DESIGN OF PILE FOUNDATIONS FOR THE SAND CREEK BYWAY, SANDPOINT, IDAHO

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The realignment of U.S. Highway 95 in Sandpoint, Idaho, is being designed on a site underlain by glaciolacustrine soil that includes very thick deposits of soft, sensitive clay to estimated depths of greater than 200 m. The highway realignment will include construction of numerous embankments, mechanically stabilized earth (MSE) retaining walls, and pile-supported bridges. The area of the proposed new highway corridor also includes existing historic and commercial buildings, a railroad, sensitive environmental areas, and economically vital tourism/recreational assets.

The evaluation of pile axial capacity included the use of a variety of methods, including total stress, effective stress, and empirical methods using the results of piezocone soundings and conventional borehole and laboratory tests. These methods were used along with the findings from an instrumented static loading test to design the piled foundations for the bridges. A critical factor throughout the design of the pile foundations was controlling settlements to levels that could be tolerated by the bridges. Also, because of the proximity of the pile foundations to the existing structures and the proposed new embankments, the project has required careful consideration of the potential interaction between these elements. The analysis of piles also considered the potential for settlement of embankments, drag loads on the piles, and downdrag.

INTRODUCTION

Washington Infrastructure Services and CH2M HILL, under contract to the Idaho Transportation Department (ITD), are completing design services for the U.S. Highway 95 (U.S. 95) realignment in Sandpoint, Idaho. The highway realignment has required balancing many engineering issues along with many sensitive environmental, cultural, and social/political factors. The geotechnical issues have created difficult challenges because of the soft, sensitive soil present to great depth.

SITE AND PROJECT DESCRIPTION

U.S. 95 is a vital link for commerce between Canada and the United States, and is the major north south route from North Idaho to South Idaho. In the current configuration, U.S. 95 travels through the City of Sandpoint downtown grid along one-way streets that are the main route for traffic in the city and through traffic. The realignment will provide an alternative, non-stop route for through traffic.

Surface Conditions

Sandpoint, Idaho, is located on relatively flat ground adjacent to Lake Pend Oreille, a large and very deep natural lake formed by glaciation. Figure 1 shows the project and vicinity. Sand Creek, a tributary to Lake Pend Oreille, flows to the south through the City and turns east to drain into the lake within the limits of the project. South of the Sand Creek inlet into the lake, a 100-meter (m) to 150-m- wide peninsula and causeway extends into the lake approximately 1.7 kilometers (km). U.S. 95 and the Burlington Northern Santa Fe (BNSF) Railway are both currently located on the peninsula. Immediately south of the Sand Creek inlet, the existing U.S. 95 turns west into Sandpoint, but the railroad crosses the creek and continues north along a natural terrace separating Sand Creek and the Lake. The terrace is approximately 100 to 300 m wide, and the ground surface is 5 to 14 m above the water.

Project Description

The realignment will move the highway to the terrace east of Sand Creek, parallel to the railroad. A two-lane section with an auxiliary lane in the southbound direction will be used

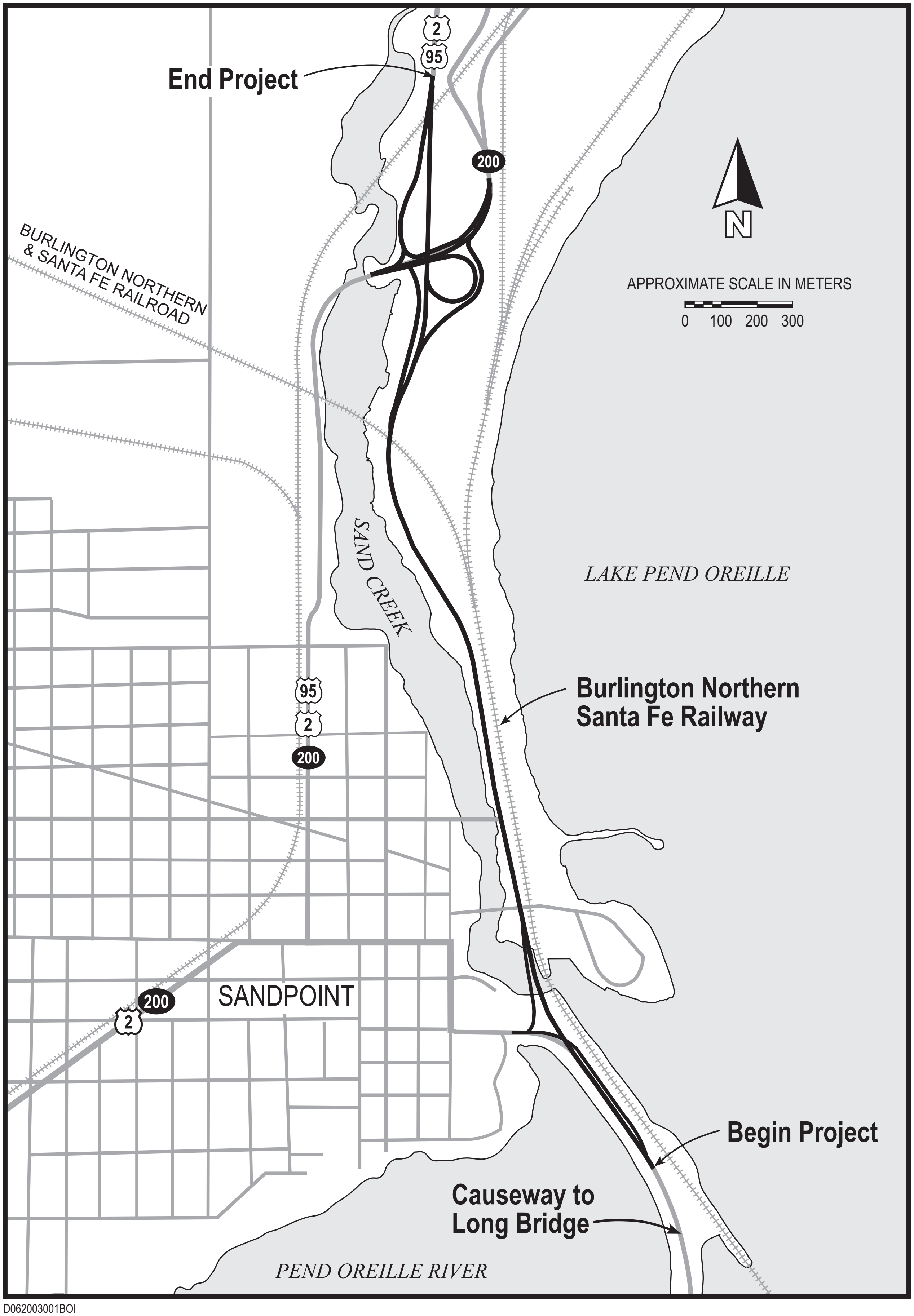

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Figure 1. Vicinity/Site Map

along most of the project limits. The project will require a new bridge to cross Sand Creek. This bridge will be a horizontally curved steel girder structure with five spans, and a total length of 320 m. Figure 2 is a Site Plan and Elevation showing the proposed Sand Creek Crossing. Other bridges will be required north of Sand Creek to cross a local city street, and another U.S. Highway at the north end of the project. Because of the adjacent Creek, MSE walls are required at numerous locations along the alignment, including the approaches to bridges. Some of these walls will reach maximum heights of 8 to 11 m.

From the outset of the project, it was recognized that pile capacity, drag loads, and settlement (including downdrag) would be very important issues for bridge foundations. The design team and ITD developed a geotechnical engineering scope to meet that challenge by including a series of field explorations and laboratory testing along with a full-scale pile loading test. The tests were completed to evaluate the compressibility of the soil and the load transfer characteristics of piled foundations.

SUBSURFACE EXPLORATIONS

Soil borings were advanced near the proposed southern interchange by ITD and other consultants in 1997, in support of the U.S. 95 project, and for a track-widening project for the adjacent BNSF railroad. ITD also advanced cone penetration soundings at the site in December 1997, using a 10-square-centimeter (cm^2) electric cone.

Field explorations and testing completed by CH2M HILL included soil borings, vane shear testing, piezocone penetration (CPTU) soundings, and test pit excavations.

Cone Penetrometer Tests

CPTu soundings were completed at 42 locations throughout the project. Generally, the CPTu soundings were made prior to advancing soil borings, so that potential sample intervals could be determined before-hand. All but one of the cone soundings were made using a standard 10-cm^2 cone. One sounding was completed using a 15-cm^2 cone. The cone soundings discussed herein included a pore pressure element located on the “shoulder” or the rod section immediately behind the cone. This arrangement is usually referred to as the U_2 position.

Soil Borings

A total of 19 soil borings were drilled to depths ranging from 11 to 80 m using casing advancer methods. Samples were collected from Standard Penetration Testing (SPT) and from thin-walled “Shelby” tubes assisted by a hydraulically actuated piston.

SUBSURFACE PROFILE

A typical subsurface profile from the CPTu testing is included as Figure 3. In general, the profile is composed of four soil units, in order of increasing depth: an upper terrace of thin, interbedded layers of sand, silt, and clay; a thick clay unit; a silt unit; and a lower clay unit. Depth to bedrock is unknown, but estimated to be in excess of 200 m (Breckenridge and Sprenke, 1997).

Upper Terrace

In the upper 5 to 11 m of the subsurface at the site, the soil borings and piezocone soundings encountered sandy and silty soil with relatively thin interbeds of clay and silt. This terrace unit overlies the thick clay unit, with a bottom elevation that ranges from 624 m to 631 m over the length of the project. The piezocone records indicate that there are random layers of soft to medium silt and clay within this upper unit that are typically less than 1-m thick. Atterberg limits testing in the material generally indicates non-plastic to low-plasticity material. In some areas of the project, portions of this material are fill.

Clay

Below the upper terrace material, the explorations encountered a thick, very soft to soft clay layer to an elevation ranging from 586 m to 597 m. The plasticity of the clay was observed to be generally low to medium. Thin (less than 1.5-m-thick) layers of loose to medium dense sandy silt and silt were encountered within the clay layer. These layers appear to be consistent between exploration locations.

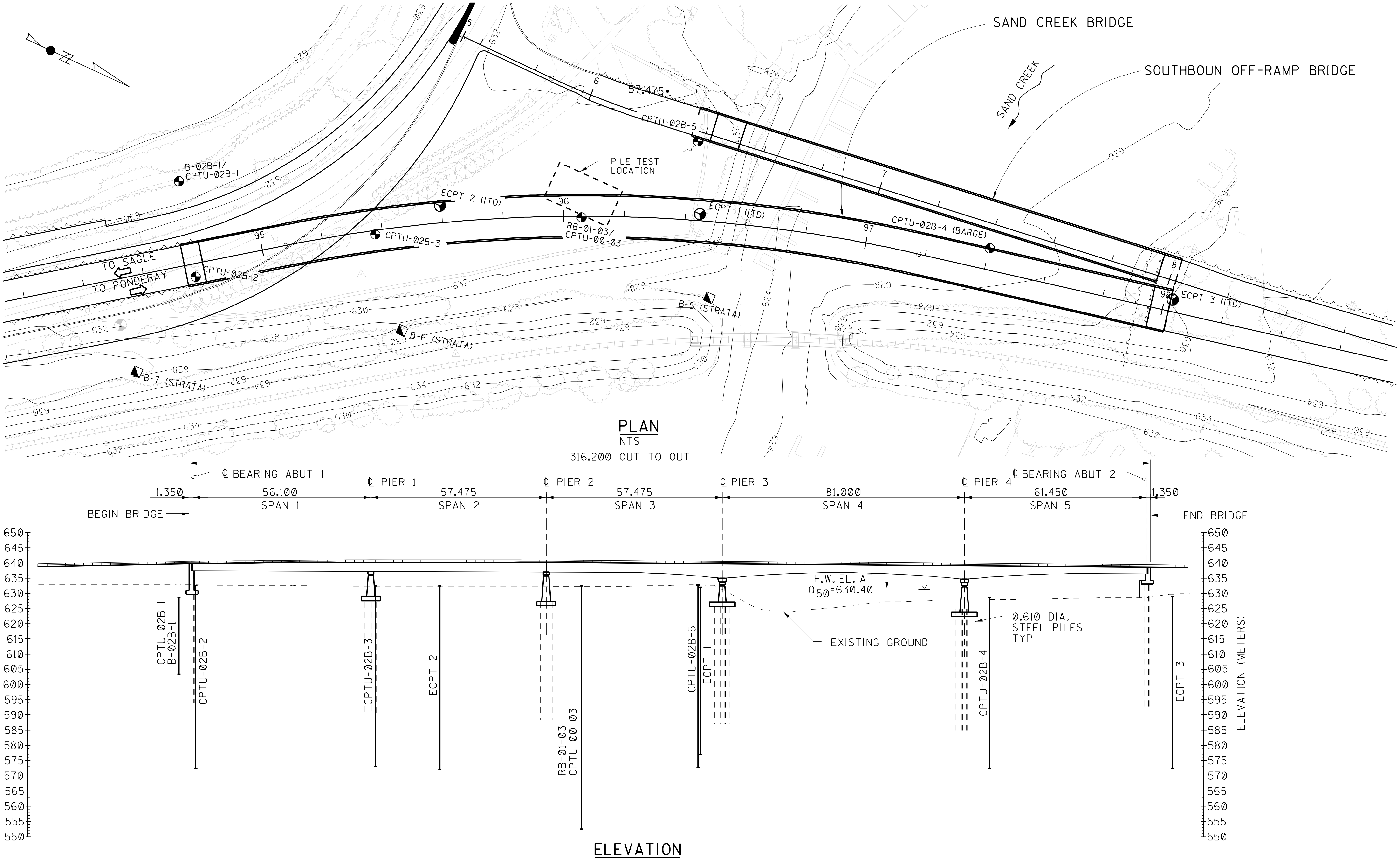

FIGURE 2
SITE PLAN AND ELEVATION
SAND CREEK CROSSING
ALL DIMENSIONS ARE IN METERS

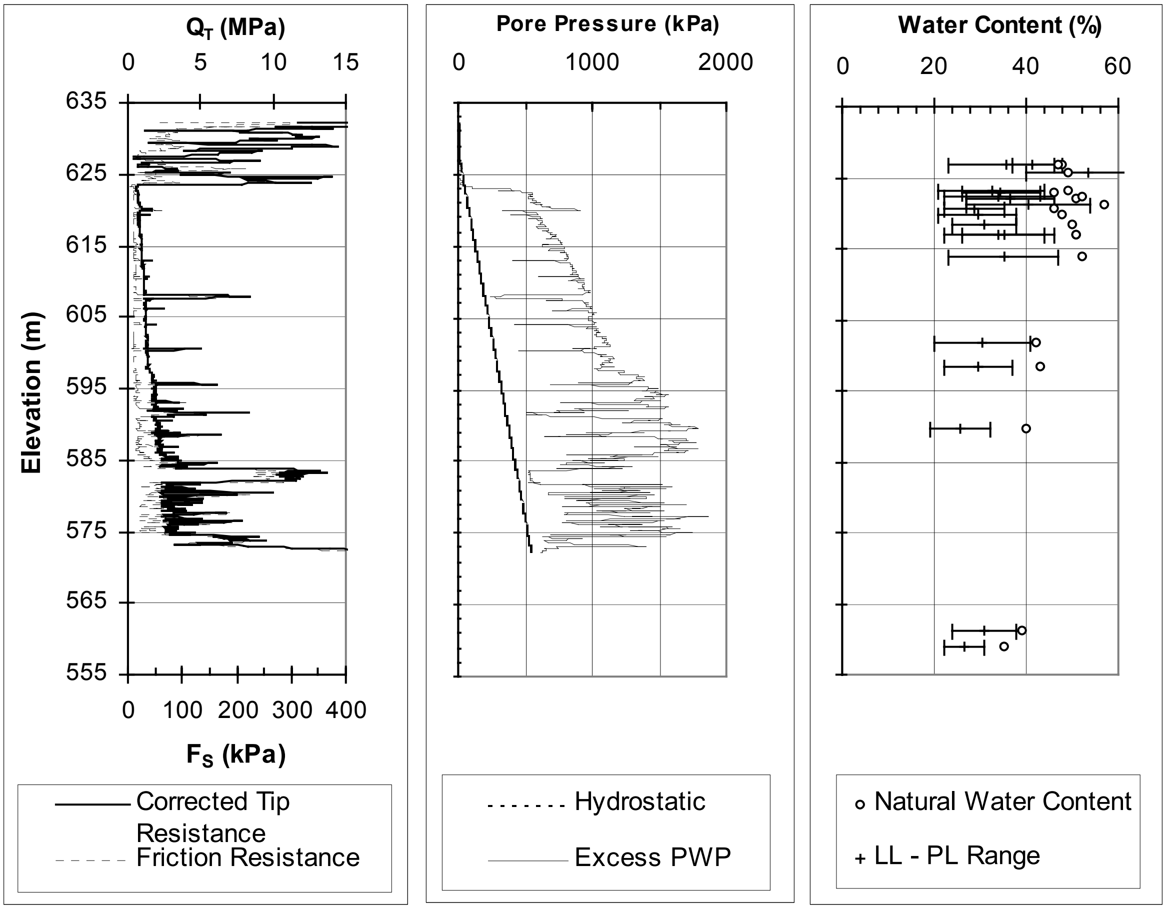

FIGURE 3
Soil Profile at Sand Creek Crossing.

Silt

Below the clay unit, the explorations encountered an alternating sequence of low-plasticity silt, silt with sand, and sandy silt and clay. This sequence is referred to collectively as the silt unit because it is predominantly composed of low-plasticity material. Based on blow counts, the consistency of the silt unit ranges from soft to very hard. Particular interbeds of clay and sand were identified within the silt unit across the project, at consistent elevations and thicknesses. The bottom of the silt layer, where encountered, was between elevation 566 and 570 m.

Lower Clay

Below the silt unit, soft to medium clay was again encountered in the explorations to the termination of the explorations at a maximum depth of 80 m below ground surface (approximate elevation 552 to 556 m).

Groundwater

Groundwater was encountered in the upper terrace at an elevation of approximately 634 m in the north end of the project, and at levels that coincide with seasonal lake level fluctuations in the south end of the project (approximately 626 to 628 m).

In the silt unit and lower clay unit, the piezometers indicate artesian conditions, with an artesian head 1.0 to 2.5 m above the ground surface.

SOIL TESTING

Vane shear testing was performed in two soil borings. Vane shear test data were used in evaluating the undrained shear strength, s_u, of the clay and to provide a basis for comparison with laboratory strength tests and empirical correlations from CPTu measurements.

Five vane shear tests were performed between elevation 621.1 m and 611.7 m. The peak undrained shear strength, s_u, ranged from 33 to 39 kPa, and remolded s_u values ranged from 6 to 10 kPa. Though the shear strength was observed to drop significantly and then “level-off” at the completion of the test, the residual strength is likely lower than the reported value because the tests were completed to less than 1 full revolution, instead of 10 revolutions as is typically done for residual strength determination.

Consolidation testing on samples from the upper terrace soil and the clay layer indicate that the soil is overconsolidated. It appears that this overconsolidation is the result of unloading due to erosion of the ground surface by Sand Creek.

A significant number of triaxial shear tests were completed to interpret values of the undrained shear strength, s_u. The triaxial tests included unconsolidated, undrained (UU) tests and isotropically consolidated undrained (CIUC) tests. The CIUC tests were used to estimate effective stress strength parameters.

At the South Interchange Test Pile site, s_u from UU tests ranged from 10 to 37 kPa in the clay unit, with an average of 26 kPa. The CIUC tests indicated a range of 35 to 60 kPa, with an average of 32 for the same depth range. Values of s_u from vane testing ranged from 35 to 39, with an average of 37. A profile of estimated undrained shear strengths for the test pile site is presented as Figure 4. The profile includes estimates of the undrained strength from the CPTu advanced at the site. The profile is based on a correlation presented by Kulhawy and Mayne (1990), and is shown below:

$$s_u = (q_T - \sigma_{VO})/N_K$$

s_u = undrained shear strength

q_T = corrected cone tip resistance

σ_{VO} = vertical stress

N_K = cone bearing factor, assumed equal to 17

The effective angle of internal friction, ϕ', was determined using the maximum shear stress failure point from the test with the highest confining stress in each test series. This results in ϕ' ranging from 22 to 31 degrees for normally consolidated clay, with an average value of 27 degrees. Axial strains of 2 to 5 percent were required for the maximum shear stress to mobilize. Specimens from the upper sand, silt, and clay unit indicate a ϕ' ranging from 32 to 34 degrees.

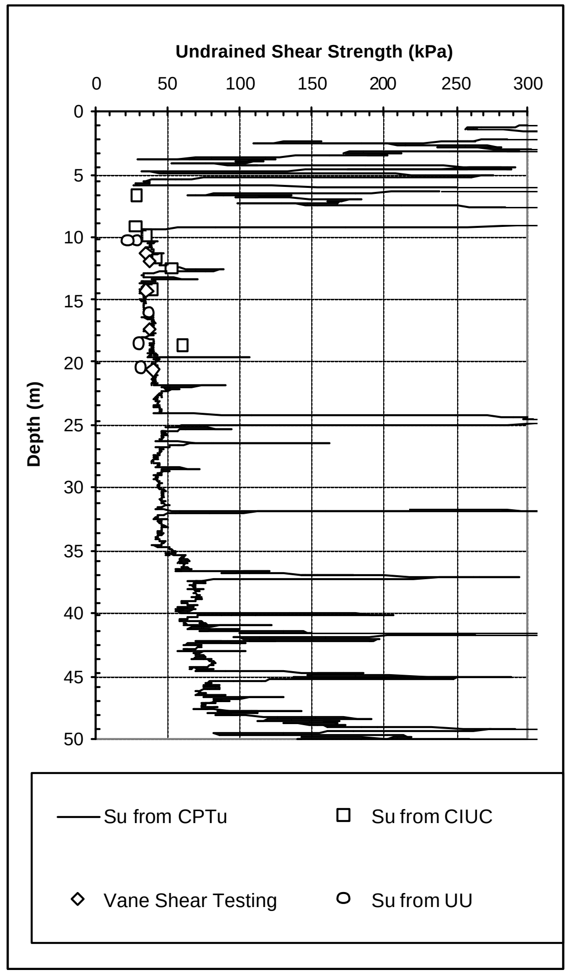

FIGURE 4

Undrained Strength Profile

FOUNDATION EVALUATION

The Sand Creek Crossing is the largest structure included in the project. The structure will be a 5-span bridge with span lengths ranging from 57 to 81 m. MSE retaining walls will be used in the approach embankments at each end. The bridge will be supported using 0.61-m-diameter steel pipe piles. The pile design is based on the unified method of analysis (Fellenius, 1999) which considers pile capacity for dead and live loads, pile structural strength for dead and drag loads, and pile settlement from pile loads and embankment loads (downdrag). To implement this analysis for the Sand Creek Crossing, the minimum pile lengths to meet the ultimate capacity requirements were determined. Then, by considering dead loads and potential drag loads, the depth to the neutral plane was estimated and compared to the estimated settlement profile to consider pile settlement (downdrag).

Axial Capacity of Piles

The project scope was developed to include a static loading test during the design phase, so that the findings could be extended to the foundations for several bridges included in the project. Findings from the test were compared to several methods of estimating pile capacity. The effective stress or "β" method was primarily used to extend the results to other locations.

Loading test

A static loading test was completed on a 0.41-m pipe pile, at a site within the limits of the proposed South Interchange. The test pile was driven to a depth of 45 m below ground surface on August 31, 2001. Vibrating wire strain gauges were placed at eight locations within the pile, and the pile was filled with portland cement grout. Instrumentation also included two telltales. The pile test was performed with a reaction frame anchored by six reaction piles. Complete details of the loading test and the findings are presented in Fellenius et al. (2003).

The static loading test was carried out on October 15, 2001. The delay between pile driving and testing was intentional, so that pore pressures from pile driving could dissipate. Pore pressure readings were measured in vibrating wire pore pressure gauges that were located approximately 1.3 m away from the test pile.

Load displacement data from the test is presented in Figure 5. The pile reached an ultimate load of 1,915 kN, after which the pile plunged and it was not possible to increase the load. In fact, the pile response softened slightly after the plunging failure occurred. The total movement at the head of the pile at failure was only about 11 mm.

The pile side and toe resistance were interpreted with consideration of the effects of residual load. The findings were then used to estimate effective stress parameters. In the effective stress method, the unit shaft resistance, r_s, and the toe bearing resistance are calculated as follows:

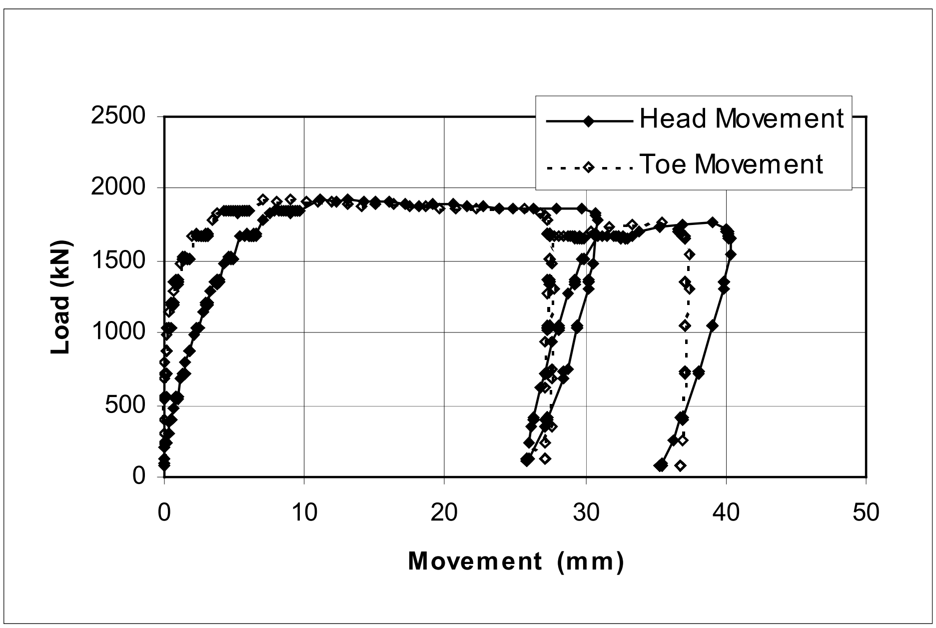

FIGURE 5
Findings from Pile Loading Test

$$r_s = \beta \sigma'_z$$

$$R_t = A_t N_t \sigma'_{Z=D}$$

r_s = unit shaft resistance

β= Bjerrum-Burland beta-coefficient (Bjerrum, et al., 1965; 1969; and Burland, 1973)

σ'_Z = effective vertical stress at depth $_z$

R_t = toe resistance

A_t = pile toe cross section area

$\sigma_{Z=D}$ = effective vertical stress at toe

N_t = toe bearing capacity coefficient (Meyerhof, 1976)

It was found that the results of the pile test were best matched with β values of 0.70 in the upper silty and sandy material, and a value of 0.11 in the clay. A toe bearing coefficient, Nt of 6 was determined.

The interpreted β from the clay is quite low in comparison to commonly used values. For clay, β is more typically anticipated to range between 0.25 and 0.35 (Fellenius, 1999).

Because the effective stress parameters are so low compared to anticipated values, it was useful to compare the findings to other methods of pile capacity calculation. The other methods included the direct CPTU method (Eslami and Fellenius, 1997) and the total stress method. Table 1 presents the results of these estimates for the test pile.

In the Eslami-Fellenius method, the difference between the corrected cone stress , Q_T, and the pore pressure response, U_2, is defined as the effective cone stress. The pile shaft resistance is then estimated as the product of the effective cone stress and a factor C_S, which is a constant that varies by soil type. Soil types are classified according to empirical information developed by Eslami and Fellenius (1997). The pile toe resistance is taken to be equal to the effective cone resistance through an influence zone that extends above and below the anticipated depth of the pile toe. The estimates using the direct cone method compare very well and indicate that the method is useful for evaluating capacity at other locations for the Sandpoint North-South Project.

An estimated undrained strength ranging from 35 to 75 was used in a total stress analysis. For this strength range, an "α" value of 1.0 is typically considered appropriate (FHWA, 1996). However, the results significantly overestimated the capacity of the pile. It was found that an α-value of 0.5 to 0.7 would be required to match the findings from the loading test.

Pile Length for Axial Capacity

The Sand Creek structure is being designed with 0.61-m piles. Based on a preliminary layout of the pile cap, the estimated unfactored dead loads range from 635 to 1,100 kN per pile and an ultimate capacity (unfactored) of 1,700 to 3,200 kN is required. The higher loads occur at the bridge pier locations where no fill is expected to occur. Using the effective stress parameters discussed above, required pile lengths ranging from about 20 m at the abutments to about 50 m at the piers were determined.

Settlement Analysis

MSE walls with heights from about 7 to 9 m will be used for the approach embankments at the south and north end of the bridge, respectively. These embankments are expected to cause 150

TABLE 1
Correlation between Loading Test and Pile Capacity Analysis

	Loading Test (kN)	Direct CPTU Method (kN)	Effective Stress Analysis (kN)[1]	Total Stress Analysis (kN)[2]
Ultimate Capacity (kN)	1,915	2,030	2,040	2595
Side Resistance (kN)	1,685	1,880	1,732	2,515
Toe Resistance (kN)	230	150	308	80

[1] – β (Upper sand/silt) = 0.6
β (clay) = 0.1
N_t (clay) = 6

[2] – α (clay) = 1.0

to 300 mm of settlement resulting from consolidation of the deep clay layer. A profile of the estimated settlement at the north abutment (Abutment 2) is shown in Figure 6.

Because of the magnitude of settlement at the abutments, it is anticipated that negative skin friction (drag loads) will develop. If the piles are not of suitable length, the embankment settlement could cause significant settlement (downdrag) of the piles.

In the unified method, drag loads and downdrag are evaluated using a load and resistance diagram to consider the magnitude of downward acting loads compared to the available resistance. Because the analysis is meant to consider long-term conditions contributing to settlement, the relevant downward loads include the dead load at the head of the pile and the negative skin friction (drag load) along the entire length of the pile. In the load and resistance diagram, these are summed from the head of the pile downward. If the toe resistance and positive side resistance are summed from the toe of the pile upward, the depth to the "neutral plane" can be established at the intersection of these two curves. The neutral plane represents the depth at which the downward loads are at equilibrium with the positive resistance. Theoretically, the pile movement is equal to the soil movement at the neutral plane. Therefore, the settlement of the pile head is equal to the sum of the movement at the neutral plane, load transfer movement, and the elastic compression of the pile. Both of the latter two components of settlement are relatively small.

Figure 6 includes a load and resistance diagram for a 21-m pile, which is the minimum length to develop the required capacity at Abutment 2. The depth to the neutral plane for this pile is estimated to be about 4 m. The estimated settlement at this depth is 200 mm. It is desired

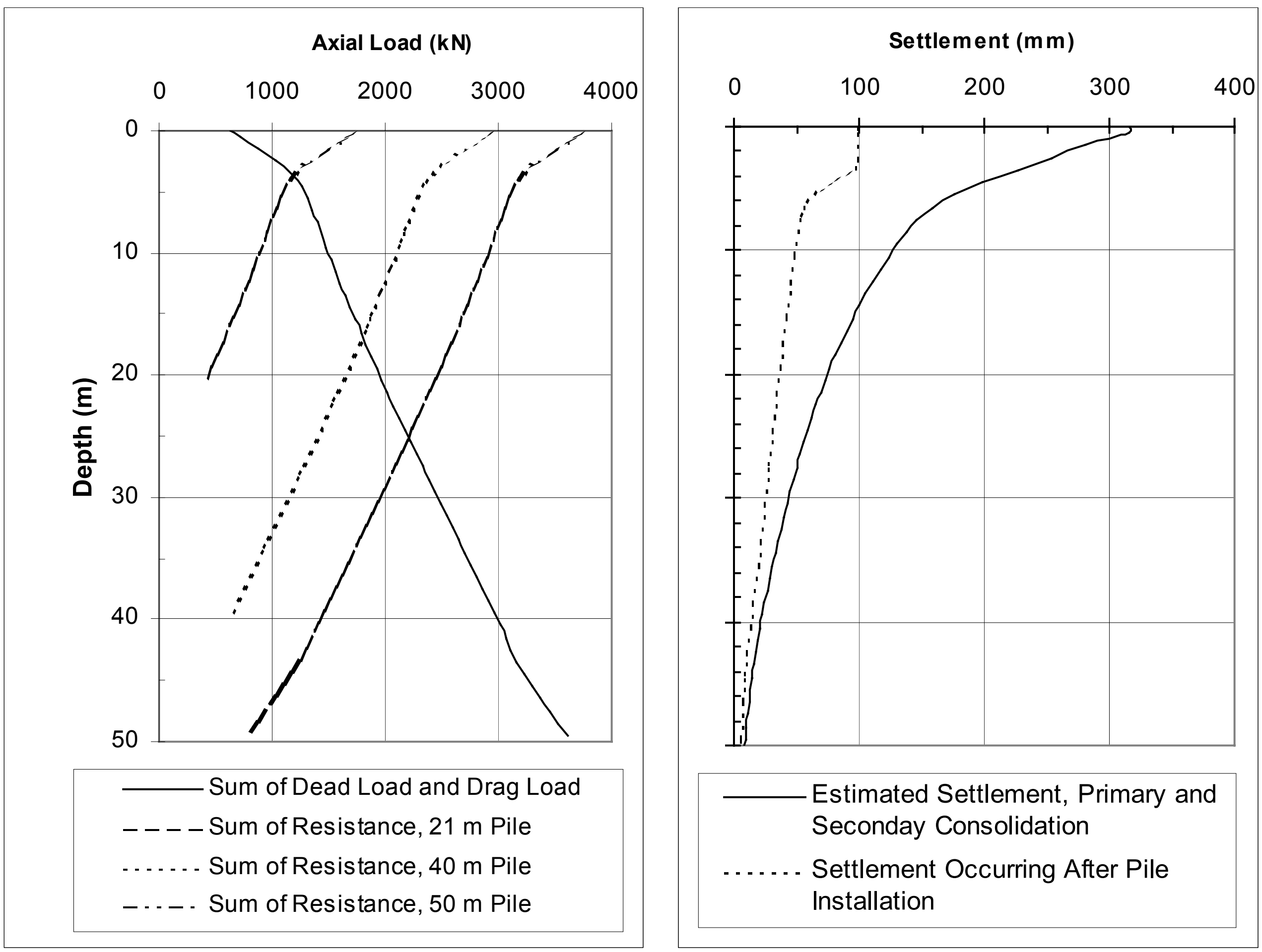

FIGURE 6
Load and Resistance Profile and Settlement Profile

to limit differential settlement to a maximum value of about 50 mm. Pier 4, the closest pier to Abutment 2, is expected to undergo very little settlement, because there are no approach embankments nearby. Therefore, a longer pile is desired to limit settlement.

Load and resistance diagrams for a 40-m and a 50-m pile are also shown in Figure 6. It is estimated that the depth to the neutral plane for these two piles is 18 m and 25 m, respectively. The estimated settlements are 80 mm and 50 mm.

Prefabricated vertical drains (wick drains) will be used to accelerate the consolidation of clay beneath the embankments. If the foundation piles are driven after the embankment settlement reaches a value corresponding to 90 percent consolidation (t_{90}), then the settlement affecting piles will be less than the values discussed above. The settlement at the neutral plane occurring after reaching t_{90} and driving piles is estimated to be 50 mm and 35 mm for 40-m and 50-m piles, respectively.

Differential settlement between the pile-supported bridge abutment and the embankment approach fill must also be limited to maintain a relatively smooth transition from the approach fill to the bridge deck. This can be partly addressed by using an approach slab for the structure. Briaud (1997) recommends that rotation of approach slabs not exceed 1/200. For the pile lengths discussed above, the settlement of the bridge structure is anticipated to be on the order of 25 to 50 mm. The embankment surface is anticipated to experience a total settlement on the order of 100 mm, including secondary compression. Therefore, an approach slab length of 10 m is required for the structure.

SUMMARY AND CONCLUSIONS

Design of pile foundations for the Sand Creek Byway presented challenges to the design team because of the softness of the material and the interrelationship between pile capacity, pile settlement, and embankment conditions. The use of effective stress analysis and the unified pile design concept allowed geotechnical engineers on the project to evaluate these factors in a rational methodology.

Important conclusions from the project include:

- For unusual soil conditions such as these, site-specific pile loading-test data are very important in estimating the pile capacity. Findings from the loading test performed at the Sandpoint Site indicated pile design parameters significantly lower than those that might have been selected from published values for other sites.
- The success of existing pile capacity predictive methods can vary, depending on the soil conditions at a particular site. For this project the CPTu predictive method was very successful, while capacities predicted using undrained strengths, as might be normally be done for this type of soil, led to unconservative capacity predictions. At other sites a different conclusion could be reached – clearly indicating that it is beneficial to consider alternate capacity predictive methods when evaluating pile response.
- Because of the varying embankment conditions and lower foundation loads occurring at the abutments, selection of the design pile length will be primarily a function of the allowable settlement that the structure can tolerate rather than the plunging capacity of the piles. The differential settlement of the structure between abutment and pier locations, along with the expected differential settlement between the structure and the approach embankment, must be considered.
- The anticipated sequence of bridge construction is important to understanding the settlement that may affect piles. In this case, because wick drains are being used to speed the consolidation of the clay, driving piles after the embankment reaches a settlement corresponding to average of 90 percent of consolidation is expected to limit differential settlement to tolerable levels.

ACKNOWLEDGEMENTS

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