Soils

OKLAHOMA RESEARCH PROGRAM
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# INVESTIGATION OF BEHAVIOR OF AN EMBANKMENT

## FINAL REPORT

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by

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Oklahoma Research Program
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# INVESTIGATION OF BEHAVIOR OF AN EMBANKMENT

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## INTRODUCTION

The Citadel building of the Salvation Army, located at the northwest corner of Cheyenne Avenue and Easton Street in Tulsa, Oklahoma, has undergone severe structural damage since its construction in the late 1950's, from cracking of interior walls and movement of floor slabs, grade beams, and roof beams. During the period March, 1969, to July, 1970, a highway embankment about 25 ft high was constructed immediately to the north of the building, as part of a grade separation on I 244 over Cheyenne Avenue. It has been suggested that the structural damages may be attributable, at least in part, to the presence of the embankment.

Seeking to establish the facts of the matter, the State of Oklahoma, Department of Highways contracted with the School of Civil Engineering at Oklahoma State University, under Project No. 71-02-03, to investigate behavior of the embankment and determine if a causal relationship existed between construction of the embankment and structural damage to the Salvation Army Citadel.

The authors, acting respectively as project director and principal investigator, undertook the investigation on July 1, 1971. Results of the investigation are contained herein.

# RELEVANT INFORMATION CONCERNING THE OSHD EMBANKMENT

The embankment in question is located immediately north of the Salvation Army Citadel, along the route of I 244, and is part of the west access to the west-east/east-west four-lane grade separation bridge (OSHD Structure No. 17) over Cheyenne Avenue. Information concerning pre-design subsurface exploration, design, and construction of the embankment was obtained from official OSHD plans and from discussions with Mr. Jerry Shepherd, Soils, Foundations, and Liason Engineer of the OSHD Materials Division, who was extremely helpful in obtaining any information desired by the authors.

## Subsurface Exploration Prior to Embankment Construction

Subsurface exploration at the embankment site was carried out by the sounding crew of the OSHD Bridge Division, sometime before final FHWA approval of design plans in June, 1967. A total of nine borings were made west of Cheyenne Avenue and north of the Salvation Army Citadel. Holes were advanced by auger until hard strata were encountered, and cores of the hard strata were taken. The soil profile reported for all holes was "soft sandy clay to medium hard shale." The original ground elevation in the general area was approximately El. 716, and "medium hard shale" was encountered at about El. 700 to 708 at the west edge of Cheyenne Avenue, with the shale rising to about El. 710 to 715 some 65 ft west of the west curb of Cheyenne Avenue and 120 ft north of the Citadel.

## Detail of the Embankment

The portion of the I 244 embankment in question rises approximately 25 ft above original ground level in the vicinity of the Salvation Army Citadel. Parallel with the east edge of the Citadel, where the embankment abuts the grade separation bridge over Cheyenne Avenue, it is approximately 225 ft wide at the bottom and 125 ft wide at the top. Approximately 140 ft west, parallel to the west edge of the Citadel, the embankment is about 275 ft wide at the bottom and 190 ft wide at the top.

The toe of the south embankment slope terminated above five feet north of the Citadel north wall, and this slope is faced with concrete to prevent erosion. A concrete ditch has been placed between the Citadel and the slope toe to carry runoff to the alley west of the Citadel. The east end of the embankment slope, near Cheyenne Avenue, is contained by a circular retaining wall connected to the west abutment of the grade separation bridge. The relationship between Citadel and embankment is shown in Figs 1 and 2.

The circular slope retaining wall has a multiple foundation; the section nearest the bridge, 26.95 ft in length, is founded (at El. 705) on eleven 10BP42 steel H-piles driven into the underlying shale. The remaining 28.0 ft of the circular wall is founded on a spread footing at El. 708, designed for an allowable bearing pressure of 0.7 tons/ft$^2$.

Embankment construction was begun on March 17, 1969 and about 80% of the required fill was placed by April 23, 1969, when construction was temporarily halted. The remaining fill was placed between June 8, 1970 and July 29, 1970. In the vicinity of the Citadel, the embankment is complete except for I 244 surfacing.

Before the investigation, little data were available concerning material used in the embankment; it was assumed to be of quality acceptable to the OSHD Resident Engineer in charge of embankment construction. Detailed information on embankment materials was obtained by the authors and will be presented later. The _1967 Oklahoma State Specifications for Highway Construction_ governed placement of fill materials.

## Structural Damage Observed for the Embankment

The entire embankment in the vicinity of the Salvation Army Citadel was carefully scrutinized by the authors. Of particular interest were any indications of embankment movement. As the slope north of the Citadel is covered with concrete, clues to embankment movement should be easily found.

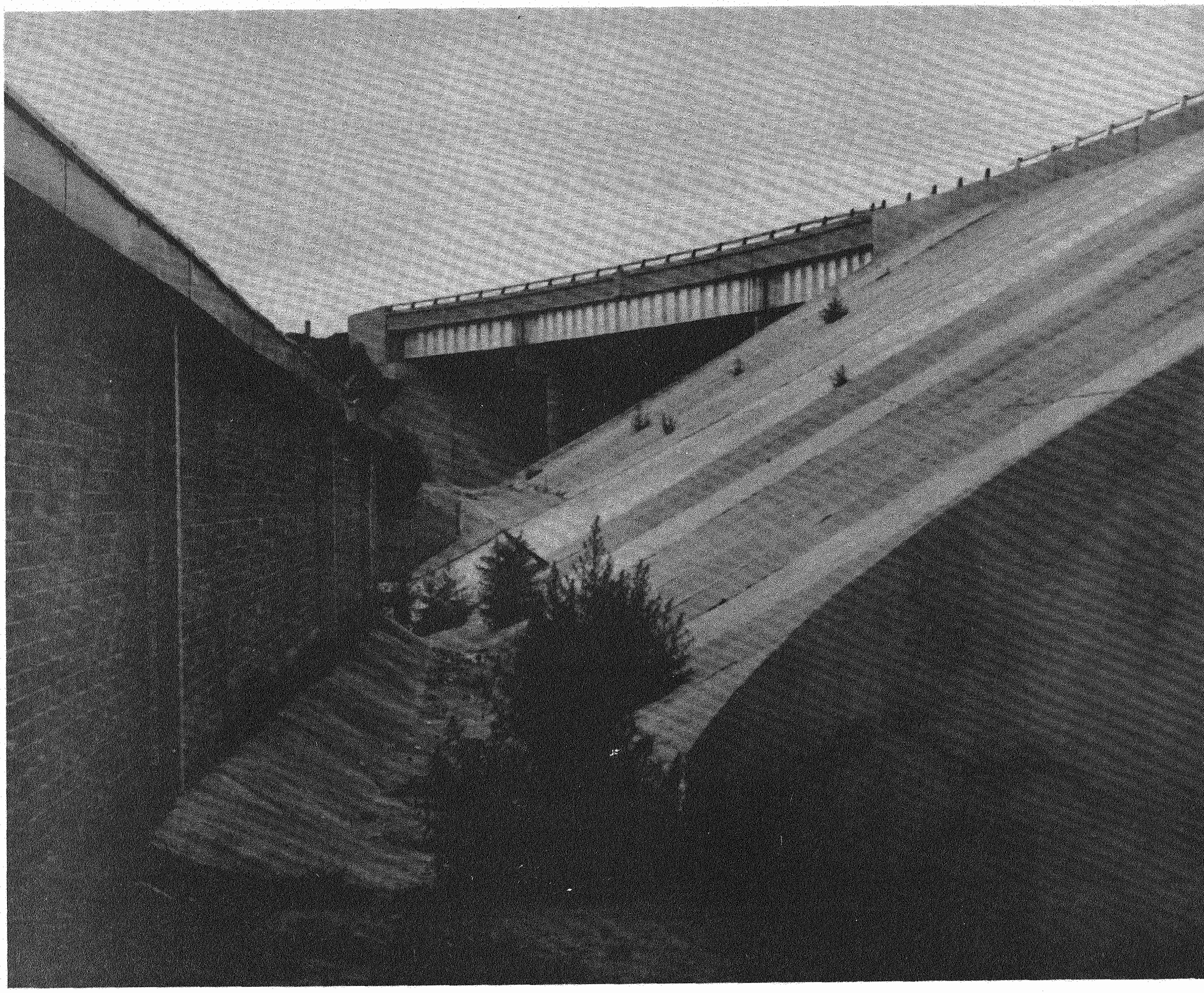

Figure 1. Relationship Between Salvation Army Citadel and OSHD Embankment, Looking West from Ground Level

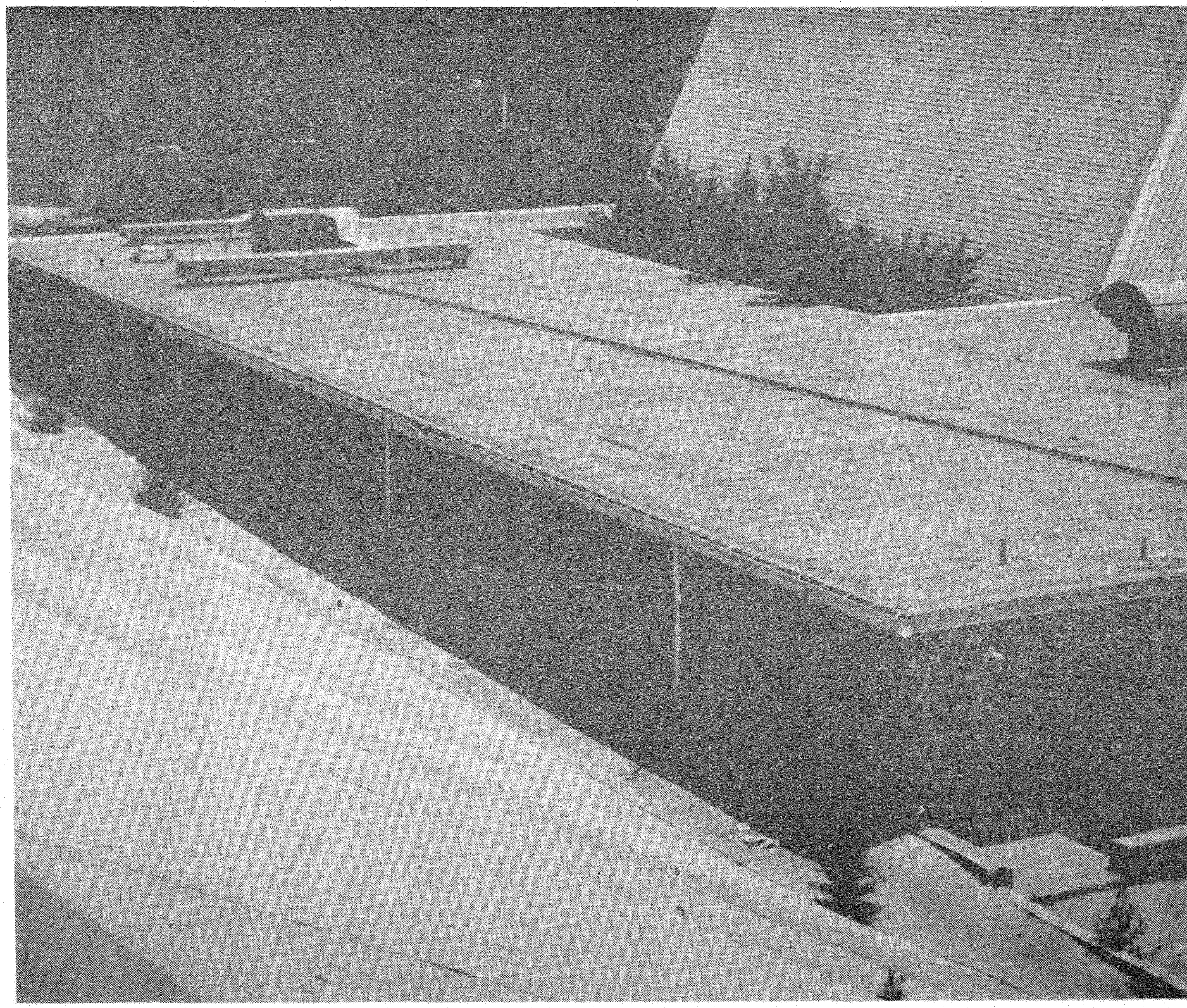

Figure 2. Relationship Between Salvation Army Citadel and OSHD Embankment, Looking East from Top of Embankment

All concrete slope panels were inspected for cracking and relative displacement, and tar-filled expansion joints between the panels were inspected for squeezing or tensile cracking. The lightly reinforced concrete ditch between the Citadel and the embankment slope toe was carefully inspected for cracking and distortion which might be caused by slope movement. Construction joints and facing of the circular slope retaining wall and bridge abutment were also inspected.

Except for some slight vertical cracking in the circular retaining wall, movements noted appeared to result only from concrete expansion and contraction. Vertical cracking in the circular slope retaining wall was observed near the transition from pile foundation to spread footing; use of a dual foundation for a continuous structure makes it likely that differential movements will occur. However, the observed differential movement was less than 1/8 in.

Inspection of the other (north) slope of the embankment produced similar results. Thus, it appears that the embankment itself has undergone little, if any, movement since its construction.

## RELEVANT INFORMATION CONCERNING THE SALVATION ARMY CITADEL

Information concerning site layout, design, and construction details of the Salvation Army Citadel was obtained from the Plot Plan and Foundation Plan & Details for the structure, provided by McCune & McCune, and dated August 2, 1957. Additional information concerning the building was provided by Mr. James G. McDonald of McCune & McCune.

### Description of the Site Before Citadel Construction

The site of the Salvation Army Citadel was originally a residential area, with 110 ft frontage on Cheyenne Avenue and 140 ft frontage on Easton Street. Original ground contours, converted from relative to absolute elevation, were taken from the Plot Plan and are shown in Fig 3, superimposed on the outline of the Citadel. As

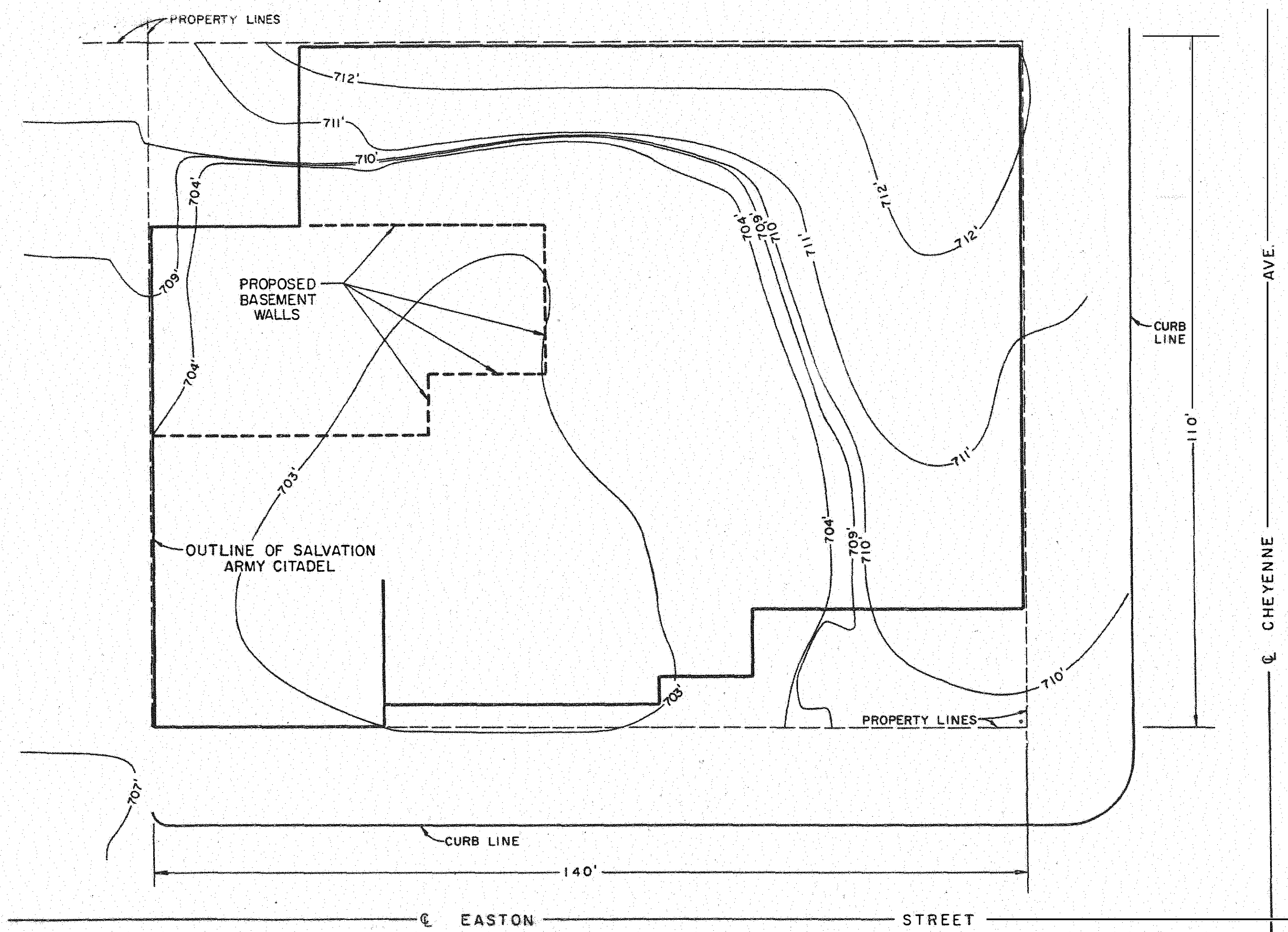


Figure 3. Original Ground Contours at the Salvation Army Citadel Site

the direction of movement; the cracking and patches are shown in Fig 5. Similar cracking was observed at the northwest top corner of the building (kitchen storage area) and at the southeast top corner outside Classroom No. 1.

Vertical hairline cracking was also noted at the ends of the facing outside the south wall of Classroom No. 1, but the horizontal cracking noted in the concrete block walls was not noted in the exterior brick facing, even opposite the west wall of the kitchen and the north wall of the living quarters. A tendency for outward bulging of the upper west wall and south wall brick facing was noted, but this could possibly be attributed to poor bricklaying technique.

### Summary

While structural damage exists over a considerable part of the building, the greatest damage is concentrated in three areas. Areas of greatest damage are the north wall of the living quarters, the west wall of the kitchen, and the south wall and roof of Classroom No. 1. Behavior at these three areas will be analyzed in detail in a later section.

## SUBSURFACE EXPLORATION AT THE SITE

Besides the pre-construction explorations by the OSHD Bridge Division and McCune & McCune, additional explorations were made by the OSHD and the authors. One set of soil borings was made on May 25, 26, and 27, 1971 by the drilling crew of the OSHD Materials Division, under the general supervision of Mr. Jerry Shepherd. A Failing 1250 drilling rig was used to advance a 5-inch fishtail bit and cuttings were removed by compressed air lift. Borings were made adjacent to the northeast and southeast corners of the Salvation Army Citadel and through the embankment along the W-E centerline of I 244, north of the northeast and northwest corners of the Citadel. Borings were logged by textural classification and ASTM Standard Penetration Tests were made on strata encountered. Gow spoon samples were taken and final boring logs prepared by correlating field logs with laboratory test results on obtained samples.

Borings were terminated upon encountering hard shale strata. The logs obtained were in close agreement with those obtained by the authors at the same locations, as described below.

On July 12, 13, 14, and 26, 1971, a total of seven borings were made by the authors at the site; site layout and boring locations are shown in Fig 6. Borings were made by the drilling crew of the OSHD Materials Division, using a Failing 1250 drilling rig to advance a 5-inch fishtail bit. Cuttings were removed by compressed air lift. Drilling and sampling operations were conducted under the supervision of the authors and Mr. Jerry Shepherd of the OSHD Materials Division. Borings were logged by the authors.

At Borings No. 1 through 6, continuous Shelby tube "undisturbed" sampling was carried out where practicable, using tubes made from 3.00 in. OD and 2.875 in. ID seamless steel tubing, cut from tubing stock by the OSHD Materials Division. Both blunt and sharpened tubes were used, and all tubes were coated before use with Ultraflex Amber microcrystalline wax in the Soil Mechanics Laboratory of the OSU School of Civil Engineering, to aid in obtaining and extracting more nearly undisturbed soil samples.

Two samples 12 in. in length were usually obtained every 3.0 ft by hydraulic push. A total of 69 Shelby tube samples were taken at the six borings; an exact sampling log is included as Appendix 1. The large number of samples was taken to insure that the subsurface profile and engineering properties of the soils could be determined as completely as possible.

At Boring No. 7, made to the north of the embankment, the subsurface profile was determined but no Shelby tube samples were taken. All borings were terminated upon encountering hard laminated blue shale or clay-shale.

## RESULTS OF FOUNDATION EXPLORATION

### General

Principal shallow subsurface formations in the Tulsa area are those of the Pennsylvanian Period of the Paleozoic Era. These formations are fairly extensive and their existence is well-known

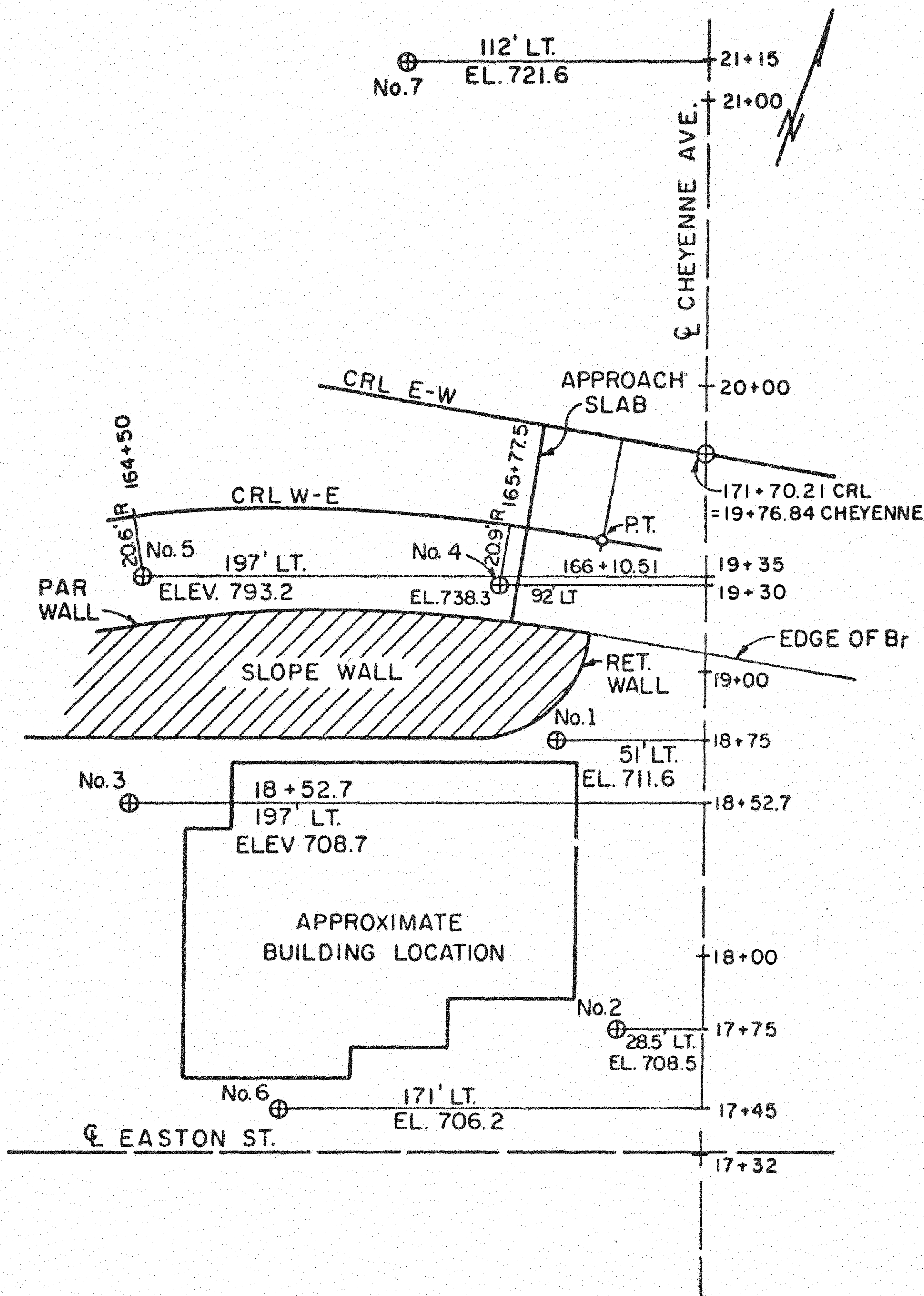


Figure 6. Site Layout and Boring Locations

to the authors. The profile typically encountered below topsoil consists of a layer of stiff to very stiff yellow/tan laminated clay, weathered from its parent material, a yellow/tan laminated clay-shale of very stiff to hard consistency that is the next formation encountered. Clay-shale is the name given to strata that are in an intermediate stage of weathering or consolidation between clay and shale; they are usually too hard to be sampled by conventional Shelby tube methods and too soft to be cored using conventional soft formation bits.

The yellow/tan laminated clay-shale usually becomes harder with depth, and takes on a yellow/tan/blue color. Next, blue to blue-gray laminated clay-shale or shale of hard to very hard consistency is encountered and may be up to several hundred feet in thickness. The blue shale is generally encountered between 10 and 40 ft below the surface in the Tulsa area.

Laminations of all strata are usually horizontal, and both clay-shale and shale formations are subject to rapid degradation when exposed to open weathering. Clays weathered from the parent shales are usually highly overconsolidated and preconsolidation pressures may range between 2 and 5 tons/ft$^2$.

## Soil Profile Around the Citadel

Borings No. 1, 2, 3, and 6 were made at the four corners of the Citadel, on public property just outside the Citadel property lines, as shown in Fig 6. Field Boring Logs from these holes (shown in Figs 7 and 8) indicate a pattern of varied Quaternary alluvial deposits overlying the hard laminated blue clay-shale/shale of the Pennsylvanian Period. Below the silty topsoil in Borings No. 1, 2, and 6, and below the concrete pavement and sand cushion in Boring No. 3, assorted layers of moist to very wet sandy clay, clayey sand, silty sand, silty clay, clayey silt, fine and coarse sand, and clay were found. Transitions from one soil type to the next were gradual rather than distinct, except when the hard shale strata were encountered. The nature of most soil was basically stiff and sandy, and most sandy strata were dense enough so that Shelby tube samples could be obtained, even below the water table.

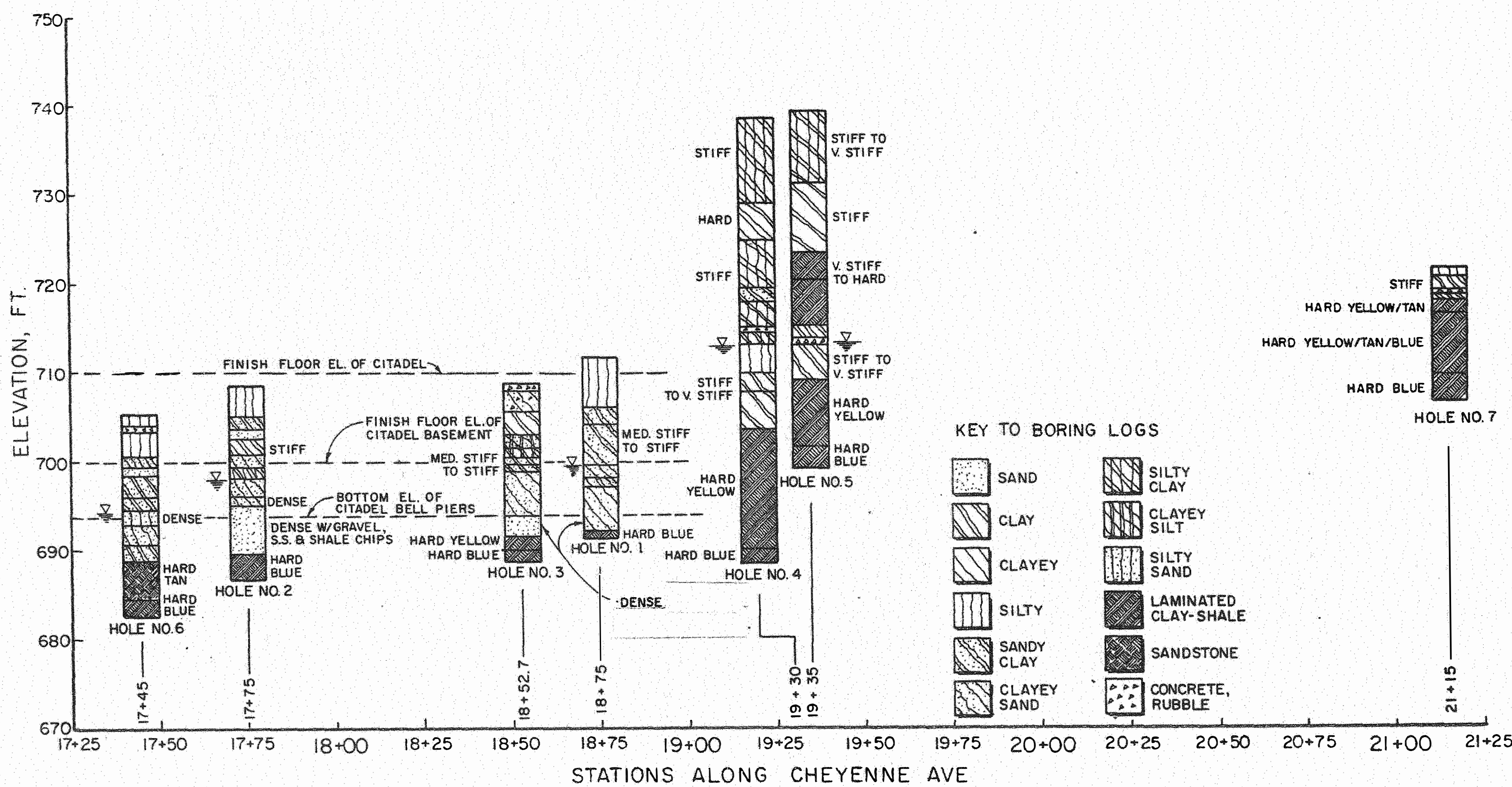


Figure 7. Field Boring Logs, Referenced to Stations Along Cheyenne Avenue

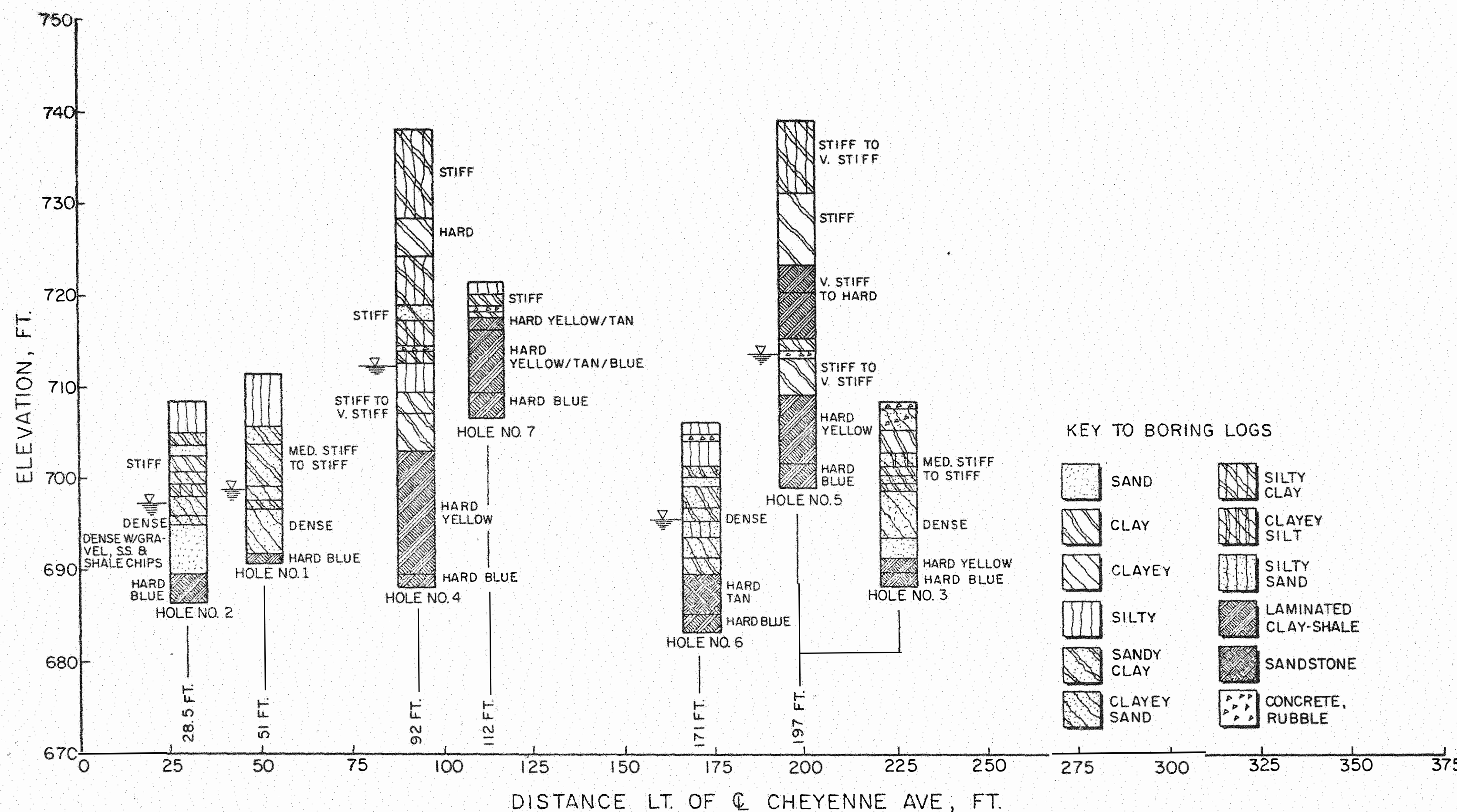


Figure 8. Field Boring Logs, Referenced to Distance Left of Cheyenne Avenue

The clay content of the basically sandy soils was very sticky, yellow/tan and blue/gray in color, and undoubtedly was eroded and weathered from nearby parent Pennsylvanian Period shale. In many cases, clay "spots" of blue color were encountered, as if a chip of blue shale has been surrounded and weathered.

The soil profile encountered is what one might expect to find in a sedimented stream channel. This hypothesis is further reinforced by the fact that the first Pennsylvanian Period strata encountered in Borings No. 1, 2, and 6 was the hard blue clay-shale/shale, indicating that the softer yellow/tan strata above it had been eroded by water action. In Boring No. 3, 1.7 ft of stiff laminated yellow/tan clay or clay/shale was found above the blue shale, indicating that perhaps the stream bank began at this location.

Blue clay-shale or shale was encountered at El. 692.3 in Boring No. 1, El. 689.5 in Boring No. 2, and El. 689.7 in Boring No. 3. In Boring No. 6, a layer of hard tan sandstone was encountered at El. 689.5, overlying the blue shale which began at El. 685.2. Water was encountered in all four of the borings around the Citadel, and Borings No. 1, 2, and 6 made water during exploration. A flow of several gallons per minute was blown from Boring No. 2 for approximately five minutes, and water in Boring No. 6 just above the sandstone caused sloughing of sandy material, requiring considerable cleaning of the hole during drilling. The water table at the site was found to exist between El. 695.2 and El. 699.2, 24 hrs. after drilling.

## Soil Profile Through and Under the Fill

Borings No. 4 and 5 were made through the fill to the hard blue shale of the Pennsylvanian Period, and were located about seven feet north of the south top edge of the fill and as close as possible to north-south lines through Borings No. 1 and 3. Boring No. 7 was made at ground level immediately north of the fill and approximately half-way between Borings No. 4 and 5. Exact locations are shown in Fig 6.

As may be noted from the boring logs of Figs 7 and 8, the fill is composed primarily of stiff to very stiff, yellow and yellow/tan, silty shaley compacted clay. The fill material was probably obtained from cut at nearby locations on I 244. In Boring No. 5 layers of brown clay, probably from the upper part of a cut, and hard blue and yellow/tan clay-shale, probably from the lower part of a cut, are intermixed with the yellow silty shaley clay.

Climatic conditions and highway construction procedures in Oklahoma usually result in soil compaction dry of Standard Proctor (AASHO T-99) optimum moisture content, and the fill material appeared to be dry of optimum, except for a few thin layers at about optimum moisture. Thus, it is doubtful that the fill has gained or lost an appreciable amount of moisture since its construction.

The concrete/rubble layer found at El. 714.8 in Boring No. 4 and El. 713.9 in Boring No. 5 probably indicates the original ground level. This indication is further confirmed in Boring No. 4 by the silty topsoil existing between El. 713.0 and 709.6, similar to that found in Boring No. 1, except much stiffer.

Subsurface strata found in Boring No. 4 are typical of Pennsylvanian Period deposits; below the topsoil yellow/tan clay is encountered, becoming yellow/tan clay-shale with increasing traces of blue. Hard laminated blue shale is found at El. 689.8. Subsurface strata in Boring No. 5 are similar, except for a lack of topsoil. Blue shale is encountered at El. 701.6. Two weeks after exploration, the water table in both borings was found to exist at El. 713, approximately the original ground line.

Boring No. 7 also exposed typical Pennsylvanian Period deposits, and blue shale was found at El. 710.1, or 11.5 ft below the surface.

## Summary

Based on soil profiles obtained during exploration and on soil descriptions and classifications determined during the laboratory testing program, it appears that the Salvation Army Citadel is built on an old stream channel sedimented with assorted Quaternary alluvial deposits overlying Pennsylvanian Period hard, laminated blue clay-shale/shale. The highway embankment proper is built on Pennsylvanian Period deposits of stiff to very stiff laminated yellow/tan clay overlying very stiff to hard laminated yellow/tan clay-shale overlying hard to very hard laminated blue shale. Thus, the fill appears to be located on the bank of the old stream, and the transition from bank to stream channel occurs somewhere under the south slope of the fill.

## LABORATORY TESTING OF SOIL SAMPLES

Laboratory tests on Shelby tube soil samples were carried out in the Soil Mechanics Laboratory of the OSU School of Civil Engineering, by the authors and Mr. Donald R. Snethen, soils doctoral candidate and Research Assistant on the project.

An extensive series of tests were conducted, with a view toward establishing relevant engineering behavior of all soils encountered in exploration. Tests conducted included natural moisture content, Atterberg limits, compaction, consolidation, direct shear, and unconfined compression. In most cases, strength and consolidation tests were carried out on 2.875 in. OD samples to minimize sample disturbance from trimming.

## RESULTS OF LABORATORY TESTING

Test data were reduced and evaluated by the authors and Mr. Donald R. Snethen. Test results are presented and discussed below by Boring No., and will be referred to in later sections.

### Boring No. 1

Test results from Boring No. 1 are shown in Table 1. Despite differences in color and texture during exploration, a rather uniform plasticity profile exists, with natural moisture contents near or slightly above the plastic limit. Between El. 705 and 692 the profile may be classified CL-SM or SM-CL by the Unified Soil Classification System.

The direct shear data substantiate Field Boring Logs of Figs. 7 and 8, as below El. 706 a pattern (simplified) of sandy clay overlying clayey sand is noted. Relatively high values of cohesion and

TABLE 1. SOIL TEST RESULTS - BORING NO. 1

| Elevation (ft) | Boring Depth (ft) | $w_{nat}$ (%) | LL | PL | PI | Direct Shear c (psi) | Direct Shear $\phi$ (deg) | Unified Classification |
|---|---|---|---|---|---|---|---|---|
| 707.6-706.6 | 4.0- 5.0 | 20.9 | NP | NP | NP | 0.5 | 44.0[1] | ML |
| 705.7-704.7 | 5.9- 6.9 | 18.8 | 31.9 | 16.7 | 16.2 | 26.0 | 38.3 | CL-SM |
| 704.1-703.1 | 7.5- 8.5 | 18.5 | 33.4 | 15.9 | 17.5 | 24.0 | 43.0[1] | CL-SM |
| 702.6-701.6 | 9.0-10.0 | 19.9 | 35.6 | 16.0 | 19.6 | 20.5 | 34.2 | CL-SM |
| 701.1-699.1 | 10.5-11.5 | 18.3 | 34.6 | 16.7 | 17.8 | 7.5 | 37.2 | SM-CL |
| 698.6-697.6 | 12.0-13.0 | 17.4 | 32.2 | 15.1 | 17.1 | 38.5 | 33.0 | CL-SM |
| 697.1-696.1 | 13.5-14.5 | 16.5 | 32.0 | 17.3 | 14.7 | 5.0 | 45.0[1,2] | SM-CL |
| 695.6-694.6 | 14.5-15.5 | 17.7 | 37.2 | 17.8 | 19.4 | 0.0 | 44.0[1,2] | SM-CL |
| 694.1-693.1 | 16.0-17.0 | 18.1 | 36.4 | 18.6 | 17.8 | 8.0 | 48.5[1,2] | SM-CL |
| 693.6-692.6 | 17.5-18.5 | 18.1 | 32.5 | 17.6 | 14.9 | 8.0 | 48.5[1,2] | SM-CL |
| 692.1-691.2 | 19.0-19.9 | 15.3 | 42.2 | 26.1 | 16.2 | 21.2 | 36.8 | CL |

[1] Interlock noted in these samples

[2] Small rocks on failure plane

friction angle $\phi$ for the sandy clays, as well as the very high friction angle $\phi$ for the clayey sands is indicative of very dense sandy material. Interlock effects (increase of volume during confined shear) were noted in the dense sand samples, and give some of these soils an exceptionally high friction angle $\phi$. Data for the upper laminated clay-shale at El. 692.1-691.2 are typical of those for this material.

Boring No. 2

Test results from Boring No. 2 are shown in Table 2. As indicated in the Field Boring Logs, the upper strata are very stiff to hard sandy clays and clays, becoming sandier with depth. Sandy soils encountered were very fine, clayey, and dense to about El. 695, where dense, coarse, water-bearing sand containing gravel, shale, and sandstone chips in addition to rather sticky clay was encountered. As noted in Table 1, all clayey sands had appreciable cohesion (15.2 to 30.0 psi) and friction angles approaching and even exceeding $45^{o}$. The upper strata grade from CL-SM to CL to SM-CL by the Unified System while the lower strata grade from SM-SF to SC-SF, nonplastic (no plastic limit obtainable) despite their noticeable clay content.

Boring No. 3

Test results from Boring No. 3 are shown in Table 3. Upper strata at this location are composed of clay and clayey silt, grading to dense fine sandy clay/clayey sand below El. 701, mostly nonplastic as no plastic limit could be obtained. Dense, coarse sand was encountered below El. 694; the fraction passing the U. S. No. 40 sieve had some plasticity. Direct shear tests indicated little cohesion, but friction angles consistently exceeded $45^{o}$, from effects of interlock. By the Unified Soil Classification System the profile grades from CL through ML to SM-SF, then through SM-CL to SC.

Boring No. 4

Test results from Boring No. 4 are shown in Table 4. As may be seen, the fill is composed of stiff to very stiff clay of moderate

TABLE 2. SOIL TEST RESULTS - BORING NO. 2

| Elevation (ft) | Boring Depth (ft) | $w_{nat}$ (%) | LL | PL | PI | Direct Shear c (psi) | Direct Shear $\phi$ (deg) | Unconfined Compressive Strength $Q_u$ (psi) | Unified Classification |
|---|---|---|---|---|---|---|---|---|---|
| 704.5-703.5 | 4.0- 5.0 | 11.2 | 28.4 | 18.8 | 9.6 | | | 106.6 | CL-SM |
| 703.0-702.0 | 5.5- 6.5 | 20.4 | 42.0 | 22.7 | 19.3 | | | 54.0 | CL |
| 701.5-700.5 | 7.0- 8.0 | 17.9 | 37.3 | 21.9 | 15.4 | | | 63.7 | CL |
| 700.0-699.0 | 8.5- 9.5 | 17.7 | 34.9 | 19.4 | 15.5 | 15.2 | 44.1[1,2] | | SM-CL |
| 698.5-697.5 | 10.0-11.0 | 16.3 | NP | NP | NP | 0.0 | 50.0[1,2] | | SM-SF |
| 696.5-695.5 | 12.0-13.0 | 15.4 | NP | NP | NP | 30.0 | 45.0[1,2] | | SM-SF |
| 690.5-689.5 | 18.0-19.0 | 14.7 | NP | NP | NP | 20.1 | 45.0[1,2] | | SC-SF |

[1] Interlock noted in these samples

[2] Small rocks on failure plane

TABLE 3. SOIL TEST RESULTS - BORING NO. 3

| Elevation (ft) | Boring Depth (ft) | $w_{nat}$ (%) | LL | PL | PI | Direct Shear c (psi) | Direct Shear $\phi$ (deg) | Unified Classification |
|---|---|---|---|---|---|---|---|---|
| 705.2-704.2 | 3.5- 4.5 | 18.3 | 40.8 | 21.0 | 19.2 | | | CL |
| 702.2-701.2 | 6.5- 7.5 | 14.1 | 29.6 | 21.7 | 7.9 | | | ML |
| 700.7-699.7 | 8.0- 9.0 | 19.6 | NP | NP | NP | 0.0 | 47.6[1] | SM-SF |
| 699.2-698.2 | 9.5-10.5 | 19.0 | NP | NP | NP | 0.0 | 47.8[1] | SM-SF |
| 697.7-696.7 | 11.0-12.0 | 17.0 | NP | NP | NP | 5.9 | 50.2[1] | SM-SF |
| 696.2-695.2 | 12.5-13.5 | 21.6 | 28.7 | 19.0 | 9.7 | | | SM-CL |
| 694.2-693.2 | 14.5-15.5 | 22.2 | 30.7 | 17.2 | 13.5 | 1.2 | 47.8[1] | SC |
| 692.7-691.7 | 16.0-17.0 | 20.8 | 27.9 | 18.4 | 9.6 | 5.2 | 36.9 | SC |

[1]Interlock noted in these samples

TABLE 4. SOIL TEST RESULTS - BORING NO. 4

| Elevation (ft) | Boring Depth (ft) | $w_{nat}$ (%) | LL | PL | PI | Direct Shear | | Unconfined Compressive Strength $q_u$ (psi) | S.P. Compaction | | Unified Classification |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | c (psi) | $\phi$ (dep) | | $\gamma_{d(max)}$ (pcf) | $w_{opt}$ (%) | |
| 735.3-734.3 | 3.0- 4.0 | 17.5 | 43.8 | 23.3 | 20.5 | | | 56.2 | 100.2 | 19.8 | CL |
| 732.3-731.3 | 6.0- 7.0 | 16.4 | 44.3 | 25.1 | 19.2 | | | 49.8 | | | CL |
| 729.3-728.3 | 9.0-10.0 | 15.2 | 39.6 | 19.9 | 19.7 | | | | 101.8 | 21.2 | CL |
| 726.3-725.3 | 12.0-13.0 | 13.8 | 31.8 | 20.6 | 11.2 | | | 31.2 | | | CL |
| 723.3-722.3 | 15.0-16.0 | 13.7 | 35.4 | 20.6 | 14.8 | | | 58.3 | 107.1 | 18.6 | CL |
| 720.3-719.3 | 18.0-19.0 | 17.4 | 36.3 | 25.0 | 11.3 | | | 22.0 | | | CL |
| 717.3-716.3 | 21.0-22.0 | 15.4 | 41.5 | 23.1 | 18.4 | | | 45.9 | 102.4 | 21.1 | CL |
| 711.8-710.8 | 26.5-27.5 | 17.4 | 28.6 | 18.6 | 10.4 | 14.0 | 39.4 | | | | ML |
| 705.3-704.3 | 33.0-33.5 | 17.6 | 45.8 | 27.7 | 18.1 | | | 52.4 | | | CL |
| 703.8-702.8 | 34.0-35.0 | 14.7 | | | | | | 46.6 | | | CL |

plasticity, at natural moisture contents below the plastic limit and dry of optimum moisture. Below the fill, a thin layer of silty material overlies very stiff clay of moderately high plasticity, weathered from parent clay-shales and existing at a natural moisture content considerably below the plastic limit. Except for the ML silty stratum at about El. 711, the entire profile may be classified CL by the Unified System.

Boring No. 5

Test results from Boring No. 5 are shown in Table 5. The fill material at this location is slightly stiffer than found at Boring No. 4, being composed of very stiff to hard clay of moderately high plasticity, existing at natural moisture contents considerably below the plastic limit and dry of optimum compaction moisture. A still harder layer of fill material between El. 723.5 and 715.2 was too hard to be sampled. Just above the original ground surface, stiff clay of high plasticity is encountered and the material below ground level is very stiff to hard clay of moderate plasticity; below El. 709 it was too hard to sample. Except for the CH strata at the bottom of the embankment, the entire profile may be classified CL by the Unified System.

Boring No. 6

Test results from Boring No. 6 are shown in Table 6. Soils encountered in this boring were the weakest at the site, with an upper profile of sticky sandy clay grading to sticky clayey fine sand with enough clay content to produce Atterberg limits, then grading back to sticky fine sandy clay. However, the lower strata were dense enough to exhibit high angles of internal friction $\phi$, from $38.7^{\circ}$ to $40.4^{\circ}$. The profile below El. 699 grades (by the Unified System) from CL-SM to SM-CL and back to CL-SM just above the tan sandstone shown in Figures 7 and 8.

Swell Pressure and Consolidation Data

Swell pressure and consolidation tests were made on samples from

TABLE 5. SOIL TEST RESULTS - BORING NO. 5

| Elevation (ft) | Boring Depth (ft) | $w_{nat}$ (%) | LL | PL | PI | Unconfined Compressive Strength $q_u$ (psi) | S.P. Compaction | | Unified Classification |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | $\gamma_{d(max)}$ (pcf) | $w_{opt}$ (%) | |
| 736.2-735.2 | 3.0- 4.0 | 16.5 | 42.0 | 20.8 | 21.2 | 71.5 | 102.0 | 14.7 | CL |
| 733.3-732.2 | 6.0- 7.0 | 14.2 | 45.3 | 25.2 | 20.1 | | | | CL |
| 731.7-730.7 | 7.5- 8.5 | 17.6 | | | | 67.0 | | | CL |
| 730.2-729.2 | 9.0-10.0 | 18.2 | 45.2 | 21.6 | 23.6 | 60.7 | 99.1 | 22.5 | CL |
| 727.2-726.2 | 12.0-13.0 | 14.5 | | | | 64.3 | | | CL |
| 725.7-724.7 | 13.5-14.5 | 14.4 | | | | 45.4 | | | CL |
| 724.2-723.2 | 15.0-16.0 | 8.4 | 32.9 | 22.5 | 10.4 | | 106.7 | 18.8 | CL |
| 715.2-714.2 | 24.0-25.0 | 22.4 | 52.2 | 27.4 | 24.8 | 31.5 | 94.0 | 24.5 | CH |
| 712.2-711.2 | 27.0-28.0 | 11.8 | 37.9 | 23.3 | 14.6 | 59.3 | | | CL |

TABLE 6. SOIL TEST RESULTS - BORING NO. 6

| Elevation (ft) | Boring Depth (ft) | $w_{nat}$ (%) | LL | PL | PI | Direct Shear c (psi) | Direct Shear $\phi$ (deg) | Unified Classification |
|---|---|---|---|---|---|---|---|---|
| 699.2-698.2 | 7.0- 8.0 | 15.5 | 33.3 | 16.9 | 16.4 | | | CL-SM |
| 697.7-696.7 | 8.5- 9.5 | 18.3 | 32.8 | 18.0 | 14.8 | 10.0 | 10.5 | CL-SM |
| 694.7-693.7 | 11.5-12.5 | 24.6 | 27.7 | 18.1 | 9.6 | | | SM-CL |
| 693.2-692.2 | 13.0-14.0 | 23.5 | 26.3 | 18.4 | 7.9 | 10.0 | 40.4 | SM-CL |
| 691.7-690.7 | 14.5-15.5 | 24.8 | 31.1 | 18.2 | 12.9 | 8.0 | 38.7 | CL-SM |

TABLE 7. RESULTS OF SWELL PRESSURE AND CONSOLIDATION TESTS

| Boring No. | Depth (ft) | Elevation (ft) | Swelling Pressure (tsf) | Overburden Pressure $P_o$[1] (tsf) | Preconsolidation Pressure $P_c$ (tsf) | Coeff. of Compressibility $C_c$ | Void Ratio at $P_o$ | Void Ratio at $P_c$ |
|---|---|---|---|---|---|---|---|---|
| 1 | 4.0- 5.0 | 707.6-706.6 | | 0.27 | 1.9 | 0.095 | 0.620 | 0.605 |
| 1 | 9.0-10.0 | 702.6-701.6 | | 0.57 | 3.0 | 0.133 | 0.630 | 0.580 |
| 1 | 16.0-17.0 | 695.6-694.6 | | 0.99 | 3.2 | 0.104 | 0.517 | 0.495 |
| 1 | 17.5-18.5 | 694.2-693.2 | | 1.04 | 3.8 | 0.084 | 0.517 | 0.491 |
| 4 | 22.5-23.5 | 715.8-714.8 | 1.85 | 1.35 | | | | |
| 4 | 26.5-27.5 | 711.8-710.8 | | 1.62 | 3.2 | 0.133 | 0.555 | 0.545 |
| 4 | 32.5-33.5 | 706.8-705.8 | 3.10 | 1.98 | 3.1 | 0.108 | 0.466 | 0.448 |
| 5 | 24.0-25.0 | 715.2-714.2 | 2.35 | 1.47 | | | | |
| 5 | 27.0-28.0 | 712.2-711.2 | 2.50 | 1.65 | | | | |

[1]Neglecting submergence effects and assuming $\gamma$ = 120 pcf.

Borings No. 1, 4 and 5. Data were obtained to estimate probable consolidation of the embankment and underlying soil. Soils from Boring No. 3 were judged too sandy for consolidation testing; a trial test on one of the samples from Boring No. 3 indicated only elastic compression of the sample and little detectable consolidation behavior.

Data obtained from the tests indicate that all samples were highly preconsolidated. However, all samples from Boring No. 1 and the 26.5 - 27.5 ft depth sample from Boring No. 4 did not have distinct re-compression or virgin compression curves, their e-log P curves were basically curvilinear. Such behavior is caused by the high sand/silt content of the samples, and determination of preconsolidation pressure $P_c$ by the Casagrande method is not reliable in this case. Of more interest is the fact that all samples had rather low compressibility ($C_c \doteq 0.1$) and existed in-situ at relatively low void ratios. The void ratios did not change very much for the range of pressures between overburden and preconsolidation pressures.

The remaining four tests were conducted on material from Borings No. 4 and 5 at the bottom of the embankment and on the first very stiff clay stratum encountered below the embankment. All tests indicated swelling pressures greater than existing overburden. These soils will thus swell instead of consolidating if they are exposed to sufficient moisture.

## Summary

From test results on soil samples obtained from peripheral borings it may be deduced that, though variable in consistency, color, and composition, the soil underneath the Salvation Army Citadel is basically sandy in nature and its clay content is moderately plastic, wet, and sticky. Most sandy material grades fine to very fine, but some coarse sandy strata were encountered. Practically all materials exist in dense to very dense condition and have high confined shearing resistances and low compressibilities.

The OSHD embankment is primarily composed of fairly uniform stiff to hard clay of moderate plasticity, overlying material of similar characteristics. The fill material exists at moisture contents considerably below the plastic limit, and will tend to swell instead of consolidating as moisture becomes available.

## ELEVATION, DISTANCE, AND MOVEMENT REFERENCE POINTS ESTABLISHED AROUND THE EMBANKMENT AND SALVATION ARMY CITADEL

Several reference points were established around the embankment and Salvation Army Citadel to determine movements of the embankment, building, and surface soil. Shortly after preliminary discussions with the authors in May, 1971, the OSHD established a line of points of known elevation along the concrete drainage ditch between the Citadel and embankment south slope, to determine if horizontal movement of the embankment was causing soil compression and upthrust against the Citadel north wall. Also, points of known location were established on the circular retaining wall at the east end of the embankment, to determine if the wall was moving. To date, periodic measurements at these reference points have indicated no change.

After further discussion between the authors and the OSHD in June, 1971, Plaster of Paris patches were placed across cracks noted in the exterior brick facing, as was shown in Fig 5. Hairline cracking was noted across these patches on July 16, 1970, when the maximum daily air temperature was over $100^{\circ}$ F. On July 25, 1971, after arrival of a cold front, the maximum daily air temperature was in the vicinity of $72^{\circ}$ F. Inspection of the patches shown in Fig 5 (northeast Citadel corner) on that date revealed that the upper portion had moved between 1/16 in. and 3/32 in. to the south. The next day, July 26, 1971, with the maximum daily air temperature above $90^{\circ}$ F, the upper portion of the wall had returned to its original position and the cracks in the plaster patches had closed. No other type of movement has been noted to date at the various plaster patch locations.

During the last week of July, a line of elevation and distance control points was established in front of the south wall of Citadel

Classroom No. 1. From these points, movements of the south wall relative to the ground in the vicinity may be determined, as well as movements of the ground surface relative to distant reference points. To date, no movement has been noted, but only a very short time period has elapsed.

Continued periodic measurements and observations of these references, followed by immediate data reduction, plotting, and evaluation, will give greater insight into actual movement conditions occurring at the site.

## ANALYSIS OF EMBANKMENT BEHAVIOR

In the analysis, possible relationships between the construction of the embankment and the following major types of observed structural damage to the Salvation Army Citadel have been investigated:

1. Relative horizontal displacement along mortar joints of the Citadel north concrete block wall in the living quarters area.
2. Relative vertical displacement along the west concrete block/tile wall in the kitchen area and diagonal cracking in the northwest and southwest kitchen corners.
3. Southward horizontal movement of the south wall (Classroom No. 1--Fig 4) grade beam away from the floor slab, without moving the rest of the surrounding structure.

The possibility that this damage could have been caused by slope movement and horizontal squeezing of any soft soils under the building from the weight of the embankment has been specifically considered, along with the possible influence of a sloping surface of shale on which the piers may rest.

Since there is no implication that the OSHD is responsible for excessive deflection of the prestressed "double-tee" beams forming the roof of Classroom No. 1, this aspect is only briefly mentioned.

### Possible Behavior Conditions for Embankment

From a soils engineering viewpoint, four possible modes of behavior could cause deformation of the embankment and/or the soil located under and adjacent to it:

1. Elastic deformation of the embankment and underlying soils under their own weight.
2. Horizontal deformation of the soils under the embankment, as a result of horizontal strains induced by vertical consolidation under the weight of the embankment, and perhaps horizontal deformation of the embankment from its own vertical consolidation.
3. Slope failure of the embankment, along a roughly circular or log spiral slip plane, or along a plane of noticeable weakness.
4. "Creep" failure of the embankment by long-time slow-rate deformation of subsoils under and adjacent to the embankment, as a result of shearing stresses caused by the weight of the embankment.

Conditions necessary for occurrence of these modes of behavior will be discussed and related to conditions actually existing at the site.

## Elastic Deformation of the Embankment

Elastic vertical deformation of the embankment under its own weight would have occurred during, or immediately after, construction. While the prediction of such deformations is an inexact process, an estimate may be made.

The approximate overburden pressure at the bottom of the embankment (assuming an average compacted density of 120 pcf) is 3000 psf or 1.50 tons/ft$^2$ (tsf). The average compressive strength of the embankment material in Boring No. 4 is 43.9 psi or 3.12 tsf. According to Terzaghi, the "elastic subgrade modulus" for such material may be taken as about 150 tons/ft$^3$. Thus an elastic deformation of 0.01 ft could be expected for the bottom 1.0 foot of embankment under the 1.5 tsf pressure. Estimating the average vertical deformation of the embankment at half this value, an elastic vertical deformation of (.005 ft/ft) (25 ft) or .125 ft total probably occurred. If Poisson's ratio for the material is taken as 0.35 and an average embankment width of 175 ft is assumed near Boring No. 4, an average horizontal deformation of 0.31 ft should have occurred in the embankment proper, half to each side.

However, as the toe of the embankment is located 5 ft north of the Citadel, this deformation should have little effect on the building, simply resulting in slightly flatter (and thus more stable) embankment side slopes than were originally designed. Also, the embankment slopes would resist horizontal movement of the fill proper by berm action.

In Boring No. 5 the approximate overburden pressure is also 1.50 tsf at the original ground level, but the average compressive strength of the embankment material is at least 56.7 psi or 3.96 tsf, as strata between El. 723.2 and 715.2 were too hard to sample. Therefore, slightly less embankment deformation should have occurred near Boring No. 5.

It is also probable that horizontal deformation from vertical elastic compression occurred under the fill proper, but fill is founded on relatively stiff materials with a minimum unconfined compressive strength of 3.35 tsf and the embankment side slopes would also tend to resist lateral elastic deformation of materials underlying the fill proper by berm action. In any case, the most convincing proof of lack of such subsoil elastic lateral deformation is the Citadel itself. Such behavior should have caused noticeable and irregular southward movement along the entire bottom portion of the north Citadel wall. Also, horizontal movements from vertical compression should have caused noticeable upward buldging immediately adjacent to the fill. Such movements were not noted. In fact, the only noticeable relative wall movements occurred in the living quarters, opposite the end of the embankment where elastic vertical deformation of both the embankment and subsurface soils would be reduced by vertical frictional forces developed along the bridge abutment.

All evidence and logic point to the very low compressibility of the embankment materials, and of the hard clay-shales on which the principal parts of the embankment rest. Thus, it is concluded that elastic deformation of the embankment and underlying soil, occurring at the time of construction, had no noticeable effect on the structural integrity of the Salvation Army Citadel.

## Horizontal Movement of Embankment and Embankment Subsoils From Consolidation

The second case of behavior to be considered is horizontal or lateral expansion of embankment subsoils from vertical consolidation under the weight of the embankment. Vertical consolidation of the embankment itself will also be considered.

### Behavior of the Embankment Proper

The subsurface profile in Boring No. 4 indicated that the water table exists at the ground surface, fulfilling one of the theoretical requirements for subsoil consolidation (as contrasted with the more rapid compressional process of compaction). However, only the silty and silty clay topsoil encountered from El. 713.0 to 707.7 was at a natural moisture content high enough to fulfill the theoretical requirements for consolidation. Consolidation test data on this material was compared with that of similar soil found in the upper part of Boring No. 1, 55 ft to the south.

As described previously, e-log P curves for both samples were curvilinear, thus exact preconsolidation pressure determinations are not possible. Data from the soil in Boring No. 1 indicates a preconsolidation pressure $P_c$ of 1.9 tsf as compared with the preconsolidation pressure of 3.2 tsf obtained for the sample from Boring No. 4. Thus it appears that, in a relative sense at least, additional compression of the silty topsoil under the embankment resulted from placement of the embankment, and this consolidation appears to have been completed.

From consolidation test data on the sample from Boring No. 5 it may be conservatively assumed that the silty strata from El. 713.0 to El. 707.7 consolidated a total of 0.086 ft or 1.03 in., under the central part of the fill. Vertical consolidation of this magnitude will produce negligible horizontal deformations, especially outside the fill proper.

Swelling pressure/consolidation test data on the stiff yellow/tan laminated clay encountered at El. 707.8 in Boring No. 4, starting from natural moisture content, indicated a swelling pressure of 3.1 tsf or 6200 psf, compared to a maximum overburden pressure of 2.1 tsf or 4200 psf for the embankment and overlying subsoil. This means that, if allowed free access to water, this soil will swell against the weight of the embankment instead of consolidating. The soil is extremely impermeable, however, and its existence at a relatively low natural moisture content may indicate that neither swelling nor consolidation has actually occurred. As strata below El. 707.7 increase in strength and no wet or water-bearing strata were encountered it is doubtful that these strata have consolidated or swelled in any appreciable amount.

In Boring No. 5 the water table also exists at the original ground surface, but no topsoil was encountered, and the stiff laminated yellow/tan clay at the original ground level had a swelling pressure of 2.5 tsf or 5000 psf, starting from natural water content. As the material at ground level was the softest encountered, it is also doubtful if the subsoil at Boring No. 5 consolidated (or swelled) to any extent.

The embankment itself rests above the water table; thus its compressibility is inhibited by lack of sufficient pore water to cause high degrees of saturation. Furthermore, overconsolidated in-situ clays remolded by excavation and recompacted dry of optimum are likely to develop considerable swelling pressure; values of 1.85 tsf or 3700 psf and 2.35 tsf or 4700 psf were obtained from samples of the compacted silty shaley clay at the bottom of the embankment in Borings No. 4 and 5 respectively. These data, plus the overall low natural moisture content of the fill, indicate that little, if any, consolidation of the fill itself has occurred.

While it is unlikely that the water table will rise above its current location, future accumulation of capillary moisture will cause vertical swelling of the embankment and not consolidation. Though difficulties may be encountered with the I 244 surfacing from this behavior, there will be no effect on the Salvation Army Citadel.

## Behavior of the South Embankment Slope

While no borings were made through the south slope of the embankment, inspection of soil borings at the toe and crest plus the profile of Fig 9 leads to the conclusion that the south slope rests partly on stiff to hard laminated clays and clay-shales (noted in Borings No. 4 and 5) and partly on dense sandy material (noted in Borings No. 1 and 3). If anything, the sandy material located under the south slope should be denser than that found in the old stream channel; its location at the stream edge would have subjected it to more cycles of flooding and drying.

Dense sandy soils exhibit little, if any, tendency for long-term consolidation; most deformation takes place shortly after the load is applied. Also, when confined, such soils deform only slightly under static loading. Consolidation test data on these soils, at El. 695.6-694.6 and El. 694.1-693.1 in Boring No. 1, indicated rather low compressibility, especially in the range of loads provided by the lower portion of the embankment south slope. Assuming that a 10 ft thick layer of alluvial soil with the average consolidation properties previously described exists under the midpoint of the slope, where the additional foundation pressure would be 0.75 tsf at the original ground surface, the vertical consolidation may be estimated conservatively at less than one inch. Again, negligible horizontal movement would result from such small vertical movements.

As the laminated clay soils under the upper part of the slope are more likely to swell than consolidate, it is also doubtful that the subsoil under the south embankment slope (or the slope itself) has consolidated to any extent.

## Summary

Based on results of engineering tests, it is doubtful that the embankment or the soil underneath it has consolidated (or swelled) to any appreciable extent. If negligible vertical movement from this source has occurred, then even less horizontal movement is to be expected.

This contention is further substantiated by observing the

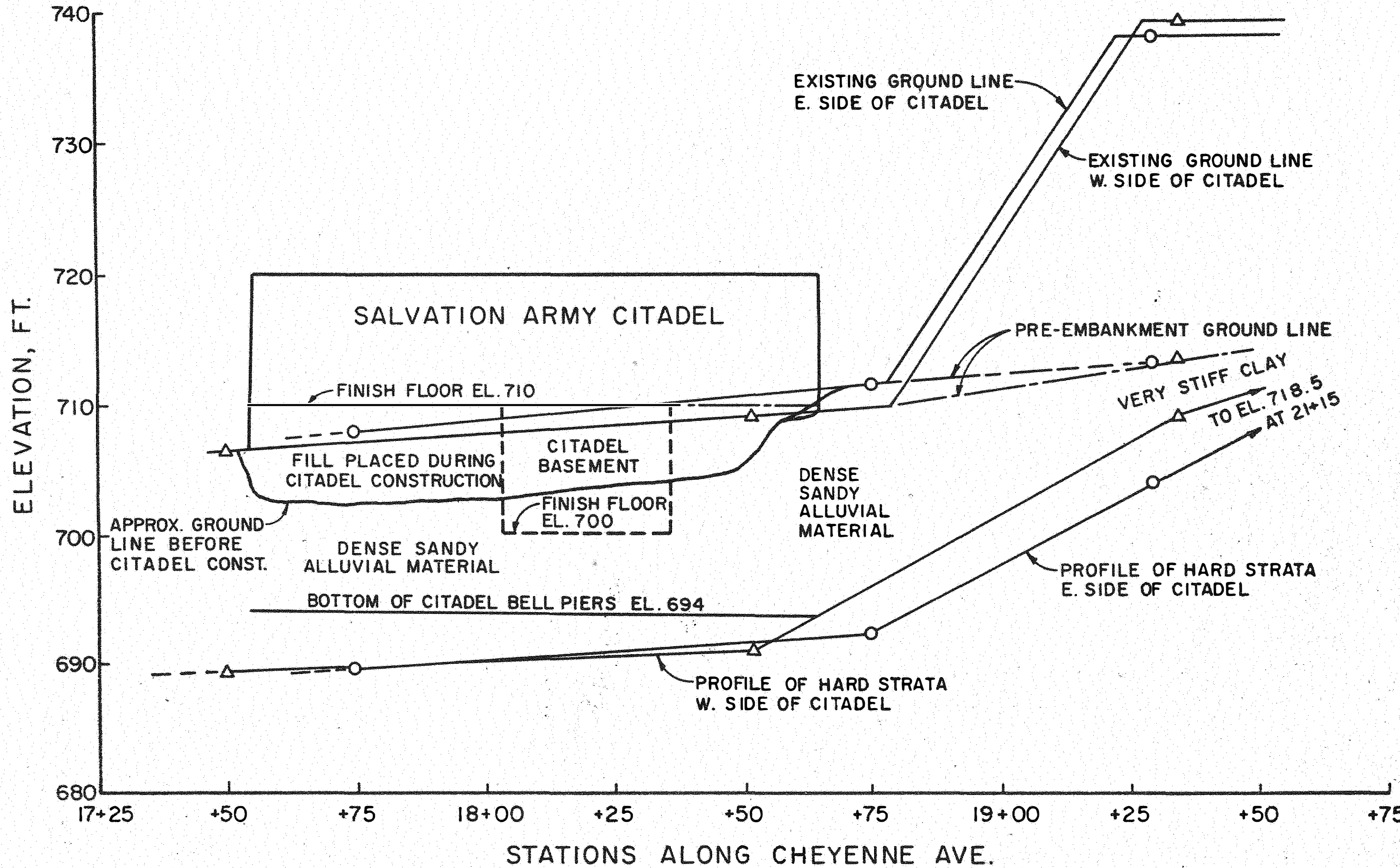


Figure 9. North-South Profile Under Citadel and Embankment

Salvation Army Citadel. As the most compressible soil strata are located near the original ground line, any appreciable vertical consolidation and resulting horizontal movement should have produced noticeable inward deflection of the Citadel north wall grade beam, displacement or buckling of the north edge of the floor slab abutting the grade beam, and damage to the lower part of the outside brick facing.

This combination of behavior has not occurred, and relative displacement of the upper and lower portions of the north wall was noted only in the living quarters, at a location where an appreciable portion of embankment weight is nullified by vertical frictional forces along the retaining wall and bridge abutment. While a slight horizontal crack runs the length of the north wall, it has existed long enough to receive several coats of paint. Furthermore, if horizontal movement of upper subsoil under the embankment (and thus the Citadel) occurred, the horizontal movement should be resisted by the structural basement of the Citadel, producing upward bulging under the kitchen area and also rotation of the kitchen wall grade beam. This behavior was not noted by the authors.

Thus, it is concluded that consolidation effects on both the embankment and embankment subsoil are of negligible magnitude, and have had no noticeable effect on the structural integrity of the Salvation Army Citadel.

## Slope Stability of the Embankment

Two possible cases of slope failure could exist for the south slope of the embankment. The first case would be horizontal sliding of the slope along the pre-embankment ground line, from lateral soil pressures induced by the embankment. The other possibility is a classic circular or log spiral failure in the slope material itself, existing slightly above, at, or below the toe of the slope. These two cases will be analyzed below. Movement of the alluvial soils under and adjacent to the embankment and possible slip failure along the deeper clay-shale strata will be discussed in the next section.

## Horizontal Sliding Stability of the South Slope

A stability analysis was made for the worst possible case of incipient sliding, assuming maximum at rest earth pressure ($K_o = 1.0$) from the embankment proper acting on the slope. Sliding was assumed to occur along the original ground line. Sliding resistance was based on direct shear test data from the silty topsoil found in Borings No. 1 and 4, as this was the weakest material encountered along the original ground line.

The analysis indicated a worst-case factor of safety against incipient movement of 2.9. This factor of safety indicates that little actual movement could have taken place, as any sliding at all would reduce the lateral earth pressure on the slope from at-rest toward active values. For this strong cohesive embankment material, the active earth pressure is likely to be tensile, and could conservatively be taken as zero. Thus it is hardly possible for the south slope to have moved to any extent by sliding along the pre-embankment ground line.

## Classic Slope Stability of the South Slope

Several classic stability analyses were made for the south slope of the embankment, using conservative values of $c = 3000$ psf and $\phi = 0^{\circ}$ and several slip circle locations. The minimum factor of safety against slip was found to be 6.4, high enough to indicate that little, if any, shearing displacement in the slope could have occurred.

## Summary

The embankment slope was found to be very stable with respect to sliding or classic slope failure. The high factors of safety against failure indicate that negligible deformations of the slope have taken place under existing conditions, and thus slope movements could hardly have had noticeable effect on the structural integrity of the Salvation Army Citadel.

Long-Term Creep Deformation of Subsoil Under and Adjacent to the Embankment

Many cases of subsoil "creep" under and adjacent to a natural slope or artifically constructed embankment have occurred. In more-or-less homogenous soil deposits, this type of behavior is noted in soft saturated clays.

When confronted with a situation, as was the case here, in which a high embankment has been constructed over subgrade soil strata described as "soft" or "mucky", one's first thought is of the possibility that the soft soil will be squeezed laterally from beneath the embankment. A natural consequence of such behavior would be the displacement of adjacent structures supported in or on the slowly creeping soil stratum. This whole phenomenon falls in the realm of plastic behavior, and requires the presence of a material to which plastic properties may properly be ascribed. If plastic action of this type occurs, the greatest displacements occur in regions of maximum shearing stress (i.e., beneath the fill slope) and are propagated from those regions. This means that the nearest parts of the nearest structure are the first to be affected by the creep movements. It is inconceivable that remote parts of an adjacent structure could be displaced by greater amounts than those parts in closest proximity to the source of disturbance.

Thus, in the present case (or in any similar case), any conclusion regarding the existence of soil creep, or squeezing action, must be based on two considerations: (1) the properties of the soil strata, i.e. is plastic behavior possible or likely, and (2) the nature of the displacements, distortions or damages that are evident in adjacent structures. These two considerations are addressed in the following statements:

1. As may be noted from the Field Boring Logs of Figs 7 and 8 and the soil testing results of Tables 1, 2, 3, 6, and 7, soils under the building can hardly be characterized as soft. Almost all subsurface soils above clay-shale/shale are basically sandy. These soils may be drilled easily by auger or fishtail bit, and, when brought to the surface, may be manually deformed quite

bases are sliding down such a slope is made moot by the fact that the piers are terminated before they reach the hard stratum (see Fig. 9).

3. Considering that a structural basement and 94 out of the 97 total drilled bell piers penetrate the subsoil under the building between the embankment and south Citadel wall, it is unlikely that forces can be developed against the three piers under the south wall grade beam without also being developed against other piers or basement walls, and moving them southward also. It is much more likely that the intervening basement walls and piers would be moved southward while the south wall would tend to remain stationary. In the still more unlikely event that "squeezing" occurred in the few feet between the bottom of the piers and the shale strata (see Fig 9) and then turned upward just under the south wall, resulting wall movement should be upward and tilting inward at the top, as well as translating southward. No such movements have occurred.
4. Citadel structural movements resulting from such subsoil movement should also have included southward movement of the lower part of piers below the north Citadel wall grade beam, producing outward rotation of the top of the wall. Also, bulging should have occurred under the floor slabs north of the north basement wall. The deep grade beam under the west kitchen wall would have been expected to move upward slightly and tilt, as southward soil movement is redirected by the north basement wall. The south part of the Salvation Army Citadel should be affected much less than the north and central portions, because of resistance provided by intervening piers and basement wall.
5. As structural movements of the nature described do not appear to have occurred, it is concluded that any hypothesis based on creep movements or flow of the subsoil mass must be disregarded.

Summary

Considering the type of soils existing under the embankment and Salvation Army Citadel and the type of movements that have occurred in the Citadel, it is unlikely that significant horizontal movement of subsoil, by either soil creep or simple shear deformation, has been caused by the construction of the embankment.

## ALTERNATE HYPOTHESES FOR STRUCTURAL DAMAGE TO THE SALVATION ARMY CITADEL

It is not sufficient to show that an extremely doubtful relationship exists between construction of the embankment and damage to the Salvation Army Citadel. The close proximity of the two structures, as shown in Figs 1 and 2, will continue to raise questions, particularly in the minds of laymen, concerning the causes of structural damage to the Citadel. Thus the work cannot be considered complete without a careful study of the damaged condition of the Citadel, with the view of offering logical and rational explanations of the causes of damage.

Causes of particular structural damage are often difficult to determine, and the authors are at a disadvantage since they were not present during Citadel construction, have not had access to field notes or other construction records, and have not been able to observe the sequence of structural damage development throughout the building. Nevertheless, after reviewing the existing damage and examining the _Plot Plan_ and _Foundation Plan & Details_ for the structure, boring logs of subsoil under the building, and the condition of other older masonry structures in the vicinity, reasonable hypotheses may be made concerning actual causes of structural damage.

### Small Interior Differential Movements of Citadel

As noted in the section on Citadel structural damage, a general pattern of masonry cracking from small differential vertical movements exists throughout the building. Such behavior indicates differential movement of Citadel piers and grade beams.

Several factors appear responsible for this behavior. First, as noted previously, no original Citadel boring was carried below

El. 691, thus the hard blue shale of the Pennsylvanian Period was not found. The deepest boring was carried to 20.0 ft below the surface, approximately 1.5 ft above the shale at this point. The five boring logs shown on the original Plot Plan for the Citadel do not indicate the presence of any hard strata.

As a result, piers were apparently founded at El. 694, one to four feet above the hard blue shale. While founded in material that should, on the average, provide the estimated 2 tsf allowable bearing capacity, the placing of numerous piers in an alluvial deposit is likely to result in their separate location on foundation materials of different stiffness and compressibility, resulting in different amounts of soil compression under design loading. The use of numerous pier bell sizes all founded at the same depth and presumedly carrying the same unit bearing pressure further aggravates the problem. It is a well-known fact that, in a homogenous soil deposit, a narrow footing will cause less soil compression than a wide footing when both are carrying the same bearing pressure. The same is true in varied alluvial deposits. Finally, it should be noted that the Foundation Plan & Details called for the various piers to be drilled in sandy material and underreamed in same below the water table. While vertical open holes could be maintained with casing, it is doubtful that underreaming could have been done with any precision, because of sloughing of the submerged sandy soil. As a matter of fact, Mr. McDonald has indicated verbally that some of the holes were cased, and that pier shaft diameters were increased to provide larger bearing areas without a necessity for underreaming. In the absence of opportunity to study careful field records describing the changes, it is impossible to estimate the extent to which these changes provided bearing pressures that were equal to those proposed for the belled piers. However, in view of the difficulties encountered in construction and the uncertainties involved in estimating the loads that will be delivered to the various piers by the superstructure, it is highly probable that substantial differences in soil bearing pressures exist beneath the various footings. These differences would also contribute to differential settlement.

It is therefore the opinion of the authors that the minor differential vertical movements experienced by the structural frame of the Citadel are not greater than could have been expected for the type of foundation employed and construction difficulties encountered.

Horizontal Cracking of Concrete Block Walls

Hairline cracking at the lower third point of exterior concrete block walls was noted along the north and west sides of the Citadel. Noticeable relative movement exists in the living quarters. Except for the west kitchen wall, all cracking was observed on walls which supported the "double-tee" roof beams. As mentioned previously, the prestressed "double-tee" beams ranged from 35 to 50 ft in span, and were tied to the concrete block walls. A tar and gravel roof is placed directly on the "double-tee" beams, providing poor insulation against ambient temperature changes. The rather sparse amount of roof gravel and relatively large amount of exposed dark tar noted by the authors probably increases thermal changes to which the beams are exposed.

The coefficients of thermal expansion for steel and concrete are similar; thus prestressing does not provide resistance to thermal deformation of the roof beams. Such deformations are cyclic in nature, as the beams grow longer in summer and shorter in winter, and when tied to rigid masonry walls may cause horizontal cracking and relative displacement of the walls. The senior author has encountered several instances of this behavior in Oklahoma when precast, prestressed "double tee" beams (or even expanded steel joists) were used in long span across masonry walls.

The roof of the living quarters, at the northeast corner of the Citadel, is formed by prestressed "double tee" beams 50 ft in length, running north-south, and framed solidly into the inverted "vee" chapel at their south end. Thus, nearly all of the thermal expansion/contraction of these beams must cause displacement of their north ends, where they rest on the north Citadel wall. Inspection of the aluminum sheathing nailed to the exterior "double tee" beam on the east Citadel wall reveals a history of cyclic longitudinal movement. Most of the nails holding the sheathing have been worked out; the remaining ones are bent from movements of the "double tee" beam relative to holes in the aluminum sheathing.

The 50 ft long "double tee" beams spanning the living quarters and restrained from movement at their south end could easily undergo a seasonal length change of 1/4 in., the amount of relative displacement noted in the living quarters wall. West of the living quarters, where the "double tee" beams span only 35 ft and are free at each end, little, if any, relative wall displacement can be seen.

It is therefore the opinion of the authors that horizontal cracking in the north and west exterior concrete block walls was probably caused by thermal movements of the precast, prestressed "double tee" roof beams, and that the 3/16 to 1/4 in. relative wall displacement in the living quarters, as well as the exterior cracking shown in Fig 5, has probably been caused by thermal expansion of "double tee" roof beams.

## Settlement of West Kitchen Wall

It was noted previously that the grade beam under the west kitchen wall had settled and that this movement was not in accord with those to be expected from either embankment or subsurface soil movement. However, the west kitchen wall is founded on a 6.75 ft deep grade beam, which forms the east wall of a four-foot wide stairway leading down into the Citadel basement at El. 700.

In basically sandy soils, bearing capacity is provided by interparticle frictional resistance, and this resistance is a function of normal pressure in the area of applied foundation loading; such normal pressure is usually provided by soil overburden.

Construction of the basement entrance outside the west wall effectively removes considerable overburden from one side of the center footing on which the kitchen wall is founded, reducing the bearing capacity and increasing the compressibility of the soil below the footing. The bearing capacity on the exterior side of the pier would be reduced to a value substantially less than that on the inward side, and that settlement would be greater than for those piers where greater soil confinement exists. It would be expected that the greatest settlement would occur where overburden removal was greatest, near the bottom of the stairs, and that the grade beam and wall would rotate slightly inward because of reduced bearing capacity on the exterior side. The damages observed and photographed by the authors support this hypothesis.

It is therefore the opinion of the authors that removal of west kitchen wall footing overburden by construction of a basement entranceway is as likely to have led to the observed damage of the Citadel west kitchen wall as any other factor.

Horizontal Movement of South and West Wall Citadel Grade Beams

As noted previously both the south and west exterior wall grade beams in Citadel Classroom No. 1 have moved outward away from the floor slab. The south wall grade beam has moved the most. As discussed in the last section, it is not possible to ascribe this movement satisfactorily to the influence of embankment construction, and an alternate hypothesis for this movement will be presented.

As noted on the Citadel Plot Plan and shown in Fig 3, between 5.6 and 6.6 ft of fill was required over the southwestern two-thirds of the Citadel building site. Assuming that the last six inches or so would be a sand cushion for the Citadel floor slab, between 5 and 6 ft of fill would be required.

No indication was given concerning the type of fill used, except for a notation about a topping of "black dirt" between the building and property lines. As no boring was made by the authors inside the property lines and no fill was needed very far outside the property lines (see Fig 3) except for establishing new sidewalks, the authors were not able to obtain samples of fill material for testing.

However, only a limited number of cheap and readily available fill materials are available in the Tulsa area. Because of both detrimental organic content and economics, it is doubtful that the contractor would backfill with any of the various loamy A-horizon soils which may be roughly classed as "topsoil." The other types of available fill are basically granular soils from Arkansas River bottom deposits and cohesive soils weathered from parent clay-shales and shales of the Pennsylvanian Period. As considerable difficulty would be encountered in placing and compacting an extensive fill of granular material 5 to 6 ft deep, it is likely that cohesive material was used for the fill. The normal tendency for Oklahoma contractors is to compact dry of optimum moisture, and one may hypothesize that the Citadel fill was compacted in this manner.

Since the water table is close to the surface, it may be expected that cohesive fill material under the covered area would accummulate capillary moisture with time, and undergo an increase in volume. Vertical swelling of compacted soil is a well-known and expected behavior. However, soil volume change occurs in three dimensions; for compacted Oklahoma clays the lateral unit expansion is approximately one-half the vertical unit expansion. Lateral subsoil expansion often causes cracking of basement walls and is also a major cause of highway pavement failure in Oklahoma. Conditions necessary for detrimental lateral expansion are a large area of material of relatively shallow depth. Vertical swelling is often negligible.

In this instance the area of compacted fill under Classroom No. 1 is about 45 ft north-wouth and between 37 and 45 ft east-west. Expansion of the fill material is restrained on the north by the walls of the structural basement and on the east by the rest of the Citadel; expansion to the south and west is restrained only by the grade beams and surrounding soil.

It should be noted that a lateral expansion of only 0.1% over the 45 ft north-south length of fill would produce the 1/2 in. of south wall movement noted by the authors. Vertical swelling of twice that amount (0.2%) over the 6 ft thickness of fill would produce a rise of only 0.14 inch, probably undetectable, especially if it occurred more or less uniformly. It should be noted, however, that instances of apparent small differential floor slab movement were noted, at locations where transitions between fill and natural ground were shown on the Citadel Plot Plan.

Before swelling can occur, swelling pressures are developed and then reduced when volume change occurs. As mentioned previously, the south and west walls of Classroom No. 1 offer less resistance to movement from lateral pressure than the other sides of the room.

Swelling pressures on these two sides would be resisted by the grade beams and supporting piers on which the grade beams rest. As the piers were not drilled into the shale, cantilever resistance will not occur and resistance to outward wall movement must occur from development of passive resistance in front of the grade beam and by lateral pile action of the drilled piers.

As may be seen from the profile of Fig. 9, most of the south wall grade beam is above ground, thus the majority of swelling pressure must be resisted by lateral pile action of the piers under the grade beam. While the other walls of the Citadel have piers spaced at relatively close intervals the south wall has only three piers, at 18 ft intervals, and the upper portion of these piers are located in the weakest soil ($c = 10.0$ psi, $\phi = 10.5^{\circ}$) found at any point around the Citadel. Along the west wall, piers are spaced at 9.3 ft intervals, probably accounting for the smaller grade beam movements observed there.

It is therefore the opinion of the authors that horizontal movements of grade beams in Classroom No. 1, and especially the grade beam carrying the south wall, are more likely to have resulted from lateral expansion of fill material placed during Citadel construction than from movements induced by the I 244 embankment.

## CONCLUSION

The authors have carried out an extensive exploration, testing, and analysis program to determine behavior of the Salvation Army Citadel and I 244 embankment. In addition, the authors have observed and inspected both the embankment and the Citadel and have held discussions with various interested parties.

Based on observations of behavior, results of exploration, soil testing, movement analysis, and the engineering judgment of the authors, as presented in this report, the following may be concluded:

1. There is no evidence to indicate that either the I 244

embankment or the subsoil under and adjacent to the embankment has moved appreciably.

2. Alternate and rational explanations exist for structural damage sustained by the Salvation Army Citadel; these explanations are not related to construction of the embankment.
3. No causal relationship, direct or indirect, was found to exist between construction of the I 244 embankment and structural damage sustained by the Salvation Army Citadel.

## APPENDIX 1

## SHELBY TUBE SAMPLING RECORD

## APPENDIX 1 - SHELBY TUBE SAMPLING RECORD

| HOLE NO. | DEPTH, FT. | TYPE OF PUSH* |
|---|---|---|
| 1 | 4.0 - 5.0 | Easy |
| 1 | 5.9 - 6.9 | Easy |
| 1 | 7.5 - 8.5 | Easy |
| 1 | 9.0 - 10.0 | Easy |
| 1 | 10.5 - 11.5 | Easy |
| 1 | 12.0 - 13.0 | Easy |
| 1 | 13.5 - 14.5 | Easy |
| 1 | 14.5 - 15.5 | Easy |
| 1 | 16.0 - 17.0 | Easy |
| 1 | 17.5 - 18.5[1] | Medium |
| 1 | 19.0 - 19.9[2] | V.Hard |
| 1 | 20.5 - 21.1 | V.Hard |
| 2 | 4.0 - 5.0 | Easy |
| 2 | 5.5 - 6.5 | Easy |
| 2 | 7.0 - 8.0 | Easy |
| 2 | 8.5 - 9.5 | Easy |
| 2 | 10.0 - 11.0 | Easy |
| 2 | 12.0 - 13.0 | Easy |
| 2 | 13.5 - 14.5 | Easy |
| 2 | 18.0 - 19.0 | Easy |
| 2 | 20.0 - 21.0[3] | Hard |
| 3 | 3.5 - 4.5 | Easy |
| 3 | 5.0 - 6.0 | V.Easy |
| 3 | 6.5 - 7.5 | V.Easy |
| 3 | 8.0 - 9.5 | V.Easy |
| 3 | 9.5 - 10.5 | Easy |
| 3 | 11.0 - 12.0 | V.Easy |
| 3 | 12.5 - 13.5 | Easy |
| 3 | 14.5 - 15.5[4] | Easy |
| 3 | 16.0 - 17.0 | Medium |
| 4 | 3.0 - 4.0 | Easy |

*V.Easy - less than 100 psi hydraulic pressure, Easy - 100 psi, Med. Easy - 150 psi, Medium - 200 psi, Medium Hard - 250 psi, Hard - 300 psi, V.Hard - 350 psi (truck lifted)

[1] Tube fastening bolt twisted - not high strength steel
[2] Sample not retrieved, Shelby Mandrel broken, left in hole
[3] Pushed into chunk of concrete
[4] Bent sharpened tube on a rock

| HOLE NO. | DEPTH, FT. | TYPE OF PUSH* |
|---|---|---|
| 4 | 4.5 - 5.5 | Easy |
| 4 | 6.0 - 7.0 | Easy |
| 4 | 7.5 - 8.5 | Medium |
| 4 | 9.0 - 10.0 | Medium |
| 4 | 10.5 - 11.5 | Med.Easy |
| 4 | 12.0 - 13.0 | Med.Easy |
| 4 | 13.5 - 14.5 | Easy |
| 4 | 15.0 - 16.0 | Easy |
| 4 | 16.5 - 17.5 | Easy |
| 4 | 18.0 - 19.0 | Easy |
| 4 | 19.5 - 20.5 | Easy |
| 4 | 21.0 - 22.0[5] | Easy |
| 4 | 22.5 - 23.5 | Easy/Hard |
| 4 | 25.0 - 26.0 | Easy |
| 4 | 26.5 - 27.5 | Easy |
| 4 | 28.0 - 29.0 | Easy |
| 4 | 29.5 - 30.5 | Easy |
| 4 | 31.0 - 32.0 | Easy |
| 4 | 32.5 - 33.5 | Medium |
| 4 | 34.0 - 35.0 | Hard |
| 5 | 3.0 - 4.0 | Medium |
| 5 | 4.5 - 5.5 | Medium |
| 5 | 6.0 - 7.0 | Easy |
| 5 | 7.5 - 8.5 | Med.Easy |
| 5 | 9.0 - 10.0 | Easy |
| 5 | 10.5 - 11.5 | Easy |
| 5 | 12.0 - 13.0 | Easy |
| 5 | 13.5 - 14.5 | Easy |
| 5 | 15.0 - 16.0 | Med.Hard |
| 5 | 24.0 - 25.0 | Med.Hard |
| 5 | 27.0 - 28.0 | Med.Hard |
| 5 | 28.5 - 29.4[6] | V.Hard |
| 6 | 4.0 - 5.0 | V.Easy |
| 6 | 7.0 - 8.0 | Easy |
| 6 | 8.5 - 9.5 | V.Easy |
| 6 | 10.0 - 11.0 | V.Easy |
| 6 | 11.5 - 12.5 | V.Easy |
| 6 | 13.0 - 14.0 | V.Easy |
| 6 | 14.5 - 15.5 | V.Easy |
| 6 | 16.0 - 16.2 | V.Hard |

[5] Pushed into chunk of concrete
[6] Sample did not stay in tube

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