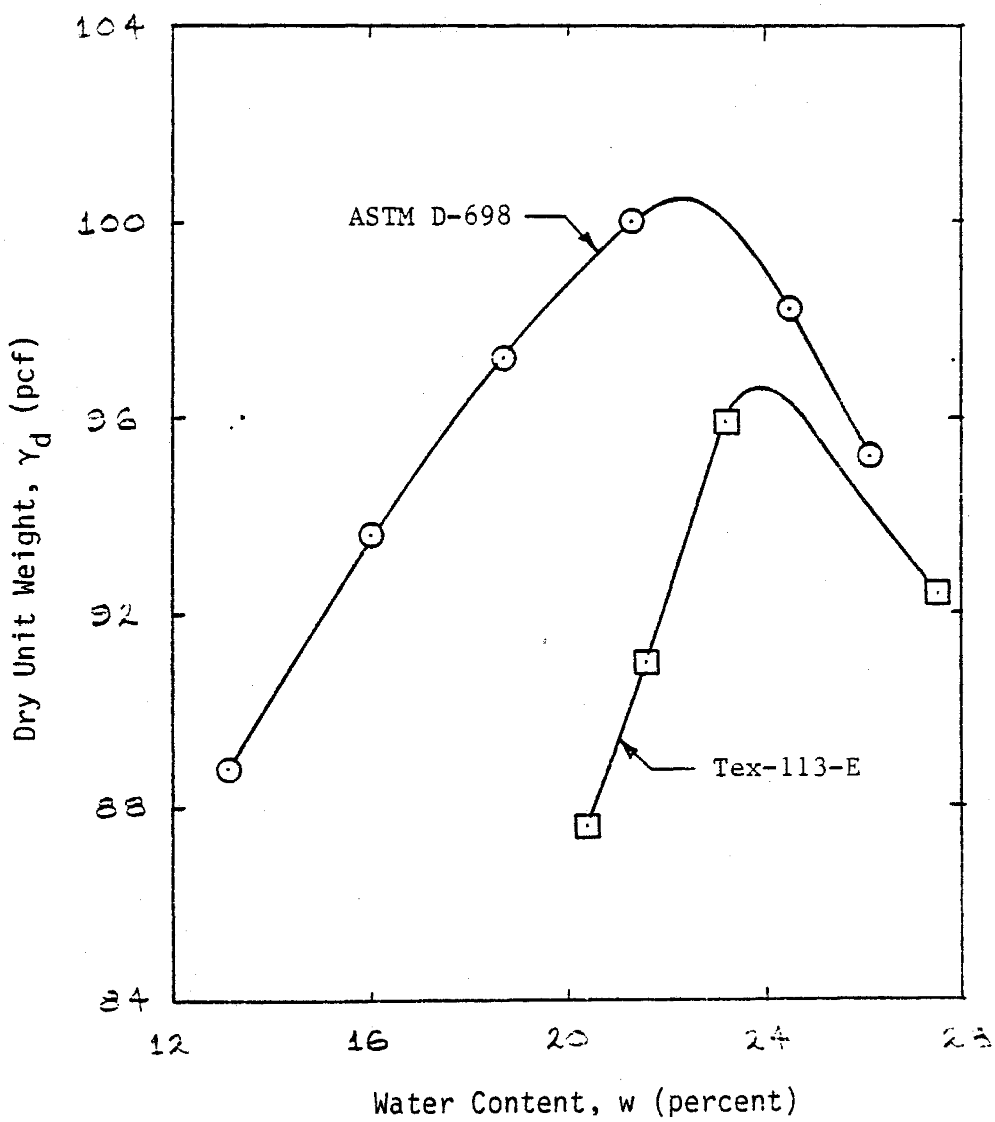


Figure 1 - Compaction Moisture-Density Curves
From Red Clay at Interstate 610
and Scott Street Slide

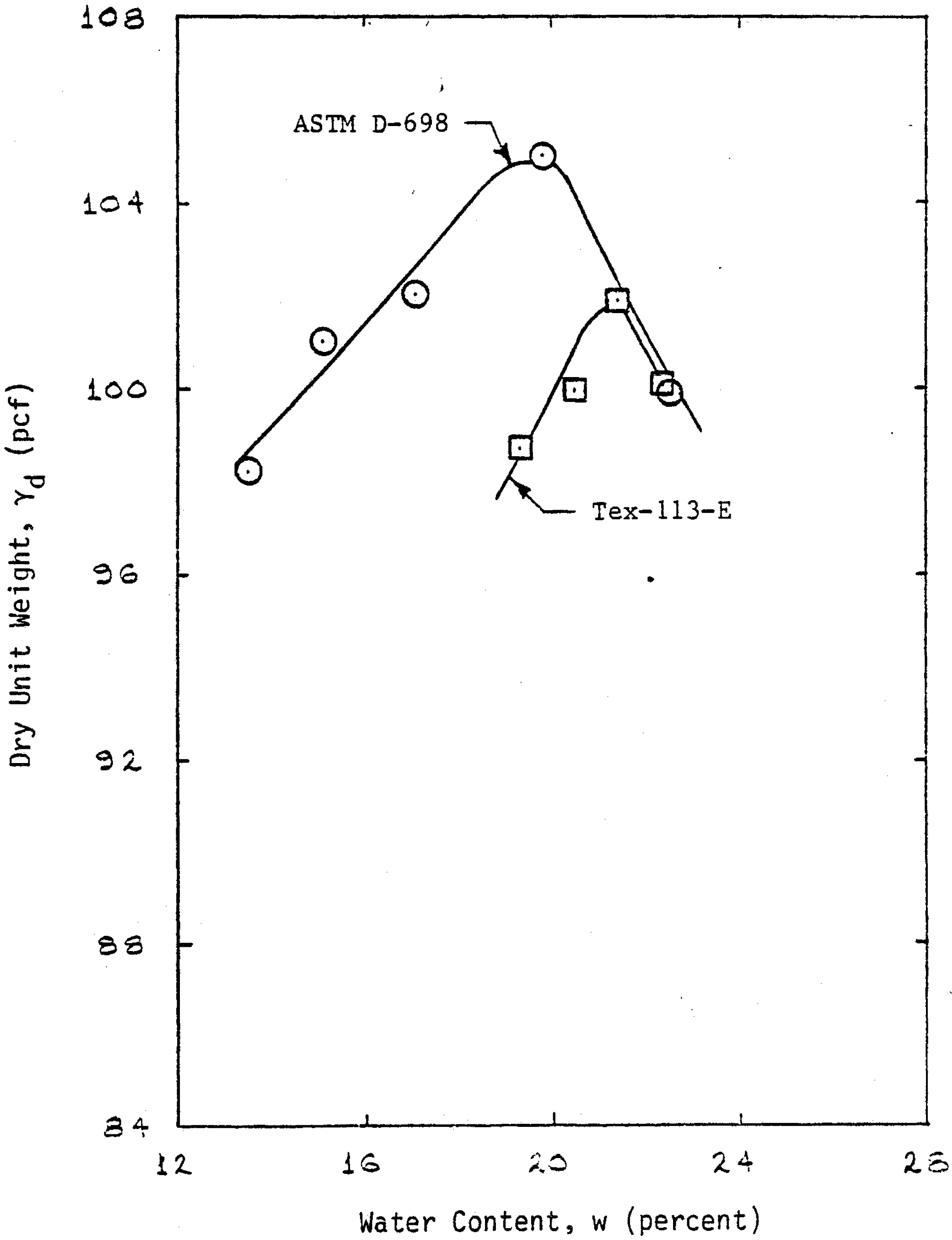


Figure 2 - Compaction Moisture-Density Curves From Grey Clay at Interstate 610 and Scott Street Slide