PERFORMANCE OF AN ANCHORED SHEETPILE WALL

by

S.N. Endley, Ph.D., PSI/National Soil Services Div., Houston, TX
Wayne Dunlap, Ph.D., Texas A&M University
Chris Snow, Haley & Aldrich
David Knuckey, P.E., Port of Freeport, TX
Jean-Louis Briaud, Ph.D., Texas A&M University
Lee Lowery, Ph.D., Texas A&M University

Presented at Texas Section ASCE Meeting
Spring 1991
San Antonio, Texas
INTRODUCTION

In 1985, the Port of Freeport initiated plans to develop a wharf and dock facility for the American Rice Co. The final structure, completed in 1987, included a 660 ft. long dock with a water depth of 32 ft., MLW. Plans included dredging the harbor to a depth of 38 ft. (plus 2 ft. of overdredging) in the near future. The dock consisted of a sheetpile bulkhead and a relieving platform, a common structure along the Texas Gulf Coast (Figure 1). A unique feature of the dock was the use of auger-cast concrete piles to bear the weight of the dock loads. Not only did the auger-cast piles provide a significant savings over conventional driven piles, they eliminated the possibility of vibration and backfill displacement which might have created additional lateral forces against the sheetpile bulkhead.

The principle advantage of the relieving type structure is that the dock loads are carried by the underlying piles rather than the loads being transferred to the backfill and then to the sheetpile bulkhead. The possibility that some of the lateral load from the backfill may be carried by the closely spaced bearing piles to further reduce the forces on the bulkhead seems not to have been considered in the past. On the other hand, auger-cast piles have less lateral load capacity than most conventional piles.

As a result of the uncertainties concerning lateral pressures in relieving type platforms, particularly with the use of auger-cast piles, the Port of Freeport authorized a study to evaluate the new dock structure with a view that possible savings could result with future docks. The study included instrumentation of the bulkhead and tie rods, measurements during and after construction, and analyses of the results before and after harbor dredging. This paper discusses several aspects of the instrumentation, measurements and analysis.

SOILS DATA

The plant site had been thoroughly investigated for the design of several silos, processing facilities and the wharf building. Consequently, only four soil borings were needed for the dock structure. The underwater bank of the ship channel sloped up gently to land from a maximum channel depth of -32 ft. MLW. The soils consisted of stiff overconsolidated clays and dense sands overlain by recent river deposits. Both total stress and effective stress tests were performed to evaluate the end of construction and long term stability conditions for the bulkhead design. The design parameters used are presented in Table 1.

The sand used for the backfill behind the sheetpile bulkhead was sampled and tested before and during backfill operations. Its characteristics are also given in Table 1.
DESIGN CONSIDERATIONS

Based on the soils data provided, the firm of Dunbar & Dickson designed the sheetpile bulkhead using the free earth support method applying Rowe's moment reduction. The results called for a sheetpile section with a section modulus of 44 in$^3$/ft. which extended to an elevation of -71 ft. MLW. Additional calculations were made using the p-y approach.

No economical U.S.-made sheetpiles were readily available with the required section modulus and subsequently a French-made Larssen Vs. section was selected which had the properties shown in Figure 2. These sections are supplied with two piles crimped together and they are driven two at a time.

INSTRUMENTATION

The instrumentation (Figure 3) included strain gages on the sheet piles at three locations along the dock and strain gaged load cells on the tie rods at three locations. Pressure cells were placed at three locations on the sheetpiles to measure backfill pressure, and inclinometers were placed at six locations to measure the movement of the sheet piles. Micro-Measurements weldable strain gages were used.

As shown in Figure 4, each instrumented sheetpile had seven strain gage locations spaced at intervals of 10 ft.; the gages were placed on the backfill side of the piles. After the gages were applied and waterproofed, the gages and connecting wires were protected by a 2 in. by 2 in. angle which was welded to the sheetpile. The void between the pile and the angle was filled with a waterproofing compound as further protection. The wires were terminated in watertight pipe cannisters to await connection to the readout devices once the piles were driven.

Earth pressure cells were installed on the sheetpiles to measure the backfill pressures. These were oil-filled diaphragm type cells with pressure transducers to measure the pressure in the diaphragm. Pressure calibrations and temperature corrections were supplied by the manufacturer; these were spot checked at the site by lowering the cells to a known water depth. To minimize the "foreign body" effect, a 9 1/2 in. diameter pocket was machined in the sheetpiles at each pressure cell location (Figure 5). To circumvent the effect of driving stresses on the pressure cells, they were epoxied into the pre-machined pockets by a diver after the sheetpiles were driven.

No. 18 Dywidag bars spaced at 6.5 ft. centers were used as tie rods. Two strain gaged load cells were installed in three of the tie rods. The load cells consisted of 4 ft. sections of the Dywidag bars which were gaged and then calibrated in a universal testing machine (Figure 6). They were placed in the tie rods with coupling nuts at approximately 20 ft. from the bulkhead on one end and 21 ft. from the tie beam on the other end.

-2-
Inclinometer casings were installed after the sheetpiles were driven. The casings were grouted into steel pipes which had been welded to the sheetpiles before driving, as shown in Figure 7.

Plans called for protecting the instrumentation wiring in gage boxes to be located inside the transit storage shed. The sheetpile strain gage wiring was installed in PVC conduit embedded in the lower concrete portion of the relieving platform. Wiring from the pressure cells and the dockside tie rod load cells was placed in the same conduit. At the shore side retaining wall of the relieving platform, the conduit was turned up into the wall to exit at the top of the wall, there to extend into the transit shed. Thus, the wiring was also encased in concrete for further protection.

CONSTRUCTION SEQUENCE

Construction of the sheetpile bulkhead began in August 1986 with a barge-mounted dragline stripping approximately 2 ft. of soft detritus from the underwater slope. Immediately thereafter, the sheetpiles were driven. Strain gages on the sheetpiles were checked for continuity upon completion of the driving. After the whalers and tie rods were installed, the tie rod load cells were also checked and then protected from the ensuing backfill operation by encasing the load cells in pipe. At this point, some gages on the sheetpiles were found to be drifting but still operable while all the tie rod load cells were functional. Inclinometer casings were grouted in place and the base readings were made and the earth pressure cells were installed.

All the backfill operations were conducted from barges using draglines to place the backfill material. The process of dropping the sand from a 4 cu. yd. bucket approximately 10-15 ft. above the water surface proved to be devastating to several tie rod load cells as indicated by continuity checks made immediately after the backfilling operation. Most of the damage was to the wiring and it was possible to conduct on-site repairs. At this time readings were also made on the inclinometers and sheetpile strain gages to determine the effect of backfilling operations on the sheetpiles.

Following backfilling operation, the auger-cast piles were installed on 6 1/2 ft. centers. It was immediately observed - based on the grout records - that the piles were expanding significantly in the sand backfill. Consequently, the pile operations were temporarily suspended while the backfill was densified using vibrators. The vibrators were kept at least 10 ft. from the bulkhead to ensure that no additional pressure was exerted against the sheetpiles.

INSTRUMENTATION RESULTS

The first data analyzed were the inclinometer records. Throughout the analysis of these records there was little difference between the six inclinometer
readings, and for most part the data for inclinometer No. 2 will be used for illustrative purposes.

The initial readings of the inclinometers after backfilling (Oct. '86) showed rather alarmingly high sheetpile deflections - of the order of 4 to 6 in. in the south (toward the water) direction (Figure 8). In the east-west direction deflections averaged about 1-1/2 in. toward the east Figure 9. However, evaluation of the strain gage data did not indicate significantly high stresses associated with these high deflections.

The next set of readings were taken in Dec. 1986, immediately after the auger-cast piles were completed. On average, there was a slight decrease in deflection of the order of 1/2 in. in the south direction (Figure 10).

In Nov. 1987, after completion of the dock and transit shed, another set of readings were taken. These readings showed no substantial change from the previous readings. Since the performance of the bulkhead indicated no change owing to the weight of the completed structure, but the deflections were still considered high, it was thought that the deflection measurements might be in error. Consequently, a diver made measurements which extended to the mudline.

Although the diver's readings and the inclinometer readings were not a perfect match as shown in Figure 11, they compared closely enough to convince us that the inclinometer readings were indeed correct.

ANALYSIS OF THE DATA

At this stage, the available data were analyzed in detail to ascertain the cause for the large deflections. The first clue came from the pressure cell readings, shown in Figure 12 in terms of an earth pressure coefficient based on the effective stresses. The readings taken immediately after the backfill placement (Nov. 86) showed values near 1.0 at a depth of 20 ft. below the top of the sheetpile and 0.8 at 30 ft. depth. Although these values decreased somewhat after a couple of months (Jan. 87), they were still much higher than is usually assumed for granular materials, and they were certainly much higher than the value of 0.33 assumed in the design of the bulkhead.

At this stage, several of the sheetpile strain gages had become inoperable, some due to the contractor's operations, some to moisture penetration. However, none of the operable gages indicated stresses which were higher than 20 ksi. To further determine the behavior of the sheetpile bulkhead, the displacements of the bulkhead obtained from the inclinometer data were used to obtain bending moments and stresses. This was accomplished by first determining the radius of curvature at each foot of length (station) along the pile, and then calculating bending moments (M) at each station based on the sheetpile moment of inertia (I) and modulus of elasticity (E). The initial attempt with this method produced very erratic results, mainly because small inaccuracies in the deflection data were

-4-
magnified in the numerical differentiation and produced large changes in the moments. An alternative approach was subsequently developed in which the curvature was smoothed numerically over a 6 ft. length of pile, thus cancelling the effect of the measurement errors in the data. The equation used to obtain the moments at, say, the 11 ft. station was:

\[ M = \frac{EI(D_8 - D_{10} - D_{12} + D_{14})}{(2)(2)(12)^2} \]

where the term \( D_i \) indicates the deflection at the station indicated. Stresses were obtained by dividing the moment by the section modulus.

The results of these calculations for bending moment and stresses are shown in Figures 13 and 14, respectively. Figure 15 compares the stresses obtained from the sheetpile strain gages with the inclinometer-determined stresses at three locations. These figures show that the stresses are approaching the yield stress of 50 ksi in the sheetpile.

The major problem with the high stresses was whether the planned deepening of the harbor in front of the bulkhead would increase the stresses beyond the yield stress. Since the original design utilized the p-y approach as an aid in determining the stresses, this approach was again taken to understand the cause for the high deflections. Using typical values for K of 0.5, the manufacturer's pile properties and the stiffness of the tie rods, several cases were run, none of which showed deflections near as high as the observed deflections. Subsequently, the value of K was changed to 1.0, which is about the average value measured immediately after completion of the backfill. It was obvious that the tie rod tension had an influence on the computer deflections, and several assumptions were made for the tie rod stiffness including one which had some initial slack in the rods before the wall moved enough to start tensioning the rods. The only assumptions which gave calculated deflections close to the measured ones (Figure 16) utilized a moment of inertia which was one third of the manufacturer's suggested value (the logic for this will be discussed later) and a sand layer extending to 8 ft. below the mudline; this was based on a sand layer which occurred in one of the borings.

The high K values measured by the pressure cells persisted for well over one year, at which time the wires to the cells were accidentally cut. With time, the pressures decreased about 15% but the values still ranged between 0.65 to 0.80. One possible explanation is that the process of dumping the sand from a significant height above the water momentarily created a dense semi-fluid which exerted a high enough pressure to cause the large deflections of the sheetpile. As the pore water escaped from the sand and it transformed into a solid mass, the pressure against the sheetpiles would decrease and the piles would attempt to at least partially spring back. However, as they attempt to do this the sand is now passively resisting the movement. Of course the passive resistance is much higher than the active resistance. Thus, the high earth pressures may actually be passive pressures rather than active ones.
This still does not totally explain the high deflections. In Bowles' text on Foundation Engineering (1), there is a discussion regarding the location of the neutral axis for sheetpiles as practiced in the U.S. and Europe (Figure 17). By crimping two sections together, the manufacturers of the Larssen piles infer that the neutral axis is located along the line of the interlocks, which also happens to be the position of maximum shear stress. There has been some doubt that the crimping can resist these shear stresses and prevent movement between the adjacent piles. If the piles act individually, and the neutral axis shifts up or down as shown by Bowles, then the moment of inertia is also reduced to about one half of the values presented by the manufacturer. The p-y studies showed that these lower values, in conjunction with the high earth pressures, would give deflection values near the measured ones.

Apparently, the question regarding the location of the principle axis has been raised before. In a paper by Phelan and Nucci (2), the effect of crimping on the Larssen sections was discussed, but only from the viewpoint that the crimping was adequate to resist movement along the interlocks.

The problem seems to be more fundamental, however. Figure 18 shows the results of calculations to obtain the moment of inertia for the sheetpiles. Position A-A is the principle axis recommended by the manufacturer. In reality, the principle axis should be located along B-B, at an angle of 28 degrees with the line of the interlocks. This would explain the significant East-West movement measured by the inclinometers. In fact, if the angle of the principle axis is based on the geometrical relationship between the measured East-West and North-South deflections (using average values), then 12 degrees should be used (position C-C). Axes B-B and C-C give pile properties which are fairly close to those for a single sheet pile with principle axis at D-D.

Table 2 shows the results of p-y calculations using the two section modulii believed to be the closest to reality. The section modulus of 26.3 in$^3$/ft., which corresponds to an inclination of the neutral axis of 28 degrees, seems to provide a deflection which is closer to the measured deflections before dredging. The calculated stress with this section modulus is about 28 ksi, well below the yield stress. The results show that the stresses after dredging 8 ft. would increase to somewhat less than 41 ksi, still below the yield stress of 50 ksi., even though the deflections would increase to nearly 7.5 in.

Based on these calculations, it was recommended that the dredging be allowed to proceed, although the uncertainties associated with the sheetpile properties dictated some caution. Consequently, the dredging was performed in two stages - 4 ft. at a time - with deflection measurements made after the first stage to ensure that sheetpile movement was within the calculated limits.

The deflection measurements showed that neither stage of the channel deepening produced significant changes in the deflections (Figure 19), justifying the decision to continue with the dredging. However, there is still some uncertainty regarding the earth pressures from the sand backfill, particularly
whether the passive pressure was actually acting on the bulkhead. If it were, then the small additional deflections associated with the dredging could actually have released some pressure on the sheetpiles. In the extreme case, the backfill pressures would revert to the active case, with pressures of approximately one half of those measured initially by the pressure cells. Unfortunately, the loss of the pressure cells prevents us from knowing whether this was the case.

Figure 20 shows the calculated stresses in the sheetpile at inclinometer No. 2 after the first and second stages of the dredging using the manufacturer's recommended properties. There seems to be some deterioration in the inclinometer casing with time, resulting in the jagged nature of the stresses. If these peaks and valleys are discounted, the maximum stress would be approximately 40 ksi. With a moment of inertia of 284 in^4/ft. the maximum stresses decrease to about 35 ksi (Figure 21), and with a value of 162 in^4/ft., the stresses reduce to less than 30 ksi (Figure 22).

CONCLUSIONS

Data analysis on the project is still ongoing, but on the basis of the measurements, the following conclusions are appropriate:

1. High lateral earth pressures resulted from the backfill construction methods, which, in turn, produced large sheetpile deflections and stresses. Less aggressive placement methods - such as hydraulic dredging or tremieing - would reduce the lateral earth pressures.

2. The installation of piezometers to measure backfill pore pressures during and after construction would help to evaluate the effect of the construction method on the lateral pressures in the backfill.

3. The evidence from the measurements indicates that the manufacturer's values of section modulus and moment of inertia are not correct for the Larsen Vs. sections. Crimping (or even welding) the two sections does not alter the properties since the adjacent uncrimped interlocks located in the plane of maximum shear stress are free to slip. Regardless of this, the principle axis for the two sections crimped together is not parallel to the interlock.

4. The question of responsibility for instrumentation on a construction project is age-old, but the contractor must be made to share the blame for damage to instrumentation.

ACKNOWLEDGEMENTS

We gratefully acknowledge the support of the Board of Directors of the Port of Freeport for funding the instrumentation program and allowing the presentation of results.
Mr. Carl Frederickson performed the strain gaging of the sheetpiles and the tie rod load cells, and Mr. Richard Bartoskewitz supervised the installation of the instruments. McClelland Engineers and McBride-Ratcliff and Associates performed the inclinometer measurements.

REFERENCES


### TABLE 1
**DESIGN SOIL PROPERTIES**

<table>
<thead>
<tr>
<th>Depth (MSL)</th>
<th>Type</th>
<th>Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-32'</td>
<td>Sand backfill</td>
<td>Grain size</td>
</tr>
<tr>
<td></td>
<td></td>
<td>max = #40 sieve</td>
</tr>
<tr>
<td></td>
<td></td>
<td>min &lt; 5% passing #200 sieve</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\theta' = 20^\circ$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$K_o = 0.33$ to 0.50</td>
</tr>
<tr>
<td>&gt;32'</td>
<td>Stiff tan and gray clay</td>
<td>$S_u = 1.0$ tsf</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\theta' = 25^\circ$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$C' = 0$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$K_o = 0.3$ to 1.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$K_p = 1.0$ to 4.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\gamma = 122$ pcf</td>
</tr>
</tbody>
</table>

### TABLE 2
**RESULTS OF ANALYSES USING VARIOUS SECTION MODULUS VALUES**

#### Before Dredging

<table>
<thead>
<tr>
<th>Case</th>
<th>Section Modulus (in.³)</th>
<th>Deflection (in.)</th>
<th>Bending Moment (in. lb.)</th>
<th>Stress (lbs. in.²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>36.1</td>
<td>2.56</td>
<td>749,200</td>
<td>20,750</td>
</tr>
<tr>
<td>II</td>
<td>26.3</td>
<td>4.23</td>
<td>735,700</td>
<td>27,970</td>
</tr>
</tbody>
</table>

#### After Dredging

<table>
<thead>
<tr>
<th>Case</th>
<th>Section Modulus (in.³)</th>
<th>Deflection (in.)</th>
<th>Bending Moment (in. lb.)</th>
<th>Stress (lbs. in.²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>III</td>
<td>36.1</td>
<td>4.20</td>
<td>1,083,000</td>
<td>30,000</td>
</tr>
<tr>
<td>IV</td>
<td>26.3</td>
<td>7.37</td>
<td>1,067,000</td>
<td>40,570</td>
</tr>
</tbody>
</table>
Figure 1. Components of Wharf and Dock No. 3, Port of Freeport

Figure 2. Properties of Larssen Vs. Sheetpile
Figure 3. Instrumentation Plan

Figure 4. Sheetpile Strain Gage Detail
ELEVATION
NORTH SIDE (SOIL SIDE)
OF SHEET FILE

Figure 5. Detail of Pressure Cell "Pocket"

Figure 6. Tie Rod Load Cell
Figure 7. Details of Inclinometer Casing Installation

DEFLECTION, INCHES (SOUTH - NORTH)

Figure 8. Measured North-South Deflections from Inclinometer No. 2, October 1986
Figure 9. Measured East-West Deflection from Inclinometer No. 2, October 1986
Figure 10. Measured Deflections (North-South) from Inclinometer No. 2, December 1986
Figure 11. Comparison of Diver's Data with Inclinometer Data
Figure 12. Measured Earth Pressures
Figure 13. Bending Moments Calculated from Deflections, October 1986

Figure 14. Stresses Calculated from Deflections, October 1986
Figure 15. Comparison of Stresses from Strain Gages and Inclinometers, October 1986

Figure 16. Results of Calculated Deflections from p-y Approach
Figure 17. Approaches Used to Determine Neutral Axis (from Bowles)

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>A - A (without crimping)</td>
<td>527</td>
<td>59</td>
</tr>
<tr>
<td>B - B (with crimping)</td>
<td>355</td>
<td>26</td>
</tr>
<tr>
<td>C - C (with crimping)</td>
<td>284</td>
<td>36</td>
</tr>
<tr>
<td>D - D (without crimping)</td>
<td>161</td>
<td>25.5</td>
</tr>
</tbody>
</table>

Figure 18. Calculated Principle Axes for Larsen Vs. Pile
Figure 19. Comparison of Deflections Before and After 2-Stage Dredging

Figure 20. Stresses Calculated Using Moment of Inertia from Manufacturer
Figure 21. Stresses Calculated Using Moment of Inertia Along Axis B-B

Figure 22. Stresses Calculated Using Moment of Inertia for Single Pile