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Clay Shale Foundation Slide at Waco Dam, Texas

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SYNOPSIS A major slide occurred during construction of the dam in 1961. It was caused by a combination of unusually high pore pressures in the clay shale foundation and a low residual shear strength. The dam was completed with wide berms to provide stability. Since completion there has been a very slow decrease in foundation pore pressure.

DESCRIPTION OF PROJECT

Waco Dam and Reservoir, a water supply and flood protection project, is located on the Bosque River at the northwest edge of the city of Waco, in central McLennan County, Texas. The Bosque River watershed is approximately 89 miles (143 km) long, 19 miles (31 km) wide and it includes the drainage systems of both the North and South Bosque Rivers which unite a short distance upstream of the dam. The dam was constructed about 1/2 mile (0.8 km) downstream of the city owned Lake Waco Dam which was in existence during design and construction of Waco Dam, but was breached upon completion of the new dam.

Waco Dam is a rolled earth fill about 18,000 ft (5.5 km) long with a maximum height of 140 ft (43 m) above streambed. It includes a 560-ft (171 m) long ogee type, gate controlled spillway on the left abutment and a 20-ft (6.1 m) diameter gate controlled outlet works located adjacent to the left bank of the old Bosque River channel. (Fig. 1)

FOUNDATION CONDITIONS - ORIGINAL UNDERSTANDING

Throughout investigations and design studies for Waco Dam, geotechnical personnel assumed that all of the foundation for the outlet works and spillway and all the embankment foundation except the upper portion of the steep right abutment would be on a single, hard, dense, shale formation. The shale throughout the left abutment and river valley sections was masked by a 10-to 40-ft (3-to 12-m) section of alluvial overburden. The shale was usually penetrated only 10-to 15-ft (3-to 4.6-m) by borings spaced on nominal 500-ft (152 m) centers. A few select borings in the embankment foundation penetrated 35 to 50-ft (11-to 15-m) of shale. The borings were logged in the field by an inspector or geologist, then submitted to the design office where they were prepared in final form and the information therefrom developed into geologic profiles, sections and the geology sections of design memoranda by an office geologist. Based on the information developed
from the logs and a limited amount of testing on the shale samples (mostly unconfined compression tests) - it was assumed that all the structures and the embankment were underlain by a single shale formation - the Eagle Ford shale of the upper Cretaceous system.

Geotechnical personnel were aware that the project was within the influence of the Balcones fault zone but no evidence was detected in any of the borings that faulting occurred in the structure foundation areas. A statement appears in the Earthen Dam Design Memorandum that "It is possible that some structural displacement, associated with the Balcones faulting, may be present in the shale at one or more points beneath the proposed embankment, even though none was noted from examination of the core from the borings." The main concern in undetected fault planes or zones was related to leakage along the fault planes rather than foundation stability.

BASIS OF ORIGINAL DESIGN

Design of the embankment segment, from stations 34+00 to 79+50 within which area the failure occurred was for a rolled earth fill with lv on 3h upper and lv on 4.5h lower upstream slopes and lv on 2.5h upper and lv on 3h lower downstream slopes. Embankment seining called for an upstream and central impervious core with a downstream compacted shale zone. An inclined and horizontal drainage blanket was incorporated in the downstream section of the dam. A 20-ft (6.1 m) wide cut-off trench with lv on 2h sideslope was excavated to shale along the dam axis and backfilled with impervious material. The original design, Figure 2, was based on the interpretation that the weakest foundation material was a 40-ft (12.2 m) thick clay layer between the river and the right abutment. The design strength of the foundation soil was $c = 1.5$ t/sf (144 kPa) for the unconsolidated undrained condition. The analyses indicated that the embankment would have an adequate degree of stability on this foundation and also within the fill itself, for the anticipated loading conditions of construction, steady seepage, rapid drawdown and critical pool level.

The foundation shales were not recognized as controlling the stability of the selected section. Laboratory tests were performed on shale samples from the outlet conduit site at station 24+00 and from the spillway site at station 85+75. Tests included unconfined compression tests and direct shear tests both parallel and perpendicular to the bedding of the shale. Samples of the shale were also taken from the embankment foundation between these two structures. Nothing was observed within the state of the art knowledge to indicate that there would be any problems with the shale as a foundation for the embankment. This view was supported by the construction of another dam of similar height a few years earlier on the Eagle Ford shale in North Texas. No foundation stability problems were noted on that dam.

Because of the high degree of overconsolidation resulting from loading in geologic times, and the very stiff to hard consistency (in soils terms) of the clay shales, the potential for pore pressure development was not recognized. Furthermore, the state of the art did not permit an appropriate assessment of the development of positive pore pressures in the clay shales. Therefore, no predictions were made and no piezometers were installed in the original construction.

CONSTRUCTION PHASES

First construction at Waco Dam was for an initial embankment section between stations 117+00 and 144+30 on the left abutment. Contract for this work was awarded in July 1958 and completed in November 1958. A second contract was awarded in September 1955 which included embankment construction from stations 34+00 to 79+50, 93+30 to 120+37, 143+00 to 180+45 and partial excavation for the spillway. The remainder of the embankment including the river closure section was to have been constructed under a third embankment contract. A contract was awarded in November 1959 and completed in November 1962 for construction of the embankment contract was about 8 percent complete. A contract for construction of the spillway was awarded in March 1961.

At the time of the partial embankment failure in October-November 1961, the second embankment contract was about 90 percent complete, the embankment was about 85 ft (26 m) high and 13 ft (4 m) below final grade in the failure area. The outlet works contract was essentially complete and the spillway contract was about 8 percent complete.

DESCRIPTION OF SLIDE

The evidence of a potential slide was first recognized on 4 October 1961 when a horizontal crack 900 ft (264 m) long parallel to the axis of the dam was found on the downstream slope of the embankment. A review
of construction disclosed that there had been signs of movement on 17 September 1961, when survey stakes set for placement of riprap on the upstream slope were found to be 0.56 ft (0.17 m) below the established elevation. This was discounted at the time as a survey error. Some cracks that appeared were attributed to shrinkage of the clay fill, and they were filled and covered by final slope trimming. On 6 October, stakes for riprap were found to be 0.9 ft (0.27 m) below the correct elevation. This suggests an average rate of movement on the upstream slope of the dam of 0.047 ft (14 mm) per day since 17 September 1961. With the 300-ft (274 m) long crack, a few small diagonal cracks appeared at the downstream toe of the dam, approximately over the locations of what was later determined to be the two faults. At this time, the full extent of the sliding mass was not apparent. (Fig. 3)

Since the rate of movement was very slow, a large number of surface reference points was established on the embankment and on the original ground surface beyond the toes of the dam. Elevations were determined by differential levelling and horizontal locations were determined by measuring offsets from an initially established line of sight parallel to the axis of the dam. The vertical movement of a point on the upstream slope of the dam at station 55+00 is shown on Figure 4. (The reference points were installed on 12-14 October 1961).

On 7 October, the small cracks at the toe of the dam could be traced to a distance of 300 ft (91 m) downstream from the toe.

On 10 October, a crack about 5 in (0.13 m) wide was found beneath the riprap on the upstream slope of the dam. This crack was parallel to the axis, and about 70 ft (21 m) upstream from the axis. This crack was ultimately found to be 700 ft (213 m) long, spanning the distance between the two faults. It had not been apparent earlier because of the rough surface of the 24-in (0.01 m) thick riprap blanket. It was the scarp of the failure surface.

On 13 October, the limit of movement was found about 500 ft (150 m) downstream from the downstream toe of the dam. At this time, the crest of the dam had possibly subsided about 1.5 ft (0.46 m) based on the evidence of the stakes that had been set for riprap placement.

As shown by Figure 4, the movement accelerated until 27 October when the motion became slower and essentially stopped by 1 November. From 25 to 27 October, a point on the axis of the dam subsided about 7 ft (2.1 m) and the point on the upstream slope at the limit of the failure surface subsided about 11 ft (3.4 m). The total measured movement at the failure surface on the upstream slope was 22 ft (6.7 m). The total horizontal movement of a typical point at the downstream limit of the sliding mass was 4 ft (1.2 m).
During the period of 23-27 October 1961, bulges and cracks developed in the area between the downstream toe of the dam and the previously found downstream limit of movement. Until 23 October, this area appeared to have moved as an intact horizontal plate. The ultimate configuration of the failed embankment is shown in Figure 5.

The base of the sliding mass was at elevation 370, approximately in the middle of the Pepper clay shale stratum (Fig. 5). The location and shape of the failure surface were inferred before the sliding movement stopped by projecting the measured surface movements of the failure blocks into the foundation. This was later confirmed by core holes drilled after movement ceased. The surface of sliding had a spiral shape from its scarp on the upstream slope, down through the cutoff trench, becoming horizontal at elevation 370, about 45 ft (14 m) below the original ground surface. The sliding surface rose to the ground surface at an average distance of 770 ft (235 m) from the axis of the dam. The total length of the failure surface was approximately 940 ft (287 m).

Inspection holes, 34 inches (0.86 m) in diameter, were drilled from the crest of the dam through the cutoff trench into the shale foundation. They showed that the failure surface was slickensided, but tight, and that there was an excellent bond between the compacted fill and the shale foundation. Figure 6 shows the failure surface encountered in one of the inspection holes.

MOVEMENT AT UPSTREAM TOE

A slight bulge developed on the natural ground surface at the upstream toe of the dam. It rose about 1 ft (0.3 m) and extended about 100 ft (30 m) upstream from the toe. There were not any cracks connecting this movement with the slide scarp or other cracks on the upstream slope. Two factors contributed to the difference between upstream and downstream movements: (1) the upstream slope was considerably flatter than the downstream slope, resulting in lower shear stresses in the foundation, and (2) the faults converged in the upstream direction. Since the faults acted as walls to confine the slide, the convergence helped to restrict upstream movement.

POST SLIDE INVESTIGATIONS

Investigations to determine the cause of the slide and to evaluate foundation and embankment conditions for redesign of the project were initiated on 10 October and continued until the project was successfully completed in January 1965. The investigations include drilling large diameter inspection holes, core borings, holes for electrical resistivity logging, borings to sample foundation materials and for installation of piezometer and slope indicators. Approximately 550 borings were made for a total of over 55,000 ft (16,775 m) of drilling. Piezometers were installed in 165 of the borings. Extensive laboratory testing was performed on embankment, soil and shale foundation materials. Detailed investigations were made to develop all available background data on soil, ground water and geological conditions in the project area.

A board of consultants consisting of Dr. Arthur Casagrande, consulting geotechnical engineer with Harvard University, Mr. H.M. Hill, consulting structural engineer of Minneapolis, Minnesota, and Mr. E.B. Barvel, consulting engineering geologist of Upperville Virginia was appointed to advise and consult with Corps of Engineers representatives on the investigative program and redesign of the embankment and spillway. The Board and Corps of Engineers representatives met on four separate occasions to evaluate the investigation and testing programs and to establish criteria for redesign of the embankment and spillway.

FOUNDATION CONDITIONS – REVISED UNDERSTANDI

Preconstruction investigations and design studies led to the assumption that all of the
embankment and structure foundations except the upper portion of the steep right abutment where the Austin chalk outcropped would be on the Eagle Ford shale and that, while no faults were located by the investigations, some might be present in the embankment foundation area since it was located within the influence of the Balcones fault zones.

Patterns of distress in the slide area together with results of intensive investigations soon after the slide occurred revealed that several geologic formations as well as significant faulting were involved in the embankment foundation area and that the faulting had a significant influence on the location and magnitude of the slide (Fig. 3). The investigation revealed that the normal sequence of upper Cretaceous beds at the project, in ascending order, is the Georgetown limestone, Del Rio shale, Pepper shale, Eagle Ford shale and Austin chalk. It also revealed that three en echelon faults, downthrown to the southeast occur in the river valley section. Two of these faults, designated as the north and south faults cross the centerline of the dam at stations 58+40 and 51+50, respectively. Their location is generally reflected by the slide area. As a result of these faults the relatively soft Pepper formation was down faulted between the harder calcareous clay shale beds of the Del Rio formation to the north and the much harder calcareous shale of the Eagle Ford formation to the south. The primary plane of the third fault crosses the dam axis at station 9+10. Displacement in this fault, which is actually a complex series of small en echelon faults, is about 75 ft (23 m) with downthrow to the southeast.

The north and south faults diverge from a distance of less than 100 ft (30 m) apart just downstream of the old Lake Waco Dam, or about 0.5 mi (0.8 km) upstream from the project, to a distance of about 1,500 ft (450 m) apart at a point 2,200 ft (670 m) downstream of the dam axis. This gradual separation, or divergence, is shown in Figure 1.

The investigations revealed that from the north end of the dam to the north fault at station 58+40 the embankment is founded on the moderately hard, calcareous clay shales of the Del Rio formation and that all of the spillway except the stilling basin is founded in the Del Rio. Excavation for the stilling basin exposed the upper surface of the Georgetown formation, the only place at the project where the hard argillaceous limestones of this formation were encountered.

Between the north and south faults at stations 58+40 and 51+50 the embankment foundation is on the relatively soft, black, waxy, clay shales of the Pepper formation. From the south fault to station 4+00 on the right abutment the embankment is founded on the stiff to hard, calcareous shale of the Eagle Ford formation, beyond which point to the end of the dam, the embankment is founded in the fairly soft, highly fractured Austin chalk. The entire outlet works structure is founded on the Eagle Ford. Generalized geologic features and embankment centerline profile are shown in Figure 7.

FORE PRESSURE OBSERVED

After the slide movements ceased, piezometers were installed to determine the distribution and amount of pore pressures in the clay shale foundation materials. Most of the piezometers were standard 1-1/2 in (32 mm) diameter well points 2 ft (0.6 m) long. A few were air-actuated piezometers and a few were porous stones with plastic tube risers. They were placed in 8 in (200 mm) diameter borings, and the screened tips were surrounded with sand. A 3 ft (0.9 m) deep seal of tamped bentonite balls was placed above the filter, and the remainder of the hole was filled with tamped clay or a cement-bentonite slurry. The

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[Diagram showing geologic profile and embankment centerline]
Riser pipes were 1/4 in (6.4 mm) or 3/8 inch (9.5 mm) standard steel pipe. They were protected by a 1-1/2 in (32 mm) diameter casing. Piezometers were installed to depths as great as 265 ft (81 m).

The piezometers showed a unique distribution of pore pressures. At the Pepper-Del Rio contact at elevation 340 msl beneath the slide, excess pore pressures at the axis of the dam were about 100 percent of the pressure applied by the embankment (Fig 8). Beneath the toes of the dam, the excess pore pressures were about 40 ft (12 m) above the normal ground water level.

Fig. 8. Estimated Pore Pressure, Prior to Slide

At the mid-Pepper, elevation 370 msl, pore pressures at the axis were about 80 percent of the embankment load. Beneath the toe of the dam, the pore pressures were about at normal ground water level. Prior to the slide they were estimated to have been 15 ft (4.6 m) above normal ground water levels. The cracking and bulging in this area associated with the slide had permitted relief of whatever pore pressure existed at the beginning of the slide.

Pore pressure observed after reconstruction of the embankment are shown on Figure 9 for the Pepper-Del Rio contact and for the mid-Pepper. The pore pressure distribution is unusual, both with regard to the higher pressures at greater depth, and the wide distribution of the pressure beyond the toes of the dam.

Research that has been performed by the Corps of Engineers in recent years has shown that the pore pressure development in anisotropic clay shales is a function of the ratio of axial to radial stiffness, measured in a laboratory triaxial compression test, within the elastic range. Simplified computations of stress distribution beneath the load of the embankment would show a reduction in incremental stress with greater depth if the foundation material were homogenous. However, the Pepper shale varies in character with depth, and the Del Rio shale immediately underlying the Pepper is more rigid. These circumstances make it possible for larger excess pore pressures to be developed at the Pepper-Del Rio contact than are developed at shallower depths in the Pepper. The results is a pore pressure gradient upward from the Pepper-Del Rio contact to the top of the shale beneath the overburden. The Pepper-Del Rio contact is relatively more pervious than the Pepper shale. This permits the contact to act as a distribution system, spreading the excess pore pressure far beyond the toes of the embankment.

To investigate the possibility that there were artesian ground water pressures before the dam was built, an extensive survey was made of water wells within a distance of at least 10 miles (16 km) of the damsite. Also, a test hole was drilled to a depth of 620 ft (189 m) at the downstream toe in the middle of the slide area to determine pressures within possible aquifers. These investigations showed that the ground water levels before construction of the dam were at or near the original ground surface. The conclusion was that all of the excess pore pressures observed were caused by the weight of the dam.

Fig. 9. Measured Pore Pressure, End of Construction
Investigations after the slide revealed the presence of six significantly different foundation materials:

a. Overburden clays
b. Eagle Ford shale, upper member
c. Eagle Ford shale, lower member
d. Bentonite seams, Eagle Ford shale, lower member
e. Pepper shale
f. Del Rio shale.

The weakest foundation material is the Pepper shale, which was involved in the slide. This discussion is limited to the Pepper shale.

Laboratory tests performed to determine the shear strength of the foundation materials included unconfined compression, triaxial compression and direct shear. Because of the obvious significance of surfaces of weakness and of horizontal bedding planes, most of the testing was done in direct shear. To evaluate the effect of time and of rate of drainage tests were made on specimens 0.5 in (13 mm) and 0.25 in (6.4 mm) thick, and at rates of strain from 0.05 to 0.000007 in/min (0.4 to 2.8 X 10^-7 mm/min). Some specimens were precut to provide a shearing plane that might be analogous to a previously broken or sheared surface in the field. All direct shear specimens were 3 in (76 mm) square.

The 0.25 in (6.4 mm) thick specimens tended to break across the corners giving a low apparent cohesion. The 0.5 in (13 mm) thick specimens sheared consistently through the specimen. The typical consolidated-drained shear strength of the intact Pepper shale was found to be: $\phi = 14$ degrees, $c = 0.4$ tons per square foot (38 kPa). The rate of strain had little influence on results.

The direct shear tests performed on precut specimens of the Pepper shale showed friction angles ranging from 7 degrees to 9 degrees, with no cohesion. This is the laboratory test condition that can be related to a material that has been broken prior to construction, and to the condition of the Pepper shale after the slide. Table I shows a comparison of design shear strengths of the various foundation materials.

<table>
<thead>
<tr>
<th>Material</th>
<th>Friction Angle</th>
<th>Cohesion Per Sq Ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eagle Ford, upper</td>
<td>15</td>
<td>0.4</td>
</tr>
<tr>
<td>Eagle Ford, lower</td>
<td>34</td>
<td>0.4</td>
</tr>
<tr>
<td>Bentonite layers</td>
<td>19</td>
<td>0.4</td>
</tr>
<tr>
<td>Pepper, intact</td>
<td>14</td>
<td>0.4</td>
</tr>
<tr>
<td>Pepper, residual</td>
<td>8</td>
<td>0.0</td>
</tr>
<tr>
<td>Del Rio</td>
<td>19</td>
<td>0.4</td>
</tr>
<tr>
<td>Clay Overburden</td>
<td>25</td>
<td>0.0</td>
</tr>
</tbody>
</table>

A drained direct shear test on a sample of the Pepper shale mixed into a slurry at the liquid limit, then consolidated and sheared under a normal stress of 3.3 tsf (314 kPa) indicated a friction angle of 16 degrees. This represents an effective shear strength, which is about twice as great as the residual strength.

The difference in stability of the embankment sections on the different foundation materials can be explained by the difference in pore pressure, the varying weakening effect of the faults and the changes in embankment cross sections.

**REDESIGN OF EMBANKMENT**

The pore pressure observed after the slide movement stopped were extrapolated to the conditions believed to exist when the slide began. This produced a pore pressure beneath the axis of the dam at the mid-Pepper of 80% of the fill pressure. This was then used in an analysis of stability along the observed sliding surface to determine the shear strength required for a safety factor of one. The average strength determined for the complete sliding surface within the Pepper shale was $\phi = 8$ degrees. This value was used in analysis of a new embankment section with berms added. The selected section is shown in Figure 10. Pore pressure were extrapolated to the loading condition resulting from the redesigned section.

**Fig. 10. Redesigned Embankment Section, As Built**
The analysis of the section with berms produced a safety factor of 1.15. This would not ordinarily be considered sufficient, but because of the extensive knowledge of the subsurface conditions and because of the use of a field strength derived from analysis of the slide, it is considered to be adequate.

It should be noted that there was evidence of progressive failure during the slide in which the peak shear strength was not mobilized simultaneously along the entire failure surface. Based upon the observation of early slide movement principally beneath the limits of the slide, it is considered to be adequate.

Between the downstream toe of the dam and the ultimate limit of sliding downstream, the upper half of the Pepper was an intact plate, in which the resistance to passive failure was greater than the horizontal sliding resistance along the weak plane at the mid-point of the Pepper.

Although the slide movement in the upstream direction was minor compared with that downstream, an equal berm was put upstream to assure adequate buttressing and blanketing of the disturbed foundation of the dam. Both upstream and downstream berms were extended in the direction parallel to the axis of the dam to a distance of 200 ft (61 m) beyond the faults.

RECONSTRUCTION

The embankment within the slide area was excavated to elevation 450 msl, and the open cracks were grouted with a cement-bentonite grout to provide a degree of stabilisation of the slide blocks. Beneath the downstream berm, vertical sand drains 18 in (0.45 m) in diameter were constructed in both faults. A gravel blanket was placed over the faults. This provided a controlled outlet for any seepage that would pass from the reservoir along the faults. The berms and the central portion of the dam were constructed to elevation 475 msl. The central portion to elevation 490 was then constructed at a rate of 5 ft (1.5 m) of elevation per month. It was next raised to elevation 499 msl at 1 ft (0.3 m) in 3 to 5 days. Construction was stopped for about 2 months while a thorough review was made of all instrumentation. The remaining 1 ft (3.35 m) of fill were added over a period of 40 days.

As the fill was added, pore pressure increases beneath the axis of the dam in the mid-Pepper were about 70% of the fill pressure. Pore pressures at the end of construction are shown in Figure 9. In the 19 years since completion pore pressures have diminished an average of about 25%. Total horizontal movement of the downstream slope of the embankment as the fill was added was about 0.23 ft (70 mm). Horizontal movement near the downstream end of the berm was 0.18 ft (55 mm). The embankment in the slide area was completed to design grade in August 1964. Downstream horizontal movement from 1964 to 1983 has ranged from 0.08 ft to 0.13 ft (24 to 40 mm).

CONCLUSIONS

There are three factors that contributed to the inadequacy of the original design:

1. Incomplete knowledge of the geologic structure and stratigraphy of the foundation. However, even if these features had been fully recognized, the design of the dam would not have been different because of the following two factors.

2. Incomplete understanding of the shear strength characteristics, including the unusually low residual strength, of the foundation clay shale under all conditions of loading and strain.

3. Inadequate state-of-the-art knowledge about pore pressure development and distribution in clay shale foundations. We now recognize the influence of vertical and horizontal stiffness on the development of pore pressure in anisotropic clay shales. However, the presence of geologic structure and boundary conditions that are difficult or impossible to define may make it impossible to predict field pore pressures reliably.

The experience at Waco Dam, and subsequent related research, has provided us with greatly improved insight into the problems of predicting pore pressures, estimating shear strength, and analyzing stability of embankment dams on clay shales. As a result of the lessons learned, we have successfully completed a substantial number of additional dams on similar clay shale foundations.

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