SEISMIC RETROFIT OF AN UNDERGROUND RESERVOIR

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ABSTRACT

Seismic Upgrade of the University Mound Reservoir North Basin included 542 micropiles with 1335 KN (300 kip) design capacity installed in restricted overhead conditions. The North Basin Reservoir is a 200,000 M^3 (53 million gallon) underground water reservoir which stores water for emergency or life safety situations. This paper presents a case history of micropile installation for the seismic retrofit project, highlighting the solutions employed to create drilling access and detailing the unique load testing requirements and performance.

Micropiles were installed on the reservoir side slopes to provide foundations for new shears walls located around the site perimeter. The 200 mm (8 in) diameter piles were installed using cased rotary drilling methods to advance through fill embankments and then bonded in dense clayey sand of the Colma formation. Along the southern edge of the site, pile bond zones encountered variable lengths of the Franciscan rock formation. The micropiles were reinforced with double-corrosion protected all-thread bar and all piles were post-grouted.

All 542 piles were tested in tension to 1780 KN (400 kips) with acceptance based on the FHWA (1999) criteria of evaluating actual apparent free lengths and creep data. This approach is typically applied to ground anchor applications and contrasts with the lower percentage of testing and load-deflection performance measures commonly employed for micropiles. The extensive data set from University Mound allowed for detailed comparison of performance tests, particularly across the east side of the reservoir, which contained variable fill depths and micropiles bonded into both Colma Sand and Franciscan Rock.

Another unique challenge to this project was the access. All micropiles were located on the reservoir's 18 degree sideslope. Temporary platforms provided a stable working area for the drill rig and workers, and were continually moved around the site to create access as drilling progressed. In addition to the sloped condition, the reservoir roof limited overhead clearance down to a limiting condition of 2.3 M (7.5 ft) headroom. Detailed work sequencing and co-ordination with the excavation and demolition contractors was required to ensure drill rig could access each work location, with the corners providing specific challenge due to their dual camber.

This was a challenging project due to the large pile quantity, high capacity and limited headroom. However the requirements for testing of every pile, coupled with working on an 18 degree slope were particularly unique elements of this work.

BACKGROUND

University Mound North Basin Reservoir is located in San Francisco, California. This reservoir was originally constructed in 1885 and provides an off-line water storage facility within city limits. The San Francisco water supply originates in the Sierra Nevada Mountains and is conveyed almost entirely by gravity for 320 KM (200 miles) through a complex system of pipelines, tunnels, reservoirs and treatment plants. The route crosses three major active faults – the Calaveras, Hayward and San Andreas. In order to accommodate this challenging seismic environment, the water supply system includes a number of off-line reservoirs and includes built-in

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redundancy through parallel conveyance lines. University Mound North Basin is one of 80 projects performed under San Francisco Public Utility Commission's $4.6 Billion Water System Improvement Program which upgrades the service and seismic safety of the City water supply.

The reservoir is located in the south-eastern corner of the city and was constructed on a slope using excavation on the south and west sides, combined with embankment fill on the northeast corner. The perimeter embankment was raised 1.8 M (6 ft) in 1924, and subsequently a roof was added in 1962 and concrete lining installed to improve containment and water quality. The reservoir is 229 M (750 ft) from north to south and 168 M (550 ft) from east to west and has 200,000 M^3 (53 million gallon) capacity. The reservoir sides slope from El. 51.1 M (167.6 ft) down to invert at 44.5 M (145.9 ft) at a grade of 33%. The reservoir floor is nominally level.

SUBSURFACE CONDITIONS

In general subsurface conditions can be described as fill, overlying native clayey and silty sand, in turn overlying bedrock. However, due to the reservoirs large size and location on a slope, the relative depth and thickness of these strata vary widely across the site. Figure 1 shows a plan layout.

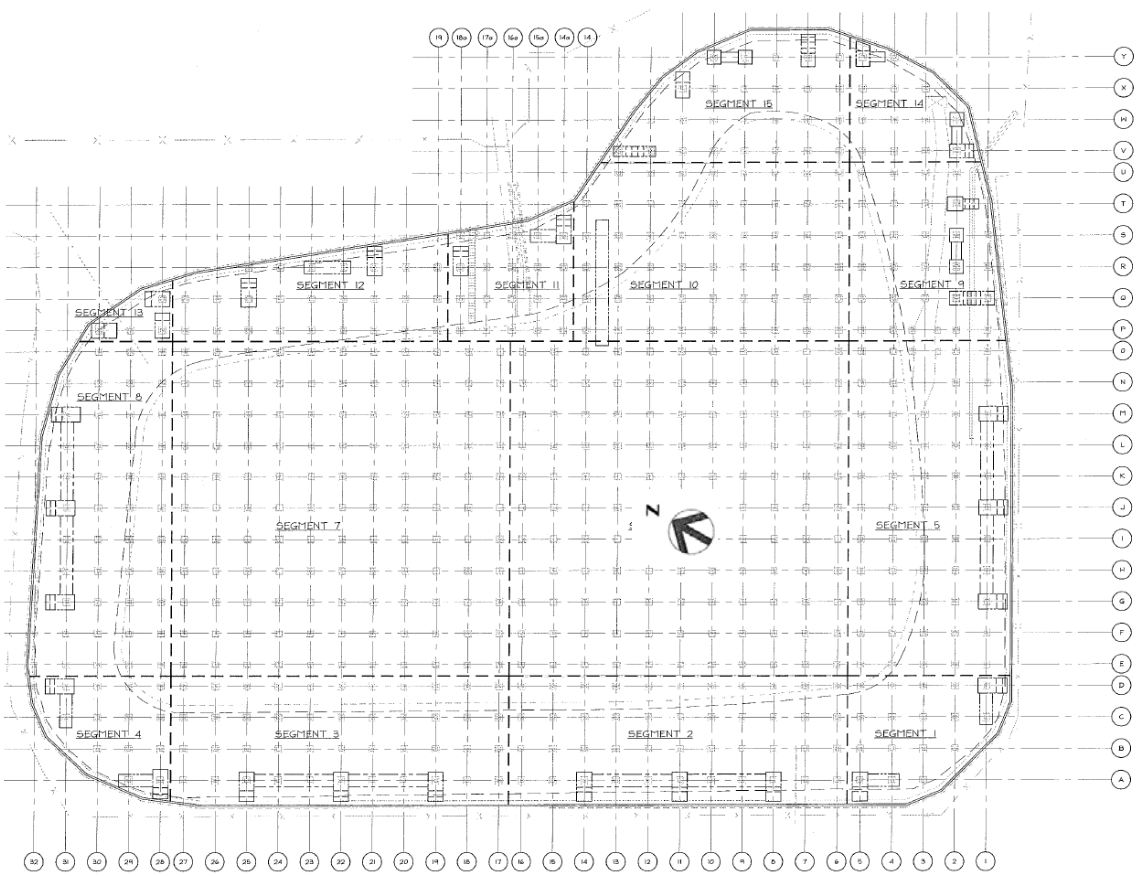

Figure 1. University Mound Plan Layout

The native ground slopes from south to northeast across the site. The existing grade is at El. 52.7 M (173 ft) in southwest corner, sloping down to El. 45.4 M (149 ft) at northeast, with a corresponding 7.3 M (24 ft) high embankment fill placed to contain the reservoir. The shallowest bedrock was observed at El. 46.6 M (153 ft) in the southeast corner, and sloped northwards to El. 6.1 M (20 ft) at northeast corner. Borings along west side extended down to El. 39.6 M (130 ft) but did not encounter rock.

Embankment fills, up to 7.3 M (24 ft) were placed to contain the east and north sides of the reservoir. The greatest fill depth occurs at northeast corner, and thickness tapers both south and west from this location. The fill materials are similar to the native soils, comprising medium dense to dense silty sand and sandy clay, with traces of gravel. SPT ‘N’ values in the fills were typically 20 to 40 blows / ft.

Medium dense to very dense sandy soils of the Colma Formation (Quaternary Period - Pleistocene Age) are present throughout the site. The Colma ranges from clayey to silty sand, and strength increases with depth. An upper zone of medium dense material ranges from 1.5 to 6 M (5 to 20 ft) thickness, with typically SPT ‘N’ values of 15 to 25. The underlying Colma Formation is dense to very dense material, with ‘N’ values exceeding 50, and based on triaxial testing indicates friction angle of 31 to 35 degrees, and cohesion of 24 kPa (500 psf).

Bedrock at the site is comprised of the Franciscan Formation (Late Jurassic and Early Cretaceous). This complex unit is a tectonic mélange consisting primarily of sandstone (greywacke) and shale, with mafic volcanic rocks and occurrences of serpentinite. The relatively young rock has been subject to intense shearing and deformation from regional seismic activity. The Franciscan composition and engineering properties are extremely variable and within this project site exhibited behavior ranging between that indicative of dense clayey gravel and hard, moderately fractured rock. The project investigation borings were typically terminated a maximum of 3 M (10 ft) into rock, where encountered, and described the Franciscan as Basalt; intensely to closely fractured, moderate to deeply weathered and ranging from weak to moderately hard. SPT ‘N’ values ranging from 64 to 148 were recorded in this zone. Rock strength testing was not performed or reported.

A monitoring well at the northeast corner of the site recorded groundwater levels between El. 36.6 and 38 M (120 and 125 ft), and some groundwater was observed perched on top of rock in the southeast corner during investigation.

RETROFIT SCHEME

The University Mound Seismic Upgrade aimed to preserve the integrity of critical water supplies in the event of a severe earthquake. The primary concern was the roof and its supporting columns, located on a 7.6 M (25 ft) grid across this site. The structural upgrade involved two key elements; reinforced concrete floor to ceiling shear walls connecting existing columns on the reservoir side slopes in radial and circumferential orientations which were founded on micropiles, and two 60 M (200 ft) square stainless steel brace frames that were constructed on the reservoir floor using grade beam foundations. The balance of this paper is focused on the micropile foundation system constructed on the side slopes.

A total of 542 micropiles were required for the retrofit program. All piles were designed for a 1335 KN (300 kip) seismic load, applicable in both tension and compression. The piles were designed by the owner as 57 mm (2.25 in) Gr150 threadbar only piles, with minimum 3 M (10 ft) unbonded length. Specifications required increased unbonded length in northeast segments of site to ensure no load transfer occurred in the embankment fill materials. Bond length was designed by the

Contractor to meet loading requirements. Piles were anchored into caps using a plate and double-nut connection. The micropiles were grouped in pile caps constructed around existing columns. A typical pile cap configuration for a radial shear wall included 16 micropiles, with 3 steps in the concrete footing elevation in order to accommodate the side slope (see Figure 2, below). For circumferential walls, typically 4 micropiles were installed per column, with 2 tension-only micropiles located at mid-point between columns. The specifications required that all piles were load tested in tension to 1780 KN (400 kips).

MICROPILE CONSTRUCTION

Key challenge for micropile design and construction was the variable site geometry. Typically both the Colma and Franciscan formations provide good bonding materials, but each pile required evaluation to ensure that bar configuration was compatible with the headroom and access constraints.

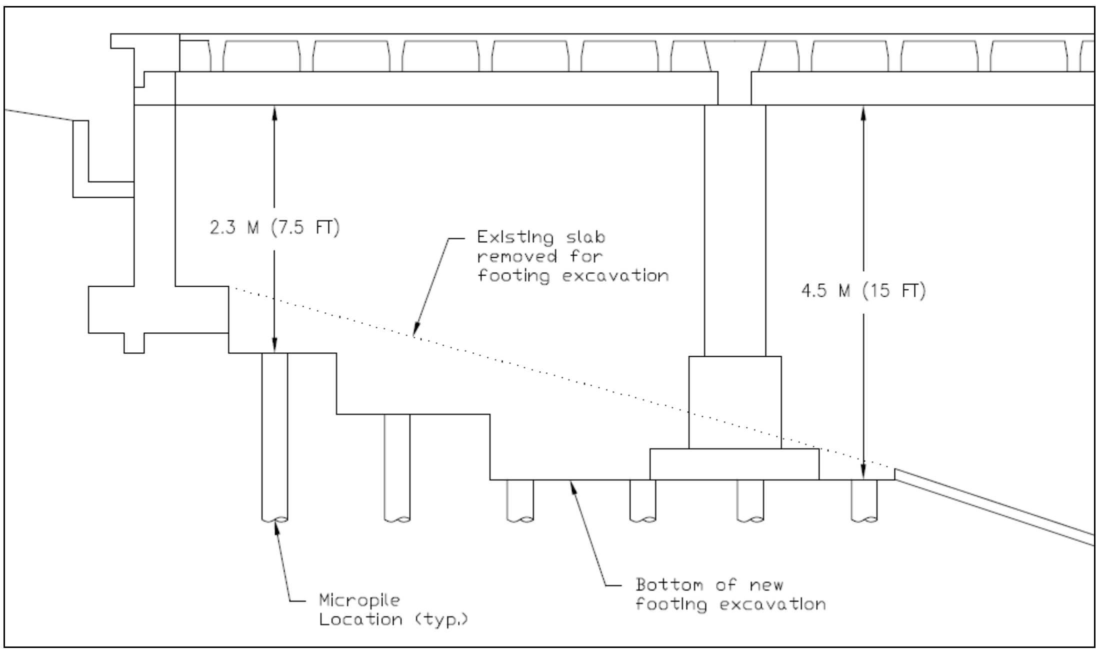

Figure 2. Typical Cross Section of Footing and Pile Layout

The pile caps were located within a 3:1 side slope of the reservoir. At the reservoir floor, headroom was 8.5 M (28 ft) however; most pile caps were located in the upper third of the slope, resulting in headroom as low as 2.3 M (7.5 ft). The pile caps were stepped, such that lowest headroom conditions applied only to the upper tier, and then increased for the lower piles within each cap. In order to minimize the number of bar couplings and hence cost of each pile, a range of bar configurations were employed and selected based on headroom conditions at each drilling location.

A series of tapered drilling platforms were constructed to allow for equipment operation on the reservoir side slope. The up-slope edge of each platform was located on the edge of new pile cap, while adjustable legs on the down-slope edge were configured to match the double camber slope conditions at each cap location. Tightly controlled working procedures and spoil management were employed to maintain safe personnel and equipment access on the sloped working area.

Every pile had a seismic design capacity of 1335 KN (300 kips), and required testing to 1780 KN (400 kips). The specified minimum unbonded length was 3 M (10 ft), with minimum bond length of 9 M (30 ft). The Contractor selected bond lengths ranging from 9 M (30 ft) to 12 M (40 ft) dependent on evaluation of ground profiles around the site perimeter. Along the south and west sides of the reservoir, where pile caps were founded in dense native soil, the specified minimum unbonded length was used uniformly. In the northeast quadrant of the site, the piles were drilled through embankment fills and unbonded length was increased to a maximum of 5 M (16.4 ft) in order to ensure bond length was set in native ground. Due to variable elevation of pile caps, and stepped configuration within individual caps, the unbonded length was checked for each pile and detailed schedules were developed in order to minimize pile lengths while maintaining groups with consistent pile configuration for ease of construction. For site management, the work area was divided into 15 Segments as shown in Figure 1. Extended unbonded lengths were required to accommodate embankment fill in Segments 8, 13 and 12.

LOAD TESTING

The specifications required testing of every micropile without applying load to existing columns or footings. The testing protocol involved loading all piles to 1780 KN (400 kips) in tension, with 5% of piles subject to performance testing and 1 each extended creep test required. The procedures were specified under FHWA 1999, which is typically applied to anchor or tie-down testing, with acceptance criteria defined in terms of creep and apparent free length. Apparent free length (L_a) is defined as length of micropile reinforcing that is, based on elastic movements at the test load, not bonding to surrounding grout or ground. Acceptance criteria were:

- L_a exceeds Jacking Length + 80% Design Unbonded Length
- L_a is less than Jacking Length + Unbonded Length + 50% Bond Length
- Creep at 1780 kN (400 kips) < 2 mm (0.08") per log cycle

In order to accommodate testing of all piles on the project, a number of different testing configurations were employed. The simplest cases applied when a line of piles could be tested by reacting against adjacent micropiles. This approach was combined with variants including cantilevered compression and tension loads spread over a range of piles, or distributed into the sloped reservoir slab in order to meet the project requirements.

A total of 542 pile load tests were performed for the project, however the analysis presented in this paper describes the pre-production test and then focuses on performance testing, specifically along the east wall of the reservoir. This area has been selected for further evaluation since fill-soil-rock interfaces are well defined along this perimeter.

PRE-PRODUCTION & TYPICAL PERFORMANCE TEST RECORDS

At the Contractor's option, one sacrificial pre-production test was installed to confirm selected drilling methods and assumed geotechnical load transfer. Production piling commenced immediately after successful testing of the verification pile. The pre-production test pile was installed adjacent to Segment 4 to verify assumed design parameters. This was the only location available to the contractor at this preliminary stage of work.

A 200 mm (8 in) diameter micropile was drilled to 16.7 M (55 ft) below grade. Reinforcing steel bar was installed with a 12 M (40 ft) bonded section, 4.5 M (15 ft) unbonded section and 2.3 M (7.5 ft) jacking length. The pile was tremie grouted after the bar was installed and post grouted 24 hours later. After curing for 7 days, the micropile was subject to a performance test up to a maximum load of 450 kips. The results demonstrated excellent pile performance satisfying both creep and elongation criteria at project test load of 1780 KN (400 kips). The pile was reloaded to 2000 KN (450 kips) and performed well; still meeting the specified criteria for apparent free length and creep.

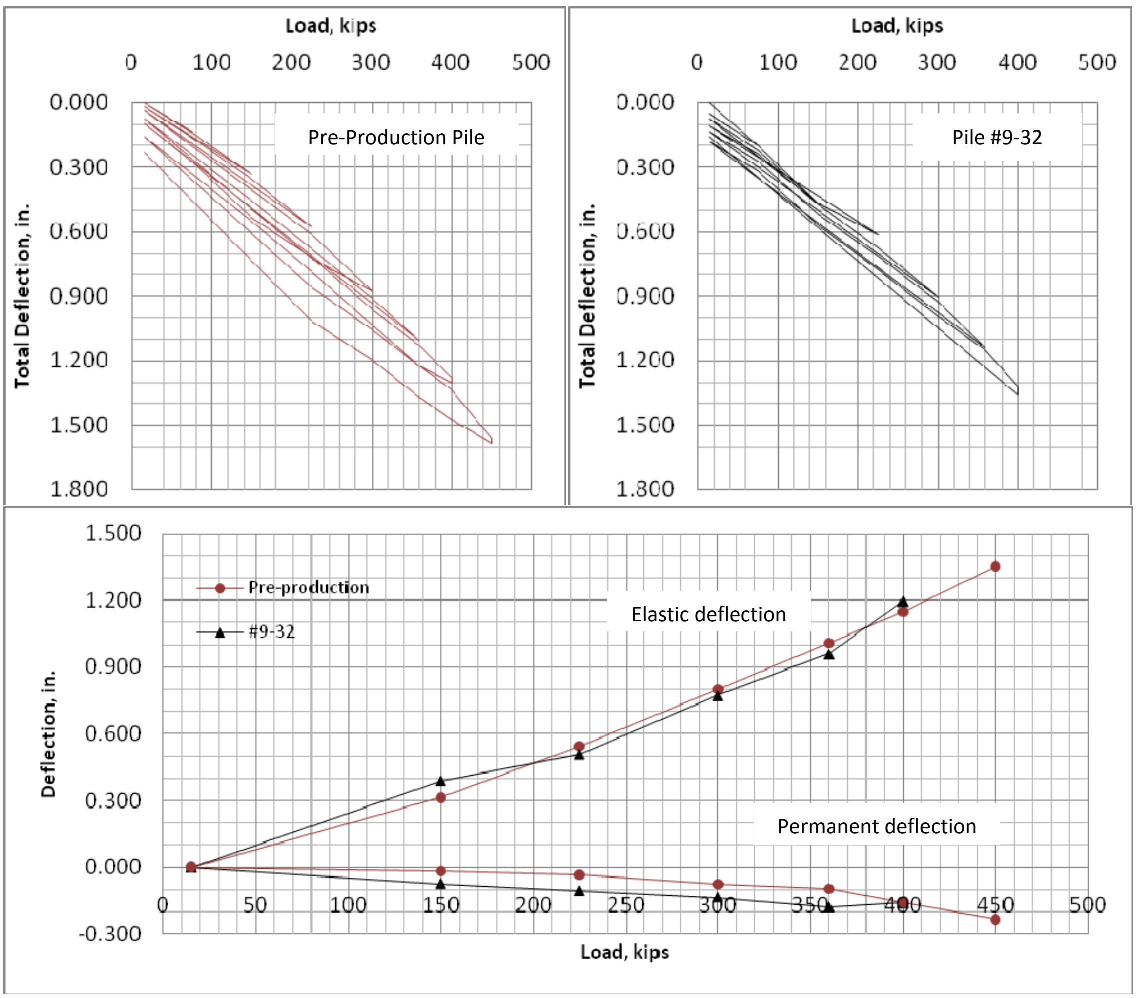

Figure 3. Performance Test Data: Pre-Production Pile & Pile 9-32

Figure 3 compares performance test data from the pre-production test pile with results from production pile #9-32 (pile naming format is “segment - pile number”). Both piles were installed with the same bonded and unbonded lengths and were located on complete opposite sides of the site. Although the pre-production pile was bonded into the Colma Sand while pile 9-32 was bonded in Franciscan bedrock, both tests exhibit similar load-deflections performance. At 1780 KN (400 kips) the pre-production pile had an apparent free length of 8.5 M (28 ft) and permanent set of 4 mm (0.16 in) while pile 9-32 had an apparent free length of 8.8 M (29 ft) and the same permanent set of 4 mm (0.16 in); these data are indicative of almost entirely elastic pile behavior. Even when loading of the pre-production pile was increased to 2000 KN (450 kips) the creep was only 6 mm (0.024 in) in 10 minutes, indicating that although slope of the load-deflection curve increased slightly over the final load increment, the pile was not approaching geotechnical failure.

The test pile verified the contractor's design but, due to the yield limitations of the pile reinforcing steel, loading did not result in geotechnical failure. Due to the stringent creep based acceptance criteria and potential for variable ground conditions across this large project site, the contractor elected to proceed with construction without modifying the original pile configuration.

PERFORMANCE TESTING – EAST WALL OF RESERVOIR

Figure 4 combines the results from the 13 performance tests that were conducted along the east wall (Segments 9-15). This area has been selected for detailed evaluation since the soil borings indicate rock elevation is highest closer to the south wall and decreases moving north, while conversely embankment fill height is greatest at the north and tapers to zero the closer to the south wall. Effectively, the unbonded length and length of bond in Colma (as opposed to Franciscan) increase from south to north.

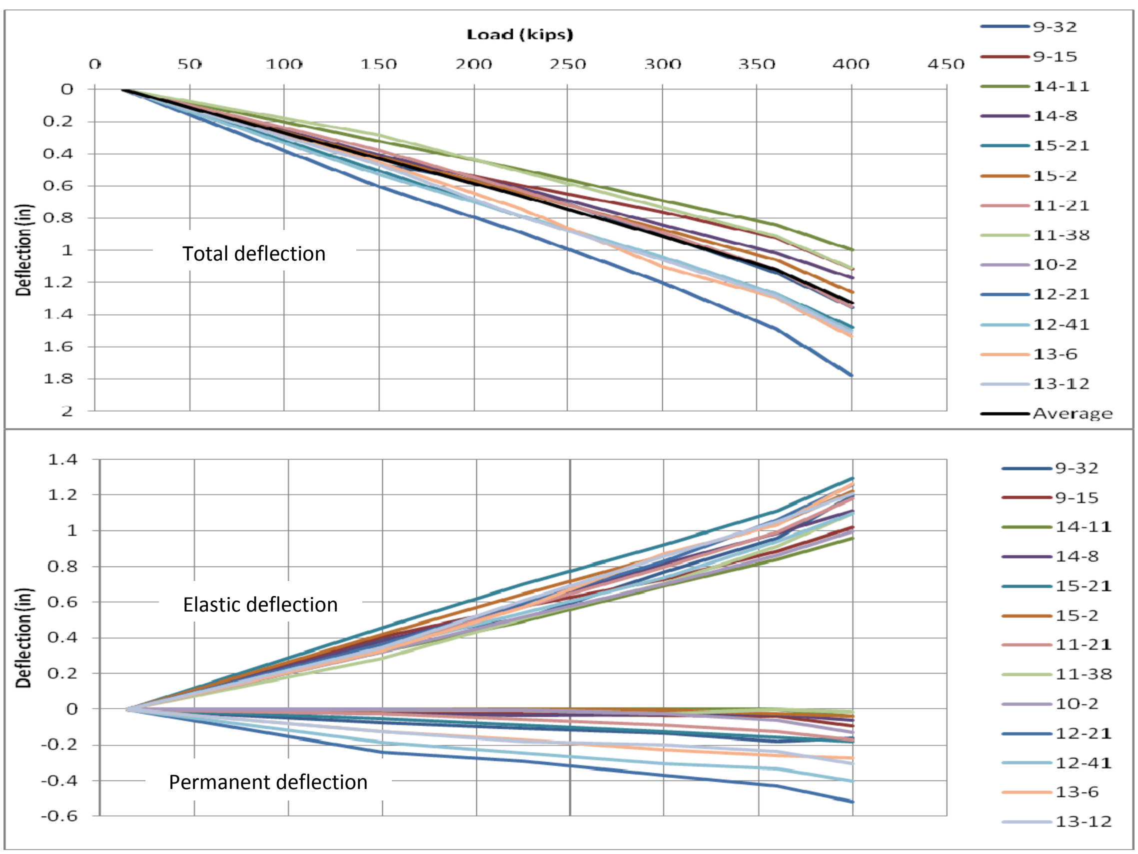

Figure 4. East Wall Performance Test Data Summary

Visual analysis of Figure 4 shows that test results exhibited a consistent form of load deflection curve, but with total deflection ranging from 25 mm (1.0 in) to 46 mm (1.8 in) at test load. For further analysis, performance test summary data has been plotted against the distance from the southern wall of the reservoir.

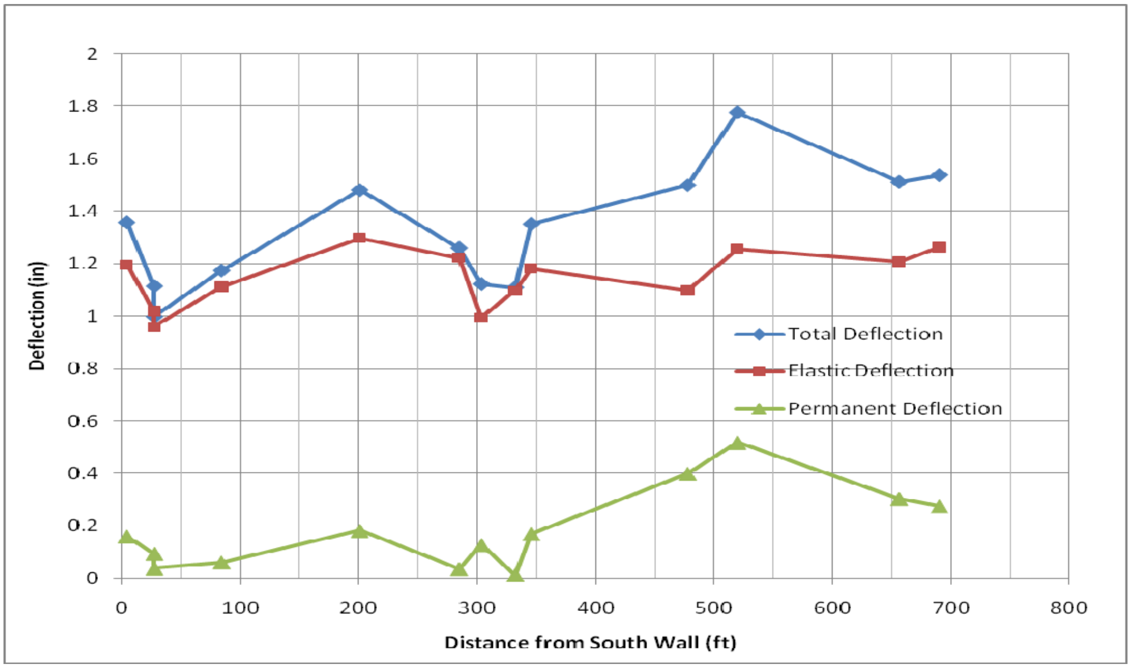

Figure 5. Pile Deflection vs. Distance from South wall of reservoir

Figure 5 compares the deflection (total, elastic, permanent) at 1780 KN (400 kips) for each performance test to the distance each pile is from the south wall of the reservoir. Trend noted in this data are:

- Total Deflection increases south to north
- Relatively uniform elastic elongation of piles
- Greater permanent set in piles at north end, particularly zone at +/- 500 ft

We note that although unbonded length increased at north end due to thickness of embankment fill, the actual unbonded plus jacking length in testing configurations was relatively uniform, ranging from 5.5 to 6.5 M (18 to 21 ft) for all piles except the northern most two data points. This explains the uniformity of elastic elongation amongst the performance test data sampled.

The trend of increase in permanent set moving northwards suggests that more movement is required to mobilize friction in the dense Colma sand formation compared to the Franciscan rock.

Figure 6 plots the creep performance per log cycle of time for piles at 1780 kN (400 kips) relative to distance from reservoir south wall, with vertical axis set to 2 mm (0.08 in) which is the maximum acceptance criteria. We do not note any clear trend in magnitude of creep along this selected wall alignment, nor do any piles exhibit high creep movements indicative of piles approaching geotechnical failure.

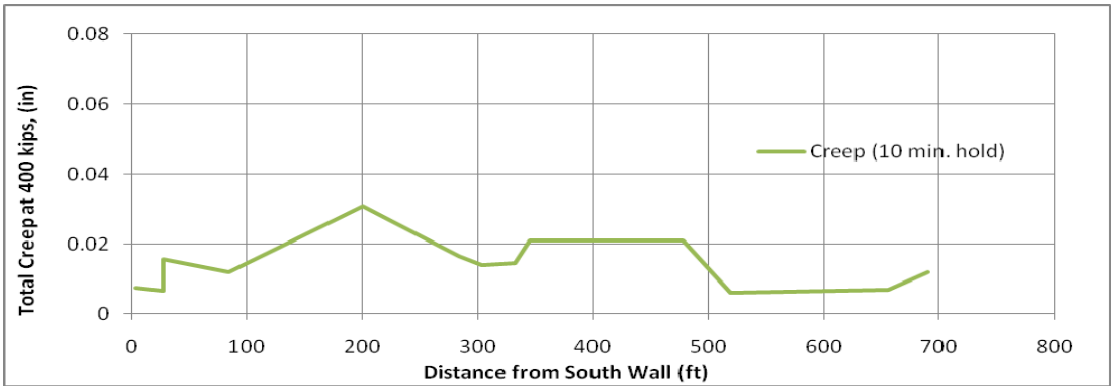

Figure 6. Pile Creep at Test Load vs. Distance from South wall of reservoir

The load testing data demonstrates very good load transfer in both the Colma and Franciscan formations, and piles did not approach failure in either bonding strata. Due to the extremely stringent acceptance criteria of creep testing at full seismic load, the Contractor selected a relatively conservative bond stress for pile design and piles were not loaded to a level which demonstrated any significant variance in load transfer between these two strata.

The acceptance criteria did not set a maximum deflection at test load, which is slightly unusual, since the performance of a structure subject to seismic loading may range between nominal damage and barely serviceable in response to pile deflections ranging up to 50 mm (2 in). In general the load-deflection performance of micropiles is of critical importance in the design of seismic systems and this topic is discussed in further detail by Jameson, Panian & Rudolph (2008) and Jameson (2009).

SUMMARY

The University Mound project is of specific interest due to its large magnitude, challenging access and the requirement to test 100% of piles with anchor-type acceptance criteria. The project was successfully completed, ahead of schedule and on budget. Creative access solutions, combined with careful site management enabled work in a safe and efficient environment. The stringent pile acceptance criteria were controlled by creep testing at full seismic load and correspondingly, the Contractor utilized relatively conservative load transfer values. All production piles exhibited excellent load transfer in both the dense sand and weathered rock bonding strata resulting in on-time and on-budget delivery for the project.

REFERENCES

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