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Seattle Freeway Construction, Interstate Highway 5, 1960-1966

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INTRODUCTION

The 65-mi-long portion of Interstate Highway 5 (I-5) commonly known as the Seattle Freeway extends from Tacoma on the south to Everett, about 20 mi north of Seattle. About 20 mi of the route are within the city limits of Seattle (Figure 1). This paper deals primarily with design considerations, construction problems, and remedial solutions, many of which were considered to be innovative or state of the art in the early 1960s when this project was constructed. Many of the design and construction approaches developed for this project were so successful that they have become nearly standard procedures in the Seattle area and are being used on the I-90 project currently under construction.

The initial fieldwork on the project began in 1949, when the Washington State Department of Highways (WSDOH) and Seattle City Engineering Department conferred and selected the present route as being less disruptive to occupied property than other parallel alignments farther east along the crest of Beacon Hill and on Capitol Hill. A considerable portion of the chosen alignment was known to have experienced shallow slides in the weathered, weakened surficial soils and therefore was expected to require costly and extensive stabilization procedures not needed on the other routes. However, the land was not considered suitable for home or commercial building sites, and consequently the acquisition costs were low enough to offset the costs of remedial measures and to outweigh the costs of acquiring more stable, but more highly developed lands to the east or west.

Subsequent to the selection of a route, a preliminary boring program was formulated, and in 1951 the Raymond Concrete Pile Co. was engaged for drilling and sampling several exploratory holes in the James Street area, immediately west of Harborview Hospital. Two additional borings were drilled where the route crosses the Lake Washington Ship Canal. This program was conceived and inspected by the WSDOH District 1 soils crew, and samples obtained from the program were tested at the WSDOH headquarters laboratory in Olympia. Shear strength tests on samples of the hard clay indicated it to be sufficiently strong in its present condition for the anticipated cut depths and slopes.

At the time of this initial study, tunneling was contemplated in the James Street vicinity; however, a tunnel did not appear feasible, and in 1952 several additional borings were drilled in other critical areas to the north and south of the study area. These indicated that open cutting was feasible.

The WSDOH appointed an engineering firm consisting of a joint venture of the Tudor and J. E. Greiner engineering companies as consultant and agreed to furnish all existing boring and laboratory test data to the firm's geologists and soil specialists for review. The consultant agreed with the WSDOH that construction in the proposed route was feasible but noted concerns about several areas, some of which will be discussed later in this paper. As one example, Tudor and Greiner were concerned about permeability and stability of the soils immediately east of Angle Lake, several miles south of Seattle. At that point the freeway would cut into soils that underlie the lake, and engineers feared that heavy seepage might drain the lake or cause catastrophic slides. Additional borings in the lake area indicated the presence of till with very low permeability, which allayed fears of leakage.

In 1954, it appeared that the only way that the project could be financed would be by designating it a toll road; at that time this was a popular method of financing construction in many of the densely populated cities in the eastern states. The toll facility had been planned as a six-lane project in rural localities. An additional lane in each direction was planned for the urban portions, and sparse access and exit facilities were scheduled for the downtown business sections. However, in 1956 the Federal Highway Act established 90 percent federal financing for major projects, and plans were made by WSDOH to reconfigure the route to accommodate traffic heavier than originally contemplated.

Accordingly, additional lanes were incorporated in the plans, primarily in the downtown section, and per-

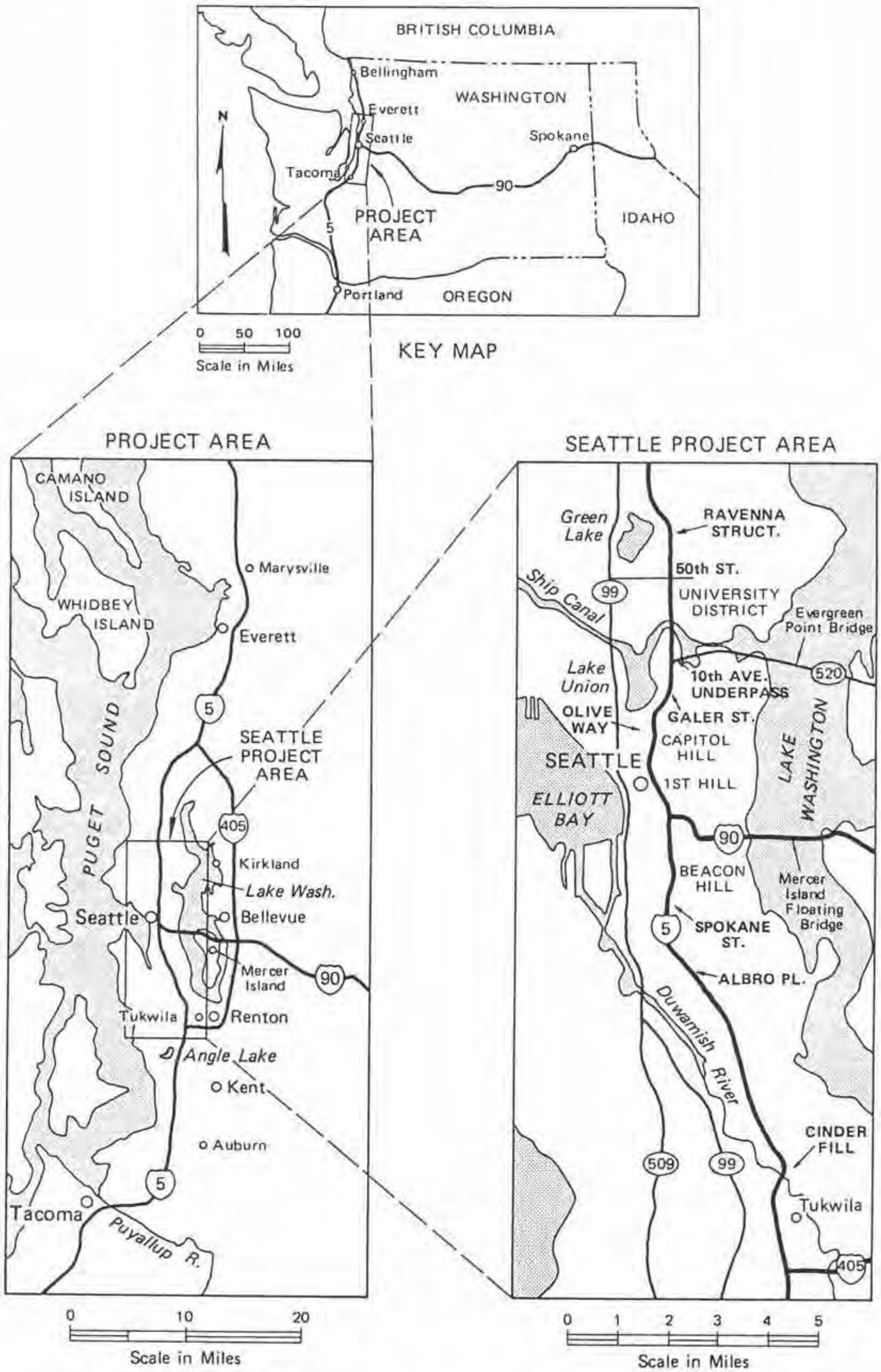

Figure 1. Location of the Seattle Freeway construction project.

mission was obtained from the Bureau of Public Roads to add a four-lane roadway between the north- and southbound lanes to accommodate the heavy traffic anticipated from the highly populated area north of the city. These were designed as reversible lanes, to be southbound in the mornings and northbound in the evenings. They extended from the Seattle central business core to a point approximately 2 mi south of the Snohomish County line, approximately 12 mi north of downtown Seattle.

The incorporation of reversible lanes necessitated a greater overall width of the roadway. In the downtown section from the ship canal to First Hill (Figure 1), the limited right-of-way width dictated the use of double-decked bridge and elevated highway structures. Considerable cut-and-cover tunnel construction was required to accommodate reversible traffic under the north- and southbound lanes and to facilitate access to city streets at specific places in the downtown area. Consequently, construction excavations became substantially larger and deeper than originally anticipated.

Prior to mid-1950, few reversible highways had been built in any part of the nation, and considerable political pressure was required in order to obtain permission to incorporate that feature in the project plans. Arguing in favor of the space-saving design was the fact that the local geography limited the number of parallel routes. Seattle is situated on a narrow strip of land between Puget Sound on the west and Lake Washington on the east; this reduces the north-south corridor width to slightly less than 2-1/2 mi at the narrowest point. At that time, the city's main traffic flow plan included three major routes: the recently completed Alaskan Way Viaduct adjacent to Puget Sound, the proposed central Seattle Freeway (I-5) a short distance to the east, and a contemplated R. H. Thompson Expressway along the east edge of the corridor close to Lake Washington. Although initial design has been completed, the Thompson route has not been built and will probably not be constructed within the foreseeable future. Consequently, the reversible lanes added to the freeway complement the two existing routes, which are now loaded to capacity during heavy traffic periods.

The major obstacles to completion of the Thompson Expressway appear to be its partial encroachment into the University of Washington Arboretum and the high construction costs of each of the alternate proposals for crossing the Lake Washington Ship Canal and the westerly approach for the Evergreen Point Bridge, immediately adjacent to the canal. Both alternative bridge or tunnel plans considered in the early 1970s appeared to have costs approaching or exceeding the entire cost of the remaining portions of the freeway route. Any bridge would have to be constructed with a clearance similar to that of the Aurora and I-5 crossings, and a tunnel or sunken tube would require water depth clearance sufficient to allow ships with considerable draft to pass. Soil borings indicated a thick sequence of soft organic soils which would have to be removed and replaced along major portions of the aesthetically preferred tunnel (Figure 2). The presence of this soft compressible soil would also complicate construction and increase foundation costs for any bridge.

PHYSIOGRAPHY AND GEOLOGY

The Seattle Freeway route lies within the Puget Lowland. The elongate north-trending lowland is located between the Olympic Mountains to the west and Cascade Range to the east. The project area lies along the western edge of a series of south-trending low hills between Puget Sound and Lake Washington. In the downtown area these hills are from north to south: Capitol Hill, First Hill, and Beacon Hill (Figure 1).

In the early 1960s, at the time of the Seattle Freeway construction, general understanding of local glacial geology held that continental glaciers had advanced from western Canada southward into the Puget Sound basin a minimum of four times. The latest of these advances occurred approximately 15,000 yr ago, and the ice retreated about 1,500 yr later. Geologic evidence indicates that the ice was at least 3,000 ft thick in the Seattle area, was about 60 mi wide at the distal end, and extended about 50 mi south to between Olympia and Centralia at its southern limit. The generally south-trending elongate hills and valleys were sculpted by the advance and retreat of this last ice mass.

The advancing continental ice sheet also blocked the normal drainage patterns to the ocean via the Strait of Juan de Fuca, causing the Puget Sound basin to fill with glaciolacustrine sediments. Massive to varved silts and clays, exceeding several hundred feet in thickness, were deposited in this large proglacial lake. These sediments were subsequently compacted by glacial loading as the ice re-advanced. These silts and clays commonly contain lenses of ice-rafted sand and gravel and scattered boulders. These are the predominant soils along the downtown Seattle section of the alignment.

Successive glacial advances deposited till, pasted and molded over the underlying ground surface. Ice loading plus the presence of natural cementation materials produced till of various degrees of hardness. Till was encountered along the freeway route, principally in the Lake Washington Ship Canal Bridge footing locations and in the deep cut sections around 45th and 50th Streets.

During the final glacial retreat, when the Puget Sound was no longer blocked, drainage from the wasting ice eroded channels in the hard clays, silts, and till. Loose sands and silts were deposited in these deep depressions, such as those prevalent in the Galer-Lakeview structure footing locations.

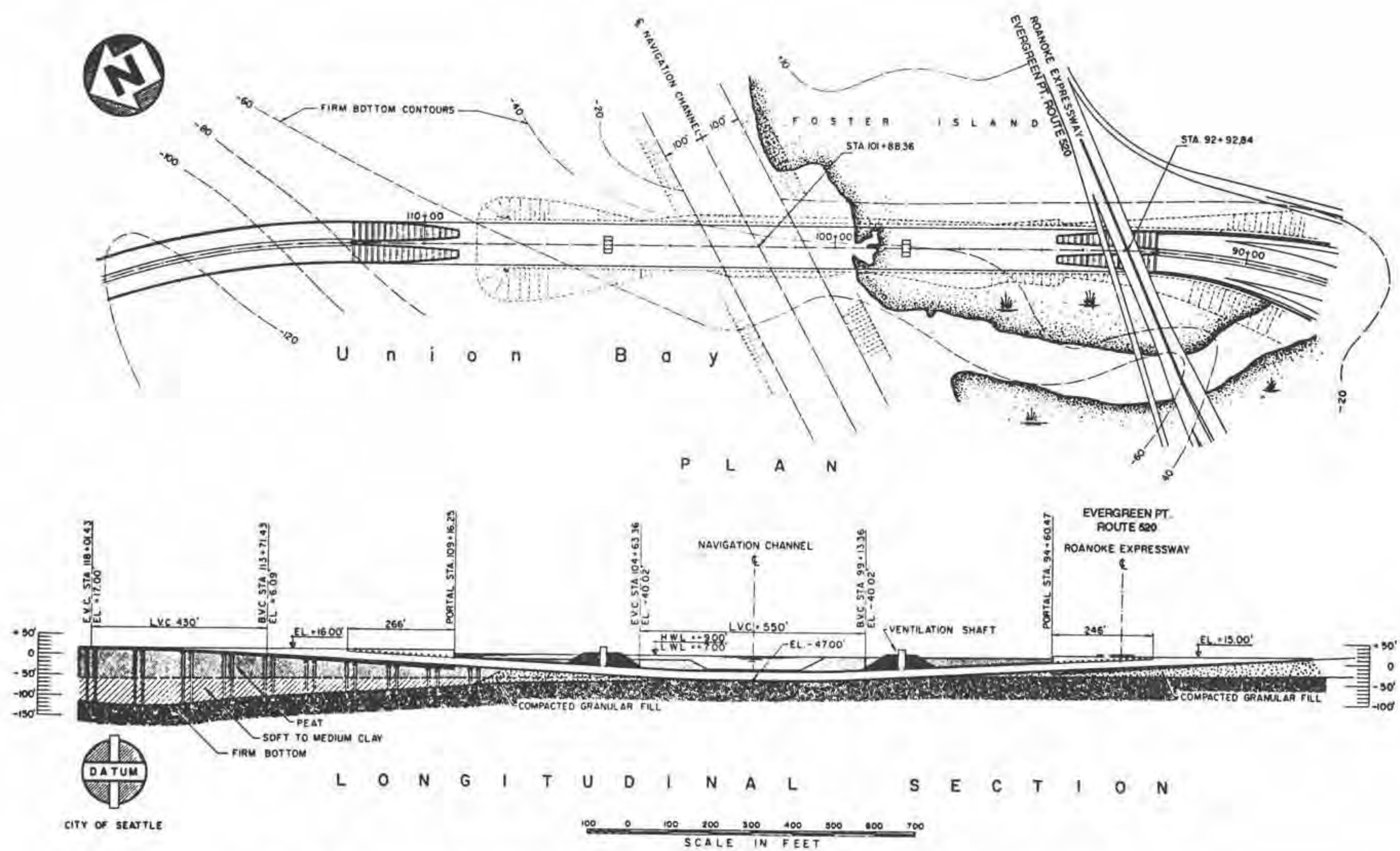

Figure 2. Plan and longitudinal section of a sunken tube tunnel for the conceptualized R. H. Thompson Expressway across the Union Bay Ship Canal. Note the thick sequence of soft soils at the north end of the alignment.

Subsequent to the last glacial retreat, depressions in the glacially sculpted topography became lakes. Large thicknesses of soft clays and silts and decaying vegetation accumulated in these lakes, resulting in near-surface deposits of soft, organic silt and clay in many areas of the city. Deposits of peat and organic silt and clay were encountered in freeway excavations at the 50th Street and Ravenna Bridge areas.

TEST CUTS

During the early stages of soils investigation along the freeway route, the potentially most adverse soil condition seemed to be present at the south end of First Hill (Figure 1) near James Street, where the greatest planned cut depth (approximately 100 ft) would be required for the desired roadway grade. Although nearly all laboratory analyses of samples obtained along the route indicated excavation of that magnitude would be feasible (Andrews, 1964), the WSDOH had no previous experience with glacially loaded, hard clays of this type. Therefore, a large test cut was proposed, and contract documents were prepared to accomplish the work.

Prior to the design of the test cut, a letter of inquiry was circulated to several state and federal agencies soliciting information that would aid in the design. Although none of the agencies contacted had dealt with clay cuts of this magnitude, several suggested supporting the clay soils with gravel buttresses and protecting cut slopes against erosion.

The test cut was begun in 1960 along the toe of Beacon Hill, south of the business district. However, close observation and monitoring of the uphill area disclosed movement in the hard clays soon after the work began. Relatively shallow excavations near the toe of the proposed cuts promptly triggered slides extending a considerable distance upslope.

During the winter of 1960 when no on-site work was being done, plans were revised to develop a method for preventing this type of failure on future cuts and to provide permanent protection for the cut slopes from seasonal weathering. During the 1961 construction season, the planned revision employed a heavy riprap buttress to support 1.3H to 1V and 1H to 1V clay cuts. The buttress was constructed with a 1.5H to 1V slope against a maximum 35-ft-high clay cut face. The length of open cut was limited to 100 ft prior to placement of buttress material. This procedure proved to be successful within the test cut section and seemed to substantiate the results of stability analyses using laboratory strength data obtained during previous investigations.

The experience gained from the test cut provided credence for using similar limited temporary excavation lengths and buttress construction for other deep excavations along the alignment, including the proposed 100-ft-deep cut at the south end of First Hill. WSDOH engineers also felt the laboratory data, borings, and test cut experience indicated that shallower cuts along other portions of the route, including Capitol Hill and the northern portion of First Hill could be supported using conventional retaining walls founded on either spread footing or piles.

Gravel- or rock-filled "key slots" were also assessed as a stabilizing procedure for smaller cuts. During the 1930s, when considerable federal financing was available for public works projects, attempts were made to stabilize some of the slide areas on Capitol Hill by using hand-dug slots or trenches, 3 to 5 ft wide, extending through the slide material down into the undisturbed soils. The slots were backfilled with cobble-sized gravel to provide drainage and to key the slide-prone material to the more stable underlying soils. Excavations on Capitol Hill during some of the earliest freeway construction disclosed that these key slots had been successful in most places, but that silt had intruded into the cobble fill to such an extent that drainage had been entirely impeded. However, the cobble fill continued to key the soil mass across the slide plane and was still partially effective.

Historically, in other parts of the county, rock blocks hand-placed in trenches had been generally successful in stabilizing slope movements. Consequently, a similar installation was incorporated into the contract for a buttress test section in the sidehill cut area immediately south of Capitol Hill. Slots a maximum of 15 ft deep were excavated. The slots were backfilled with heavy quarry rock mixed with well-graded gravel backfill to maintain long-term drainage. Key slots of this type were installed in several critical localities along Beacon Hill on subsequent contracts, and in one instance key slots successfully arrested an active slide which had developed during construction of the south slope on a major connecting highway a short distance west of the Tukwila Interchange.

CAPITOL HILL

Early in 1961, two contracts were awarded for a section of the alignment just north of the central business district. This portion of the route is in the steepest part of the west slope of the prominent, densely developed, south-trending ridge known as Capitol Hill. Deep excavation was required in the hard clayey soil that dominated the freeway alignment south of the Lake Washington Ship Canal. The abruptness of the slope and the grade differential among the northbound, reversible, and southbound lanes made it necessary to construct large retaining walls between the parallel lanes. To protect the excavations, conventional shoring was used for support of the adjacent streets and several nearby large buildings.

The contract plans required a 200-ft maximum limit on the length of open excavation that could be made for wall construction in any area until wall support was constructed. A minimum 200-ft section of undisturbed soil was specified to be retained as an intervening buttress between adjacent areas. Once the permanent slope support was installed in these separate discrete excavations, then the intervening natural soil buttress was to be excavated and the slope support members completed to form a continuous wall. This procedure was intended as a preventive measure to deter development of extensive slide movement while permanent support was placed.

Shortly after the temporary sheeting and lagging was installed, the contractor observed severe earth cracking and indications of deep-seated movement extending below the base of footings constructed as part of the wall construction. Most of the damage was limited to the street areas; however, some building damage did occur. Near Olive Way and Melrose Avenue, an open excavation on the order of 30 ft deep caused a slide scarp approximately one city block uphill immediately adjacent to a large apartment complex (Figure 3). Examination of the upper and lower limits of the movement zone at Olive Way indicated that the slide plane was 15 to 20 ft below the area where the northbound ramp was to be constructed.

CYLINDER PILE SOLUTION

In search for a method which would prevent further uphill extension of the slide area, WSDOH selected the novel use of cast-in-place concrete piles with I-beam reinforcement to stabilize the hillside. The drilling equipment and the materials were readily available in the Seattle area, having been used for deep foundations. The soil appeared to be sufficiently self-supporting for the length of time needed for each pile installation. The approximately 60-ft-deep piles were 16 in. in diameter and were placed about 3 ft apart along the uphill side of the planned Olive Way northbound ramp alignment. Initially, every fourth pile along the approximately 125-ft length of the alignment was installed to provide some measure of slide resistance prior to completion of the intervening members. This pile support provided shear resistance and prevented further movement, even though subsequent inspection of the site indicated that a number of the piles installed during the initial stages of the operation had been offset by movement along the slide plane. As more piles were added along the alignment, the cumulative supporting strength successfully prevented additional movement uphill.

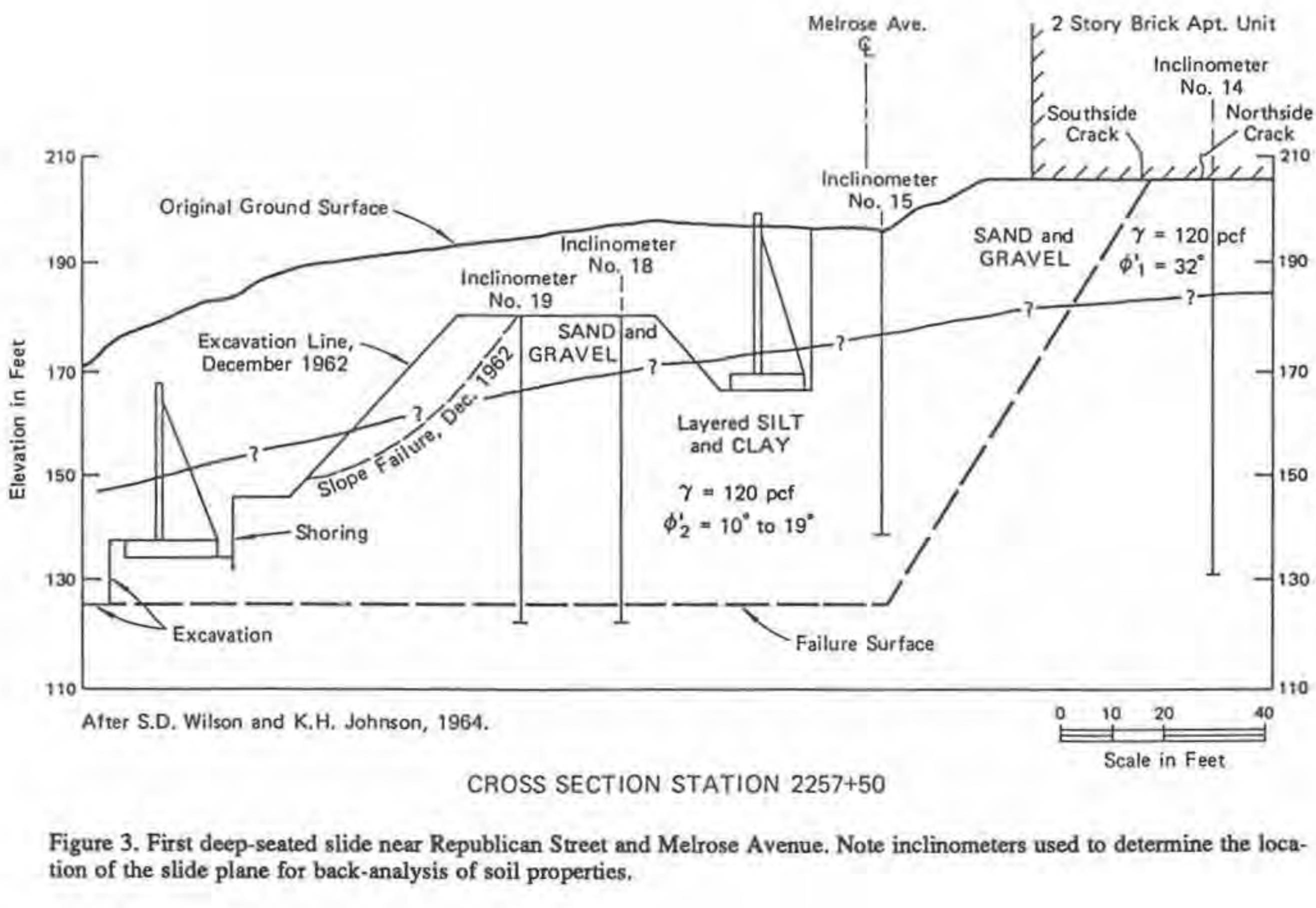

Figure 3. First deep-seated slide near Republican Street and Melrose Avenue. Note inclinometers used to determine the location of the slide plane for back-analysis of soil properties.

Also in the Olive Way area, contract plans required an additional 20-ft depth of excavation close to the base of an existing retaining wall for the construction of a reversible-lane, cut-and-cover tunnel (Figure 4). The retaining wall was supported on driven batter piles, and the wall itself had been redesigned with a solid concrete gravity block to add weight to the footing and make the batter piles more effective. However, this installation, as built, did not appear to be capable of withstanding possible slippage beneath the base of the wall. Consequently, cast-in-place concrete piles were designed and installed as an additional aid to stability; these piles were 4-1/2 ft in diameter and 45 ft deep with 6-ft center spacing. Concrete struts were placed between the tops of the cylinder piles and the base of the previously poured wall footing. The piles and struts prevented further movement.

The success of cast-in-place cylinder support piles in the Olive Way slide area indicated that a similar system could be used to stabilize the areas that had experienced movement during construction of the conventional retaining walls in the contract section immediately to the north of Capitol Hill. Accordingly, plans were revised, and the contractor agreed to replace conventional installations with an adaption of the method employed at the Olive Way location. However, the cylinders were to be drilled immediately behind the face of the planned walls and from the existing ground surface prior to any excavation. The cylinders were of the same size as those used at Olive Way, but most were spaced 7 ft center to center.

It was discovered that concrete intrusion into sand layers locally impeded drilling of adjacent piles. Subsequent piles were drilled 7 ft center to center to provide

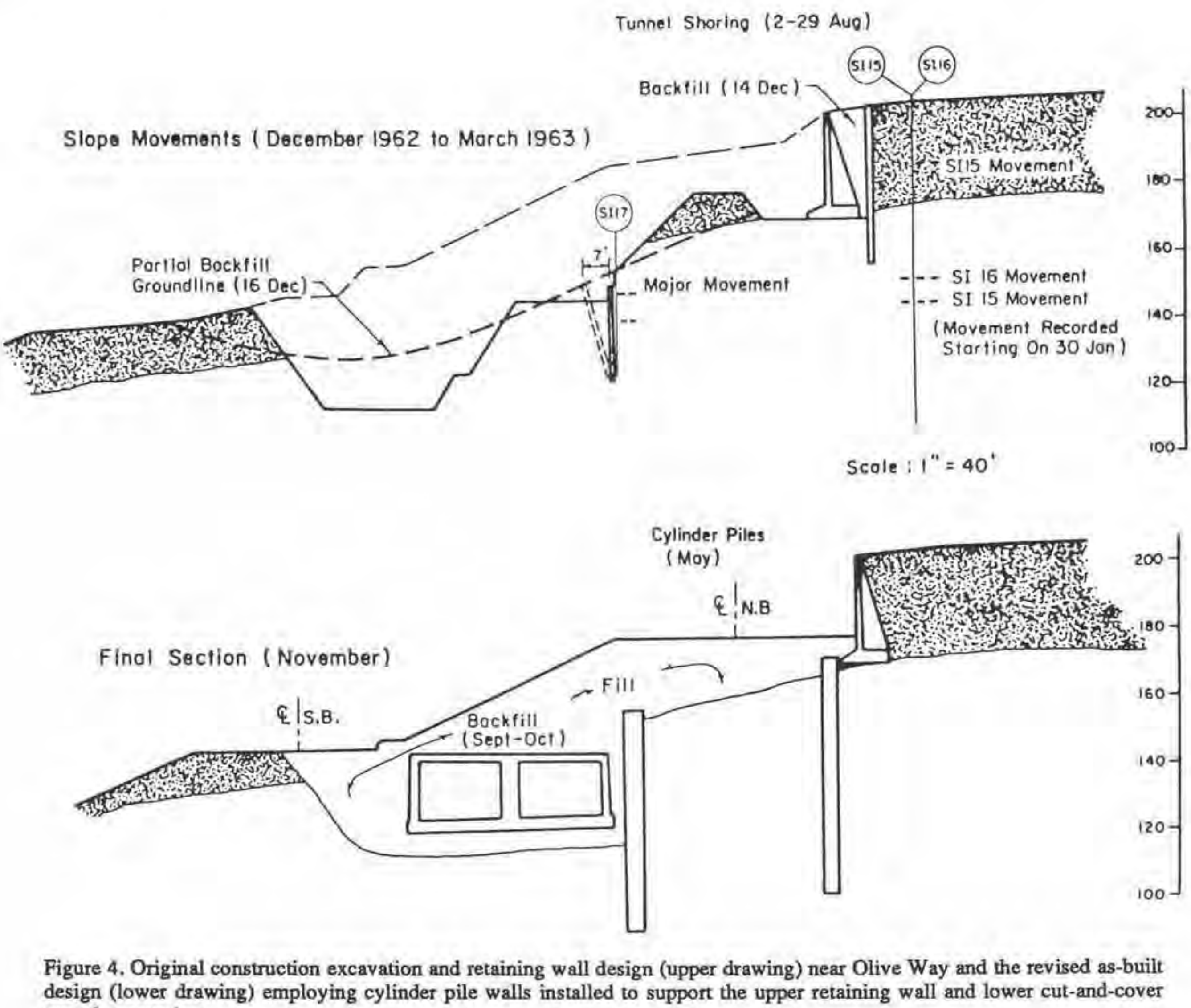

Figure 4. Original construction excavation and retaining wall design (upper drawing) near Olive Way and the revised as-built design (lower drawing) employing cylinder pile walls installed to support the upper retaining wall and lower cut-and-cover tunnel excavation.

a 1-ft gap between piles. Due to problems associated with placement of conventional steel rebar reinforcement, heavy welded steel I-beams were specified; these allowed more precise centering in the drilled shaft and provided a more efficient means for subsequent attachment of curtain walls to the front face of the cylinders.

This newly devised method of wall construction was successful in keying together potential slip planes beneath the exposed wall face and preventing significant earth movement upslope of the cuts. The cylinder piles extended below grade approximately twice the height of the exposed face. This design was initially employed in the intervening 200-ft sections of intact earth buttress specified in the original contract plans. This proved to be a safe method of construction in the highly compressed clays in many freeway excavation locations to the south. Although the cylinders themselves were expensive, considerable savings were achieved in shoring, earth excavation, and amounts of imported free-draining gravel backfill, along with a reduction in construction time.

Due to the uniqueness of the cylinder pile application to slope stabilization, selected piles were instrumented with inclinometers and SR-4 strain gauges. Analyses of deflection and strain gauge data by WSDOH and Shannon & Wilson, Inc., furnished convincing proof that this method of retaining wall construction should be employed in other critical sections of the freeway. Further analyses by the Bureau of Public Roads and its consultant, R. B. Peck, Professor of Soil Mechanics at the University of Illinois, confirmed these conclusions. On the basis of these assessments, the Bureau of Public Roads engineering staff authorized further plan changes in many of the contract sections still to be awarded.

All evidence indicated that initial soil movements were the result of deformations following the release of lateral stress, rather than any increase in shear stress resulting from a steepening of the slope (Wilson and Johnson, 1964). The initial movements were believed to have been independent of rainfall or ground-water conditions. However, the progression of movements farther uphill and continued deterioration of the hillside may have been influenced by the addition of water.

The analyses of WSDOH, Shannon & Wilson, Inc., and R. B. Peck resulted in three major design criteria (Andrews, 1964) for stabilization of the cut slopes in the hard overconsolidated Seattle clays. These criteria were:

(1) Restrict deflection of the top of each cylinder pile to 2 in. to eliminate the tendency to form surface cracks in the uphill soils which would permit the entrance of surface water and deterioration of the hard clays.

(2) Assume a plane of sliding to be either 10 ft below the lowest final ground surface at the point where the deepest excavation is to be made, or 5 ft below the depth of the deepest excavation, whichever is lower.

(3) Establish loading conditions which take into account the pressures exerted by the till cap, the varved clay, and the glacially "locked-in" soil stress.

The cylinder pile concept of retaining wall construction seemed to be appropriate in fulfilling these design criteria. The wall is placed in the soil before downslope excavation has begun, thus reducing the release of confining pressure. Adequate stiffness can be designed into the H-beam cylinder pile supports to restrict wall deflection so that soil movements do not exceed the strength of the soil.

DOWNTOWN AREA AND FIRST HILL

The successful performance of the cylinder pile-supported retaining walls in the Capitol Hill portion of the freeway indicated that this type of construction should be used wherever deep excavations were planned, particularly downtown where large buildings and numerous utilities were present. Several of the conventional types of walls, detailed in the original construction plans, were to be supported on 2-ft-diameter foundation piles because of their height and the limited bearing values of the clay soils upon which they were to be founded. Experience gained along Capitol Hill showed that cylinder pile walls could be installed considerably more quickly than conventional walls and with less interruption of traffic. Consequently, the walls were redesigned as cylinder pile structures.

The deepest cut section along the entire freeway route reached a total depth of about 100 ft in three successive steps at the south end of First Hill, directly below the Harborview Hospital complex and just north of the original shallow test cut. Based on prior experience in drilling and installing the large-diameter casings for cylinder wall construction for military missile installations, it had been established that the largest practical diameter for cylinder piles was 10 ft and the maximum depth was 120 ft. Using these size limitations and the assumed earth load design criteria formulated by Shannon & Wilson, Inc., and R. B. Peck, as many as four rows of the cylinder piles were required to insure adequate lateral support of the hillside. Three of these rows of cylinder piles would support face walls along successive steps of the hillside, and an additional row would be located uphill at approximately midpoint between the upper wall and the hospital. The upper row was constructed so that a cast-in-place cap beam tied the pile tops together at the existing ground surface. The large reinforcing I-beams were fabricated with varied thicknesses of steel along the vertical axis, to accommodate bending moments as called for by conventional design procedures. This design approach proved to be completely successful in stabilizing the largest single soil cut along the downtown Seattle portion of the I-5 alignment.

BEACON HILL

Farther south, beyond the central business and First Hill sections, major portions of the freeway along Beacon Hill were to be built with open cuts faced with riprap buttresses similar to the test cut sections described previously. Retaining walls had been planned for a few localities, and these were changed to cylinder pile walls. Experiences with the clays elsewhere along the highway indicated that it would be advisable to avoid disturbing these soils wherever possible. At only a few places on Beacon Hill it would be necessary to accommodate surface streets and interchanges; consequently, it was decided that grade and alignment could easily be revised and elevated structures substituted for surface roadways wherever feasible. Although elevated structures were more expensive than at-grade sections, they were less costly than the cylinder pile-supported walls that would have been required by the large sidehill excavations originally planned.

Near Albro Street a major bridge crossing was required, and the highway had been designed with a fairly shallow cut into the toe of a gently sloping hillside extending up several city blocks to the east. Because of its relatively flat topography, this location had not been considered to be particularly susceptible to sliding. Soon after the cut was opened, ground and street cracking developed over a large area. When the first signs of movement had been detected, several inclinometers and survey points were installed. Later investigations revealed an ancient slide directly below a steeply sloping hillside several blocks to the east. Because the bridge structure construction had not advanced to a stage where a grade change would be particularly detrimental, grades were revised as the most economical and practical corrective action. A maximum 10-ft depth of suitable soil fill was used to backfill the cut, and a rock buttress was constructed as part of the remedial action. Continued inclinometer and survey readings and visual examination of street crack patterns indicated that all detectable movement had been arrested.

RAVENNA BRIDGE

Contract plans for three parallel bridge structures at the Ravenna Boulevard overpass in north Seattle specified 4-1/2-ft-diameter, 6-in.-thick, prestressed hollow concrete piles driven to a load-bearing capacity of 260 tons. Test drilling showed that 15 to 35 ft of soft clay with lenses of sand, silt, and peat was underlain by till and dense sand and gravel of adequate bearing capacity for bridge piers. The pre-cast portion of the support column was to terminate at ground level and connect with a cast-in-place concrete member extending upward to the cross-beams supporting the post-tensioned longitudinal girders upon which the bridge deck was cast. Preliminary soil borings demonstrated the need for additional information for defining expected tip elevation. Consequently, a large number of cone penetrometer probes were performed to provide low-cost rapid determinations of the depth to firm bearing stratum.

WSDOH had not previously used prestressed hollow concrete piles. Consequently, there was no available correlative information with which to determine the tip elevations. However, the depth of the abrupt change from relatively soft to very firm soils was easily discernible. The penetrometer probe information was confirmed by full-scale load tests and proved to be very reliable for estimating the approximately 20- to 35-ft pile lengths needed at this particular location.

LAKEVIEW-GALER ELEVATED STRUCTURE

The main roadways in the Lakeview to Galer area were to be supported on three parallel elevated structures, each with 4- to 4-1/2-ft-diameter column bents, their length dependent on the abruptly sloping existing ground surface. Each drilled 4-1/2-ft-diameter cast-in-place concrete pier was to be founded on a 9-ft-diameter belled footing to provide the necessary total bearing capacity. Tip elevation of each bell was planned so that loading from one pile would not be imposed on the adjacent downhill pile. As an added precaution, to ensure that the clay was of adequate bearing capacity, the contract included special provisions requiring that a standard penetration test and an unconfined strength test be conducted in each bell by an on-site inspector assigned by the district soils section.

Extensive deep postglacial gulleying of the hard clay and silty clay and subsequent deposition of granular outwash material had taken place along this portion of the alignment. Where the clay was homogenous and relatively dry, an expandable belling tool worked quite well. Wherever moist sand or silty sand was encountered, the contractor had to resort to hand excavation and installation of wooden shoring to ensure that the bell would remain open during the concreting operation. Where extensive sloughing occurred, primarily in the granular outwash, the contractor elected to drive groups of H-piles, splayed in such a manner that the load was distributed over an approximate 9-ft diameter. To insure that the proper total bearing value for each pile group was obtained, a small number of pile load tests were conducted by the contractor. Several installations of this type of support were approved as an adequate substitute for the contracted belled piers.

50TH STREET AREA

Boring information in this irregularly shaped area (about 1-1/2 city blocks wide by 3 blocks long) indicated that soils were similar to those at the Ravenna bridge site to the north but that there was more soft clay and organic silt. The irregular distribution and compressibility of the soft soils was evidenced by extreme pavement distortion and residential building settlements. The areal extent was easily defined by visible damage, which included differential settlements of as much as 18 in. between houses and attached garages because of variations in loading.

The compressible soil was locally as much as 30 ft thick and could not be tolerated under a well-traveled facility. Consequently, full depth removal and replacement with granular, non-compressible backfill material was specified. The District 1 soils crew used a penetrometer probe to monitor these operations to insure that a false firm bottom underlain by additional compressible soils would not be mistaken for the necessary solid bottom. The roadway was constructed in a shallow cut section along the west side of the alignment and supported on a maximum of 10 ft of fill along the east side of the alignment.

POST-CONSTRUCTION MONITORING AND FIELD EXAMINATIONS

Because of the potentially unstable nature of soils along portions of the freeway route, a long-term program for monitoring slide-susceptible localities for possible latent movement was adopted.

In the areas most susceptible to landslides, inclinometer casings had been installed during construction. The inclinometer was invented by S. D. Wilson and developed by the Slope Indicator Company of Seattle. The casings consisted of a longitudinally slotted or grooved aluminum pipe inserted into a drilled hole. The void between the outer edge of the casing and the boring wall was backfilled with coarse sand or pea-gravel to provide lateral support. The monitoring probe was essentially an electrically actuated plumb bob that indicated the degree of casing deformation at set depth intervals. The traversing probe was incrementally lowered down the grooved casing in 20-in. intervals with wheels on the probe positioned in the grooves to correctly orient the probe. More than 110 inclinometers were installed along the route; most were 50 to 80 ft long. Several were placed in cylinder piles, the remainder in uphill areas. An initial set of readings was obtained immediately after installation; these and readings taken at later planned times were compared and plotted graphically to display amount and depth of movement. Readings on many inclinometers were continued for 3 or 4 yr after completion of construction in some localities.

As part of the precautionary measures taken during construction to ensure slope stability, horizontal drains were installed where moderate to heavy seepage was anticipated. These drains consisted of slotted plastic pipe inserted into machine-drilled borings. Borings were splayed out at various angles and slopes in clusters of four, placed so as to intercept any seepage in permeable strata upslope. The thin slot perforations through the tubing allowed water to enter the pipe but screened out particles that could plug the drain. These drains were monitored frequently by maintenance personnel and regularly flushed out over a period of several years.

Earth and pavement crack patterns suspected of being caused by slide movement were also visually examined periodically; some were monitored with surface measuring markers. As noted previously, movement in the areas uphill from the construction site was not a recent phenomenon, and much of the previous movement had been caused by earlier construction projects. The largest of these early projects had been the large-scale sluicing of soil from the western slope of Beacon Hill onto the tidal mudflats which at one time extended from the present freeway alignment along Beacon Hill west to the present day waterfront. Seattle clays were also suitable for brick production, and large amounts of clay were taken from the Spokane Street vicinity as recently as the mid-1950s.

LIGHT-WEIGHT CINDER FILL

Soon after the southbound lanes of the freeway were open along the base of the south end of Beacon Hill, the two outside lanes began to subside and crack and had to be closed. Examination indicated that sliding was likely occurring along a deep soil/rock contact which sloped downhill and intersected the alignment at about freeway centerline. Seepage was also observed downslope on the exposed bedrock. Remedial construction involved removal of the upper loft of displaced overburden material, installation of positive drainage, and rebuilding of the roadway up to grade. The material removed had a compacted weight of about 130 to 140 pcf and was replaced by light-weight cinders hauled in from burned slag heaps or old coal mining sites located several miles southeast of Seattle. The cinders were compacted to approximately 65 to 80 pcf. The light-weight cinders had been extensively used in the Seattle area for road and sidewalk surfacing material prior to the development of more sophisticated products. Cinder fill had also been previously used for light-weight embankments at several locations along the freeway route and on other highway projects. However, this was the first time it had been used for slide stabilization. The fill worked satisfactorily, and no subsequent sliding was evident.

PORTABLE PENETROMETER

Because of the erratic nature of the Seattle soils, WSDOH needed to verify that the design bearing values were valid at the footing elevations shown on the contract plans. Although the preliminary boring information was extensive, the planned footing elevations had to be interpolated from nearby borings spaced 100 to 400 ft apart. Consequently, engineers felt that these assumed values should be verified in the field. As a prelude to on-site inspection of footing material at individual pilings, a correlation program was worked out in which a portable cone penetrometer probe was field tested to compare its readings with results of the Standard Penetrometer Test, conducted routinely on all the preliminary borings. The makers of the portable penetrometer had furnished a chart correlating blows per foot of penetration with the Standard Penetration Test. Tests by district soils staff showed that these values were sufficiently accurate for roadway design work.

Many of the pier footings were in excess of 30 ft deep, and access was difficult for machine-mounted equipment. However, the portable penetrometer was easily transported to the site; it was hand-operated using a small-diameter point instead of the larger split-spoon sampler used with the Standard Penetration Test. The evaluation of the majority of the footing bearing soils with the portable penetrometer provided positive assurance the footings were founded on satisfactory bearing soils.

UNIVERSITY OF WASHINGTON RESEARCH PROJECT

The criteria for lateral soil pressure values employed in the design of the cylinder walls were established by R. B. Peck. However, until the Seattle Freeway construction, there had been little experience concerning deep hillside cuts in glacially overconsolidated and fissured clays. WSDOH considered it advantageous to develop more scientific guidelines for future design of similar cylinder pile walls. At the time of initial freeway construction, the only known laboratory experiments on glacially loaded clays were from the Peace River area in British Columbia; these had been conducted by the University of Washington soils laboratory under the direction of Robert Hennes. The experiments were performed on small-diameter, undisturbed samples in standard consolidation equipment. Although no positive conclusions could be formulated from this earlier work, further refinement of the experimental procedures seemed warranted. Accordingly, a contract was established between the WSDOH and the University of Washington to develop and refine a laboratory stress-meter which could measure the horizontal component of stress locked into soils after glacial retreat.

This program was developed under Memet Sherif and continued for nearly 3 yr (Sherif and Wu, 1969). Data from this test program, were not available during construction, but along with analyses of strain gauge and inclinometer readings on cylinder piles along the freeway, provided confirmation that the design criteria assigned by Peck were suitable for future projects where glacially loaded clays governed design. The experimental procedures and apparatus developed as the result of this research program were used to assess *in situ* stresses for design of the Mt. Baker Ridge Tunnel along I-90.

CONCLUSIONS

Although there were significant unforeseen problems with what was then a state-of-the-art project, they were overcome and handled with sound and innovative engineering. As a result of this project, accomplished 20 yr ago, the WSDOH developed or encouraged the development and implementation of a number of test, design, and construction procedures which have proven valid and useful on many recent projects. The use of large-diameter cylinder pile walls to retain slide-prone hillsides was first implemented on the I-5 project with great success and has subsequently been used on other major projects including the portal areas for the I-90 Mt. Baker Ridge Tunnel, completed in 1987. The WSDOH used the cone penetrometer to assess footing construction depths through soft organic soils in the Ravenna Blvd. area. Penetrometer tests are now widely used throughout the world for assessing the properties of soft or loose soils. The WSDOH also first used light-weight cinder fills to successfully replace and stabilize slide-prone areas. The result of these innovations was a successfully completed project, which continues to perform satisfactorily after 20 yr of operation.

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Aerial view to the north-northwest of the Interstate Highway 5 bridge over the Lake Washington Ship Canal, Seattle. Photograph by R. W. Galster, February 1971.

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Cylinder Pile Walls along Interstate Highway 5, Seattle

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INTRODUCTION

Cylinder pile retaining walls were adopted along the Seattle Freeway (Interstate Highway 5 or I-5) when sidehill excavations triggered slope movements. Conventional retaining walls proved inadequate because failure planes were in most cases deep-seated, passing many feet below existing and proposed retaining wall foundations.

The slopes were underlain by overconsolidated, very stiff to hard layered silts and clays, which conventional stability analyses indicated would be stable at the proposed design grades. However, even minor excavations triggered deep-seated and extensive slope movements that extended back into the slope and endangered multi-story structures located at the top of the slope.

To control the ground movements, concrete cylinder piles were installed at the toes of newly constructed retaining walls, extending below the deep failure planes. In other sections, cylinder pile walls were substituted for the proposed conventional walls. From a comprehensive study of the sequence of events surrounding the various deep-seated slides, a hypothesis for the mechanism of slope failure was developed from which a rational basis for the design of the cylinder walls evolved. The mechanism of slope failure envisioned at the time, the criteria for design that this prompted, along with some details of the design of the walls, are presented and discussed. Also presented is an evaluation of cylinder pile performance.

An analysis of the slides was the subject of a doctoral dissertation by D. J. Palladino, which subsequently was published as a paper by Palladino and Peck (1972). An earlier paper by the authors (Andrews et al., 1966) forms the basis for much of this paper. However, some information has been added to enhance the historical record, and some discussions have been deleted for brevity.

EVENTS LEADING TO CYLINDER PILE WALL CONCEPT

In the area for which the first contract (1962) was drawn up, between East Olive Way and East Galer, the freeway extends in a north-south direction along the west slope of one of several prominent till-capped hills within the Seattle city limits. Sidehill excavations were required, and various conventional retaining walls were designed to provide lateral earth support along the uphill freeway right-of-way and between northbound and southbound lanes as shown on Figure 1.

The subsurface materials comprising the hills of Seattle were deposited or modified by numerous advances and retreats of continental glaciers reaching many thousands of feet thick. Generally, the large ice masses deposited till on the upland surfaces of the hills while greatly compressing the underlying soft lacustrine sediments. The till material is characteristically an unstratified and unsorted mixture of clay, silt, sand, and gravel, with scattered cobbles and boulders, and with the smaller grain sizes predominating. This material is extremely compact, and it usually serves as an adequate foundation for major structures without the need for piles or piers.

The lacustrine deposits consist principally of unweathered silt, silty clay (CH, Unified Soil Classification System), and clay (CH), with representative Atterberg limits as tabulated below. The deposits are either massive or layered and in some places varved. The consistency of the material varies from very stiff to hard, and in the undisturbed state it exhibits a remarkably high strength. The *in-situ* properties of the material, however, are influenced by the existence of structural features consisting of joints and fissures and, generally, horizontal bedding planes. When disturbed, such as by ground movements, the fissured nature of the material creates a broken mass composed of hard blocks and chunks (Figure 2).

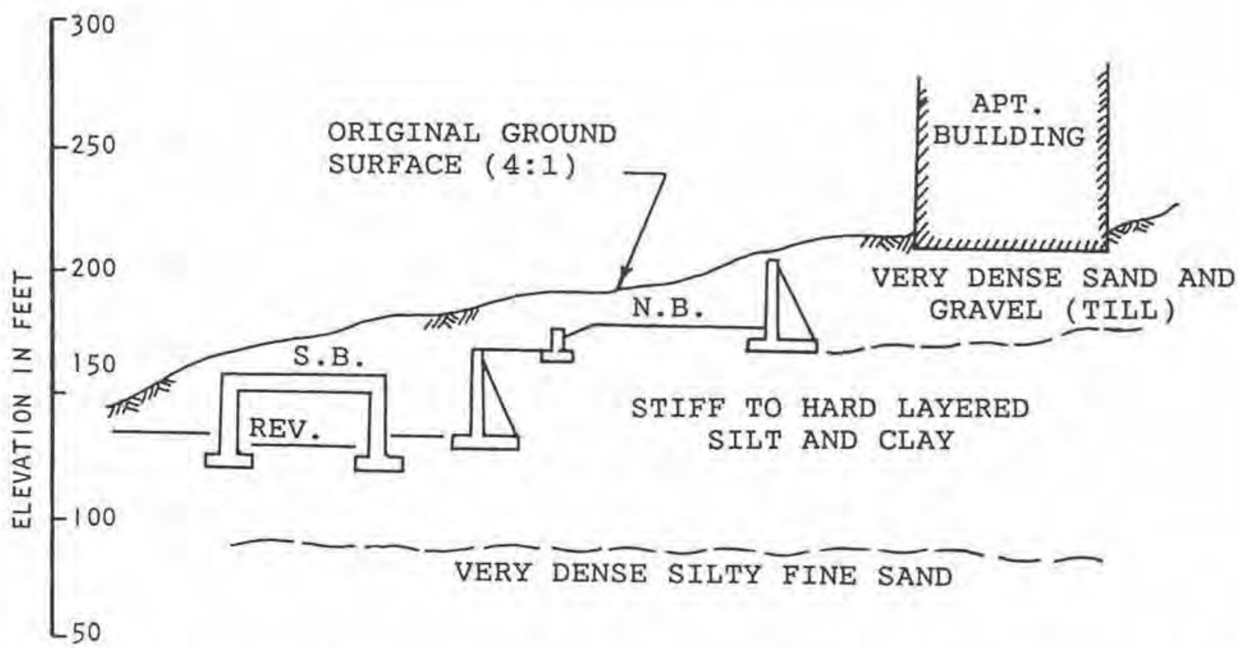

Figure 1. Cross-section A-A′ (generalized) showing the proposed construction of the Seattle freeway and the geologic profile. N.B., northbound lane; S.B., southbound lane; Rev., reversible lane.

Figure 2. Hard blocks and chunks of clay in a landslide mass. Largest chunk in the upper middle part of the photograph is about 5 ft across.

Ground water in the hillside appears as perched along the contact between the sand and gravel till and the layered silt and clay material, and in relatively pervious zones within the silt and clay. In most cases, high hydrostatic pressures were not detected in the various strata. The ground-water conditions were erratic, but in general, water levels appeared to decrease with depth of penetration of the bore holes.

Stability problems were anticipated with some of the excavation slopes, but it did not appear that the proposed excavations would extensively disturb the balance of equilibrium that existed. Prior slide movements that had occurred in the area appeared to be limited to the weathered zone of the exposed clay formation, and no serious problems were anticipated beyond the 5- to 20-ft maximum depth of the weakened zone.

Samples of the hard underlying clay were tested for shear strength and the test data indicated that the clay was adequately strong (that is, an angle of shearing resistance ϕ' of 30° or more and considerable cohesion) to remain stable under the proposed excavation slopes. However, experience showed that there were several areas in the city where cuts in the hard clay showed evidence of shallow movement, apparently caused by cyclic weathering and subsequent surface softening. These conditions had to be considered in design.

Specifications were developed, therefore, which provided a limit of 200 ft horizontally for any major retaining wall excavation and at least 200 ft between excavations. In addition, all excavations for retaining walls were to be performed either inside adequately braced cribbing or, if open cuts were used in certain specified areas, the backslope of the excavation was to be controlled by the engineer.

HILLSIDE INSTABILITY

The first indication that stability problems would be significant during construction occurred early in the first contract (1962). The first slide was followed shortly thereafter by others until five or six major zones of hillside instability had developed as shown on Figure 3. Because the conditions along the freeway are variable and because the nature of the slide movements differed from one area to another, no general description of slide activity can be expected to cover all circumstances. However, discussion of the events leading to instability for a typical hillside section will serve to point out the main characteristics of the deep-seated slide movements that were most prevalent and of primary concern.

The sequence of events associated with a typical section, cross section A-A′, is shown on Figure 4. At this section a shallow cut was made in May 1962, followed by excavation during September and October for the upper retaining wall, W-7. The wall was completed at the end of December 1962. In late fall of 1962, a major excavation was being made in the lower part of the slope, not only for wall W-12, but also probably for pier 13 of the southbound structure. On November 11 cracks appeared in the buildings across the street at the head of the slope. A slip in the back slope of the excavation for wall W-12 was noted at about elevation 150 ft. Inclinometer I-5, established at the top of the hill, indicated significant movements at about the same level (elevation 145 ft) between December 15 and 27, 1962. Inclinometer I-9, established in early 1963 near mid-height of the slope, indicated shallow movements in January and February 1963 and also movement during February at about elevation 125 ft, approximately the same elevation as the base of the footings excavated for the southbound structure. When, in April 1963, cylinder-pile excavations were made in front of wall W-12, further evidence of movement at this level was noted.

This section clearly demonstrated the deep-seated nature of some of the landsliding movements. The movements in inclinometer I-5 near elevation 145 ft in December and in inclinometer I-9 near elevation 125 ft through February could have been caused only by the excavations associated with wall W-12 and the piers for the southbound roadway structure.

Because conventional walls were already in place at this and other sections of instability, it was proposed to control the deep-seated movements by the placement of cylinder piles at the toe of the walls. At other locations, where these walls were not yet in place, cylinder pile walls were substituted (Figure 3). The cylinders installed in the first contract (1962) were 4 ft 9 in. in diameter and spaced on 6-ft centers. To extend below

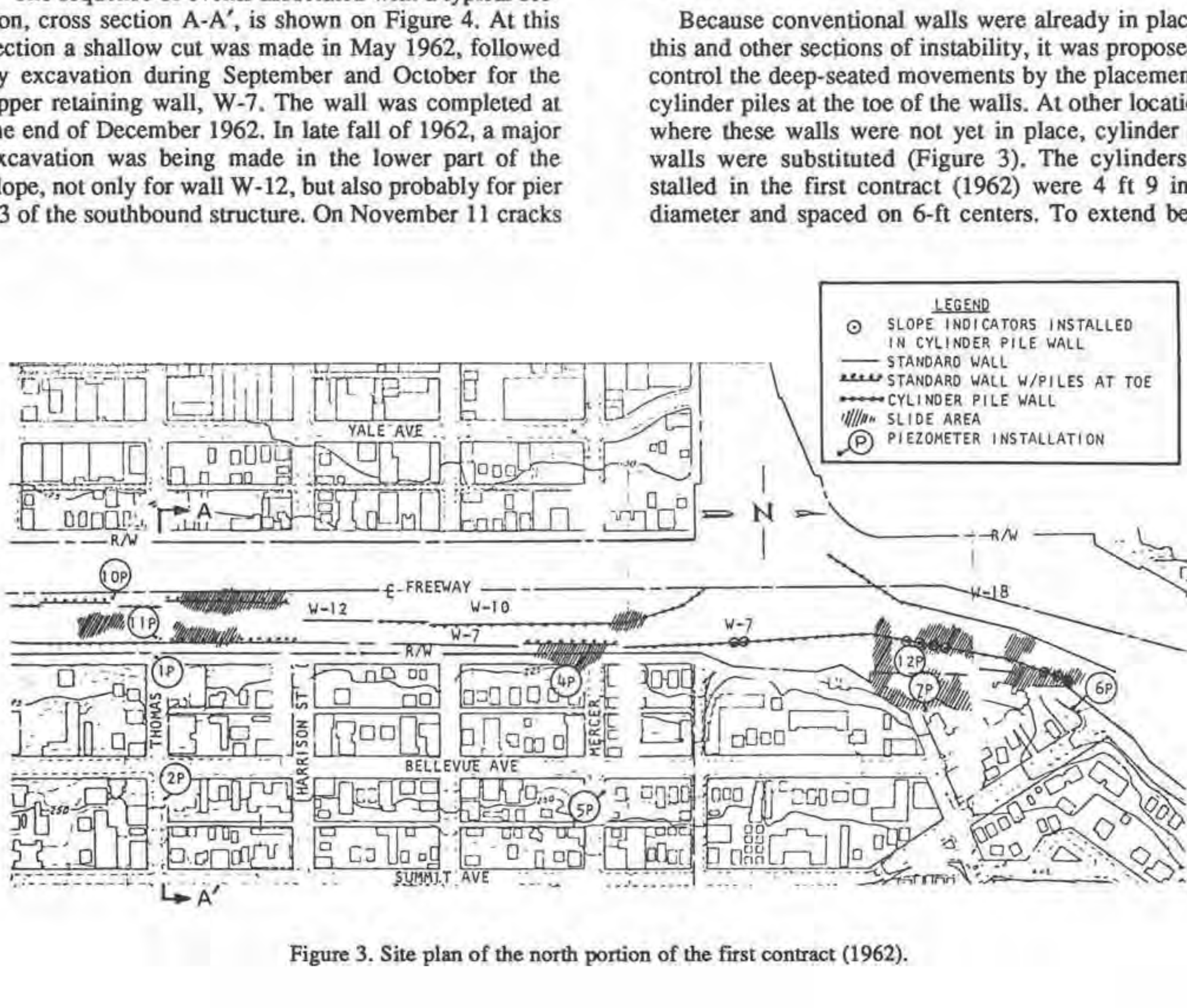

Figure 3. Site plan of the north portion of the first contract (1962).

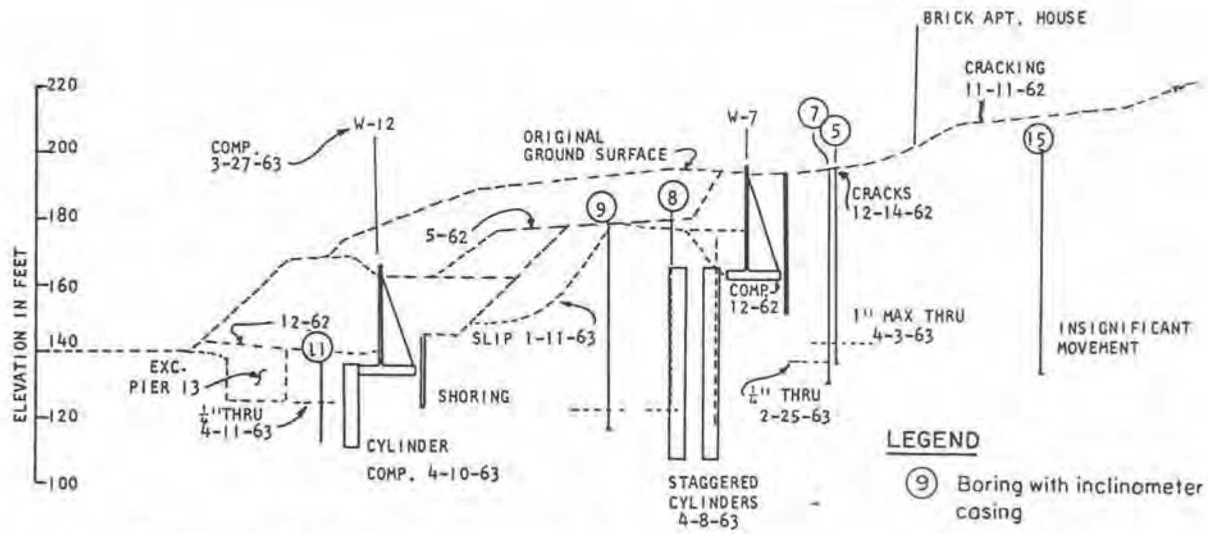

Figure 4. Cross-section A-A′ showing the sequence of events associated with slope failure. See Figure 3 for location of section. Horizontal scale same as vertical.

the lowest failure plane required cylinders as long as 81 ft. The design loading was too great to permit the use of reinforcing steel, and therefore welded steel beams were employed as shown on Figure 5.

Figure 5. Cylinder pile (diameter 4 ft 9 in.) at the toe of an existing conventional wall. Steel beam is in place and ready for concrete pour.

HYPOTHESIS AND MECHANISM OF SLOPE INSTABILITY

Substitution of the cylinder piles for conventional retaining walls and the placement of cylinder piles at the toe of in-place walls were expensive design revisions. Also, because the State was unable to find documented evidence of previous experience by others in the use of cylinder pile retaining walls and because construction through the central business district to the south still had to be accomplished, it appeared advisable to have calculations and procedures reviewed by independent consultants. The Washington Department of Highways, therefore, in 1963 requested the firm of Shannon & Wilson, Inc., of Seattle to undertake an engineering evaluation of the performance of the cylinder piles that had been constructed and to predict the long-term performance capabilities of such walls to assure stability. At the same time, the U.S. Bureau of Public Roads engaged R. B. Peck, Professor of Foundation Engineering at the University of Illinois, to evaluate the potential for similar stability problems to develop in the next freeway contract for the area just to the south and to make recommendations for an approach to construction.

The consultants were aware that it was not unusual for slopes to fail at computed factors of safety greater than 2 because similar problems of instability have occurred elsewhere in similar soils (Peck et al., 1960). At the time, no existing theory or hypothesis clearly explained the mechanism of failure in these overconsolidated soils, and therefore, the evaluation of the cylinder-pile wall concept and refinement of design criteria called for an empirical approach leading to the development of a hypothesis of slope failure taking into account the *in situ* soil characteristics.

In order to explain the mechanism of slope failure, the history of events associated with each deep-seated slide occurrence was compiled and studied. Study of these data revealed common elements associated with each failure, the more significant were as follows (Wilson and Squier, 1963; Peck, 1963):

- All slides involved movement along horizontal bedding planes.
- Initial movements occurred quite soon after excavation, and additional movements were progressive in nature.

- Ground water was present in all instances, either in overburden materials or in deposits back of the slope.
- Slides were initiated as a result of relatively minor cuts at the toe of the slope where the weight of earth removed was small compared to the mass of the subsequent slide. In several instances small, but significant movements were recorded where the excavation would not be expected to result in any significant decrease in the factor of safety.

In addition, the physical properties of the layered soils were studied in order to relate specific field occurrences of instability to the particular physical properties of the materials. It was realized that the significant properties of these soils were a result of the addition and withdrawal of considerable pressure caused by thousands of feet of glacial ice. The application and withdrawal of this great pressure created extensive fissures and joints in the silts and clays.

Slickensided surfaces were not a predominant characteristic in the clays. However, below elevation 100 ft, which is several tens of feet below the lowest excavation grade, the explorations disclosed a layer of green-gray heavily disturbed and slickensided clay. This zone, at least 10 to 15 ft thick, and possibly thicker, was observed underlying most of the project area. This zone undoubtedly had been disturbed by differential movements, most likely caused by the overriding and withdrawal of glacial ice.

Also, the state of stress in the sediments was greatly modified. The very stiff to hard consistency of the soil material permitted retention of significant residual or "locked-in" stresses. The large residual vertical stresses decreased in magnitude when the ice withdrew, but lateral stresses remained high because only minimal lateral strain was permitted. Further modification of the state of stress in the slope occurred from erosion. However, at a relatively short distance from the edge of the slope, horizontal pressures were believed to exist at magnitudes substantially higher than existing overburden pressures.

In brief, then, the layered silt and clay soils acquired characteristics and properties because of their stress history, which in combination were believed to have contributed to slope instability along the freeway. The main characteristics and properties were (1) residual or "locked-in" lateral stresses, and (2) "brittleness" or susceptibility to fracturing at low strains. These are combined with (3) layering, with horizontal planes having low frictional resistance, ϕ', and little or no cohesion after small strains.

All documentation suggested that initial movements in the slope were the result of release of lateral stress rather than any increase in shear stress resulting from a steepening of the slope. Excavations made into the slope removed lateral restraint and permitted the clay to move toward the cut. Field experience indicated that the initial movements occurred rapidly along horizontal bedding planes with differential movements in the order of one-half to several inches. The initial movements caused a pronounced reduction in the shear strength along a varve or bedding plane and produced a distinct plane of failure. Because of this undesirable consequence, further movements were progressive.

Moreover, the relief of stress caused fissures in the brittle soil to open and permitted the soil to swell and soften in the presence of water. Where the stress relief was slight, negligible swelling and distortion of the clay chunks occurred; however, where the relief of stress was great, increased swelling took place, and a general loss of strength of the soils developed as the material softened. Stress relief also contributed to the opening of tension cracks which may have extended to appreciable depths. If these became filled with water, significantly high hydrostatic pressures may have been exerted in the mass. In brief, the series of related consequences outlined above produced a sliding mass bounded at the base by a weakened horizontal section and, within the slope, by a vertical crack and a transitional surface of sliding. Depending upon the assumptions of hydrostatic pressures in the slope, post-failure stability analyses revealed angles of shearing resistance, ϕ', along the basal failure planes to be in the range of 13° to 19°.

PRINCIPAL ADVANTAGES OF CYLINDER PILE CONSTRUCTION

This explanation for the occurrences of slope failures suggested the importance of preserving the great intact strength of the overconsolidated sediments by minimizing lateral strains. Evidence obtained from the slope failures demonstrated conclusively that the strength of the sediments deteriorated progressively when a critical value of strain was exceeded.

The cylinder pile wall provides the most positive means of controlling the magnitude of horizontal movements in the slope. The installation of the walls is accomplished one cylinder at a time; excavation proceeds in a systematic fashion from one end of the wall to another. Shortly after placement, a cylinder possesses sufficient strength and is able to function as a load-carrying unit. Installation of the cylinder pile wall on the uphill side protects adjacent property while excavation proceeds downslope. Excavation may proceed to a level where an additional wall may be required; completion of this wall provides its portion of retention of the slope so that further excavation may be accomplished downslope.

In contrast to the rather straight-forward approach to cylinder pile wall construction, conventional retaining-wall installation is by necessity a more time-consuming and inefficient operation. Construction of the walls must be completed in discontinuous sections to limit the extent of excavation exposed to relief of stress. The walls must also be constructed in braced cofferdams to minimize movements in the slope materials. Pre-excavation through hard surficial materials is required at many locations. Work inside the cofferdam proceeds through a maze of bracing to completion of such operations as driving of vertical and batter piles and construction and backfilling of walls.

The complexity of the conventional retaining wall construction may make this approach more costly even though the cost of a cylinder wall from a material standpoint might be greater. In addition, it would appear that the chance of damage to adjacent property is less with the less complex method and a shorter period to completion of the retaining unit.

PRINCIPAL FEATURES OF DESIGN CRITERIA

On the basis of the foregoing assessment of the apparent advantages of cylinder pile wall construction, particularly in regards to allowing only minimum lateral strains in the layered soils and efficient construction scheduling, it was decided to substitute this wall for conventional walls in contracts for areas to the south of the first contract (1962). The criteria for subsequent cylinder pile wall design were set forth by Peck (1963).

The design procedure consists mainly of the determination of the applied and resisting earth pressures. The wall acts as a cantilever, and the deflection of the top of the pile depends upon (1) the magnitude of the applied lateral loads, (2) the flexural rigidity of the cylinder pile section, and (3) the soil resistance developed because of the displacement of the cylinder pile section.

The more important features of the design criteria are outlined briefly below for the two principal excavation conditions of excavation primarily in layered clay (Figure 6), and excavation in layered clay overlain by till of moderate thickness (Figure 7). The criteria basically consisted of establishing and considering the following:

1. *Level X-X*: Establish a potential surface of sliding X-X at depth, $d' = 10$ ft below where the deepest excavation will be made. If the deepest excavation is not backfilled at least 5 ft, or filled with a structure of equivalent weight, the depth to the assumed surface of sliding for the lower walls should be increased accordingly. This level is based upon evidence that the surface of sliding does not pass significantly below the level of the lowest excavation. The permanent cover above X-X protects against deterioration of the strength of the foundation soil with time.
2. *Pressure on Wall(s)*: The pressure above X-X against a-a representing the back of the retaining structure farthest uphill consists of:
   a. A uniform pressure, p_0, to provide a modest back pressure against the retained soil.
   b. A pressure increasing with depth, with the pressure at level X-X equal to $k\,\gamma\,h$. At higher levels, the pressure varies as a series of linear segments circumscribing a parabola. The breaks between the different segments correspond to the tops of lower retaining structures, such as b-b. When more than one wall is necessary, the load is divided according to the principles illustrated in Figures 6 and 7.
   c. A maximum permissible soil pressure q_0, at or below level X-X, and a coefficient of horizontal subgrade reaction, k_h, constant below X-X.
3. *Depth of Embedment*: The depth of embedment, d_e, should not be less than 20 ft for any cylinder unit.
4. *Limiting Top Deflection*: The top deflection of the cylinder pile should not exceed 2 in., although in select circumstances this criterion may be relaxed.

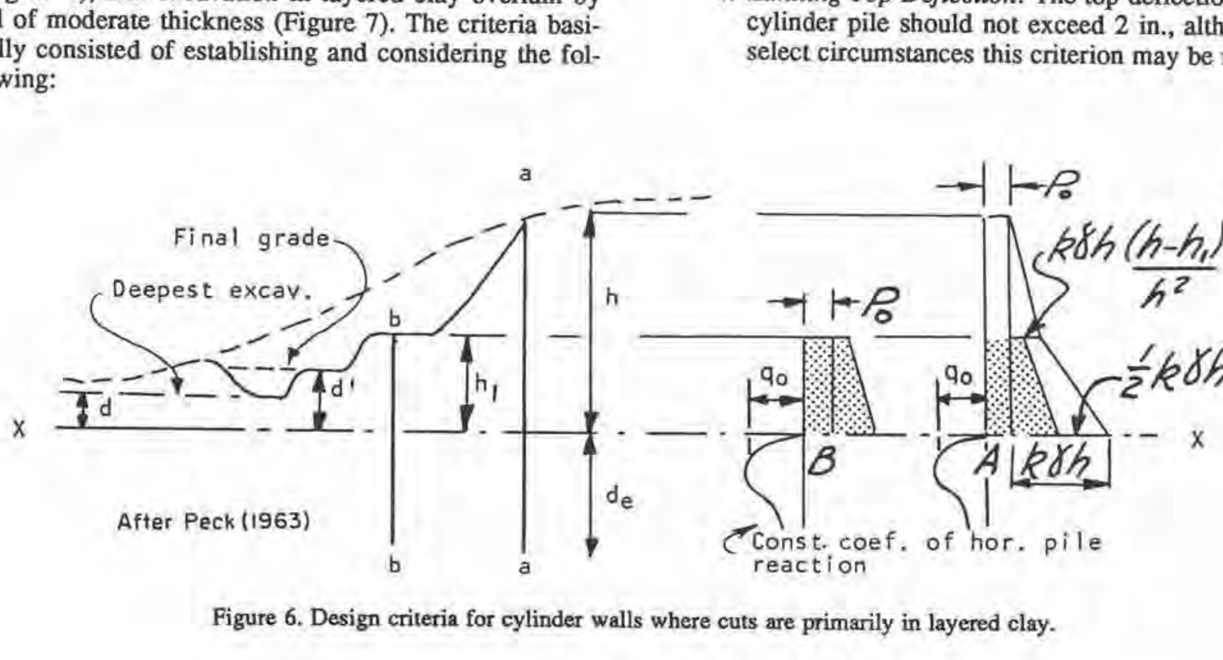

Figure 6. Design criteria for cylinder walls where cuts are primarily in layered clay.

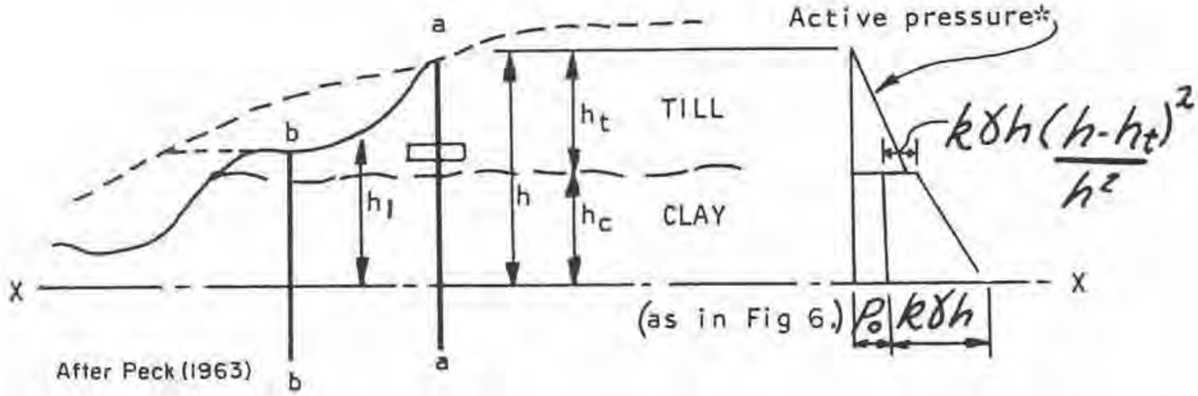

Figure 7. Design criteria for cylinder walls where cuts are in till over layered clay. *Active pressure includes effect of surcharge, if present, to depth h_t only. If the site is adequately drained, omit water pressure; if not, include hydrostatic pressure.

5. *Earthquake Conditions*: It is considered that the slopes have adequate stability with respect to earthquake forces. The stability of each wall for earthquake conditions should be checked by (1) increasing lateral pressures by 10 percent, (2) increasing q_o to 150 percent of its static value (k_h unchanged), and (3) checking that the stresses in the retaining structure are such that the factor of safety against the ultimate capacity is not less than 2.

In Figure 7, the foregoing criteria presented for conditions in Figure 6, are applicable, but in the till the earth pressure is computed by conventional procedures, taking into account any surcharge or water pressure, if present, and using a value of the angle of internal friction $\phi = 35°$. In addition, a conventional wall such as at position a-a may be founded directly on the till or hard clay if the excavation in front of the wall will nowhere extend to a level lower than 5 ft above the base of the wall before a second wall b-b is constructed. It is also required, under these circumstances, that b-b, together with any lower walls, is designed to carry the full lateral load below depth ($h-h_1$). The principal design parameters used in the structural analysis are presented in Table 1.

Table 1. Design parameters

Soil description	Atterberg Limits: Liquid limit	Atterberg Limits: Plastic limit	Natural water content (%)
Gray silty CLAY	52	31	30
Gray CLAY	70	30	35

The values of k_h and also ϕ and q_o used for preliminary evaluation of the design criteria were obtained both from field evaluations of material exposed in the excavations and from calculations based on data obtained from a 24-in. pile load test conducted by the Washington Highway Department to investigate the ultimate strength of the silt-clay soil for pile tip support. The pile tip was located at a depth of 35 ft, and skin friction was eliminated from the test results by construction procedures. The net failure load on the test pile was 110 ksf. On the assumption that the ultimate capacity of the pile is

$$Q_{Ult.} = Nc \qquad \text{(Eq. 1)}$$

where N = bearing capacity factor (N = 9 for deep piers); and c = cohesion or shearing strength of soil assuming undrained conditions.

The corresponding cohesion may be estimated conservatively as 110/9 = 12 ksf. The ultimate capacity of the soil under a lateral loading without confinement, taken conservatively at 2c, should then be about 24 ksf. It was suggested, on this basis, that the maximum permissible soil pressure, q_o, be taken equal to about 10 ksf (factor of safety = 2.4).

The same load test was used to evaluate an initial coefficient of horizontal subgrade reaction, k_h. It was determined that the settlement of the pile tip was 0.87 in., corresponding to 1,105 psi contact pressure. For a diameter of 1 ft, the value of coefficient of vertical subgrade reaction as defined by Terzaghi (1955) is then

$$k_{s1} = 1{,}105/(0.87 \times 1/2) = 2{,}540 \text{ pci} \qquad \text{(Eq. 2)}$$

The coefficient of horizontal subgrade reaction, k_h, for embedded sheet piles and closely spaced large diameter piles is

$$k_h = 1/3 \times k_{s1} \times 1/D' \qquad \text{(Eq. 3)}$$

where D′ is the depth of wall moving outward below X-

X. This value was taken equal to 10 ft, resulting in a value for k_h = 85 pci. Since the calculations are generally not sensitive to k_h, further refinement in the value of D′ was not considered necessary. Further, if the width, B, of the cylinder piles is about 10 ft, the value of the horizontal subgrade modulus, k_{h1}, becomes equal to k_h x B or 850 pci.

STRUCTURAL ANALYSIS AND IN-PLACE TESTING

The structural analysis of the cylinder wall evolved through two phases of development. The first phase, discussed in this section, was adopted by Washington State Highway personnel during the initial construction period when the slope failures required immediate solution. At this stage, the design of the walls had to proceed based on many assumptions and lacking support from field tests.

The method of structural analysis was based on assumed passive and active loadings as shown in Figure 8. The active (p_a) and passive (p_p) soil pressures were derived from the well-known expressions:

$$p_a = p + \gamma z - qu \quad \text{(Eq. 4)}$$

$$p_p = p + \gamma z + qu \quad \text{(Eq. 5)}$$

where p = uniformly distributed surcharge on the surface of stratum, psf.

γ = unit weight of soil comprising stratum, pcf.

z = depth below top of stratum, ft.

qu = unconfined compressive strength of cohesive soil, psf.

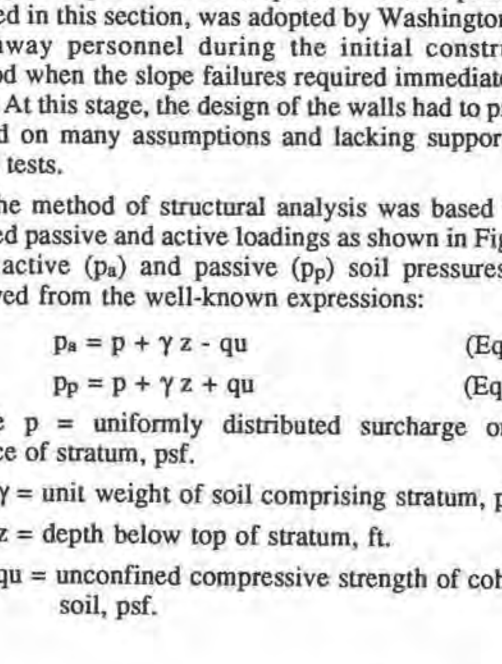

Figure 8. Initial approach to the structural design of cylinder pile walls: first contract (1962).

Most of the earlier cylinder piles were installed in areas that exhibited earth movement. Therefore, in the determination of active earth pressure, it was assumed that qu = 0, leading to design active earth pressures (equivalent fluid pressures) between 30 and 45 pcf. From laboratory test data and a review of engineering literature, a value of qu equal to 5 ksf was assumed in calculating the soil resistance, p_p, below the assumed level of failure. In addition to the foregoing earth pressure assumptions, it was specified that the maximum deflection of the top of the cylinder wall should not exceed 6 in. and the maximum deflection at the slip plane or point of passive resistance should be between 1 in. and 1/2 in.

The initial structural analysis led to the adoption of 4 ft-9 in.-diameter cylinders with a high-strength low-alloy steel (ASTM 441) beam core on 6-ft-to-7-ft center-to-center spacing. The steel beam core was a built-up section based on the design assumption that the steel beam would carry the entire load and the concrete would fill the annular space of the drilled shaft. It was contemplated that there would be some interaction between the steel and the concrete, but interaction was not considered in the initial design.

Because of the lack of design information available for this type of wall, provision was made to equip a select number of cylinders with strain gauges and inclinometer casings in order to provide performance data for evaluation. The number and location of the inclinometer casings that were attached to selected cylinder piles are shown on Figure 3.

In addition to performance data, it was also considered advisable to further investigate the k_h value of the very stiff to hard silt and clay, because the initial value of k_h, 85 pci, was evaluated from only one vertical pile load test, as discussed earlier. Therefore, a series of horizontal plate loading tests was completed at various depths inside a large-diameter shaft. The testing was accomplished in November 1963.

The test employed a 2-ft x 2-ft plate. The load versus deflection curves and a diagram showing the main details of the test set-up are shown on Figure 9. In test 1, the loading plate was placed against the soil without any confinement at the edges. At a load of approximately 50 tons, failure occurred, and clay extruded out from around the edges of the test plate. In tests 2, 3, and 4, however, the surrounding clay was confined by grouting the void between the steel liner and the soil.

Using the plot of load versus deflection, a line was drawn with a slope best representing the initial slope of the test curves. The slope of this line provided a k_{h2} = 1,165 pci and a value k_{h1}, for a 1-ft x 1-ft plate, of about 2,000 pci. From Terzaghi (1955)

$$k_h = k_{h1} \times 1/D' \quad \text{(Eq. 5)}$$

where D′ = depth of wall moving into the soil (refer to Eq. 3), which yields a value of k_h equal to 200 pci. This value was evaluated and, later, recommended for the design of subsequent cylinder pile wall sections along the freeway. (See Table 2.)

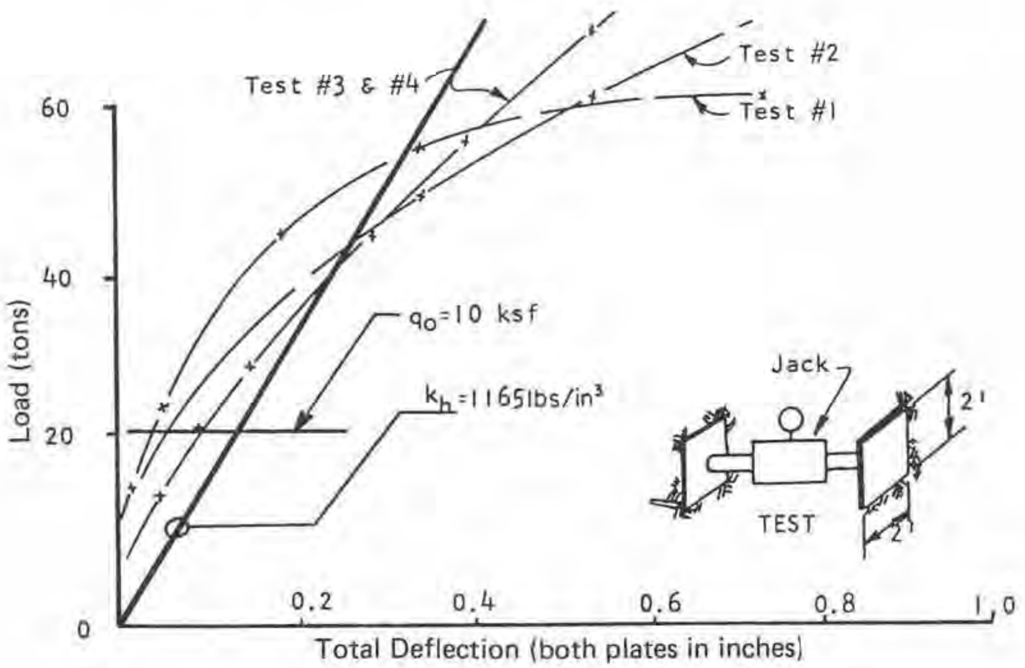

Figure 9. Load versus deflection curves for k_h test.

Table 2. Design data for cylinder pile computations

d = 5 ft	q_o = 10 kips/sq ft
p_o = 0.10 kips/sq ft	k_h = 0.085 kips/ cu in. (initial)
γ = 0.130 kips/cu ft	
$k_a\gamma$ = 0.050 kips/cu ft (k_a = 0.385)	k_h = 0.2 kips/cu in. (post-11/63)

Where k_a = a coefficient of earth pressure
γ = unit weight of soil

STRUCTURAL ANALYSIS SUBSEQUENT TO CONTRACT (1962)

For the design of cylinder pile walls to be constructed in areas south of the first contract section, the soil loading and support criteria detailed previously on Figures 6 and 7 and the design parameters in Table 2 were employed. The analysis assumed normal working stresses in the cylinder and a composite section and considered the cylinder as a flexible beam on an elastic foundation.

The basic formulae used in the analyses were developed originally by Hetenyi (1946) for a beam of semi-infinite length. By means of these formulae, expressions were developed for the cylinders to relate applied shear and moment, soil pressures, and deflections.

It was determined that the calculated value of q_o should be as high as possible. Moreover, the moment of inertia should be as small as possible to increase the design stresses in the steel section, which is the principal load-carrying member in the cylinder wall. The stresses in the steel beam for a particular value of q_o increase with the diameter of the cylinder; therefore, cylinders larger than the 4 ft 9 in. diameter used initially were investigated because of the economy due to the more efficient use of the steel.

The value of q_o for any particular cylinder is also a function of the specified limiting deflection at the top of the wall. It was found, in some instances in the design of the cylinders, that the limiting deflection of 2 in. corresponded to a value of q_o greater than the maximum allowable of 10 ksf set by the criteria. If the wall were designed for the maximum allowable deflection of 2 in., however, it was considered that the ground would yield somewhat as the pressure at the plane X-X exceeded 10 ksf and, therefore, a redistribution of the pressure would take place with depth below X-X. Thus, it appeared that the limiting value of 10 ksf would most likely not be greatly exceeded. Therefore, in some cases, design was controlled by the deflection of the top of the wall.

The cylinder walls were analyzed for both shear and moment by using the relationships and assumptions discussed above. High-strength steels were not required or economical because the large moment of inertia to resist deflection resulted in a low stress level. Therefore, an A-36 steel was used for the cylinder beams

After a review of the cost data, it was determined that a cylinder pile of a diameter greater than the 4 ft 9 in. originally used was more economical. Under design loadings and using the design criteria outlined in the foregoing paragraphs, cylinders for retaining walls reached a maximum diameter of 10 ft with cylinder spacing 12 ft center to center. In some areas, three walls of cylinders were required to carry design loads, with the dimension of cylinders in the second and third stages typically about 8 ft 4 in. and 10 ft on 10-ft and 12-ft centers, respectively (Figure 10). The length of pile reached a maximum of 120 ft in some places. In general, however, the lengths varied from 50 to 100 ft, and the exposed length of pile generally ranged from 40 ft to 20 ft.

CYLINDER PILE WALL CONSTRUCTION

The construction of a cylinder pile wall consists of drilling a hole to a depth and diameter indicated by the design, placing a steel beam and filling the remainder of the drilled hole with concrete. A continuous reinforced cap beam is then constructed throughout the length of the wall to reduce the differential deflection that may be caused by varying wall loading and soil strengths throughout its length.

No excavation below the tops of the cylinders takes place in front of or behind the wall before completion of the cap beam. After the cylinders and cap beam have attained the required strength, the material in front of the wall is excavated to grade. After the excavation has been completed, a face wall is constructed in front of the cylinders, which gives the appearance from the roadway of a conventional retaining wall (Figure 11). The face wall is attached to the steel beam by reinforcing bar ties field-welded to the beam flange. The face walls are constructed by conventional poured-in-place or precast methods. Other papers, such as Klasell (1963), Andrews (1964), and Andrews et al. (1964), discuss in more detail the design as well as the methods and problems of cylinder wall construction.

CYLINDER PILE PERFORMANCE DATA

The strain gauges and slope indicators on select small cylinders (4 ft 9 in.) and large cylinders (10 ft) were observed over a period of time. The deflection curves for selected cylinders 1 yr after construction are shown in

Figure 10. View north along the southbound lanes at the intersection of Boren Street and Pine Street. Two cylinder pile walls (10-ft diameter) were designed to accommodate the reversible lane under the southbound lane.

Figure 11. Cylinder piles (10-ft diameter) with permanent face walls under construction.

Figure 12. These curves reveal that loading on the cylinders was relatively modest.

All the instruments located in the walls throughout the freeway were monitored after the severe earthquake which occurred in the area in April 1965. The instrumentation revealed that the earthquake had very little or no effect on the performance of the cylinder pile walls.

CONCLUSIONS

A type of retaining wall has been discussed which was conceived originally as a solution to grave slope instability problems created by excavations in difficult subsurface materials. The concept evolved to the point where it was adopted to replace conventional walls in other critical sections of the Seattle Freeway. Although the material costs are higher for this type of wall than those of conventional wall construction, benefits, such as greater assurance of satisfactory performance and increasing contractor's efficiency, tended to counterbalance the higher cost.

Field test and performance data to date indicate that the walls are performing as well as or better than expected. They have been in place now for more than 24 yr, and their good performance over this period, even with a severe earthquake, confirms the adequacy of the original design assumptions.

ACKNOWLEDGMENTS

The writers take this opportunity to express their appreciation to all those who have worked on the development of the cylinder wall concept: The Washington State Highway Commission and its employees for their cooperation and assistance given the writers; Shannon & Wilson, Inc., geotechnical consultants, Seattle; R. B. Peck, University of Illinois (currently consultant, Albuquerque, NM; Howard, Needles, Tammen & Bergendoff, consulting engineers, Seattle and Kansas City; and the Federal Highway Administration for its assistance in the evaluation of the soils problems and development of the cylinder wall design.

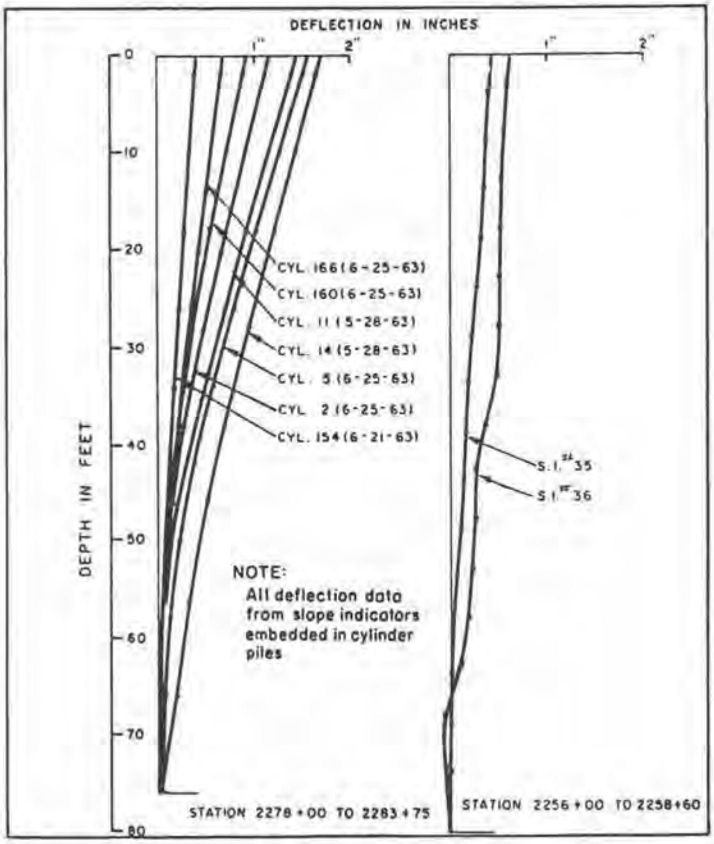

Figure 12. Deflection curves for smaller (diameter 4 ft 9 in.) cylinders constructed in first contract (1962), about 1 yr later.

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Geotechnical Impacts on Construction of State Route 167, The Valley Freeway

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INTRODUCTION

Highway planning and construction in western Washington prior to and immediately after World War II consisted principally of two-lane roads on new alignments and upgrading of existing roads by improving sight distance and intersections to accommodate heavier trucks and faster vehicles. In response to the growing need for improvements, in 1947 a Washington State Department of Highways (WSDOH) District 1 Soils and Materials section was established in Seattle; it was responsible to both the District Engineer and to the State Soils and Materials Engineer. By 1958 planning for better and safer highways was under way. At that time, the federal Interstate Highway program provided 90 percent federal funds for a 10 percent investment of state funds for planning and construction of designated interstate highways and 50-50 matching funds for feeder or collector roads.

Soil profiles were prepared for new alignments for both primary and secondary highways in western Washington, among them improvements of Primary State Highway (PSH) 5, the East Valley Highway in the Green River valley.

In the early 1950s farming in the fertile soils of the Green/Duwamish River system, which for years produced corn, beans, peas, lettuce, onions, rhubarb, and other crops, was being impacted by imported crops from California and by the slow but continuing development of land for industrial use. Truck-trailer rigs were beginning to replace the railroads in serving both industry and the marketplace. Consequently, there was a real need for an improved highway system in the Kent and Renton area.

It became apparent to highway planners that simply upgrading the two existing routes, the East Valley and the West Valley highways, would do little more than serve the then-current needs. Anticipated future development of the valley south from Renton justified construction of a four-lane limited-access highway. It was at this point that the WSDOH District 1 soils and materials laboratory became involved.

The subject of this paper, the Valley Freeway, was originally identified as PSH 5 and was constructed to replace the East Valley Highway. The freeway later became known as State Route (SR) 167 (Figure 1). This paper chronicles the investigation, design, and construction process for SR 167. The project was unique in Washington in that it had to deal with the placement of embankments as much as 30 ft high on areas of highly compressible organic soil along a large portion of the 16-mi-long route.

GEOLOGY

Geotechnical conditions throughout the valley from Renton to the King-Pierce County line are primarily related to glacial erosion and postglacial deposition. Four continental glacial advances have impacted the topography and geology in the Seattle-Tacoma area. Ice from British Columbia moved south, occupying the Puget Lowland from the Cascade Range on the east to the Olympic Mountains on the west and to some 50 mi south of Seattle. Ice in the lowland was about 6,500 ft thick at the Canadian border and thinned to about 3,500 ft in the Renton-Auburn area (Shannon and Wilson, Inc., 1968b).

The various glacial advances and retreats sculpted the deep trough which includes Lake Washington immediately north of Renton and the Green River valley to the south. This north-trending trough has been partly filled with several hundred feet of sediment supplied from advance and recessional melt-water streams. The uppermost alluvium is the result of Holocene drainage of the flanking higher land, including Mount Rainier by way of the White River. More recent soil deposits along the Green, White, and Cedar rivers include overbank, oxbow, and channel fillings of sand and silt laid down primarily during seasonal flooding. Deposits of peat and organic silt with thicknesses of as much as 30 ft represent former bogs and lakes throughout the wide, flat-bottomed river valleys. These varied soil conditions significantly influenced design and construction of SR 167.

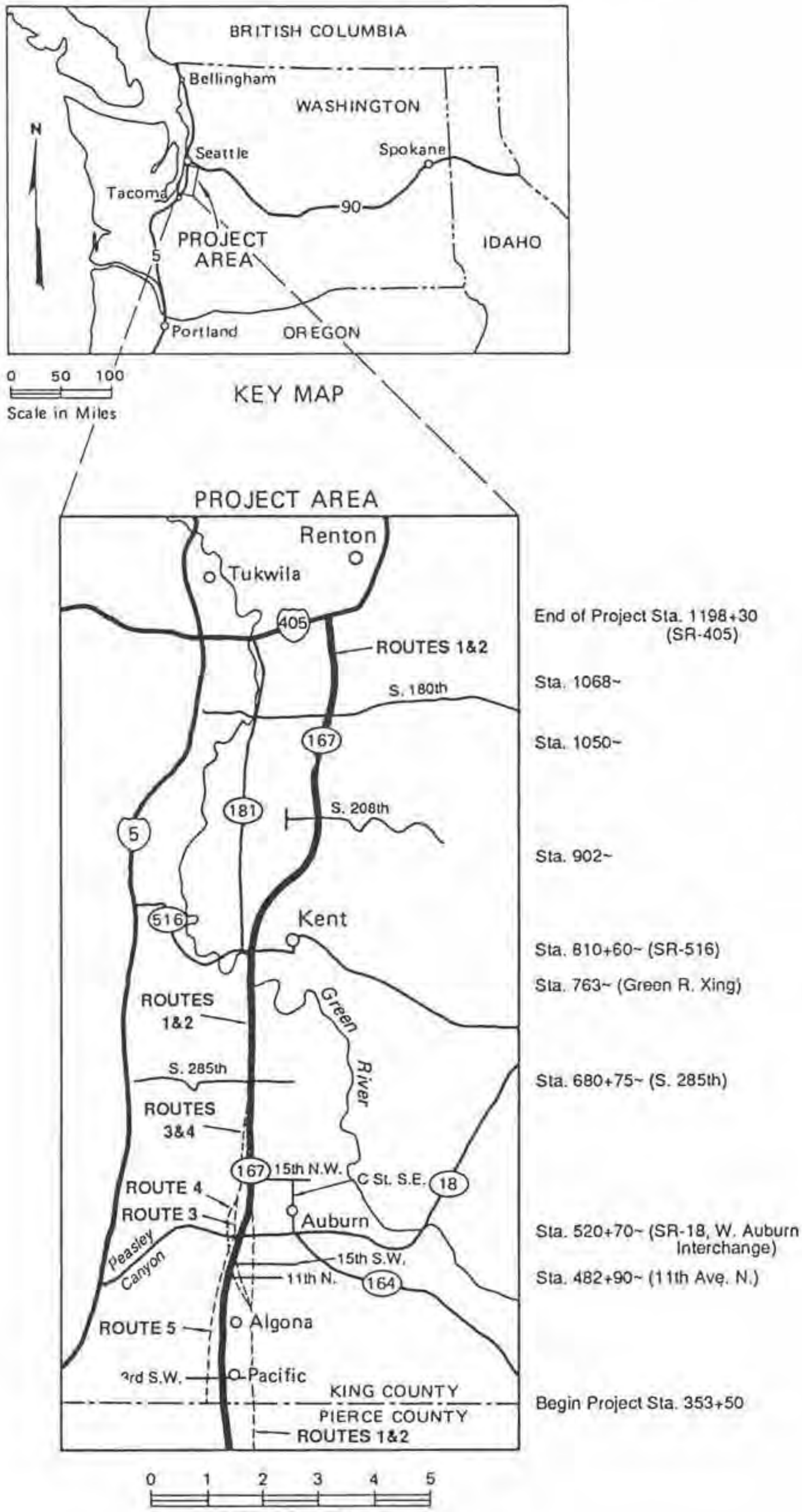

Figure 1. Location of the Valley Freeway, SR 167, project.

SOIL INVESTIGATIONS

The route selection process consisted initially of establishing a corridor. This task was comparatively easy—the corridor was to lie within the confines of the Green River valley. This valley, approximately 1 mi wide, is bounded by fairly steep hillsides on the east and west throughout the length of the project. Because prehistoric landslides and a few historic landslides were present along the flanks of the valley, a route was selected to avoid, insofar as possible, hillside cuts or fills on these slide-prone glacially overridden silt and clay soils.

Eventually five route variations were assessed for a corridor from the King/Pierce County line (Station 353+50) north to Renton (Station 1198+30), a distance of 16 mi. Initial engineering surveys were directed at establishing the alignments of preliminary routes 1 and 2 and to determine the grade for a proposed four-lane highway: four 12-ft lanes, 16-ft- and 40-ft-wide median strips, and 10-ft-wide outside shoulders. The line staked in the field was the centerline of the median. All test borings, except those for frontage roads and ramps, were tied to this survey. Stationing at selected streets and highways referenced herein is shown on Figure 1; additional positions can be approximated from the stations shown.

Soil conditions were not the principal consideration in final route selection, although they did play a significant role in design and construction of the roadway. Factors such as railroad crossings, county roads, city streets, whether to construct overcrossings or undercrossings, the location and design of major intersections, the location of the north juncture with Interstate Highway 405 near Renton, and the location of a connection to an extension south from the King/Pierce County line (designed by District 3) were the controlling factors. Subsurface explorations and geotechnical analyses were therefore not performed to select the best route, but rather to provide sufficient information to develop a roadway that could be constructed cost-effectively and accommodate the adverse soil conditions present along the alignment.

District 1 soils crews initiated a preliminary test boring program in January 1959 and continued the work through August 1959. The principal exploration tools were very simple, yet provided sufficient information for preliminary design. The tools included standard garden shovels, a 4-in.-diameter post-hole auger, a steel auger probe, a Kamo "portable" continuous flight auger, standard penetration equipment and 2-in.-diameter x 4-in.-long brass drive-tubes for obtaining fairly undisturbed samples.

Shallow holes (5-15 ft deep) were dug with post-hole augers. This method permitted detailed sampling of the upper soil strata. The post-hole auger was interchangeably used as a probe by attaching a 1-in.-diameter ships auger that had been welded to a 3/4-in. galvanized coupling.

Deeper test borings were added at locations determined on the basis of data from the shallow test borings and the projected height of embankments along the alignment. These deeper borings were drilled during July and August of 1959. Undisturbed samples were submitted to the materials laboratory for index, triaxial, and consolidation testing as they were obtained. Bridge and cantilever sign support foundations and soil pH factors for culvert design were also evaluated.

Soil profiles for routes 1 and 2 were completed, and a report dated March 31, 1961, was prepared by the District Soils Engineer. Copies of the report were transmitted to the District Engineer, the Project Engineer, and to the headquarters materials laboratory in Olympia. In May of 1961 District 1 completed examination of and reported soil conditions for frontage roads and ramps. The laboratory's response included stabilometer "R" values of samples of soils to be encountered in roadway excavation and was dated July 10, 1961.

Following the initial soil survey, the pavement determination committee in Olympia, in a letter dated June 15, 1962, agreed that the anticipated future traffic volume justified a high-use type pavement. In deference to the settlement potential (discussed later in Embankment Foundations and Characteristics of the Soft Foundation Soils), engineers selected an asphalt concrete pavement to be placed over an asphalt-treated base. Paving was to be staged by placing the gravel base, the crushed surfacing top course, the asphalt-treated base, and one-half of the thickness of the asphalt concrete pavement. The final lift of asphalt concrete pavement was to be placed 1 to 2 yr later, after most of the long-term residual settlements had occurred.

In February of 1962 headquarters drill crews initiated the second phase of foundation test borings for the alignment, beginning with the South 208th St. undercrossing structure south of Renton. The deepest test boring for embankments and undercrossings was drilled to 60 ft employing wash boring techniques. For the structures along the routes, the drill crews used wash boring with 3-in.-diameter casing, rotary drilling, and 8-in.-O.D. hollow-stem continuous flight auger. Shelby tubes were pushed to obtain undisturbed samples of the soft soils, whereas all other soils were sampled by means of the Standard Penetration Test, normally at 5-ft intervals. Most of the test borings at all bridge pier locations were drilled to depths between 90 and 115 ft, but borings in deep soft soils ranged from 120 to 150 ft deep. Reports included logs of test borings plotted on bridge location profiles, test data, and foundation recommendations.

It was anticipated that more than 7,000,000 cy of embankment fill material would be required for the project. As haul costs were a major factor, aggregate sources were sought as close as possible to the project alignment and at reasonable intervals. Therefore, concurrent with the soil survey, potential aggregate sources were sought in the Renton, Kent, and Auburn areas, both east and west of the proposed routes. Test holes, generally 18 in. in diameter and as much as 30 ft in depth, were drilled at selected locations by means of a Williams earth auger. Samples were tested, and an estimate prepared of quality and quantity of materials available from each site. To make haul distances as short as possible, nearly a dozen potential borrow sources were identified for the contractors.

DESIGN REPORT I

Design Report I was completed by the District and submitted about October 1968. This report described additional routes 3, 4, and 5, which diverged to the west or south from routes 1 and 2 at Station 650, north of Auburn. Routes 3 and 4 rejoined routes 1 and 2 near the north city limits of Algona, whereas route 5 continued on a separate alignment to the King/Pierce County line. The final approved alignment consisted of the original routes 1 and 2 from Renton to the 15th St. N.W. interchange, then southwest, crossing routes 3 and 4 near the West Auburn interchange with SR 18. The alignment then paralleled the old West Valley highway to the King/Pierce County line and a juncture with WSDOH District 3's extension to the south into Pierce County. Figure 1 shows the constructed location of SR 167 (routes 1 and 2) by a heavy black line and the alternate routes by light lines. SR 181 is not shown south of SR 516 because it parallels SR 167 to the west so closely that it would interfere with other details on the figure.

By letter of August 15, 1967, the engineering firm of Voight, Ivers, Stevens, Thompson & Associates (VISTA) was commissioned to prepare all engineering and design documents (Voight et al., 1968a, 1968b), including a report of test borings and a soils report for Section II (King/Pierce County line to 15th St. S.W.) and for Section I (15th St. S.W. to South 285th St.). Section I also included data for SR 18 from SR 167 to "C" St. S.E. in Auburn. In turn, VISTA engaged Shannon & Wilson, Inc., to complete the foundation borings and prepare foundation reports for the alignment sections I and II.

The foundation reports by Shannon & Wilson, Inc. (1968a, 1968b) included profiles derived from test boring logs, results of laboratory tests of samples, and design recommendations for embankment and bridge structure foundation treatment. These two reports covered the approved Design Report I alignment from South 285th St. to the King/Pierce County line. Of these two sections, only the portion from South 285th St. to 15th St. N.W. was previously covered by District soil investigation.

The next step was preparation of contract plans, specifications, and estimates (PS&E). The PS&E included special construction provisions imposed as a result of findings of the soil surveys and borrow pit evaluations. An overview of these conditions and the method of controlling embankment construction follows.

ROADWAY EXCAVATION

Roadway excavation was required at four locations on this project: from Station 1047 to Station 1068 (north and south of the South 180th St. interchange in Renton); Station 966 to Station 986 and Station 924 to Station 926 (sidehill cuts and fills between Renton and Kent); and the Peasley Canyon portion of the West Auburn interchange. The West Auburn interchange was cut in clayey, sandy gravel of an advance glacial outwash. The other three cuts were in alluvial sandy silts to silty sands with some gravel. Stabilometer values for these areas ranged from an "R" value (resistance) of 24 to 40. Sieve analysis for the cut soils indicated 80 to 100 percent of the foundation soil passing the 3/8-in. sieve and 32 to 50 percent passing the U.S. No. 200 sieve. Some foundation soils were saturated *in situ*, and all tended to flow or deteriorate under construction traffic in the presence of free water.

Moisture-sensitive soils were also a problem at common borrow sources. The specifications included provisions for aeration of estimated quantities of borrow for each contract; however, it was recognized by WSDOH that aeration would be effective only during extended periods of dry weather. To compensate, WSDOH established requirements for sloping the top of active borrow pit excavations or embankments to facilitate drainage and for compacting the borrow materials to the specified density by the end of each day.

A pre-existing landslide was recognized at a ramp section of the South 180th interchange. Stability of the landslide was maintained by placing a small rock buttress at the toe of the slide and protecting the slope with riprap.

EMBANKMENT FOUNDATIONS

By far the most challenging aspect of the entire project from a soils standpoint was to provide a roadway relatively free of significant residual subsidence, yet economically feasible to construct. Stabilization was further complicated by the plans to construct only two of the ultimate four lanes during the first stage of construction. To minimize the impact of the staged construction on long-term settlements, the two lanes of the first stage were designed symmetrically about the median. When a second stage embankment was added to complete the four lanes, widening of the design median widths and the shoulders would be equal on each side, thus minimizing settlement impacts on the already completed lanes.

Major portions of the alignment required embankments ranging from 1 to 35 ft high. Most of the higher embankments formed approaches for overcrossing structures. Elsewhere, the roadway is raised 3 to 5 ft above the surrounding low countryside to reduce the potential for flooding the road.

Embankment height increased from 7 ft at the south end of the project at the King/Pierce County line in the town of Pacific (Station 353+50) to 16 ft at the 3rd Ave. S.W. overcrossing structure. The roadway remains elevated to 11th Ave. North (Station 482+90) in Algona, where the embankment height drops to about 5 ft.

A significant portion of the alignment is underlain by soft, organic soils. Consequently, excessive settlement was a major consideration. A foundation soil sequence of peat and soft silt over loose to medium dense sand was recorded from the county line to approximately Station 446. Table 1 indicates peat thicknesses along this section of the alignment. Just north of 1st Ave. S.W., where the embankment height is 18 ft at Station 398+70, test boring B-90 encountered 9.5 ft of fibrous peat overlying 8.5 ft of loose to medium dense sand above a second stratum of fibrous peat. The deeper buried peat extended to a maximum depth of 30 ft, as indicated by other borings between Station 397 and Station 458.

North of Station 458 through the West Auburn interchange to Station 617, surficial peat or soft silt with thicknesses of 1 to 8 ft overlay medium dense sand and gravel (Figure 2). At the Valley Freeway crossing over SR 18 in the West Auburn interchange, fill heights increase to 35 ft high at bridge abutments. Fills are 5 to 8 ft thick from north of SR 18 to the Green River crossing at Station 763. Organic silty to peaty topsoil depths of ~2 ft with scattered pockets of peat as much as 10 ft deep prevail from Station 617 to Station 997, north of Kent.

The embankment height increases to overpass elevations beginning at the crossing of the Green River near Station 763. The route remains elevated (except for two short areas) through Kent to Station 902, north of the East Valley highway overcrossing. From Station 902 to Station 1050, south of South 180th St., embankments averaged 5 ft high.

Fibrous peat, including some buried logs, was encountered at the surface from Station 997 to Station 1050. It extended to depths of 5 to 10 ft; the maximum recorded depth of organic soil in this area was 14 ft.

From the north end of the cut at Station 1068 to the end of the project at Interstate Highway 405 in Renton, fills were generally 15 ft thick. The combined thickness of peat and organic silt averaged 5 ft from Station 1078 to Station 1109. Between Station 1109 and the end of the project, the depth of the soft soil increased to a minimum of 7.5 ft and a maximum of 16.5 ft. A City of Seattle 60-in.-diameter water line crosses the freeway alignment at Station 1121+68 where the depth of soft soil is from 7 to 9 ft. Beneath the soft soil to a depth of 45 ft, loose to medium dense sands with varied silt, clay, and gravel contents were encountered.

CHARACTERISTICS OF THE SOFT FOUNDATION SOILS

Laboratory tests by Shannon & Wilson, Inc., and the WSDOH laboratory in Olympia provided a basis for establishing special contract provisions for removal or preloading of the organic soil.

In-situ moisture content of the peat generally ranged from a low of about 100 percent to a high of 1,000 percent. Most results were in the 250 to 500 percent range. Although there were exceptions, most of the silt samples

Table 1. Organic soil thicknesses and calculated settlements for King/Pierce County Line to 15th Ave. S.W., Valley Freeway (from a letter by State Materials Engineer R. V. LeClerc, May 25, 1970)

Station limits	Embankment fill height (ft)	Thickness of peat (ft)		Embankment Settlement		Recommended soil removal (ft)	Recommended fill overload (ft)	Proposed fill slope (H:V)
		Surface	Buried	Primary average (ft)	Secondary average[a] (ft)			
353+50 to 368+00	6 to 9	1 to 5	---	2.3	0.4	None	5	6:1
368+00 to 372+00	9 to 15	3 to 5	---	2.9	0.6	None	5 to 7	3:1
372+00 to 380+00	15 to 27	5 to 6	---	3.6	0.8	None	7 to 10	3:1
380+00 to 395+00	27 to 18	3 to 8	---	4.4	0.9	None	10	3:1
395+00 to 404+00	18 to 19	7 to 11	?[b]	1.0[c]	?[d]	Full depth	10	3:1
404+00 to 430+00	19 to 25	5 to 10	3 to 10	6.0	1.0	Full depth	10	3:1
430+00 to 440+00	25 to 20	5 to 20	2 to 7	0 to 5.0	0 to 1.0	Min. 10	10	3:1
440+00 to 445+00	20 to 12	2 to 5	4 to 6	3.3	0.4	None	10 to 7	3:1
445+00 to 456+00	12 to 8	0 to 3	0 to 10	1.0 to 3.0	0.2 to 0.5	None	7 to 5	3:1 to 6:1
456+00 to 525+00	8 to 5	0 to 4	0 to 3	0.9 to 1.7	0.2 to 0.4	None	5	6:1

Notes:

(a) Secondary settlements are calculated as 10- to 20-yr projected settlements assuming no overload.
(b) Test boring B-90 shows peat in bottom; depth and limits of peat are unknown.
(c) Settlement is based on assumption of no buried peat.
(d) Settlement is based on assumption of no buried peat; therefore, secondary settlement is minor.

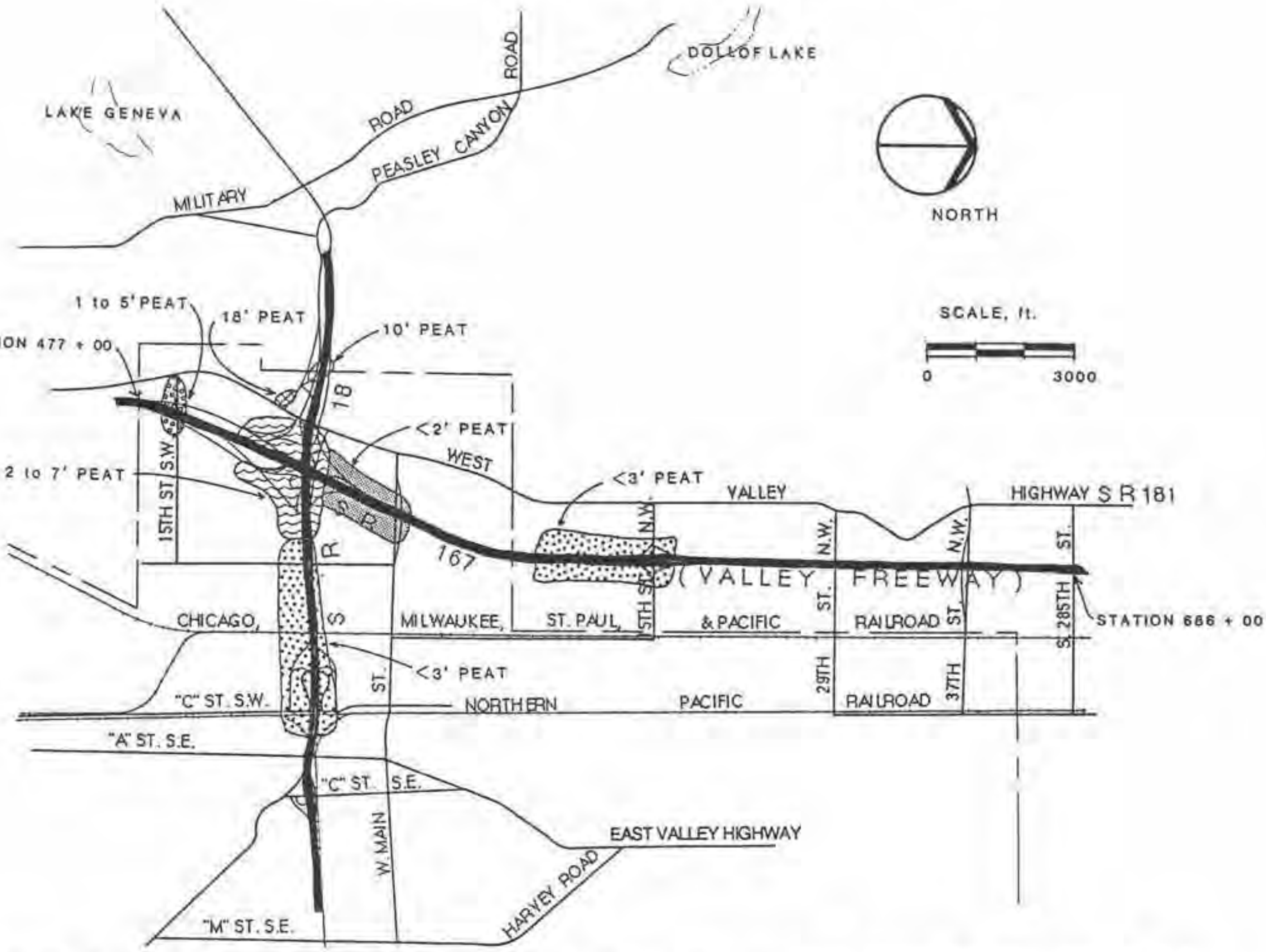

Figure 2. Depth and extent of soft organic soils near the West Auburn Interchange, Station 555+00 to 686+00, Valley Freeway.

tested at between 30 percent and 60 percent moisture. Moisture contents are a significant indicator of the relative compressibility of the site soils.

High moisture contents can indicate a high ground-water table, which can adversely impact constructing roadway embankments over these highly compressible soils. A high water table generally indicates that the organic soils have not been dewatered and consequently are likely to be normally consolidated and therefore highly compressible. A high water table also slows drainage which normally occurs during the application of additional soil loads and thus slows consolidation and buildup of strength in the organic soils. Furthermore, a high water table makes for wet excavation conditions if the organic soils must be excavated and makes it difficult to adequately compact fill material.

Over the length of the project alignment, the depth to the water table generally varied from a few inches to 6 ft; the greatest depths were recorded in August of 1959 indicating some seasonal variations. A 1- to 3-ft depth to water table prevailed through most of the embankment areas from January through June. Physical properties of typical site soils are given on Table 2.

The compression index for the organic soils was determined from consolidation tests to be approximately 0.30 in./in./log cycle pressure. This results in predicted settlements on the order of several feet for each 10 ft of fill placed over a 10-ft layer of peaty soils. In a few areas of the project alignment, embankment fills of as much as 20 ft were required over 20 ft of organic soils, resulting in calculated settlements of 4 to 6 ft.

Triaxial strength tests on the peats and organic silts and clays indicated low shear strengths, friction angles ranging from 0° to 9°, and cohesion intercepts of 0 to 250 psf (Table 2). These relatively low shear strength values showed that these soils would be susceptible to shear failures under rapid loading conditions.

Table 2. Laboratory test data for First Ave. No. overcrossing foundation investigation, Valley Freeway (from a letter by State Materials Engineer R. V. LeClerc, May 25, 1970)

Hole No.	Station	Offset	Depth	Description	% H_2O	Wet Dens. pcf	Cohesion psf	Angle of Internal Friction
H-1	433+82	58' Rt	2'-4" to 2'8"	Light brown silt w/wood chunks	70.9	94.5	120	6°
			7'-2" to 7'6"	Gray silt w/ layers of peat	144.2	80.3	75	3°
			11'-6" to 11"-10"	Gray medium sand w/organic material	43.8	104.7	80	11°
			16'-10" to 17'2"	Dark brown peat w/chunks of wood	525.3	60.5	250	9°
			21'-6" to 21'-10"	Gray layered silt, sand, wood chunks	115.7	84.3	75	7°
H-2	435+45	67' Rt	7'-8" to 8'-0"	Mottled organic silt w/wood chunks	144.5	81.9	60	3°
			11'-10" to 12'-2"	Gray silty sand w/roots	27.3	117.6	450	18°
			16'-6" to 16'-10"	Brown peat w/layer of silt and sand	140.7	79.0	90	0
			21'-10" to 22'-2"	Gray clay-silt	51.5	106.4	230	10°
			26'-10" to 27'-2"	Gray, fairly clean, medium sand	25.9	119.6	280	34°
H-3	435+47	61' Lt	6'-4" to 6'-8"	Brown organic silt	289.4	71.7	0	5°
			11'-0" to 11'-4"	Brown organic silt w/gray silt	81.1	92.2	0	6°
			16'-4" to 16'-8"	Brown organic silt	493.3	64.1	150	7°
			21'-0" to 21'-4"	Brown silt w/pockets of sand	102.9	89.5	40	6°
H-4	433+82	58' Lt	0'-2" to 0'-6"	Brown peat	326.7	65.1	50	7°
			6'-2" to 6'-6"	Brown peat - bottom 1" gray silt	193.3	74.0	0	5°

DEVELOPMENT OF A PLAN FOR CONSTRUCTION CONTROL

A major objective of the highway design was to construct stable embankments free of shear or foundation failures and with minimal long-term residual subsidence. The low shear strength of the peat and organic silt coupled with high water levels indicated the need for a controlled rate of loading to maintain stability of the foundation soil. Consolidation test data revealed that the magnitude of initial and residual settlement might amount to 10 to 30 percent of the organic soil strata thickness, which at many locations could equal several feet. While removal of the soft foundation soil was desirable from a stability and settlement perspective, it was generally impracticable economically, except beneath overpass or bridge structure foundations, due to the large thicknesses and volumes of soil involved. Therefore, other techniques for achieving stability were considered.

The selected method of minimizing potential shear failure was gradual consolidation of the foundation soils, thus increasing their shear strength by controlled loading. This was to be accomplished by (1) placement of an initial free-draining lift of embankment fill, with time delays between subsequent lifts, (2) construction of the embankment in 10- to 15-ft stages to permit consolidation and stabilization of the foundation soils, (3) construction of an extra or overload stage to overconsolidate the foundation soils, and (4) installation of instruments to indicate pore pressure changes and settlement (consolidation) at selected locations.

Embankments were to be constructed in two to three stages of 5- to 15-ft thicknesses to reduce the potential for instability while permitting adequate consolidation and increase in foundation soil shear strengths. First-stage embankments, except at the bridge ends, were to be constructed in two or three 5-ft increments with a 30-day delay between the increments. A delay of 3 to 5 months was normally required between the first and second stage. If pore pressure readings indicated that initial consolidation had progressed rapidly, then a delay was not required or could be reduced. Second-stage construction in increments of 5 ft brought the embankments to final design height. A third stage embankment of 5 to 10 ft was then generally added to overload or overconsolidate the soft organic foundation soils, thus accelerating consolidation and reducing long-term secondary settlements. After 1 to 5 months when primary consolidation and settlement was complete, this third overload stage was removed in preparation for paving. Fill from the overload stage was transferred to another area of the alignment for reuse as embankment fill.

Fill was placed in an initial lift of 2 ft of clean, free-draining, sandy gravel or gravelly sand (minimum sand equivalent value of 50) over the undisturbed valley soils. In most instances, the grass cover was not removed because the root systems helped to bind together the soil and support construction equipment. The free-draining material, classified as gravel borrow, was to function as a drainage blanket for vertical and lateral relief of pore-water pressure in the underlying soft wet soils and as a working platform for construction equipment. The planned thickness of this blanket was reduced from an original 3 ft to 2 ft in deference to the high cost and significant quantities involved. The 2-ft thickness proved to be adequate. The gravel borrow was end dumped and spread with dozers. This was followed by placement of common borrow in successive lifts.

Common borrow was placed in loose lifts about 1 ft thick and compacted to 90 percent of standard maximum density using vibratory rollers. However, due to the sensitive nature of the foundation soils, the lower 5 ft of the first stage of embankment fill was compacted with heavy rollers with the vibrators turned off.

All bridges or overpass structures on this project were pile-supported, but some abutment spread footings were placed in embankments after removal of third-stage overloads. To minimize the effects of transitions from fill to bridge structure, the upper stratum of peat and/or organic silt was removed for 100 ft from each bridge abutment. Over this 100-ft distance excavation progressed from full depth (normally about 10 ft) to the surface at a slope of about 10H to 1V. The excavations were backfilled with gravel borrow to ground surface; normal staged embankment construction was continued to the necessary final embankment height.

INSTRUMENTATION

Pore pressure and settlement monitoring instruments were installed at the base of some of the higher bridge approach fills at the south end of the project between the King County border and the West Auburn Interchange (Station 520+70) to provide an assessment of construction procedures and embankment stability. On past projects, settlements and pore pressures had been monitored with reference plates and piezometer tips, respectively, placed on the underlying soils and monitored at ground surface via vertical riser pipes extended up through the fill as the embankments were constructed. These riser pipes were necessary for monitoring the instruments, but they were extremely vulnerable to damage by the contractors' activities. Consequently, in many instances little or no information was retrieved from these installations. To reduce the potential for construction-related damage to settlement measurement and piezometer systems, devices were selected which could be monitored well away from active construction and would not require the extension of instrument components up through the fill.

In 1977 only a few systems were available which fulfilled these requirements. Initially, a simple remotely monitored, overflow fluid level settlement system—a hose level system—was designed by WSDOH and installed by the contractor at two locations. The accuracy of this type of system is influenced by temperature fluctuations and the buildup of air bubbles in the fluid lines. Consequently, WSDOH found that the accuracy and continuity of measurements were not sufficient for the project. To meet the project requirements for a monitoring system, the author designed and patented a system consisting of an anchor placed in a borehole in stable soils, generally 10 to 30 ft beneath the embankment and connected via a tensioned wire to an electrically monitored rotary potentiometer device (Figure 3). This device could be monitored remotely with a portable wheatstone bridge readout box via electrical leads extended several hundred feet away from construction activity. Accuracy and sensitivity of these settlement instruments were generally several orders of magnitude better than for the fluid level or plate settlement devices. Eight of the electrical settlement devices were installed beneath bridge approach embankments and functioned accurately for the life of the project.

Soil pore pressures were monitored with twin-tube pneumatic piezometers supplied by the Slope Indicator Company of Seattle, in lieu of the more traditional standpipe piezometer. Six piezometers were installed beneath bridge approach embankments, and all functioned satisfactorily.

Bridge approach embankments were constructed to full height plus overload fill prior to constructing the bridges. Piezometers were installed in areas of sensitive foundation soil to regulate the rate of embankment construction. Settlement devices were monitored to determine incremental and total settlement until it was determined that overload fills could be removed and first stage paving could begin. Only two sets of foundation settlement data are available (Figures 4 and 5). Measured settlements were generally within the range of consolidation-induced settlements predicted on Table 1. Both Figures 4 and 5 show the marked increases in settlement rates with increased fill height. Settlement generally appears to have taken 1 to 2 months to slow to reasonable rates. Settlements of 1 to 2 in. per month were still occurring 6 months after fill placement ceased but were decelerating rapidly. The piezometers and settlement monitoring devices provided useful data for controlling and expediting embankment fill rates and maintaining embankment stability. No embankment failures were experienced during construction of this difficult portion of SR 167.

CONCLUSIONS

A design was developed for the numerous embankment and overpass portions of the SR 167 alignment that successfully accommodated several feet of cumulative construction-induced foundation settlement. Although many design changes were made during both the plan-

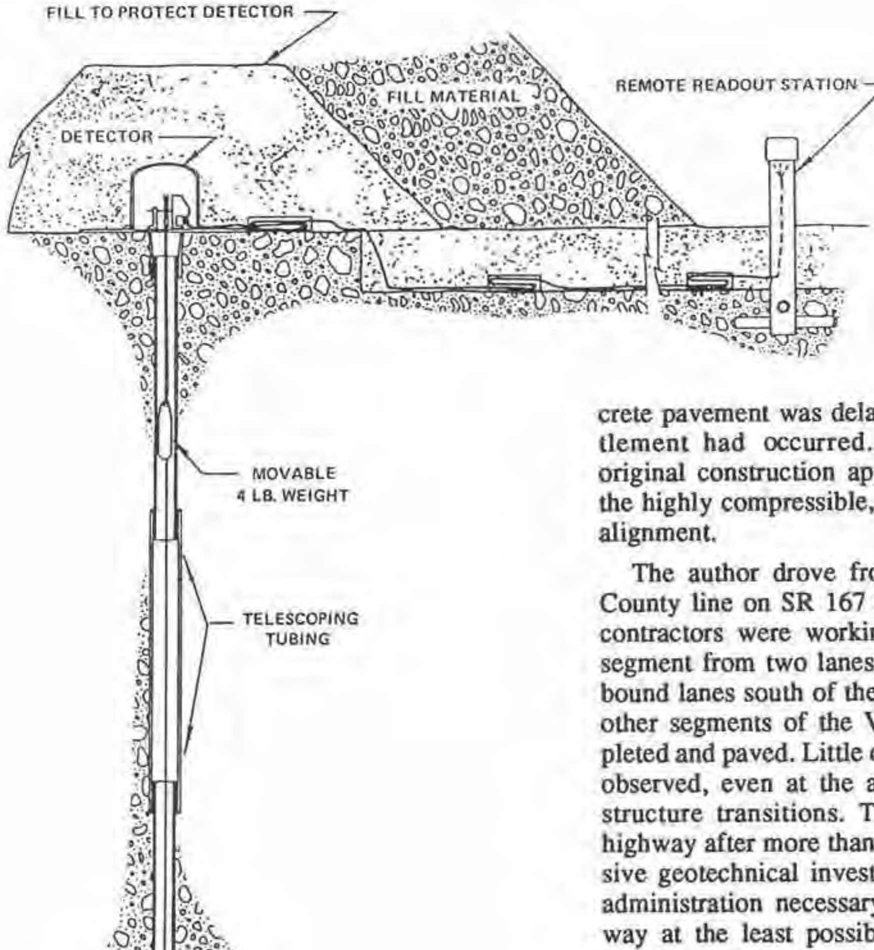

Figure 3. Electrical settlement monitoring instrumentation designed for the Valley Freeway (SR 167) project and installed in bridge approach embankments.

ning and the construction phases of the Valley Freeway project, all basic soil-related aspects of the design and construction remained essentially intact. Settlements of several feet were anticipated in the soft organic soils along portions of the alignment. Consequently, construction was staged so that most of the settlement had occurred prior to paving or placing the overpass structures. Staged construction took many forms—for example, staged embankment construction was used to gradually consolidate the soils; overload fills were used to accelerate long-term secondary settlements; some bridge abutments were supported on spread footings in the fills instead of on piling to eliminate problems with downdrag; and placement of the final lift of asphalt concrete pavement was delayed until after most of the settlement had occurred. These modifications of the original construction approach successfully dealt with the highly compressible, organic soils along the project alignment.

The author drove from Renton to the King/Pierce County line on SR 167 on July 15, 1987. At that time contractors were working to upgrade the last project segment from two lanes to four to complete the southbound lanes south of the West Auburn Interchange. All other segments of the Valley Freeway had been completed and paved. Little or no differential settlement was observed, even at the approach embankment/overpass structure transitions. The excellent condition of the highway after more than 10 yr of use justifies the extensive geotechnical investigation, planning, and contract administration necessary to provide this quality roadway at the least possible cost. The present industrial development of the valley and the expansion of housing on the surrounding hillsides and uplands are well served by this major state highway, SR 167.

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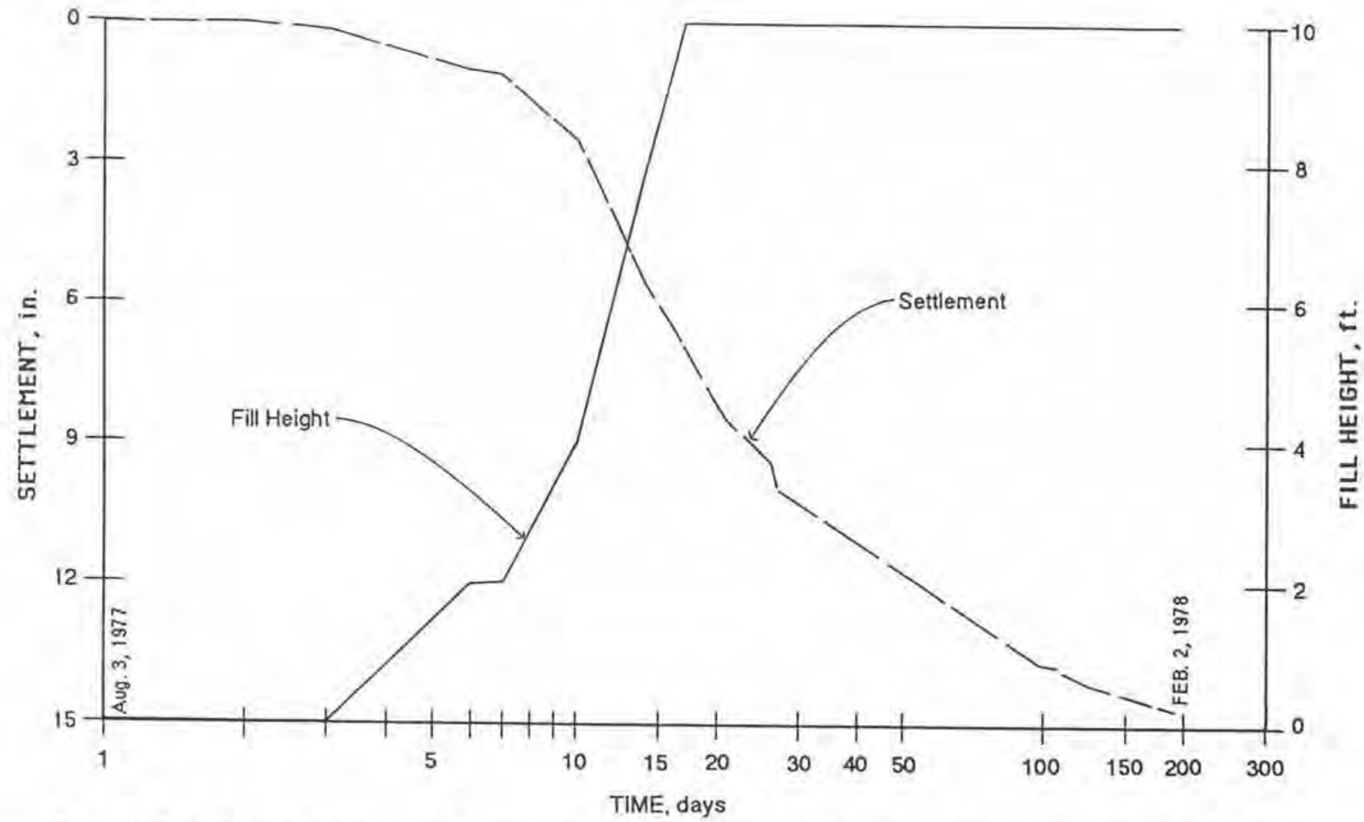

Figure 4. Plot of embankment fill height and settlement versus time at the 3rd Ave. S.W. overpass approach fill, Station 373+00 vicinity, Valley Freeway.

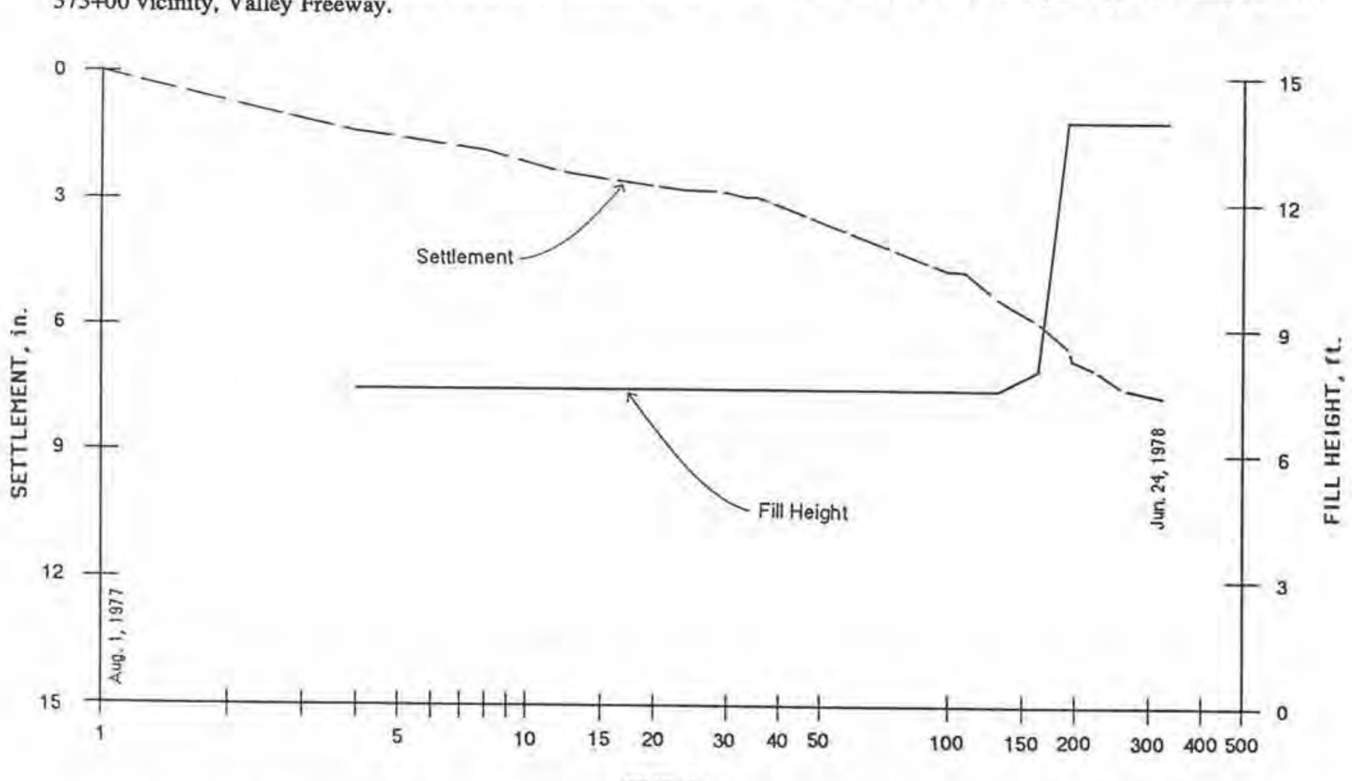

Figure 5. Plot of embankment fill height and settlement versus time at the Algona overpass approach fill, Station 448+00 vicinity, Valley Freeway.

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Design and Construction of a Highway Interchange on Soft Organic Soils, State Route 16, Kitsap Peninsula, Washington

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INTRODUCTION

The new State Route (SR) 16 interchange at the intersection of Old Clifton Road and Tremont Street is located 2 mi southwest of the town of Port Orchard in Kitsap County, Washington (Figure 1). Increased traffic flow at the old interchange, along with the high speed limits, resulted in an increasing number of accidents and created difficult access on and off the highway. The decision was made by the Washington State Department of Transportation (WSDOT) to design a safer, more easily accessible interchange at this location.

CH2M Hill, Inc., was retained to perform a subsurface investigation, to design, and to prepare plans and specifications for the new interchange. This paper discusses the geotechnical investigation and the design and construction methods used. The design was completed in June 1985. Construction of the interchange began in the fall of 1985 and was completed in spring 1987.

PROJECT DESCRIPTION

The new interchange consists of the following facilities (Figure 2):

- New alignments for Tremont Street and Old Clifton Road
- Northbound exit and entrance ramps to SR 16
- A southbound loop exit from SR 16
- A southbound approach roadway to SR 16
- Two single overcrossings for SR 16

The northbound entrance ramp to SR 16 required two concrete retaining walls, on the ramp's east and west sides. Construction of the remaining entrance and exit ramps and relocation of Old Clifton Road and Tremont Street required cuts and fills of as much as 35 ft to maintain the required highway grades. The overcrossing is a bridge with pile foundations.

SITE DESCRIPTION AND GEOLOGY

The new interchange is located approximately 650 ft south of the original intersection of Tremont Street, Old Clifton Road, and SR 16. The new alignment traverses a variety of terrains ranging from steeply sloping hillsides (that is, with 10 to 30 percent slopes) to low-lying, poorly drained, flat, wet areas.

Soft, compressible peat and highly organic silt are present in the poorly drained, low-lying areas. Where small streams drain these areas, the peat and organic silt are commonly interlayered with very fine grained alluvial deposits of silt and fine sand. The sloping areas are underlain by glacial deposits and localized areas of alluvium and organic soils. The glacial deposits are part of the Vashon Drift and consist of outwash sand and till that were deposited during the Vashon Stade of the Fraser Glaciation approximately 15,000 to 13,500 yr ago (Deeter, 1979; Molenaar, 1963). The most recent alterations to the ground surface are areas of artificial fill created to accommodate manmade structures such as SR 16.

SUBSURFACE EXPLORATION PROGRAM

A field investigation was conducted in three phases. Phase 1 was a preliminary investigation to characterize general subsurface conditions and to aid in locating the roadway interchange. The preliminary investigation, which was conducted between May 9 and May 21, 1984, consisted of a geophysical survey comprising 15,720 ft of horizontal electrical profiling, 19 mechanical auger holes, 3 hand auger holes, and 13 water jet explorations.

The phase 2 investigation was conducted between July 9 and July 20, 1984, and on July 30 and August 20, 1984, after the alignment of the roadway interchange had been selected. This alignment was chosen on the basis of property ownership. The investigation consisted of 23 test borings, 4 test pits and hand probes in the soft, compressible areas identified along the new road alignment.

The phase 3 investigation was conducted on April 22 and 23, 1985, to supplement subsurface information in two areas of geotechnical concern: compressible material at depth at the loop exit and entrance ramps for southbound SR 16, and soils at the retaining wall of the

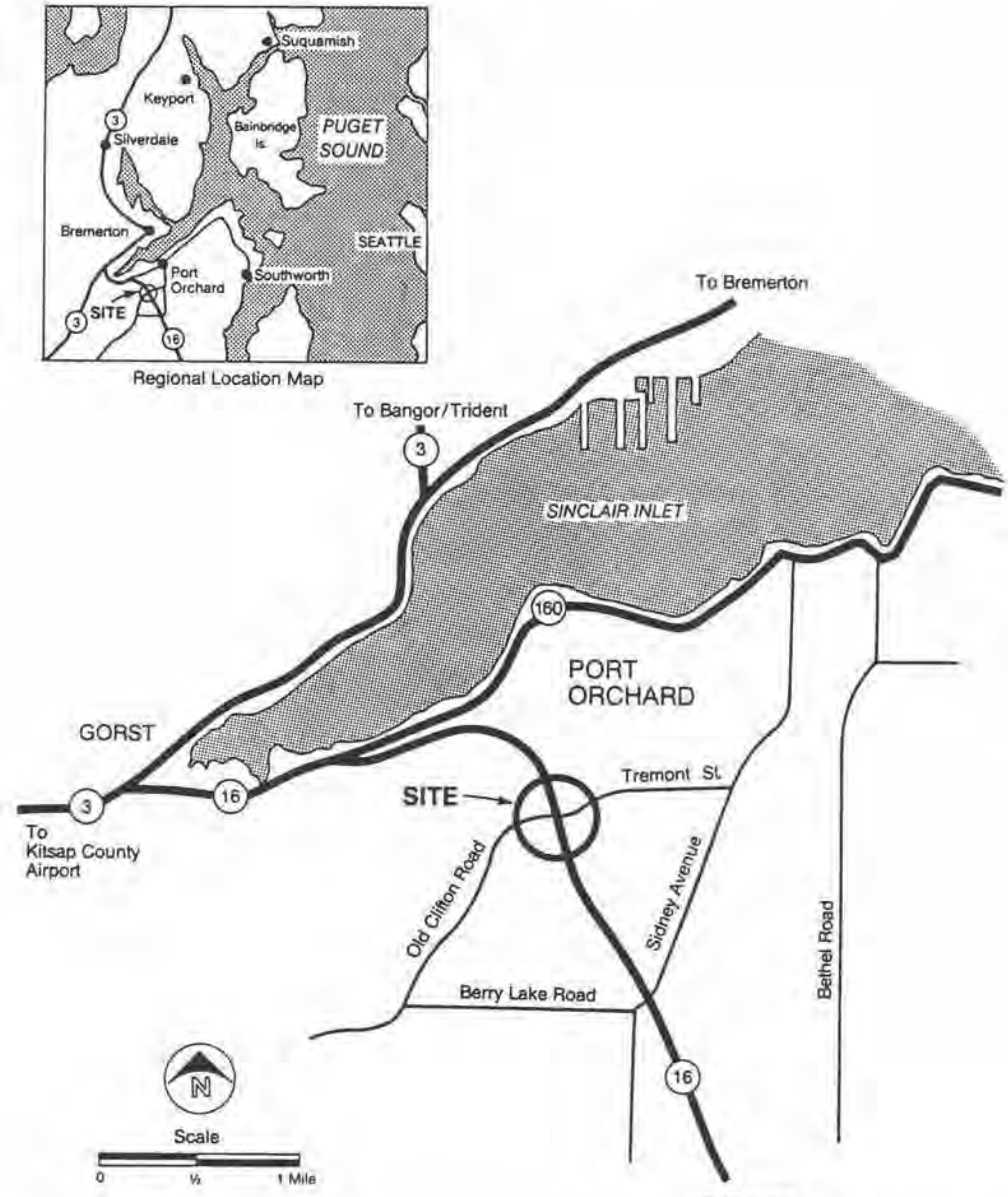

Figure 1. Location map for State Route 16 interchange, Kitsap Peninsula.

northbound SR 16 entrance ramp. This investigation consisted of eight hand and mechanical auger probes in the peat areas of the southbound SR 16 exit and entrance ramps, and four test borings at the retaining wall.

Phase 1 Investigation

Geophysical Survey

Horizontal electrical profiling (HEP) was performed by Geo Recon International to approximately locate areas of fine-grained materials. Initial vertical "soundings" at two locations of known subsurface conditions established an electrode spacing at 46 ft to provide an optimum depth penetration of 25 ft for the survey. Twenty HEP lines were run along SR 16, relocated Old Clifton Road, and Tremont Street.

Low resistivity values were considered an indication of fine-grained materials such as peat, clay, silt, or any combination of these materials. The survey results indicated that fine-grained materials underlie a large portion of the site. Low-resistivity areas generally corresponded to low-elevation areas where runoff and spring waters collect to form marshy wetlands. In general, the distribution of low-resistivity material follows the area's drainage system. Most of the site's southwest quadrant, along with portions of the northeast and northwest quadrants toward the Tremont intersection, contained low-resistivity material. A correlation of HEP results with results from subsurface borings, probes, and test pits is presented in Figure 3.

Figure 2. Stationing and proposed alignment of new interchange, State Route 16.

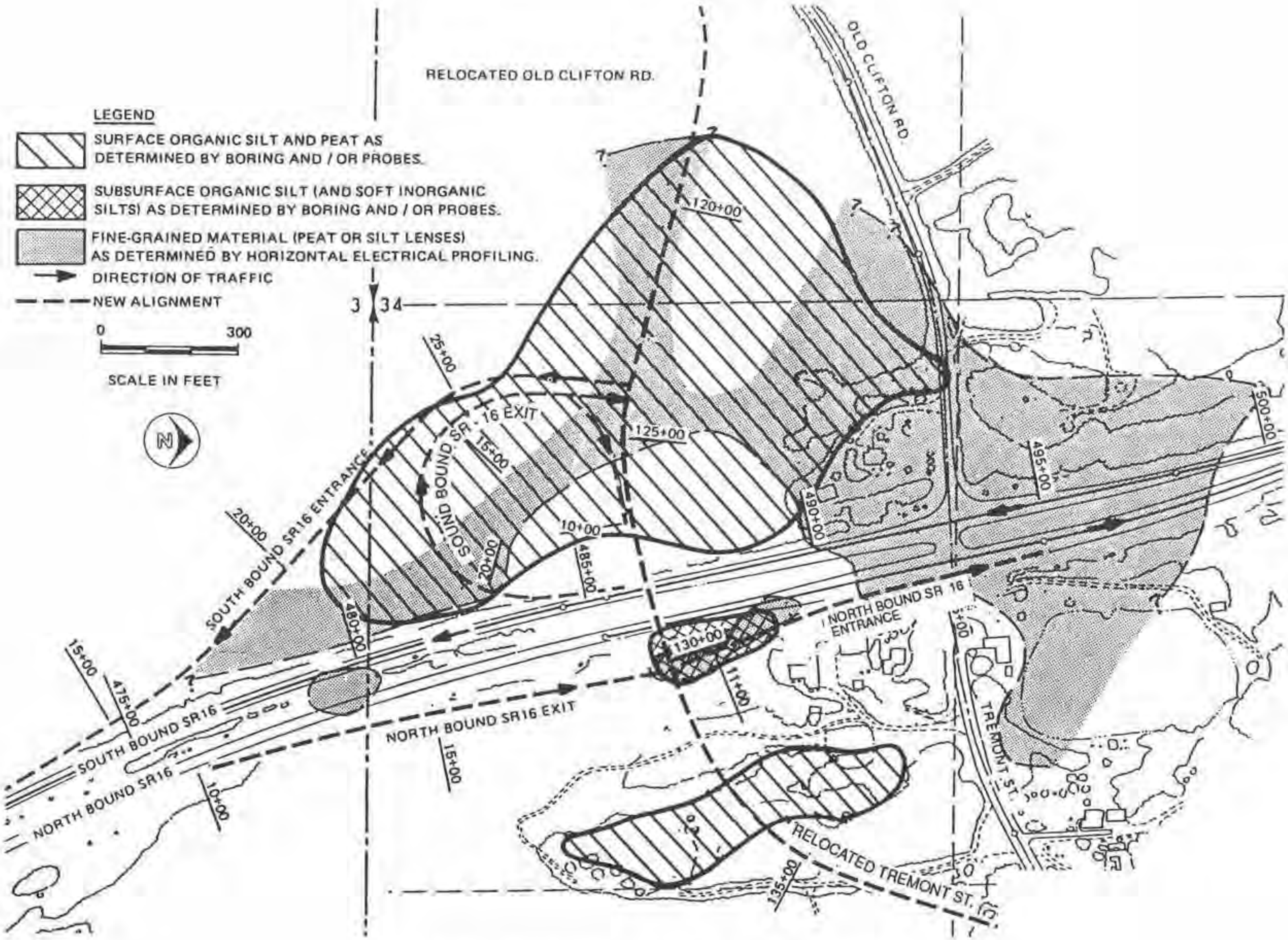

Figure 3. Surface and subsurface peat and silt deposits at interchange.

Auger and Water Jet Explorations

Mechanical or hand-operated auger borings were taken to obtain information about subsurface soils less than 10 ft in depth. The objectives of this near-surface investigation were to determine areas of soft, organic materials and to investigate soil types identified by the geophysical survey. Nineteen auger borings were made by a portable posthole digger with 3-in. solid-stem augers. The remaining three holes were made by a 4-in.-diameter hand auger.

Thirteen water jet explorations were performed in and around wet areas that were inaccessible to auger equipment. Equipment used in jetting and sampling included a gas-powered 10-in. centrifugal pump mounted either in a boat or along a shoreline, several hundred feet of hose, and 25 ft of 1-in.-diameter steel pipe with screw connections. A 1-in.-diameter, 1-ft-long sampler was attached to an end pipe section, jetted under water pressure to a desired location, and driven with a small hammer apparatus to obtain a sample.

Phase 2 Investigation

Test Borings

Test borings were drilled along the proposed alignment to delineate the subsurface profile and to obtain samples of the soil for laboratory testing.

Because of the differing terrain and subgrade conditions, four different drill rigs were used to drill the holes. For the test borings located on pavement or in easily accessible areas, a truck-mounted Mobile B-61 drill was used. In wooded or vegetated areas where roads had to be cleared for access, a Mobile B-50 drill rig mounted on a crawler tractor with a bulldozer blade was used to facilitate the drilling by clearing and leveling areas for the drill rig. In low-lying areas, where wet, compressible deposits of peat and organic silt are present, a Mobile B-61 drill rig mounted on a Nodwell FN-110 all-terrain vehicle was used for the test borings. Despite the maneuverability of the Nodwell in soft deposits, it was necessary to build log roads for access to drilling sites located on the organic soils near the new alignment for the relocated Old Clifton Road. An Acker Soil Mechanic portable drill rig was used on a steep side slope of existing SR 16 that was inaccessible to the larger drill rigs. The portable rig was disassembled and winched to the site for drilling.

Both disturbed and undisturbed samples were obtained from all of the test borings in accordance with applicable ASTM standards.

Soil Probes and Test Pits

After test borings and the preliminary investigation delineated locations and approximate thicknesses of soft organic silt and peat deposits, soil probes were performed in an effort to better define the extent of the soft deposits. The soil probe was a 1/2-in.-diameter, 11-ft-long steel bar that was manually pushed into the soil until refusal. The accuracy of the probe was checked in soft areas where information from the test borings was available regarding the thickness of the deposit.

Four test pits were excavated along the alignment for relocated Old Clifton Road. The test pits were dug (1) to determine the excavation characteristics of the soft silt and peat deposits west of SR 16, (2) to determine the excavation characteristics of the soft silt and loose sand in areas of potential high ground water on the east side of SR 16, and (3) to obtain material for pavement testing.

Phase 3 Investigation

Mechanical Auger and Hand Probes

Combination mechanical auger borings and hand probes were performed to define the extent of unsuitable material in the southbound SR 16 entrance and exit ramp area. The auger borings were performed with the portable posthole digger described for the phase 1 investigation. Augers were used until reaching either competent material or a maximum depth of 9 ft (the limit of the posthole digger). Beyond 9 ft, or if competent material was encountered, a 3/4-in. steel rod with an attachable driving hammer assembly was used to find or verify the depth and type of competent material. The 3/4-in. rods were fitted with a 1-ft-long, 3/4-in. sample tube that could be driven into subsurface materials after removing the 3-in. solid stem augers.

Test Borings

Test borings were drilled at the location of the retaining wall along the entrance ramp to northbound SR 16 to define soil conditions. Four test borings were performed during the phase 3 investigation.

SITE SUBSURFACE DESCRIPTION

Soil Profiles

The subsurface profile throughout the project site was uniform below a depth of 25 ft from the ground surface. These subsurface deposits generally consisted of medium-dense to very dense silty sand to poorly graded silty sand, with scattered subrounded gravel and cobbles. Above this depth, the soil profile varied with location and terrain.

For discussion of the upper subsurface profile, the area along the new road alignment was divided into four sections proceeding from west to east, as shown by the circled numerals in Figure 2. The subsurface information is summarized in profiles taken along the relocated alignment of Tremont Street and relocated Old Clifton Road and along SR 16 (Figures 4A, 4B, and 4C). A site plan view showing the extent of surface and subsurface organic deposits is shown in Figure 3.

A profile taken through relocated Old Clifton Road is shown in Figure 4A. The first section extends along

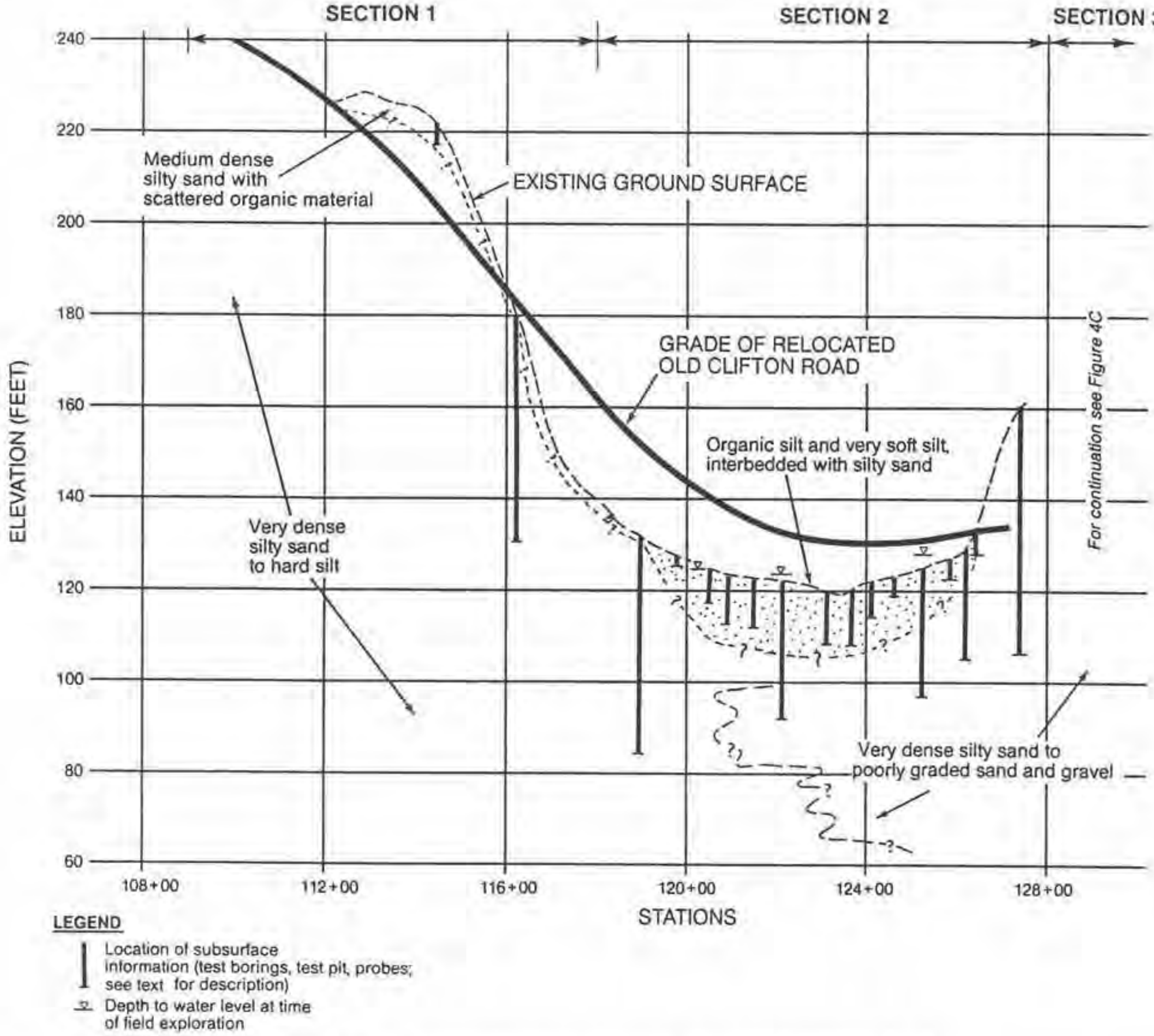

Figure 4A. Subsurface profile along proposed relocated Old Clifton Road.

the steeply sloping hillside from relocated Old Clifton Road, west of SR 16, from Station 111+00 to Station 118+50, as shown in Figure 4A. Surficial soils (approximately the upper 5 ft) at this location consist of a medium-dense silty sand with scattered wood pieces and fibers. Below this layer, subsurface materials consist of a very dense silty sand. No ground water was encountered in this area during the field exploration.

The second section is a low-lying swampy area extending from the base of the steep hillside in section 1 at Station 118+50 to the slope of SR 16 at Station 127+00 and the area south of this, which includes the southbound entrance and exit ramps to SR 16. Compressible soft organic deposits in this area varied from peat to a highly plastic, very soft organic silt with wood fibers. The lateral extent of the compressible deposit is shown in Figure 3. The thickness of this soft deposit appeared to be greatest in the center of the wetland (at relocated Old Clifton Road Station 122+00, with as much as 14 ft passed through.) The deposit thinned to less than 5 ft at the east and west edges of this low-lying area. In some parts of the low areas, the organic deposits were interbedded with loose, fine-grained silty sand. Below the soft, compressible deposits are very dense sand and silty sand to hard, sandy silt. At most locations in the swamp, the water table was near, at, or above the ground surface.

The third section is the area beneath the existing SR 16. A simplified subsurface profile of the southbound lanes is shown in Figure 4B. A previous investigation by WSDOT showed the presence of surface peat between SR 16 Stations 486+00 and 488+00, which was excavated prior to road construction. The surface peat was at an approximate elevation of 145 ft and was as

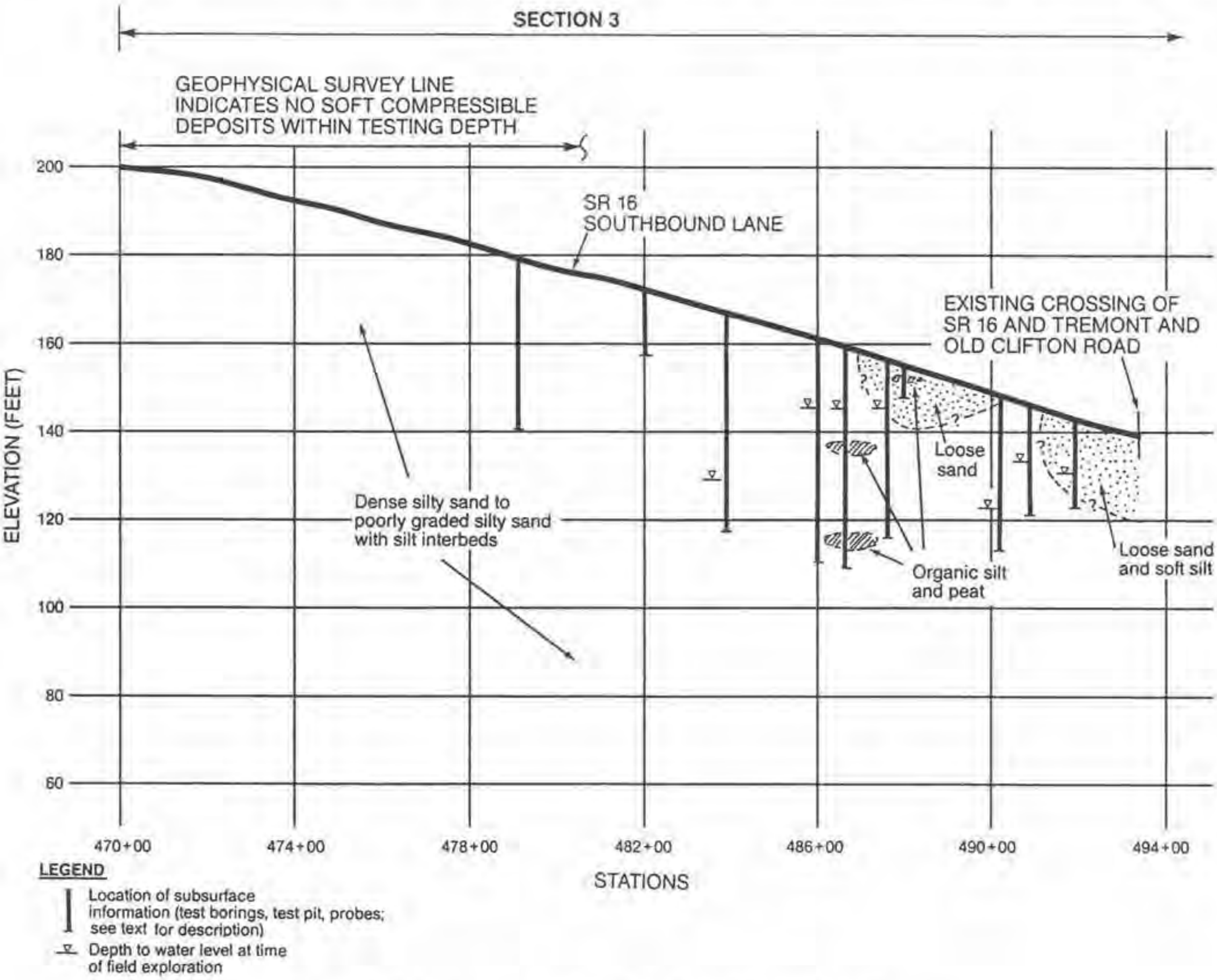

Figure 4B. Subsurface profile along State Route 16, southbound lane.

much as 12 ft thick at the thickest part of the deposit. Profiles from WSDOT indicate that the road was constructed primarily on cut along the Tremont Interchange area, except where the surface peat was removed and replaced with fill.

Test borings for this investigation showed that the upper 20 ft of soil in the fill section consist of dense to very dense silty sand to gravelly silty sand. At a depth of 20 to 25 ft (elevation 140 to 135) under a portion of the northbound lanes of the highway, an interlayered medium-dense silty sand and organic silt and peat was present; this is thought to be the remnants of the excavated peat. Deposits of loose sand and soft silt were encountered locally in the upper 20 ft in test borings completed near the existing crossing of SR 16 and Tremont Street and Old Clifton Road. Below this depth, very dense sand and silty sand was present in the remainder of the test borings except for a lens of organic silt encountered in a test boring at a depth of 45 ft.

The fourth section extends east from the edge of the fill slope off the northbound lane of SR 16 at Station 131+00 to the intersection of the existing Tremont Street with the new alignment and north/south to include most of the entrance and exit ramps for northbound SR 16. A subsurface profile along the relocated Tremont Street is shown in Figure 4C. The relocated Tremont section is in a low-lying area between two ridges. Two ponds are present in the lower portions of the area and might be the result of localized perched water conditions.

In the low-lying areas, the upper 5 to 10 ft vary from loose sand to inorganic and organic silt of varying density. Beyond this depth is very dense silty sand. Elevated areas bordering the wetlands consist of

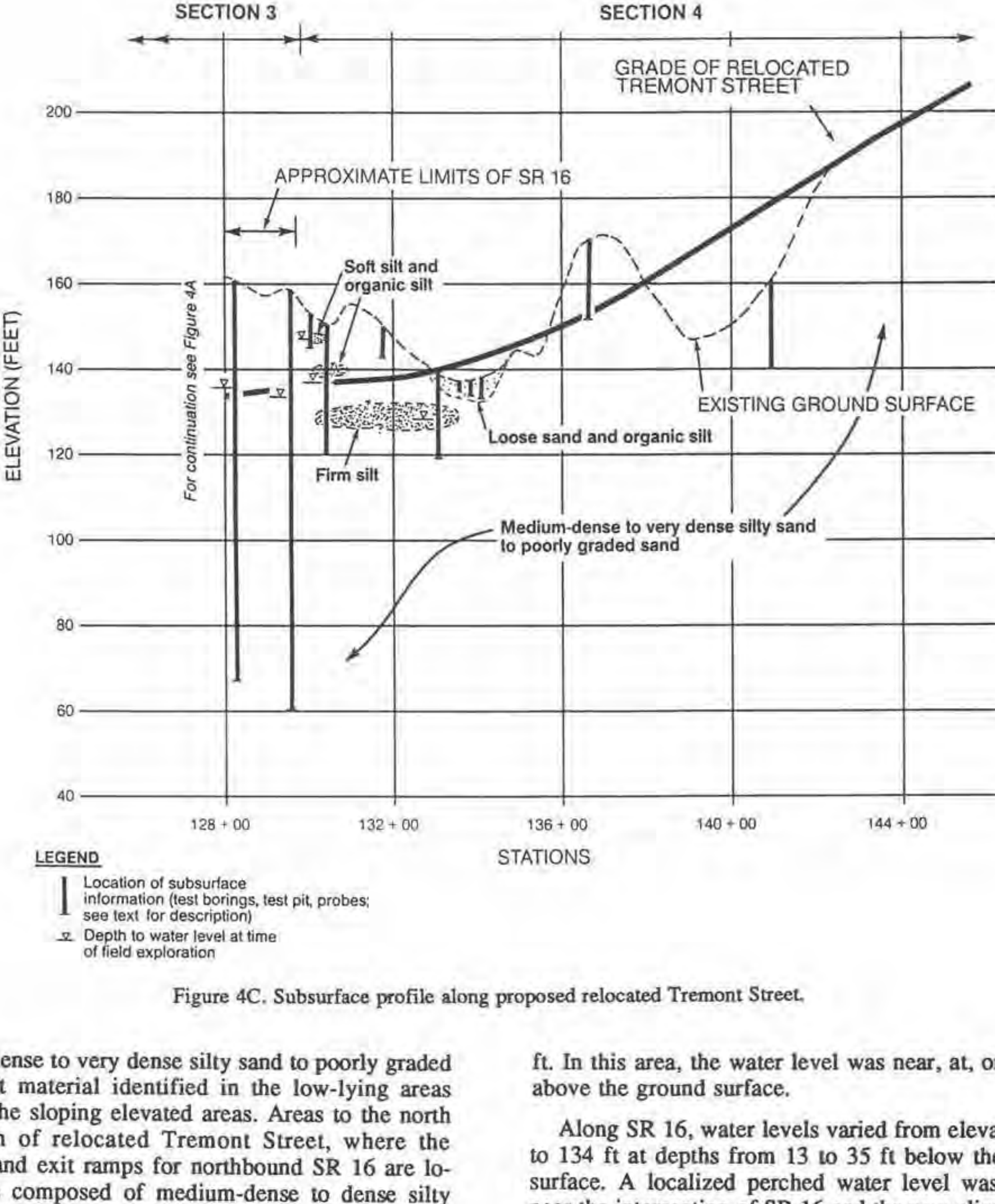

Figure 4C. Subsurface profile along proposed relocated Tremont Street.

medium-dense to very dense silty sand to poorly graded sand. Soft material identified in the low-lying areas thins on the sloping elevated areas. Areas to the north and south of relocated Tremont Street, where the entrance and exit ramps for northbound SR 16 are located, are composed of medium-dense to dense silty sand and sand with scattered silt interbeds.

GROUND WATER

Elevations of the water table as measured in test borings varied from about 120 to 145 ft. The water table generally reflects the ground surface; it is higher in the upland areas and lower in the lower areas as shown on the profiles in Figures 4A, 4B, and 4C.

Borings in the low-lying, swampy area west of SR 16 encountered water at elevations ranging from 120 to 128 ft. In this area, the water level was near, at, or slightly above the ground surface.

Along SR 16, water levels varied from elevation 123 to 134 ft at depths from 13 to 35 ft below the ground surface. A localized perched water level was present near the intersection of SR 16 and the new alignment of Tremont Street and Old Clifton Road at elevations ranging from 140 to 145 ft. East of SR 16, water was present in only two of the test borings and likely reflects perched zones.

Large areas of ponded water are present east of this portion of the alignment and might have resulted from poor drainage conditions and impermeable soil layers. It is probable that water-surface elevations in these ponds will vary by season.

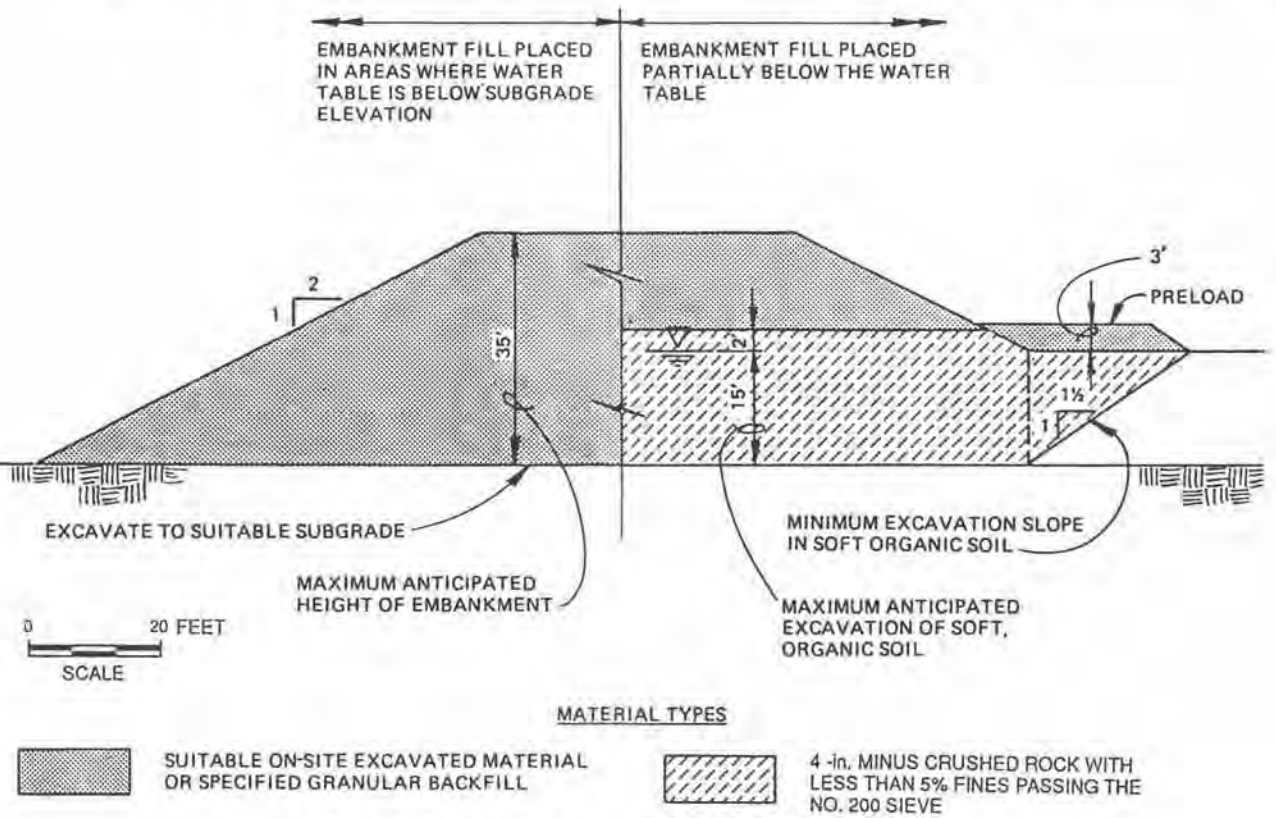

Figure 5. Embankment cross section, Tremont interchange.

GEOTECHNICAL RECOMMENDATIONS

Excavations

The subsurface deposit of organic soils and soft silt affected construction for both northbound SR 16 ramps, the concrete cantilever retaining walls for the northbound SR 16 entrance ramp, and the northbound SR 16 overcrossing.

Organic silt and peat and soft inorganic silt, because of their low strength and high compressibility, are unsuitable materials for embankment, bridge, and retaining wall foundations and for road subgrades. If left in place beneath the east and west embankment areas, these materials would have provided an unstable foundation with the potential for embankment slope failures and large differential settlements after embankment construction. Therefore, they were removed where encountered beneath embankments and roadway sections. In addition to large total and differential settlements, roadway drainage would have been a problem in these organic-rich areas, which typically have poor drainage characteristics and perched ground water. Beneath the overcrossing or concrete cantilever wall structures, subsurface organic deposits were a source of potential instability and settlement and would have necessitated the use of a pile foundation for support if left in place.

The extensive surface peat deposit west of SR 16 is characterized as a saturated, low-density (high void ratio) material of low shear strength. As much as 10 ft of this material overlies interlayered deposits of organic silt with silty sand and low-plasticity inorganic silt.

The ground water in the peat deposit west of SR 16 is close to the ground surface. Slight artesian conditions were observed locally. Observations of peat behavior during excavation were obtained from test pits excavated in the deposit. It appeared that the peat would probably not support slopes steeper than 3H to 1V (horizontal to vertical) in the upper portion of the deposit where interlayering is negligible, particularly in the areas of excavation of more than 10 ft. Dewatering of excavations in this material would have destabilized side slopes because of seepage forces and pore pressure increases within the soil mass and would also have adversely affected surrounding wetlands, which are considered environmentally sensitive. The peat was removed with a large Caterpillar 245 backhoe.

Peat was removed from beneath the embankment within the limits defined by a vertical line at the slope toe intersection with the existing ground surface, as shown on the right half of Figure 5. Peat and interlayered organic silt were excavated to competent foundation material. Excavation slopes required to remove peat within the above defined limits ranged from 1.5H to 1V to 3H to 1V.

The surface deposit east of SR 16 consists of soft organic silt overlying dense silty sand. The organic silt has properties similar to those of the peat deposits on the west side of the road: low density, low strength, and high compressibility. The organic silt was dewatered with a system of ditches and sump pumps. The dewatering lowered ground-water elevations to a minimum of 1 ft below the deepest portion of the excavation to ensure proper placement and compaction of embankment materials. The wetland ponds were also temporarily drained with ditches and sumps for road construction.

Poor subsurface soil conditions consisting of interbedded peat, sand, and silt were encountered adjacent to northbound SR 16 at the overcrossing, extending east to include the low area at the intersection of the proposed northbound SR 16 entrance and exit ramps with Tremont Street. Perched ground water in this area also necessitated dewatering for roadway excavations. Dewatering was again accomplished with ditches and sump pumps. The ground water was lowered to a minimum of 1 ft (approximate elevation 130) below the deepest portion of the excavation. To ensure adequate compaction of replacement materials, no underwater compaction was allowed. Gravel pads were built, and construction traffic was limited to minimize disturbance so that a proper roadway subgrade could be prepared.

Structural excavation for the foundations of the concrete cantilever retaining walls was affected by the subsurface peat area adjacent to northbound SR 16. The walls support a portion of the SR 16 embankment between Stations 10+60 and 12+67. More than two-thirds of the walls were located in the peat area beginning at the southern portion of the wall. Peat, other organic soils, and soft, compressible silt encountered during excavation for the walls were removed prior to their construction. Dewatering for this foundation excavation was performed in conjunction with dewatering for roadway excavation.

Unsuitable material was not encountered during excavation for the northbound SR 16 overcrossing. Design plans for the overcrossing structures called for the bottom of the abutment wall support slab (pile cap) to be approximately 17 ft below the existing embankment surface, with a 2-ft-thick gravel blanket located below the support slab. Isolated organic-rich lenses encountered at foundation subgrade were removed to a minimum of 1 ft below grade and replaced with competent granular material, compacted in place to provide a working surface for pile installation.

Cut Slopes

Permanent cut slopes in dense to medium-dense sand with varying percentages of silt and gravel were at 2H to 1V. Slope protection measures such as seeding and hydroseeding were taken to minimize erosion and reduce water infiltration for exposed cut soils.

Permanent cut slopes in the identified subsurface peat area adjacent to northbound SR 16 did not exceed 3H to 1V. Slope protection measures such as drains, riprap, vegetation mats, or slope benching were necessary to maintain slope stability where peat lenses and seeping ground water were encountered. However, at 3H to 1V cut slopes, most of the unsuitable material was removed during excavation.

In the area of the overcrossings, design requirements called for a 1.75H to 1V slope beneath the structures. These slopes were protected with a concrete facing. Because of potential instability of subsurface organic materials during pile driving, the slopes were not excavated until all piles were driven for the overcrossing foundations. Permanent cuts for the exposed slopes between overcrossing structures were not steeper than 2H to 1V. Permanent cut slopes behind the cantilever wall were at 3H to 1V to minimize the lateral load on the retaining wall and ensure a stable support for the northbound SR 16 embankment.

Embankments

Fill was placed in underwater sites with a Caterpillar 245 backhoe to build an embankment to a level above the water surface. Scrapers then dumped additional fill, and a bulldozer spread the material. The blade on the bulldozer was used to push the material down to facilitate compaction of the embankment material and displace any soft soils or slough that might not have been adequately excavated. In addition, the fill was placed from the center to the sides of the excavation to displace soft materials to the cut slopes at the side of the fill, as shown in Figure 5.

A clean, granular fill was used for the underwater embankment section because of placement and compaction restrictions. Fill placed below water was 4-in.-minus crushed rock with less than 5 percent fines passing the No. 200 sieve. When the granular fill approached the elevation of the water table and it was feasible to operate compaction equipment, the fill was compacted with a vibrating smooth-drum roller. The fill was compacted to a dense, nonyielding condition and extended to approximately 2 ft above high-water elevations to eliminate capillary action and potential frost heave in the embankment.

After the granular fill reached 2 ft above the water surface, the remainder of the fill was constructed with selected on-site excavated materials or imported granular fill. Embankment material placed above the water table was compacted to 95 percent of ASTM D1557.

Laboratory tests indicated that acceptable on-site materials consisted mainly of silty sand to poorly graded sand with gravel, with up to 35 percent passing the No. 200 sieve. Because of the difference in gradation between specified material placed below water and the overlying embankment fill placed above the water table, a geotextile was placed between the two materials to act as a filter.

It was essential that the moisture content of the on-site fill material be kept within 2 percent of optimum moisture for proper compaction of the embankment. Select on-site excavated material could be used only during the dry season when the moisture content of the material could be controlled. Because on-site excavated material can have up to 35 percent fines passing the No. 200 sieve, it would have been extremely difficult to control the moisture content during the rainy season, and the material would likely have been too wet to achieve the specified compaction.

Two types of settlement were expected to occur when the embankment fill was placed: long-term settlement of the fill itself, and settlement of deeper underlying soft layers not excavated during construction. From the results of consolidation tests conducted on soft silt, a total settlement of as much as 1.5 in. was expected in areas of the highest embankment fill in which similar material underlies the embankment. Differential settlements between embankments underlain by compressible silts and those underlain by dense sand or silty sand were also as much as 1.5 in. due to the different compressibilities of the materials. The differential settlement is anticipated to extend over the embankment area's overlying compressible material. Secondary compression of the embankment fill itself can occur over time and was estimated from NAVFAC DM 7.2 (U.S. Navy, 1982) to range between 0.1 and 0.3 percent of the height of the fill over a 3- to 15-yr time span. Where the fill height exceeds 25 ft, this secondary compression could amount to an additional 1/3 in. of settlement in 3 yr and increase to a total of 1 in. of settlement in 15 yr.

Where underwater fills were placed west of SR 16, a construction preload was placed at the sides of the fill to increase the rate of settlement of embankment zones overlying peat and to minimize any lateral and differential settlement that could occur later (see Figure 5). The preloads were constructed prior to the embankment fill and acted as partial dams to control the inflow of water into the excavation for the embankment.

STRUCTURES

The structures for the SR 16-Tremont Interchange include two overcrossing bridges for the northbound and southbound lanes of SR 16 and two concrete cantilever retaining walls. The results of field investigations performed at the bridge areas indicated that relatively competent soil conditions were present in the bridge construction area. Two areas of concern were encountered prior to constructing the structures: (1) compressible soils beneath and adjacent to the northbound SR 16 embankment with peat and/or organic silt found in thin lenses at 25 ft below the northbound SR 16 road surface, and (2) perched ground water discovered beneath both northbound and southbound embankments.

Overcrossing Structures

The superstructure for the SR 16 overcrossings consists of standard WSDOT Series 14 prestressed, precast, concrete I-girders spanning approximately 135 ft, with a cast-in-place concrete deck and diaphragms hung on each of the girders. The substructure consists of two cantilevered open-ended abutment walls with standard WSDOT wing walls to each side forming a bin-type enclosure. The abutments are located on a bench in a 1.75H to 1V slope from SR 16 to Tremont Street with standard WSDOT 20-ft wing walls located on either side of each abutment. The design earth pressures from the abutment footings were on the order of 5,000 to 6,000 psf.

A preliminary layout for the overcrossing structure called for the base of the abutment wall footing to be located approximately 19 ft below the existing embankment surface. Initially, spread footing support for the overcrossing abutments was considered appropriate on the basis of a preliminary geotechnical design. However, during the detailed design of the overcrossing foundations, it was found that pile supports would be more economical.

Foundation Development

Preliminary designs for a spread footing foundation indicated problems with bearing capacity as a result of subsurface organic deposits beneath the northbound overcrossing and slope stability problems on both overcrossings as a result of perched ground-water conditions beneath both embankments. Because the girders were near a maximum length for the overcrossing support system, the footings could not be moved away from the 1.75H to 1V slope edge to increase slope stability. Use of spread footings would have required extending the footings into deeper, more stable soil because of high overturning moments induced by seismic or static loadings. Because of its higher costs, this alternative was not selected. A pile-supported abutment that eliminated the slope stability and bearing-capacity problems associated with spread footings was ultimately chosen as the most cost-effective alternative.

Drainage

Saturated silty sands were encountered during excavation of the subgrade for pile driving. The contractor took precautions to not disturb the foundation subgrade during pile driving. This involved dewatering and placing a gravel pad below driving equipment to minimize soil disturbance.

The perched water conditions encountered in areas of both bridge locations necessitated the use of drains beneath the pile cap and behind the abutment walls to minimize hydrostatic pressures. These drains extended beneath the concrete slope protection covering the 1.75H to 1V slope to minimize hydrostatic pressures on the slope protection as well. Perforated drain pipe and "finger drain" trenches were used to convey collected water from beneath or behind abutments to the roadway drainage blanket constructed beneath the road alignment.

Clean, free-draining granular material was placed beneath and behind the overcrossing abutments and in any foundation drains. This material extended a minimum of 2 ft behind or below the overcrossing abutment wall and foundation.

Filter fabric was used to control silt migration into the abutment drainage systems and reduce the potential for drain clogging.

Concrete Cantilever Retaining Walls

The concrete cantilever retaining walls located on the northbound SR 16 entrance ramp were designed by using the WSDOT Standard Plans for cantilever walls. The walls are located in an area where it was necessary to remove unsuitable foundation material and replace it with clean, free-draining, granular material. Excavations for the walls encountered perched ground water, and dewatering was required during excavation and wall construction.

SUMMARY

The new SR 16/Tremont Street/Old Clifton Road interchange was constructed in an area plagued by geotechnical problems associated with soft, compressible organic soils and locally high ground-water conditions. A thorough geologic and geotechnical exploration program identified problem areas that could be anticipated prior to construction. As a result, the new construction was designed with the appropriate geotechnical parameters.

The construction began in the fall of 1985 and was completed in the spring of 1987. No major unforeseen problems developed. The bearing strata at the bridges were deeper than anticipated, and the pile design was modified in order to develop frictional resistance as well as end bearing. Wet and soft areas were anticipated in the plans.

ACKNOWLEDGMENTS

The author acknowledges R. F. Coon, G. W. Avolio, and J. G. Dehner of CH2M Hill, Inc., for their review of this paper. J. Peacock and R. Middlestadt, the WSDOT construction inspectors, provided reviews to check for consistency with the actual construction techniques that were used on the project.

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Geotechnical Monitoring During Construction of Two Highway Embankments On Soft Ground in Tacoma

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INTRODUCTION

This case history describes the geotechnical monitoring program for two highway embankments constructed on soft organic silt in Tacoma during 1973. These two embankments are the approaches to an elevated portion of Portland Avenue that crosses Union Pacific Railroad (UPRR) and Burlington Northern Railroad (BNRR) tracks, just north of Interstate Highway 5 (Figure 1). Studies indicated that stability would be marginal during construction and that significant settlement and potentially excessive lateral displacement would occur. The integrity of the 48-in.-diameter water main at the toe of the embankments and of the heavily used railroad tracks and underground utilities had to be maintained during construction.

Sufficiently accurate predictions of the magnitude and rate of embankment material displacements could not be made to proceed confidently without monitoring. Therefore the embankments were built in stages, and the length of time between stage additions was governed by a foundation soil monitoring program. Monitoring instruments were placed near critical sections of the embankment and the water main.

This project demonstrated that a carefully planned and well executed geotechnical instrumentation program can provide the information needed to adjust the filling rate and measure resultant soil deformations. The embankments were safely constructed several months ahead of schedule.

PROJECT DESCRIPTION

The project consisted of replacing and realigning a two-lane viaduct spanning a number of railroad tracks and the intersection of Portland and Puyallup avenues in Tacoma (Figure 1). The site is adjacent to the Puyallup River, close to the harbor, and has an average ground elevation of about +20 ft (City of Tacoma Datum). The new overpass is a 950-ft-long bridge structure with a 27-ft-high approach fill embankment on the north and a 25-ft-high embankment intersection on the south. The plan and profile of each embankment are shown in Figures 2 and 3.

The project site is in an area of weak, compressible, organic soils. Generally, 15 to 25 ft of very soft to soft, organic silt and clay overlie deposits of sand and gravel. The soft soils are river delta and tide flat deposits; some fill placed in connection with urban developments is also present. Ground water is near or only a few feet below ground surface.

GEOTECHNICAL DESIGN STUDIES AND RECOMMENDATIONS

The preliminary planning and design incorporated a maximum fill height of 38 ft and stage construction and surcharging that would require about 18 months to complete. Consultants had advised that the 18-month construction period could be reduced by as much as one-half with the installation of an extensive system of vertical and sand drains beneath each of the approach fills.

Shannon & Wilson, Inc., the project geotechnical consultant, was subsequently requested to review the preliminary work and make further investigations for final design. This included additional field and laboratory work to independently arrive at a set of recommendations for the embankment construction and anticipated performance of the adjacent 48-in. water line near the toe of each embankment.

The conclusions from this additional work were in basic agreement with many of the previous concepts, including the need for stage construction. Vertical settlements were estimated to be as much as 2 ft; lateral subgrade displacements at the embankment edges were estimated to range from 4 to 6 in. Overall stability was determined to be marginal. However, it was believed the approach fills could be built faster without the use of sand drains and still take only 9 to 13 months for completion of primary settlements. From 5 to 8 in. of post-construction settlement was expected.

Figure 1. Intersection of Portland and Yakima avenues at the south embankment (center of this 1976 photo). North embankment is to the right, above the sewage treatment plant along the Puyallup River.

The estimated heights and periods for stage construction are shown in Figure 4. The actual rate of embankment construction was to be based on a definitive program of performance monitoring with field instrumentation consisting of piezometers, settlement plates, and inclinometers.

A procedure calling for a 9-month maximum construction time was very much desired by the City. Thus, the addition of stabilizing toe berms was recommended to enable filling to proceed at a more rapid rate.

Because of the rather substantial cost involved, vertical sand drains were not to be used, unless their effectiveness could be demonstrated. The literature contains a number of examples of projects where sand drains had been used with various degrees of success; in some, no time advantage was gained. The time required for evaluating sand drains at the location was not available. Wick drains were not in use in 1973.

Estimates of settlement and lateral deformation, no matter how sophisticated, could not yield truly precise results. However, it was important to know the response of the foundation soils to embankment loading. The water main had to be operative during the entire construction period. In addition, a failure of the embankment during construction would be not only costly but also potentially catastrophic, should the water main be caused to fail.

INSTRUMENTATION PROGRAM

The final embankment design included the use of 2H to 1V slopes with 10-ft-high x 25-ft-wide toe berms without sand drains. A 9-month schedule was proposed for building the embankments in several stages; delay periods were to be governed by data from a foundation soil instrumentation program. The instrumentation consisted of 6 inclinometers, 7 piezometers, and 13 settlement plates located near critical sections of the embankments and the water main (Figures 2 and 3). The purposes of the instrumentation are:

- Inclinometers were to monitor lateral displacements of the soft foundation soils near the toe of the embankment so that both the embankment stability and the lateral soil movement adjacent to the 48-in.-diameter water main could be checked at all times near the critical sections.
- Piezometers were installed to monitor the build-up of pore pressure near the center layer of the soft foundation soil during filling.

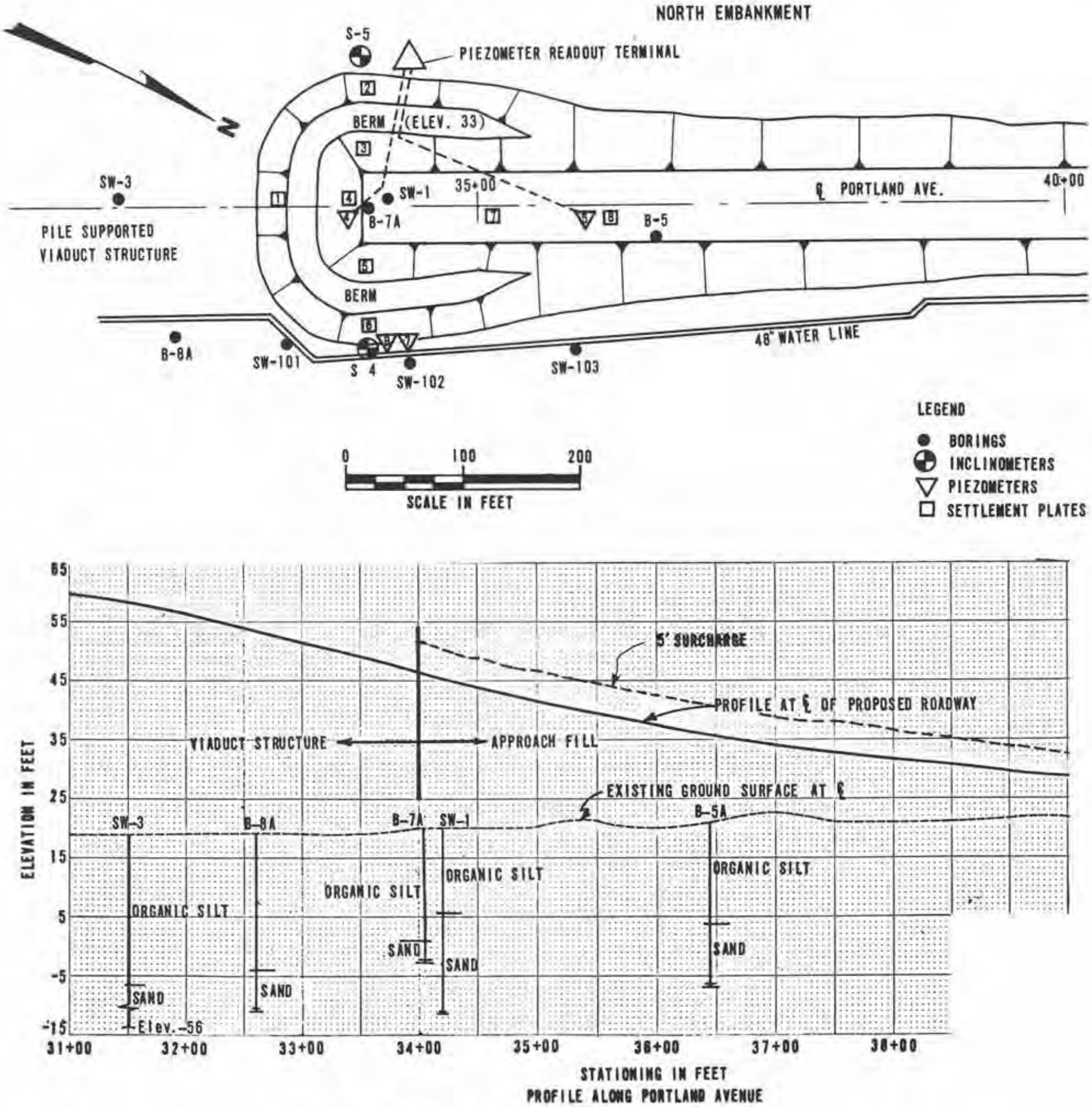


Figure 2. North embankment, plan and profile.

- Settlement plates were set at original grade with 2-in. pipe risers extending above the level of the embankment to monitor foundation settlements as fill was being placed.

INCLINOMETER MONITORING

Inclinometers measure inclination (or tilt) of a specially grooved casing in a near-vertical borehole. Measurements of change from the casing's initial position can determine lateral movements as a function of depth below the ground surface (Figure 5) and of time. The casings were installed to depths between 40 and 60 ft and extended into material that would be stable during embankment construction. Pea gravel backfill was used around the casings.

A portable Digitilt inclinometer manufactured by the Slope Indicator Company in Seattle similar to the one shown in Figure 6 was used in the monitoring program. The traversing probe has two accelerometer sensors in a waterproof housing mounted in two perpendicular sensing directions. The casing grooves control the orientation of the probe.

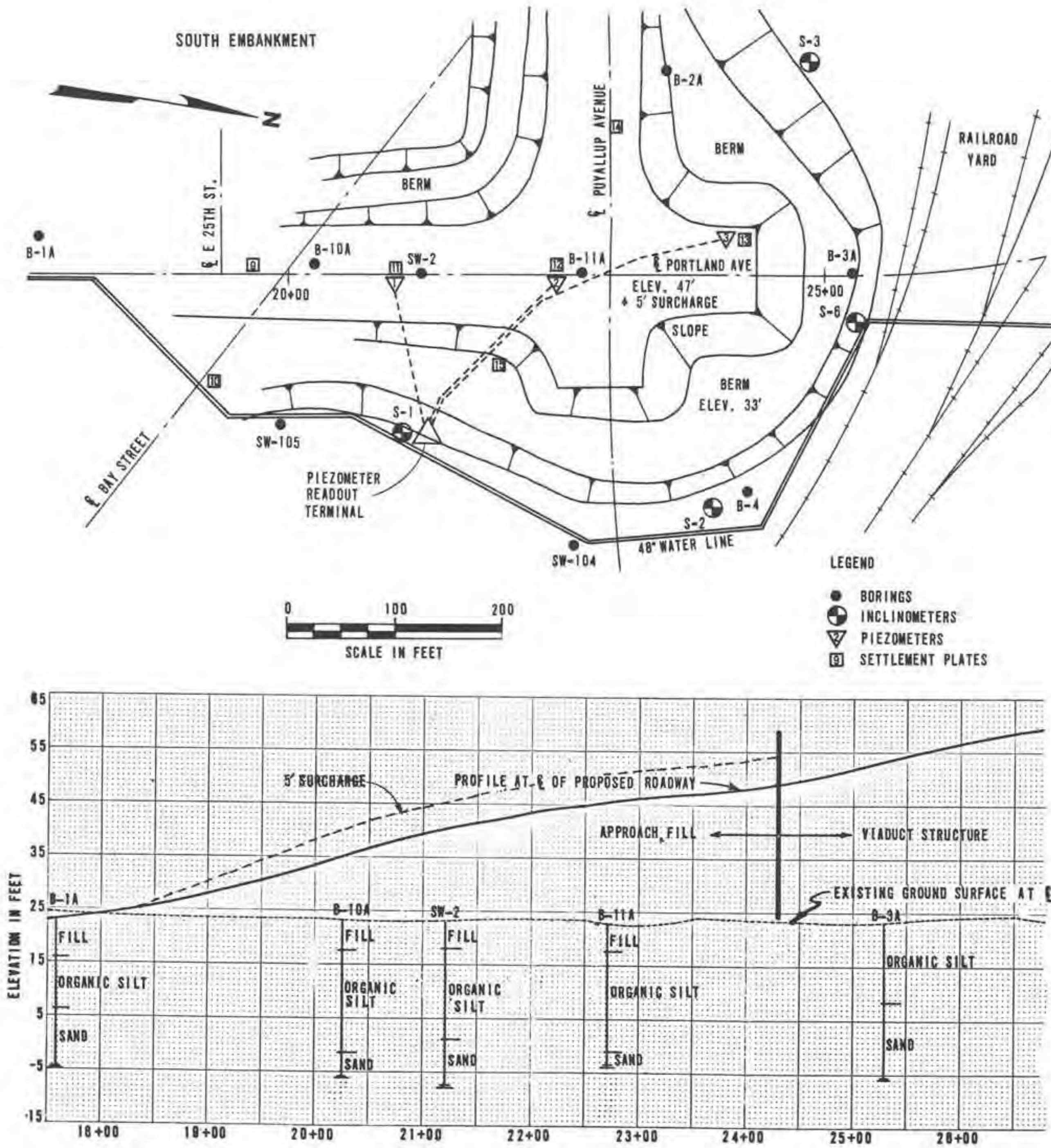


Figure 3. South embankment, plan and profile.

The probe was lowered down the casing, and an initial set of readings was obtained at 2-ft depth intervals, equal to the 2-ft gage length of the probe. Periodic readings for the same depths provided data on the location, magnitude, direction, and rate of movement of the casing. The readout was digital and was presented as a positive or negative value of twice the sine of the angle of inclination from vertical.

In the field, the data were manually recorded on forms formatted for computer input (Figure 7). A 40-ft-deep inclinometer could be read by one operator in

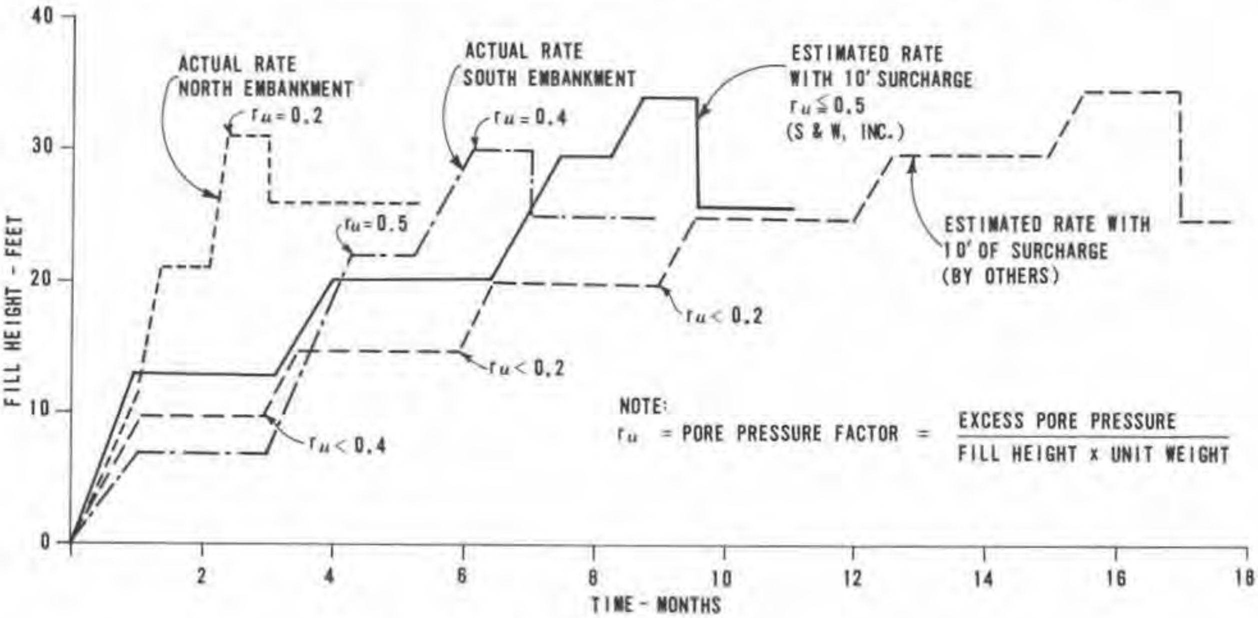


Figure 4. Estimated and actual rate of embankment construction.

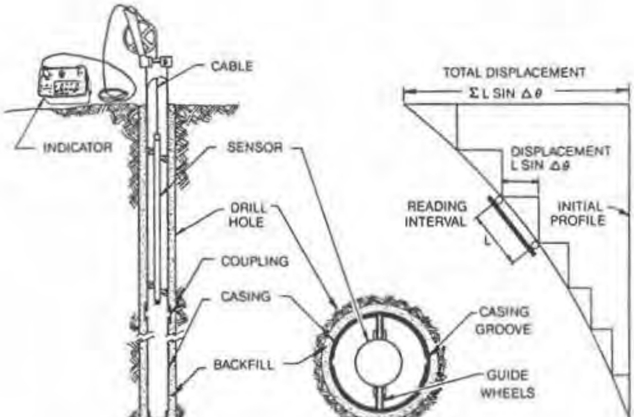
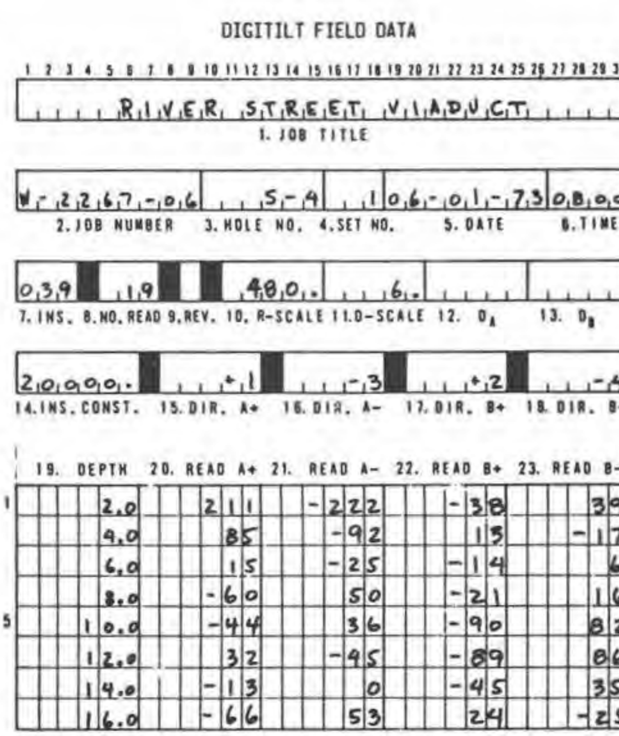


Figure 5. Inclinometer's principle of operation.

Figure 6. Digitilt Inclinometer (1987).


DIGITILT FIELD DATA

1. JOB TITLE: RIVER STREET VIADUCT

2. JOB NUMBER	3. HOLE NO.	4. SET NO.	5. DATE	6. TIME
W-2267-06	S-4	1	06-01-73	0800

7. INS.	8. NO. READ	9. REV.	10. R-SCALE	11. D-SCALE	12. D_A	13. D_B
039	19		48.0	6.		

14. INS. CONST.	15. DIR. A+	16. DIR. A-	17. DIR. B+	18. DIR. B-
20000.	+1	-3	+2	-4

19. DEPTH	20. READ A+	21. READ A-	22. READ B+	23. READ B-
2.0	211	-222	-38	39
4.0	85	-92	13	-17
6.0	15	-25	-14	6
8.0	-60	50	-21	16
10.0	-44	36	-90	82
12.0	32	-45	-89	86
14.0	-13	0	-45	35
16.0	-66	53	24	-25

Figure 7. Inclinometer field dataform, computer input format.

about 15 to 20 min; two persons could record data somewhat faster.

The average of readings taken in opposite groove directions was routinely used to eliminate possible instrument and operator errors. Also, a check on instrument performance in the field was provided by inspecting the sum of the opposite readings for consistency.

Soon after data collection began, the field data for certain depths where the largest deformations were occurring were immediately searched at each reading. In a matter of minutes, plots of movement rates could be brought up to date. After these data were reduced, the field data were keypunched and reduced by computer, and numerical data listings and data plots were included as normal output (Figure 8).

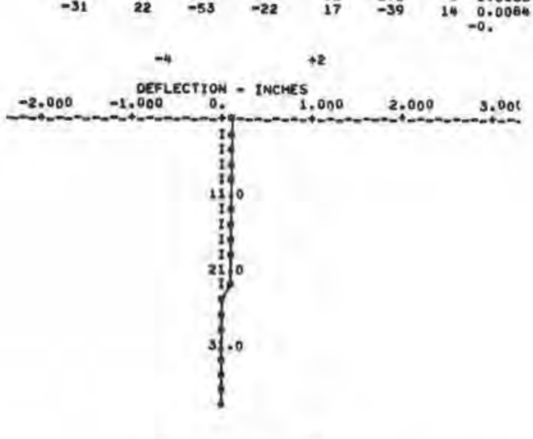
RIVER STREET VIADUCT

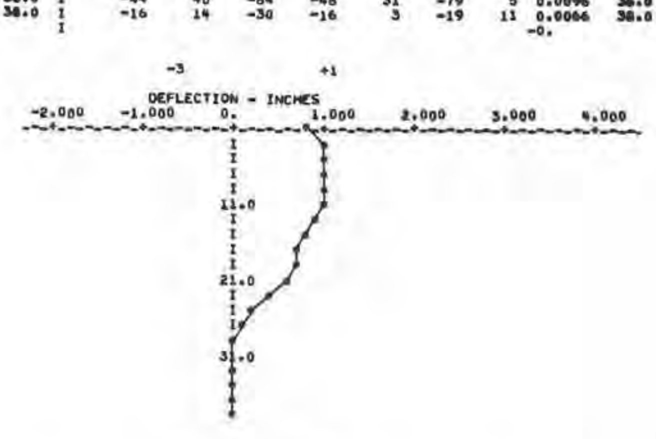
DIGITILT DATA

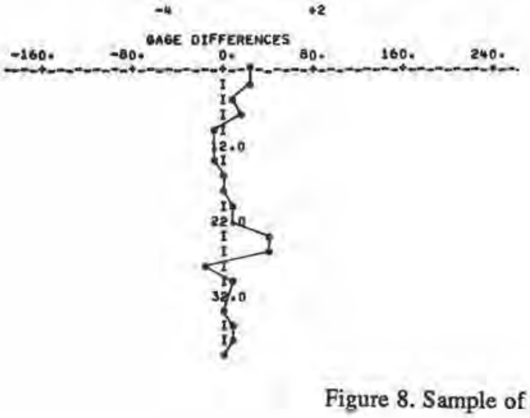
	INITIAL	FINAL
JOB NUMBER	W-2267-06	W-2267-06
HOLE NUMBER	S-4	S-4
DATA SET NUMBER	1	10
DATE	6/01/73	7/6/73
TIME		
READINGS PER DIRECTION	2	2
INSTRUMENT NUMBER	039	039
CONSTANT	20000.	20000.
ERROR ANGLE - A COMP.	-0.	-0.
ERROR ANGLE - B COMP.	-0.	-0.

INITIAL DATA			CURRENT DATA					
+1	-3	TOT	+1	-3	TOT	DIFF	DEFL. IN.	DEPTH FT.
211	-222	433	126	-133	259	-174	0.8136	2.0
85	-92	177	60	-64	124	-53	0.9702	4.0
15	-25	40	54	-62	116	76	1.0020	6.0
-60	50	-110	-65	57	-122	-12	0.9564	8.0
-44	36	-80	-36	30	-66	14	0.9636	10.0
32	-45	77	74	-81	155	78	0.9562	12.0
-13	-0	-13	40	-48	88	101	0.9084	14.0
-66	53	-119	30	-35	65	184	0.8478	16.0
-191	176	-367	-118	116	-234	133	0.7374	18.0
-324	328	-652	-237	246	-483	169	0.6576	20.0
-293	283	-576	-182	173	-355	221	0.5562	22.0
-210	195	-405	-22	11	-33	372	0.4236	24.0
-87	80	-167	35	-34	69	236	0.2004	26.0
9	-26	35	50	-57	107	72	0.0588	28.0
-23	6	-29	-8	-2	-6	23	0.0156	30.0
-166	161	-327	-176	169	-345	-18	0.0018	32.0
-123	117	-240	-125	117	-242	-2	0.0126	34.0
-93	86	-179	-89	81	-170	9	0.0138	36.0
-31	22	-53	-22	17	-39	14	0.0084	38.0
							-0.	

INITIAL DATA			CURRENT DATA					
+2	-4	TOT	+2	-4	TOT	DIFF	DEFL. IN.	DEPTH FT.
-38	39	-77	-33	17	-50	27	0.1077	2.0
13	-17	30	20	-33	53	23	0.0834	4.0
-14	6	-20	-16	-7	-9	11	0.0696	6.0
-21	16	-37	-18	1	-19	18	0.0630	8.0
-90	82	-172	-98	80	-178	-6	0.0522	10.0
-89	86	-175	-98	84	-182	-7	0.0558	12.0
-41	35	-76	-51	33	-84	-8	0.0600	14.0
24	-25	49	17	-33	50	1	0.0648	16.0
86	-84	170	77	-92	169	-1	0.0642	18.0
88	-97	185	85	-109	194	9	0.0648	20.0
12	-13	25	10	-23	33	8	0.0594	22.0
-46	41	-87	-33	16	-49	38	0.0546	24.0
-114	113	-227	-101	88	-189	38	0.0318	26.0
-189	186	-375	-203	186	-389	-14	0.0090	28.0
-141	125	-266	-146	116	-262	4	0.0174	30.0
-72	73	-145	-72	61	-133	12	0.0150	32.0
-63	60	-123	-70	56	-126	-3	0.0078	34.0
-44	40	-84	-48	31	-79	5	0.0096	36.0
-16	14	-30	-16	3	-19	11	0.0066	38.0
							-0.	







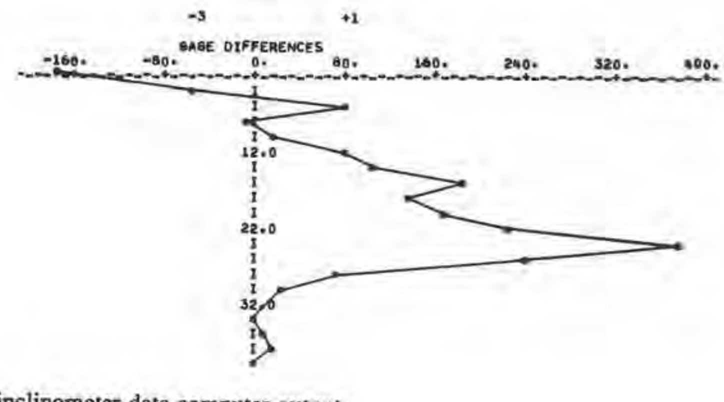
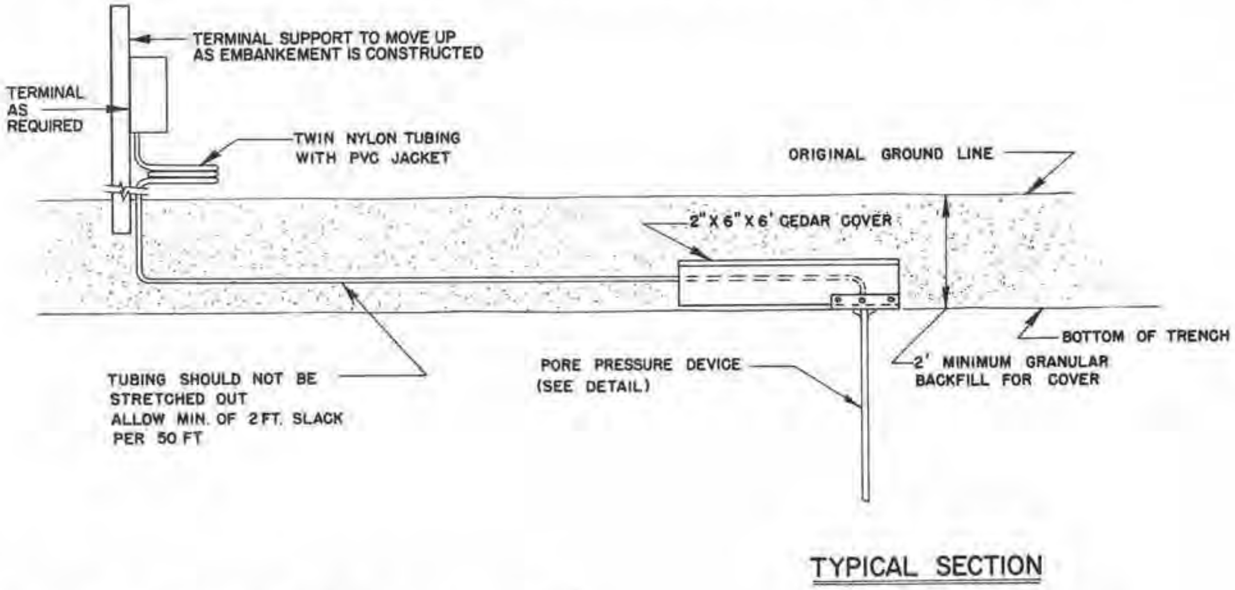


Figure 8. Sample of inclinometer data computer output.



The need for instrument data was the most critical during periods when the fill placement rates were the highest. At these times the inclinometer readings were taken twice a week, but piezometers and settlements were monitored each work day.

PIEZOMETERS

The piezometers selected for this project were of the pneumatic no-volume displacement type. This instrument has essentially no time lag in responding to actual pore pressure changes in slowly draining soils. Fast response to change was necessary because embankment stability was most critical immediately following the last lift of fill.

Each piezometer sensor was sealed inside a 2-in.-diameter driven well-point (Figure 9). The point was driven (or pushed) to the approximate center of the compressible fill layer. A special flange and protective box were installed at the top so the pneumatic tubing would not kink under the weight of fill and equipment. The polyethylene-jacketed, twin 1/8-in.-diameter nylon tubing was directly buried in a 2-ft-deep trench to a readout terminal near the toe of the embankment fill.

The readings were made by hooking a portable pressure indicator (readout box) to the terminal. Reading the piezometers was a simple task and took one person one-half hour daily. All piezometers on this project were still operating in 1976 when the last readings were taken.

SETTLEMENT PLATES

The details of a settlement plate are shown in Figure 10. These were positioned near the centerline and across sections of the highest portions of each embankment. The riser pipes were extended in 5-ft sections as fill was being placed. Riser pipes were also placed on the water main to monitor its settlements. This installation was simple and was monitored with conventional optical

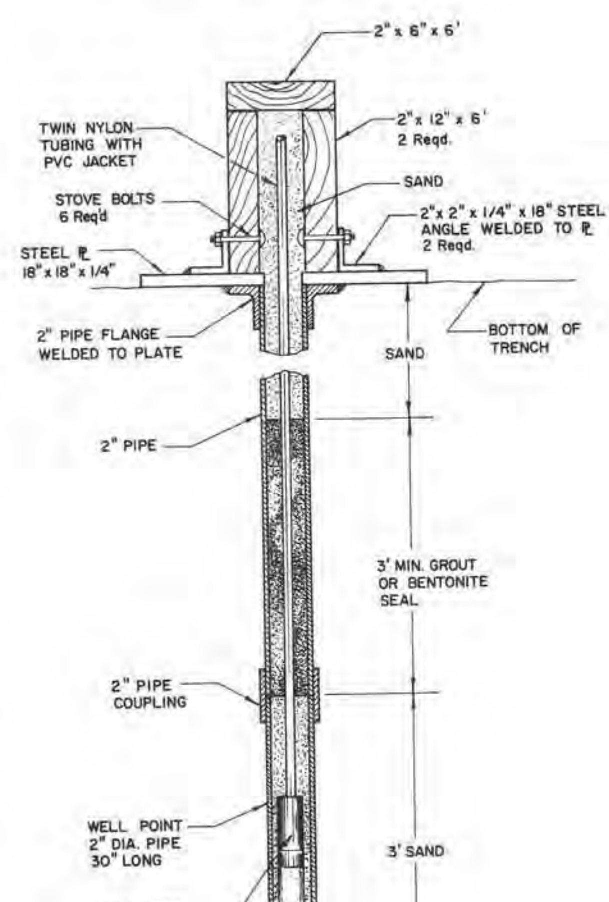


Figure 9. Schematic of piezometer installation for soft ground.

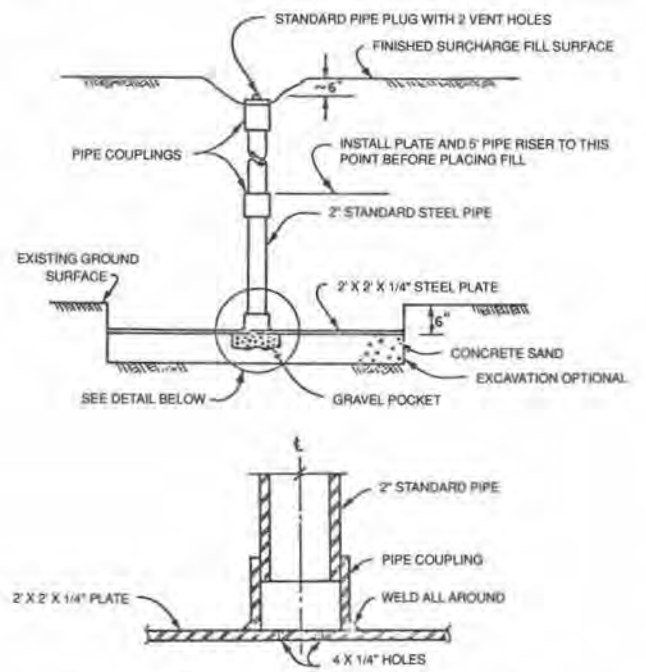


Figure 10. Schematic of settlement plate and riser.

surveying techniques used at a construction site. The precision of the monitoring increased with the number of readings being taken. Daily readings were taken during all stages of filling.

The vertical riser pipes interfered with and slowed down the fill construction, however. The contractor had a difficult time avoiding damaging the risers. Another instrument was available at that time that could have avoided the problem; Figure 11 shows an instrument that was then commercially available. The hardware cost to use that instrument would have been initially higher, but would easily have been offset by the lower cost of monitoring and increased fill production.

EMBANKMENT PERFORMANCE

Representative instrumentation data are presented in Figure 12 to illustrate the performance of the foundation soils with time.

In addition to conventional plots versus time, the data on Figure 12 are plotted versus log time. Without exception, all the inclinometers recorded performance very similar to the classic consolidation versus time relation seen in laboratory oedometer tests. While this result was not entirely unexpected, instrumentation data on a project like this generally lacks the level of accuracy required to allow such refinement in the analysis of the results. Thus, the inclinometer data provided a much better insight into the embankment performance during filling than originally expected.

The plots of fill height and pore pressure versus log time were used in correlation with the inclinometer data. These and the settlement data were not precise enough to provide extra insight from a log time plot. The conventional plots were adequate to determine accelerating or decelerating rates of settlement and changes in pore pressure; they were essential tools in the evaluation of the embankment stability and foundation behavior.

By examining both the log time relation of the lateral deformation in the depth interval of maximum move-

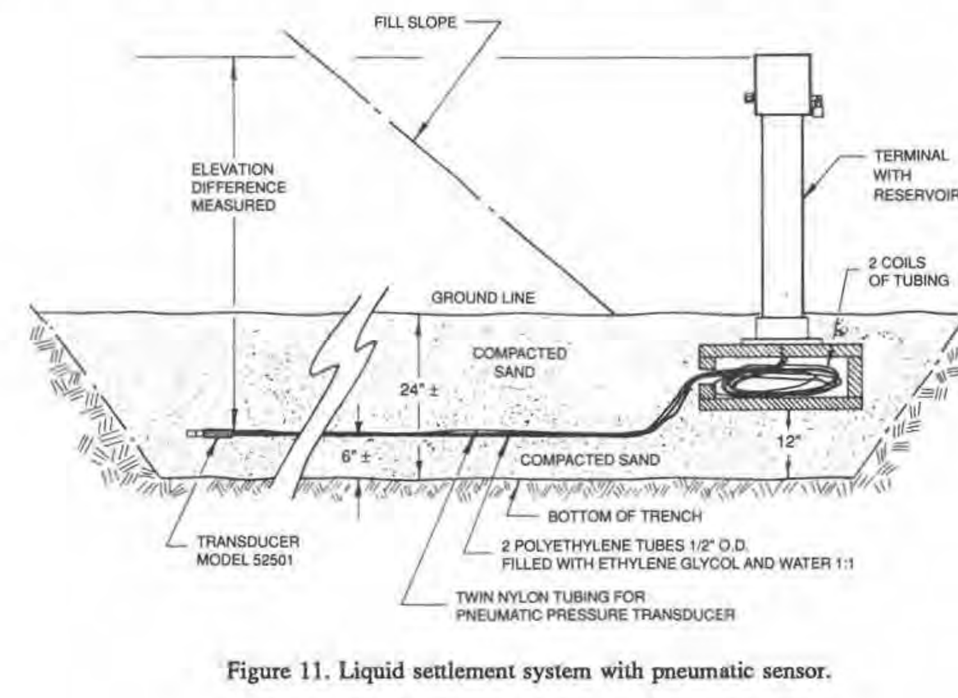


Figure 11. Liquid settlement system with pneumatic sensor.

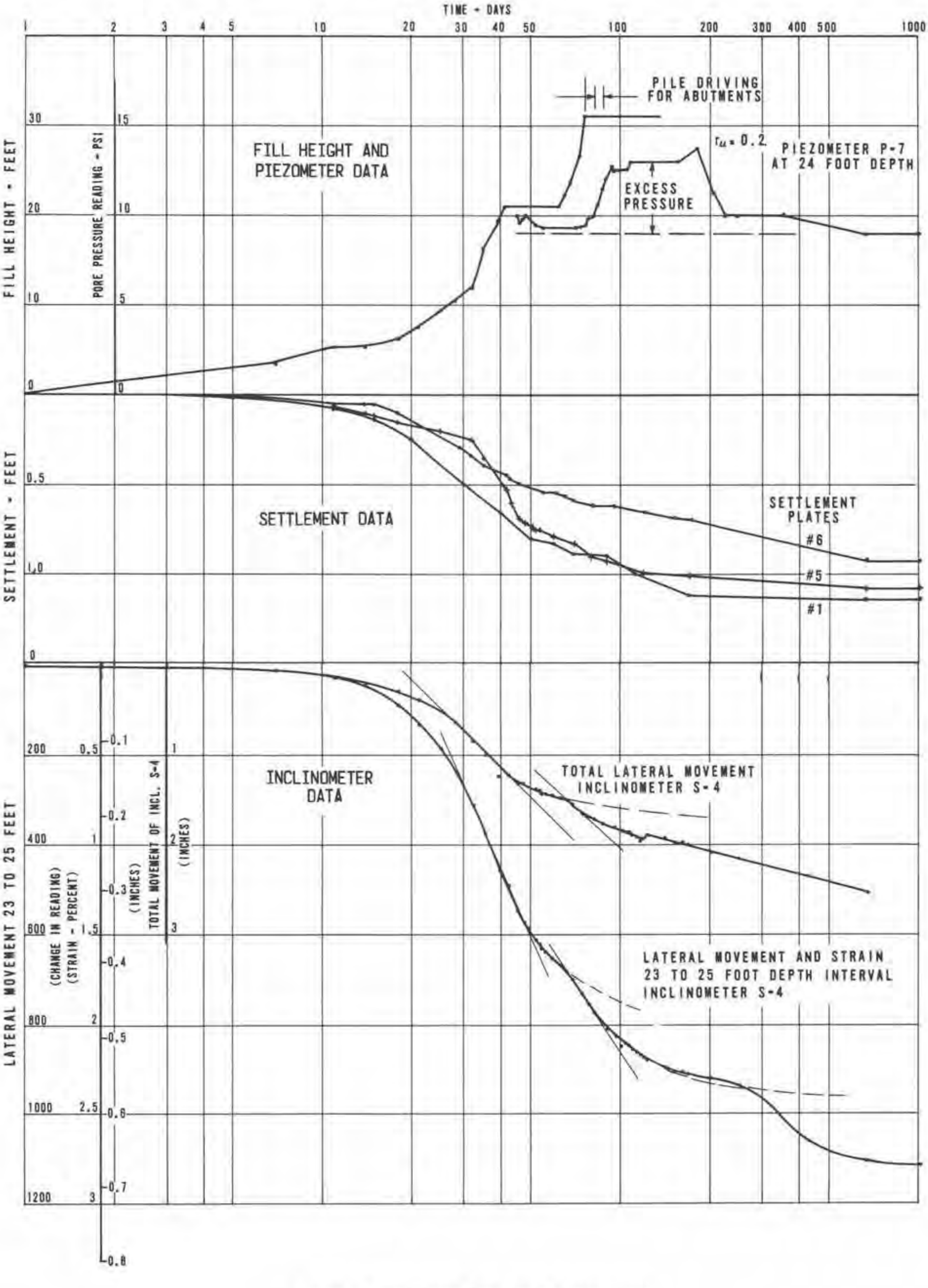


Figure 12. Instrument data, north embankment.

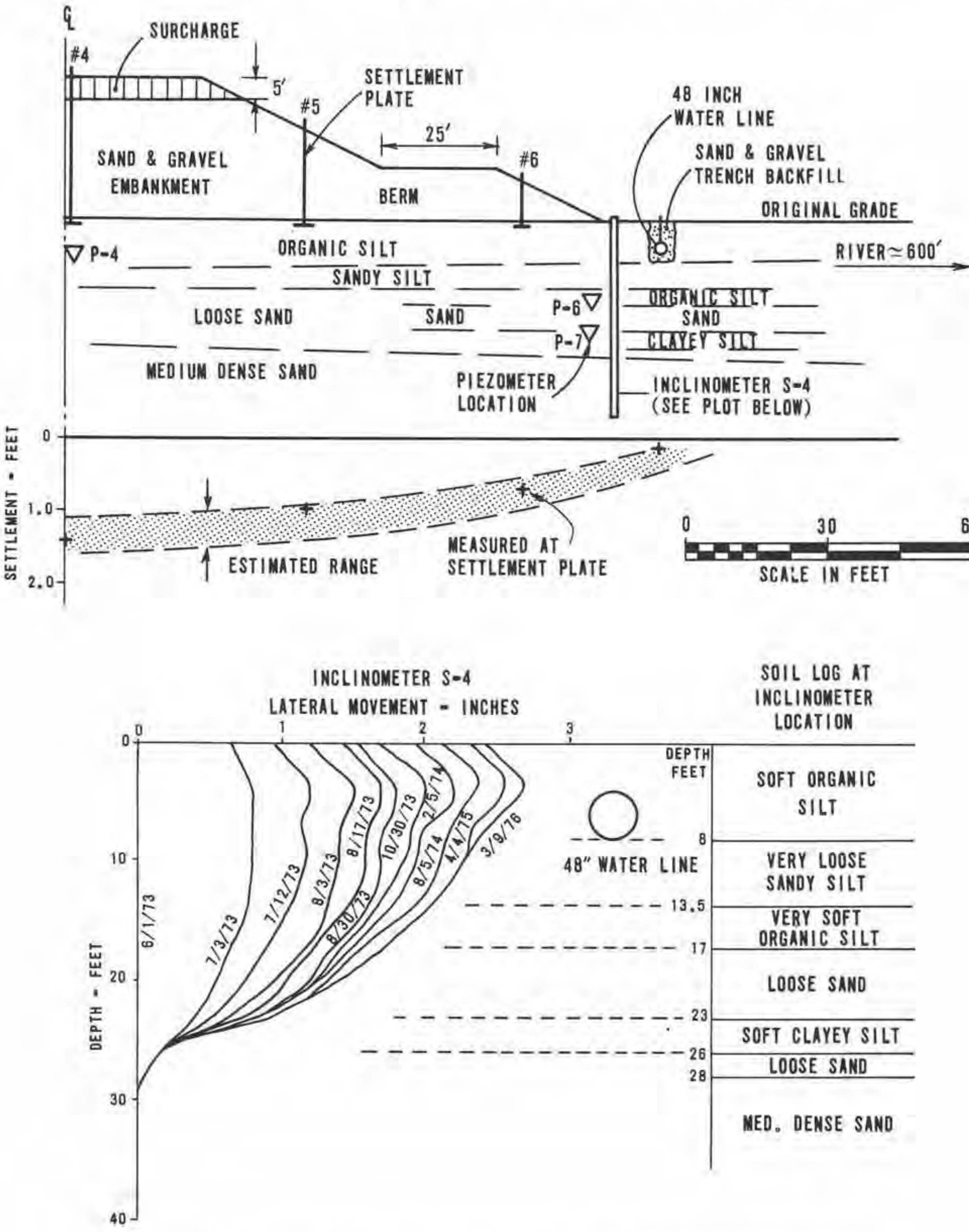


Figure 13. Inclinometer displacement and settlement plate data for the north embankment.

ment and the total movement of the inclinometer casing, it was possible to make an evaluation of the stability at the embankment toe. As long as the data exhibited a consolidation strain behavior, we were quite confident that toe failure was not about to occur. As long as there were no data indicating acceleration and the excess pore pressures were less than 50 percent of the fill pressures, filling could proceed. Generally, no more than 1 ft of fill was placed per day.

In order to obtain inclinometer data having the kind of precision attained on this project, conditions must be kept as ideal as possible. The nominal instrument accuracy for this instrument is 0.025 ft per 100 ft for a near-vertical installation. The overall accuracy during this construction project was about 0.012 ft per 100 ft. The following procedures were observed to obtain this level of accuracy:

- The casing was installed as close to vertical as possible because greater inclinations produce greater errors.
- The same portable instrument was used throughout the project and was handled with extreme care. The probe wheel assemblies were checked for excessive wear and replaced when needed. A short piece of inclined casing, fixed in place, was used to check (calibrate) the instrument periodically.
- The same personnel made readings each time to provide better precision in the procedures. The person doing the data interpretation was in charge of such personnel or made the readings himself.
- Data were recorded carefully, checked in the opposite grooves, and double checked with the previous set of readings. Any discrepancies were checked by running further precision tests.

A section through the instrumentation zone at the north embankment is shown on Figure 13. A summary of the settlements and the progression of lateral movements of the inclinometer with depth has been included. The settlements were within a fairly broad range of estimated settlements, and the lateral displacement of the soil near the 48-in.-diameter water line essentially stopped at a tolerable 2.8 in.

The north embankment was completed with only 3 weeks delay when construction had progressed to near the 2/3-height mark. The time from start of construction until completion of primary vertical consolidation of foundation soils was about 3 months less than anticipated. The south embankment was completed in 6 months with two 3- to 4-week delays.

SUMMARY

This geotechnical monitoring program was vital to the success of this construction project. The program functioned well because it was coherent and was designed and executed by the same team of engineers from start to finish. This included careful planning, design, procurement, successful installation, proper monitoring, and timely interpretation of the results.

The costs of the monitoring program was relatively modest compared to other alternatives. Where laboratory testing, analytical methods, and engineering judgment fell short, ground behavior monitoring with instruments provided the needed engineering technique.

ACKNOWLEDGMENTS

The author is grateful for the opportunity to have been of service to ABAM-Harstad, Consulting Engineers, and to the City of Tacoma on this challenging project. Special thanks to go Donald Casad and Allen Mettler, with whom all work was coordinated and who also provided all optical level data and assisted with the monitoring.

This paper was originally presented at and published in the proceedings of Fourteenth Annual Engineering Geology and Soils Engineering Symposium in Boise, Idaho, on April 7, 1976 (Mikkelsen and Bestwick, 1976). The paper was coauthored by the late L. Keith Bestwick, Senior Vice President and Manager of Shannon & Wilson, Inc., and project manager. The author dedicates this paper to his memory in tribute to the numerous challenging projects he completed in Washington State.

REFERENCE

Mikkelsen, P. E. and Bestwick, L. K., 1976, Instrumentation and performance–Urban arterial embankments of soft foundation soil. In *Fourteenth Annual Engineering Geology and Soils Engineering Symposium, Proceedings*: Idaho Transportation Department, Division of Highways, Boise, ID, pp. 1-18.

West portal drift excavations for tunnels through Mt. Baker Ridge, Seattle. These two double-lane tunnels were driven by the stacked drift method through glacially overridden hard clay. Note the large glacier-transported boulder in the foreground. 1939 photograph courtesy of Shannon & Wilson, Inc.

Stacked drift excavations for the early double-lane tunnels through Mt. Baker Ridge, Seattle. The small drifts were excavated to maintain the stability of the clay soils and reduce settlement of overlying structures. The clay core was used to support the forms for the concrete lining. 1940 photograph courtesy of Shannon & Wilson, Inc.

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Geotechnical Impacts on Design and Construction of the Mt. Baker Ridge Tunnel, Interstate Highway 90, Seattle

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Shannon & Wilson, Inc.

INTRODUCTION

The design and construction of tunnels are more thoroughly dominated by the varied and non-uniform behavioral characteristics of the geologic medium than are any other civil structures. Not only are tunnels founded on and supported by soil or rock, as are most other civil structures, but they must in turn support the geologic medium on both sides and above. The bulk of the material handled during construction is soil or rock excavated from the advancing tunnel rather than concrete or steel, as in most other construction projects. The method selected by the contractor for excavating the tunnel heading must be capable of both overcoming the strength of the soil or rock and of supporting the tunnel face and periphery until permanent tunnel support is installed. Any water problems encountered during tunneling are magnified by its occurrence not only at the bottom of the final structure, but also in the walls and crown. Therefore, the design and construction of a tunnel is as much a geotechnical problem as it is an architectural, structural, or construction problem.

Design and construction of the Mt. Baker Ridge Tunnel in Seattle (Figure 1), was heavily influenced by geotechnical considerations. This paper discusses the geotechnical conditions, the design and construction approach, and the impacts of geotechnical conditions on construction.

The tunnel was recently completed at a cost of $36.5 million, including $600,000 in resolved claims, for the Washington State Department of Transportation (WSDOT). The actual cost of the completed tunnel was $1.8 million below bid price due in part to lower than anticipated inflation combined with innovative contracting procedures for the sharing of risk, reduction of claims, and resolution of disputes.

With an inside diameter of 63.5 ft and an outside diameter of 82.5 ft, this tunnel has the largest diameter of any soft-ground tunnel in the world. The tunnel is an important part of the last remaining 6-mi-long link of Interstate Highway 90 (I-90) through Washington state; the total cost of the highway is about $1.4 billion.

The tunnel was constructed using an innovative "stacked drift" method, in which an articulated or flexible tunnel lining, consisting of 24 concrete-filled drifts, was first constructed to form a 1,332-ft-long horizontal compression ring. This was followed by removal of the soil core (Figure 2).

Excavation began in January 1983 and was completed in May 1986. Currently, construction of the interior roadway structure is being completed under separate contract for $12.6 million. When finished, the tunnel will provide five lanes for the I-90 freeway and a pedestrian/bicycle path. Three additional lanes will be provided by the two 48-yr-old, double-lane I-90 tunnels located a clear distance of 70 ft to the south.

Design of the tunnel was accomplished by Howard Needles Tammen & Bergendoff (HNTB). The geotechnical investigations, design and interpretation of construction monitoring instrumentation, and underground engineering assistance were provided by Shannon & Wilson, Inc. WSDOT also established a Design Review Board; its members were R. B. Peck, A. A. Mathews, and C. M. Metcalf. A Disputes Review Board consisting of P. E. Sperry, R. K. Dodds, and R. E. Heuer was established by the WSDOT and the contractor, the Guy F. Atkinson Company.

GEOLOGY AND GEOTECHNICAL CONDITIONS

The following paragraphs briefly discuss the geologic history and resultant geotechnical characteristics of the site soils, which bear heavily on the selected design and construction approaches. All this information was documented in a Design Summary Report (HNTB and Shannon & Wilson, Inc., 1982) as part of the contract documents for prospective bidders. The stratigraphy and corresponding soil properties in the ridge were developed from various exploration and testing programs spanning nearly 50 yr, beginning with ex-

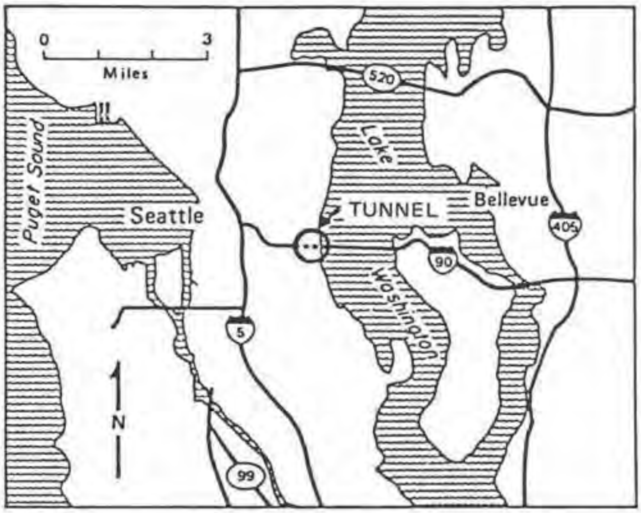

Figure 1. Location of Mt. Baker Ridge Tunnel.

ploration for the two adjacent double-lane highway tunnels. Data have been collected from 48 borings, 2 test shafts, and 3 test adits. Construction of the 3 test adits, at a cost of $600,000, for inspection during the bidding phase helped reduce the bid prices by as much as $15 million, as related by several bidders. Field tests included plate jack, borehole jack, downhole seismic, and piezometer measurements. Laboratory tests included consolidation, slake, unconfined and triaxial compression and K_o tests.

Geology

Mt. Baker Ridge is one of a series of prominent north-south-trending low ridges in the Puget Sound region that were sculpted by the last of four Pleistocene glaciations. Soils deposited by two of these glacial episodes comprise the 150-ft-high Mt. Baker Ridge.

Till and advance and recessional outwash deposited during the Vashon Stade of the Fraser Glaciation, some 13,500 to 15,000 yr ago (Mullineaux et al., 1965) mantle the hilltop. Lacustrine clays and silts deposited in a proglacial lake during the retreat of an earlier glaciation comprise the bulk of the ridge. The ridge is underlain to an explored depth of about 200 ft by laminated silts, interbedded sands and silts, and medium sands deposited during an interglacial period. Bedrock is believed to be deeper than 2,000 ft at the site.

Advance and retreat of the last glacial ice mass loaded and unloaded the underlying soils and had a significant effect on their strength, structure, and stability. The weight of an estimated 3,000 ft of glacial ice resulted in considerable overconsolidation and consequent increase in soil mass modulus. Horizontal soil stress was also increased due to ice loads. In addition,

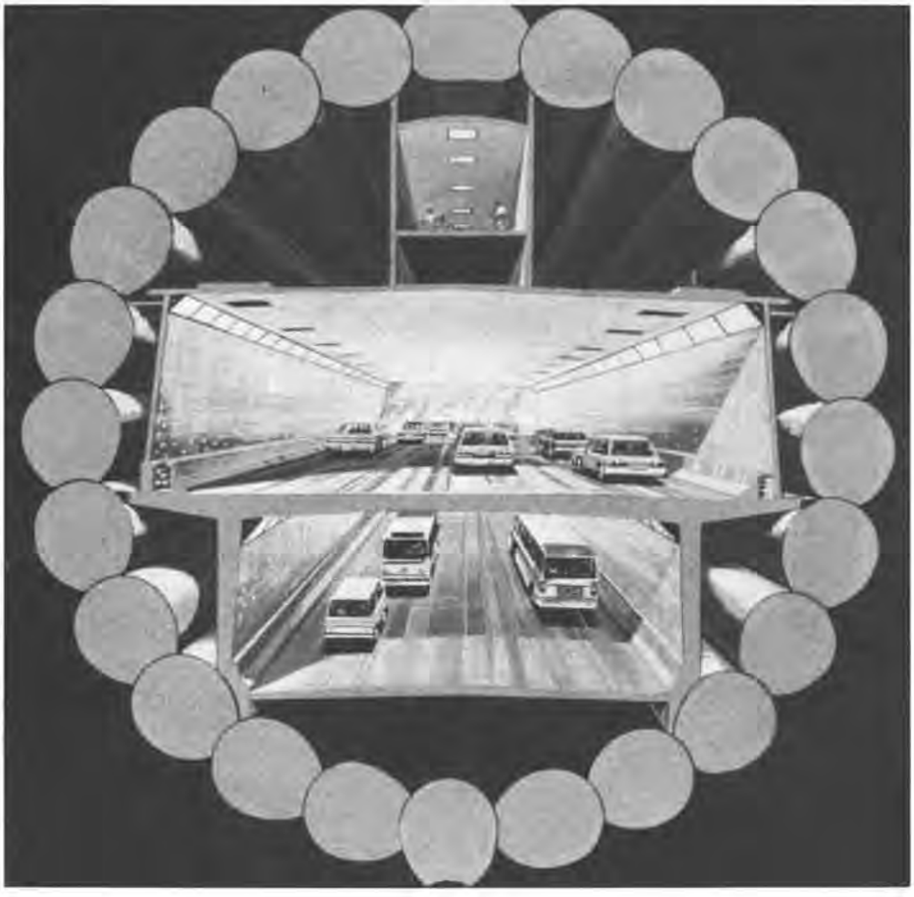

Figure 2. Cross-section of tunnel showing stacked drift tunnel lining with interior roadway and bicycle/pedestrian path.

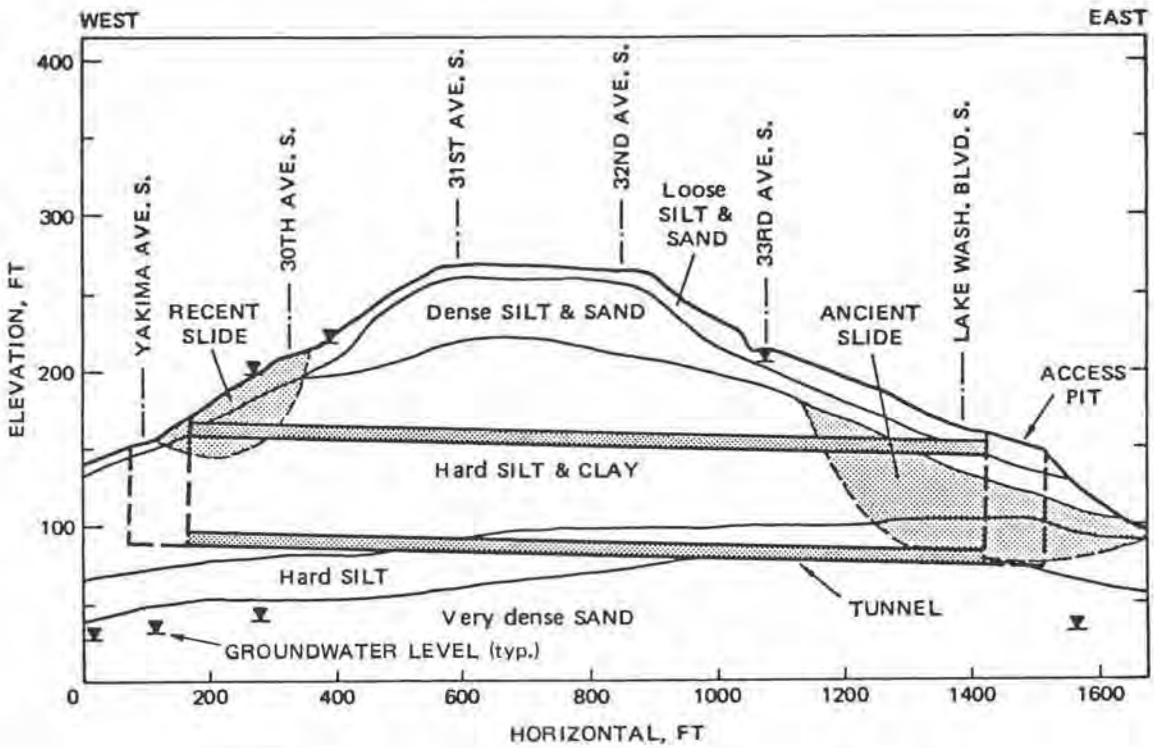

Figure 3. Longitudinal subsurface soil profile, Mt. Baker Ridge Tunnel.

Table 1. Average soil properties, Mt. Baker Ridge Tunnel area. SPT, number (n) of blows per foot from standard penetration test; d_w, wet density; WC, water content; LL, liquid limit; PL, plastic limit; PI, plasticity index; F_c, preconsolidation stress; Q_u, unconfined compressive strength

Material type	SPT (n)	d_w (pcf)	WC (%)	Atterberg Limits LL	PL	PI	Shear strength (tsf)	F_c (tsf)	Modulus (psi) Plate bearing	Q_u
Loose to Medium Dense SAND	21	--	16	32	24	8	0.85	--	----	
Upper Dense SAND	81	128	11	--	--	--	--	--	33.3	--
Upper Laminated SILT and CLAY	41	117	30	54	30	24	1.15	9	--	3.3
Non-laminated CLAY	33	117	33	64	29	35	1.6	9	11.9	3.5
Lower Laminated SILT and CLAY	32	116	33	62	29	32	1.7	18.6	21.6	2.0
Hard SILT	102	120	27	42	20	1.0	1.0	16	--	1.4
Very Dense SAND	114	119	14	--	--	--	--	--	11.4	--

glacial overriding resulted in local shearing forces, causing fracturing, warping, and mixing of soil layers. Glacial retreat unloaded the sediments, resulting in elastic rebound, local strain relief, and the creation of ubiquitous hairline fractures or fissures. Lateral support along valley walls was lost during glacial retreat, causing widespread landslides, as well as inclined and horizontal joints and shear zones, resulting in a soil mass with widely varying strength properties.

Subsurface Conditions

Mt. Baker Ridge was explored to a depth of approximately 200 ft. The major soil units are, in descending order: (1) loose to medium-dense sand, (2) dense to very dense sand and silt, (3) hard silt and clay consisting of laminated and nonlaminated subunits, (4) hard laminated silt and (5) a lower unit of very dense sand. The sequence is shown on Figure 3. Average soil design properties, as determined from a variety of laboratory and field tests, are summarized for each of the units on Table 1.

Loose Sand

Loose to medium-dense, silty sand mantles most of the ridge to a depth of as much as 25 ft. The deposit likely originates from weathering and loosening of the underlying dense recessional sand and till.

Dense Sand and Silt

The surficial sand grades with depth into a dense to very dense sand and silt with scattered gravel, cobbles, and boulders. The upper portion of this soil unit is believed to be recessional outwash, and the lower portion is a sandy, cobbly till. Locally, the till is underlain by as much as 5 ft of a slightly less dense, well sorted sand, possibly an advance outwash deposit. The outwash contains a seasonally fluctuating perched groundwater table that tends to cause the sand to flow in excavations. The till is similar to the Vashon till observed throughout much of Seattle. This till and the overlying and underlying sand units are interpreted to have been deposited during the Vashon Stade of the Fraser Glaciation. All three of these soil units are grouped together on Table 1.

Hard Silt and Clay

This unit, deposited in proglacial lakes and subsequently glacially overridden, is the soil through which most of the tunnel was excavated (Figure 3). The silt and clay unit has been subdivided into upper and lower laminated or rhythmic zones and an intermediate nonlaminated zone. The upper and lower laminated zones are composed of nearly horizontally bedded dark gray clay and lighter gray silt beds of varying thickness. Isolated cobbles and boulders as much as 2 ft across were encountered in the explorations, particularly in the nonlaminated zone. A 6-ft-diameter boulder is shown in one of the construction photos for the old I-90 tunnels.

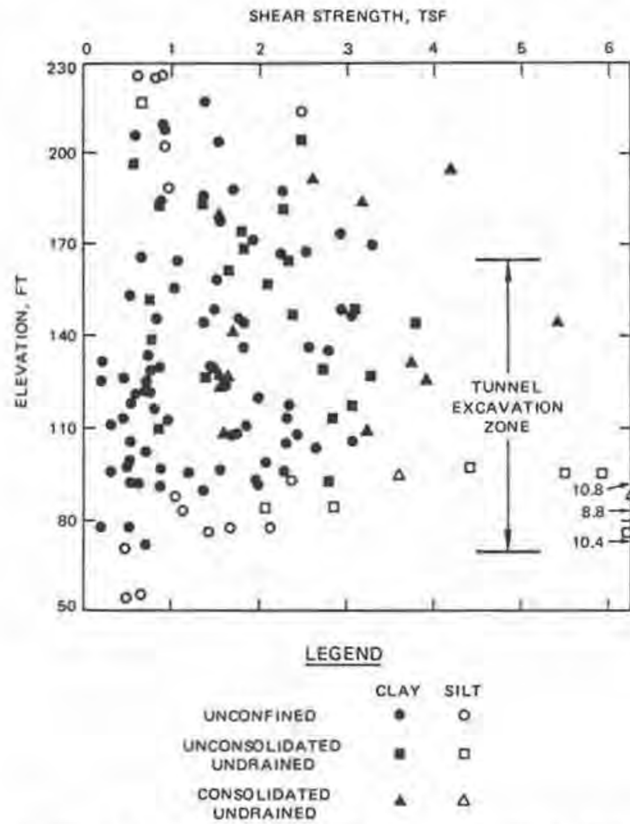

Figure 4. Soil peak shear strength versus elevation in Mt. Baker Ridge.

All three silt and clay units contain pervasive joints and small shear zones spaced from 2 to 3 ft apart. Many of the fractures are slickensided, and in some places the laminations are offset by as much as 6 in. Joint surfaces tend to dip steeply toward the nearby slopes, implying stress relief of the slopes during and following glacial retreat. Excavation-induced stress fracturing also developed in these soils in the test adit. Ubiquitous small-scale hairline fissures become apparent after wetting, causing the soil to slough or ravel. With continuous wetting and low confining pressure, these soils tend to swell slightly, as indicated by as much as 6 in. of heave in the invert of one of the test adits. The presence of various fractures causes the formation of discrete blocks and excavation overbreak under the influence of even small amounts of water.

The soil mass strength and modulus of the clay and silt are extremely varied due to the presence of fissures and joints. The wide range of peak strengths is shown on Figure 4, with the lower values more strongly reflecting the discontinuities. Effective consolidated undrained peak and creep strength tests were performed to better assess the short- and long-term strength properties for design. Envelope data for the peak and creep strengths indicate friction angles of 34° and 19°, respectively. Peak failure characteristically occurs at less than 5 per-

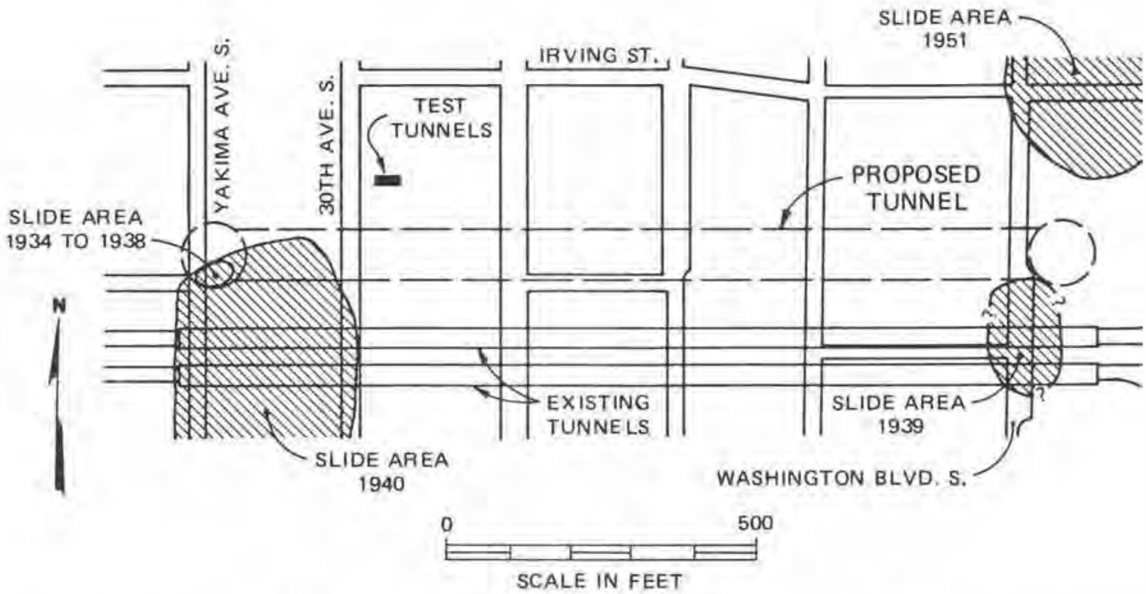

Figure 5. Historic landslide activity in the area of Mt. Baker Ridge Tunnel.

cent strain. Residual strengths have been previously assessed for overconsolidated Seattle clays and found to pertain to many deep-seated slope failures around Seattle (Palladino and Peck, 1972) involving pre-existing failure surfaces. Residual friction angles for Seattle clays have been found to range from 10° to 19°. Modulus values shown on Table 1 determined from field and laboratory tests and used for design are likely conservative. Recent self-boring pressuremeter data for the Columbia Center foundation (Grant and Hughes, 1986) indicate modulus values 2 to 3 times these values.

The overconsolidation of clays has been related by Brooker and Ireland (1965) to increased horizontal stresses. Tests indicated overconsolidation stresses on the order of 9 to 20 tsf with overconsolidation stress ratios of 2 to 4. These values correlate to K_o values (the ratio of horizontal to vertical stress) of about 1.0. Reconsolidation tests were also performed in a specially developed consolidation ring (Sherif and Wu, 1969), enabling the measurement of effective horizontal stresses in response to vertical consolidation loads on remolded samples. These tests indicated K_o values of 0.6 to 1.0. At the time of the last soil explorations in 1971, field methods for measuring K_o were not readily available. Since then, measurements with a self-boring pressuremeter within 5 mi of the project in similar soils indicate K_o values as great as 3.0.

Hard Silt

A laminated clayey silt, possibly deposited during a prior glaciation, underlies the hard silt and clay unit. The base of this deposit consists of about 2 ft of silty, gravelly, sandy till. This unit has a siltier texture and higher blow count than the overlying unit (Table 1). Otherwise, its strength properties, degree of fracturing, and general behavior in test excavations are similar to those of the overlying clayey soils. This soil unit was encountered in the lower drifts over most of the length of the proposed tunnel and in the lower one-third of the east access pit.

Lower Dense Sand

The lowermost soil unit encountered in any of the explorations is very dense, silty to clean, fine to medium sand with traces of coarse sand, gravel, and some silt seams. Although no strength values were obtained, the sand possesses some apparent cohesion, as evidenced by its ability to stand unsupported in a 4-ft-diameter test shaft while showing only minor evidence of sloughing or caving. The lower sand was encountered only in the easternmost end of the lowermost tunnel drifts and in the bottom of the east portal pit, as shown on Figure 3.

Landslides

Several landslides have occurred in the immediate vicinity of the tunnel portals, as shown on Figures 3 and 5, within the last 40 yr. These slides have occurred following the excavation for a 10-ft-high retaining wall and the excavation of cuts as deep as 30 ft for the old I-90 tunnel portals. Explorations have also indicated highly disturbed, glacially overconsolidated soils at the east portal; these may have resulted from a major landslide prior to the last glacial advance.

Ground Water

Ground water, normally a problem in tunneling, occurs only as a perched water table in the upper outwash sand that mantles the ridge and in the lower dense sand unit. This perched ground water significantly hindered and delayed construction of the access shaft for the three test adits, but it was of no real consequence during con-

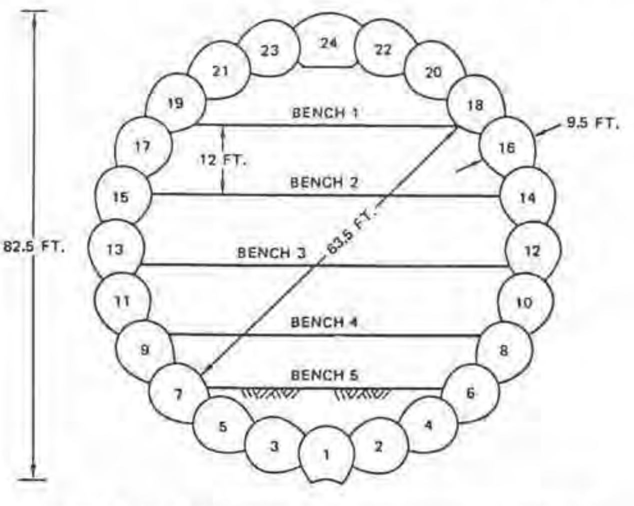

Figure 6. Drift and bench excavation sequence with pairs of drifts excavated from bottom to top followed by bench excavation.

struction of the access pits and tunnel. Ground water in the lower sand unit is more than 40 ft below the lowermost excavation level and is approximately coincident with the Lake Washington water level.

IMPACT OF GEOLOGY ON DESIGN

Early conceptual design efforts had identified several options for this transportation corridor, including a deep braced cut and several side-by-side 20- to 40-ft-diameter tunnels. Design of the Mt. Baker Ridge Tunnel as a single, large-diameter tunnel was mandated by public input as a substitute to severing the north and south ends of the ridge with a deep cut; to reduce the settlement trough width and any resulting damages to utilities and structures; and to minimize the required right-of-way.

The implemented stacked drift tunnel design overcame the extreme difficulties inherent in constructing a single large-diameter tunnel by conventional full-face methods in fractured soils. The design was developed to permit the use of conventional tunneling equipment to excavate a series of small-diameter, more easily supported tunnels; pre-forming a segmented semi-flexible compression ring to be followed by subsequent excavation of the soil core (Figure 6). More details of design and construction are presented in Holloway and Kjerbol (1986).

There are numerous advantages to this stacked drift method. Construction of the individual drifts takes advantage of existing technology and experience for small-diameter utility tunnels. Construction of successive drifts encourages the gradual release of *in-situ* stresses (high K_o) while minimizing the size of individual drift excavations and consequently reducing the impact on soil strengths and minimizing the potential for cave-ins or other adverse ground responses and resulting surface settlements. By pre-forming a lining, the large-diameter soil core is excavated beneath a protective canopy of concrete. The flexibility of the designed liner allows it to adjust to non-uniform external soil pressure, thus minimizing the moment-carrying requirements of the liner.

Soil/liner interaction was evaluated using current state-of-the-art finite element analysis (Hendron et al., 1971). Studies provided a range of design requirements for liner deformations (0.02 to 0.5 percent) and liner thrusts (10 to 1,200 kips/ft) under the effects of 10 to 100 ft of soil cover, varying liner stiffness and soil modulus, and K_o ranging from 0.5 to 2.0. Analyses indicated that the design was capable of sustaining the developed thrusts, eccentric loading, and deformations even for extreme ranges of soil parameters.

Quarter-scale model tests of two drifts and an intervening joint were performed to assess drift joint strengths and deflections for a range of thrusts, eccentric loading, and drift joint configurations. Test results indicated that the joints could sustain eccentricities equivalent to 1 percent distortion of the tunnel diameter under loads considerably higher than anticipated from the analyses.

Simple 1/50-scale model tests on a complete flexible tunnel section were performed to assess the effects of varying shallow soil cover on deformations and ultimate collapse of the tunnel. These model tests demonstrated the stability of the tunnel liner under the equivalent of as little as 10 ft of granular soil cover.

The access pit design mimicked the semi-flexible compression ring design of the tunnel. Due to the unstable nature of the hillsides, and in particular the portal areas, access pits were required that would improve stability while providing easy construction access. As with the tunnels, a portal design which involved incremental construction was desirable, thus minimizing the amount of unsupported or open excavation. Cylinder piles had proven valuable in achieving hillside stability in the past in Seattle (Palladino and Peck, 1972) and were selected for design. The cylinder piles were configured to form a vertical compression ring, to optimally resist slope deformation while permitting the construction of an access pit without initial internal bracing that would otherwise hamper construction efforts. Permanent internal bracing was required once the pits were breached at each end to permit throughgoing traffic to pass; however, the construction of this bracing was sequenced so as to minimally interfere with drift or core excavation.

Following completion of the 24 drifts, the core soils were to be excavated in lifts a maximum of 12 ft high (Figure 6). This was intended to minimize the dangers of high, unstable bench slopes in the fractured, slicken-

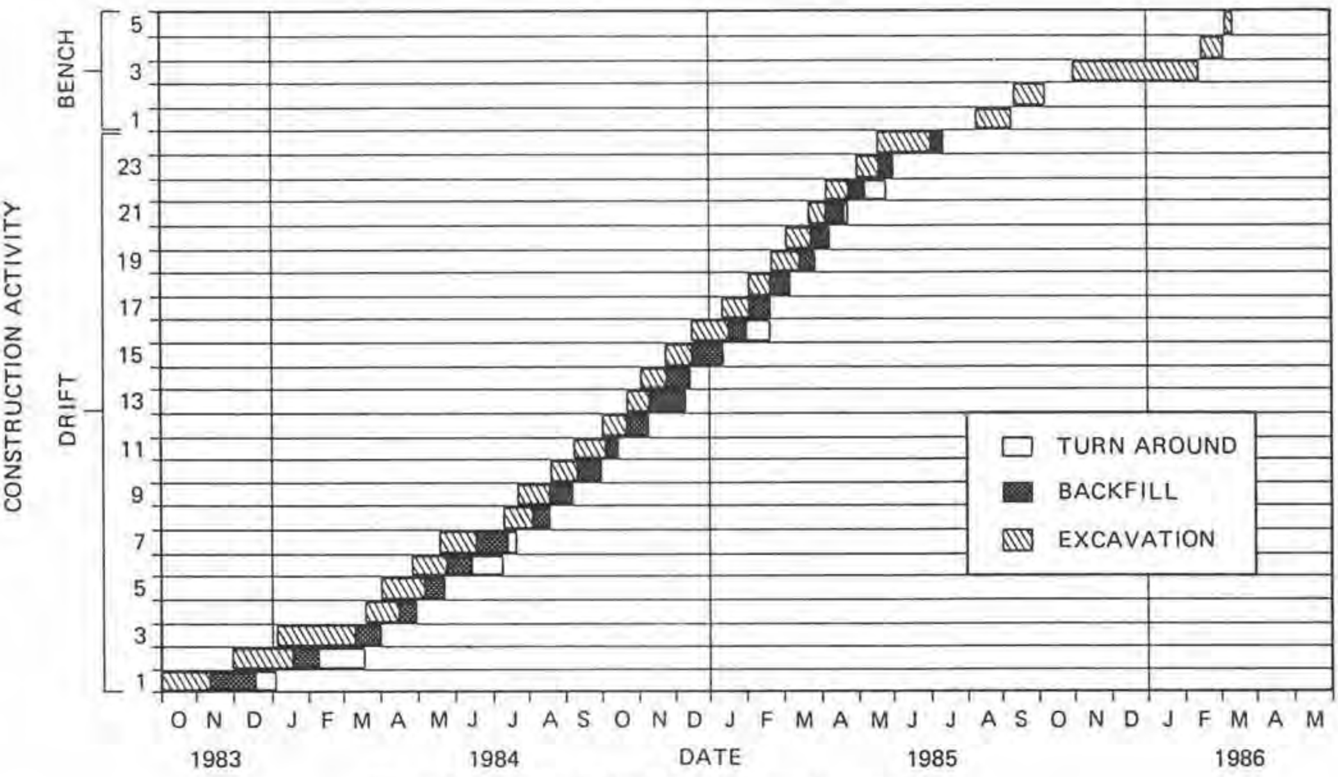

Figure 7. Mt. Baker Ridge Tunnel construction progress.

sided clays and to control the rate of deformation and stress buildup in the liner and surrounding soil.

CONSTRUCTION METHODS AND SEQUENCE

Construction began in December 1982, as shown on the construction progress diagram (Figure 7). The initial work involved the construction of a 100-ft-diameter access pit at each end of the tunnel. The contractor elected to use the secant cylinder pile design provided in the plans rather than to develop an alternative design. The intersecting 8-ft-diameter concrete piles were dug using conventional flight auger rigs and custom-designed chipping tools in the sequence shown on Figure 8. Bored pile work for each access pit took about 6 months, and both pits were complete by August 1983 (Figure 9).

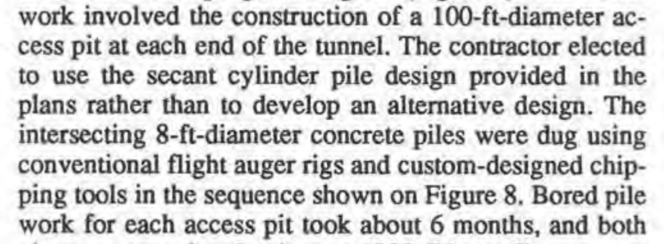

Figure 8. Sequence of access pit wall construction.

The contractor was allowed to select the construction method, shape, and temporary support for the tunnel lining drifts. The specifications required tunnel construction methods that minimized ground deformations and resulting settlements due to individual drift construction, such that soil deformations measured 3 ft above and 3 ft beside each drift were less than 1.0 and 0.5 in., respectively. The restrictions on maximum ground deformations required ground support at the heading, such as a shield and either an expanded or grouted, steel or concrete segmented liner until the concrete backfill was placed. Wood lagging was not generally permitted due to the potential for long-term deterioration and settlement.

The contractor chose to use the minimum specified number of 24 drifts and a 9.5-ft-wide horseshoe shape for his shield. This shield shape was optimal for minimizing the overall thickness of the tunnel liner and maximizing the portion of the tunnel liner circumference that was constructed in any one excavation pass. The contractor purchased two 55-ton Milwaukee Boiler digger-type open-face shields, as shown on Figure 10. Forward thrust was provided by twelve 100-ton shove jacks. The horseshoe-shaped shield had a circular inner shell which could be rotated to keep the operator's station and swing-hoe mechanism level as the angle of the axis of the horseshoe changed for each successive drift. The shields were 17.7 ft long, including a 6-ft-long, 1/2-in.-thick tail skin, making for a 2:1 length to diameter ratio.

Figure 9. Aerial view of completed west access pit; existing tunnels on the right, stabilizing blocks above portal, construction yard to bottom.

Figure 10. Breakout of drift #13 shield in east access pit.

The joints between drifts had a width of 5.5 ft to accommodate calculated maximum liner thrusts on the order of 1,000 kips/ft of tunnel. To maintain liner flexibility, the joints between successive drifts were unreinforced, the only requirement being that all contacts were clean. Drifts were temporarily supported by expanded, precast concrete segments 5 in. thick and 4 ft long with five segments per ring, as shown on Figure 11. Liner rings were seated against the exterior of the prior drift.

Each segment ring was expanded with a pair of portable 20-ton hydraulic jacks, and the expansion was maintained by driving two or three sets of wood wedges in the open joint between segments. After drift #2, the contractor also provided a vertical uplift force of 12 tons at the crown during expansion to reduce sag of the segments and to provide better support to the soil at the crown.

Approximately 38,300 segments were installed in 23 drifts. Each drift required about 330 sets of five segments each. The segments below springline of the main tunnel were reinforced only with polypropylene fibers, whereas segments installed above the springline were reinforced with #4 rebars on a 6-in. grid. Above the springline of the main tunnel, radially oriented 1-ft-long rebar pins were installed to physically attach the segments to the drift mass concrete and prevent the segments from falling and becoming a safety hazard during and after excavation of the core soils.

All drifts were excavated from the west access pit. Initially, the roadheader or boom excavator was used to

Figure 11. Temporary 5-in.-thick concrete segment lining for drift #3 with exterior of prior drift #1 on the left, wood wedges for liner expansion on the right, and grout lines in the crown.

Figure 12. Excavation of bench #3 showing drift exterior and east access pit with upper level bracing.

form individual 10-ft portals through the cylinder piles, but a change order later permitted the use of light explosives. Once started, the drifts were excavated in increments of 4 ft, followed by the erection and expansion of the segmented liner. A laser beam was used for alignment control.

With two shields, the contractor was permitted to excavate two drifts simultaneously—one on the north and one on the south side of the tunnel alignment. Once an entire drift was excavated, part of the crew moved to the other shield which had been transported back from the east access pit, reconfigured by rotating the inner shell, repaired as necessary, and set up in the west pit.

Each drift was backfilled with concrete immediately after it was cleaned out and prior to excavation of the next adjacent drift. The sequencing of drift excavation began at tunnel invert and progressed to tunnel crown (Figure 6). This allowed the visual inspection of concrete quality at the joints between drifts, particularly at tunnel springline.

Concreting was generally accomplished from the east access pit, thus efficiently separating concreting from excavation and support functions. All soil and debris were removed from the drift, and the drift was thoroughly cleaned with air and water. Concrete was trucked from a batch plant near the west portal to the top of the east access pit and fed through a series of hoppers and conveyor belts to a rail-mounted concrete pump located at drift level. The 6-in.-diameter slickline initially extended more than 1,000 ft to near the bulkheaded west end of the drift at the west access pit. As the sloping face of the fresh concrete advanced, the slickline was incrementally retracted. Five to six 8-hr working days were generally necessary to fill each drift with the required 3,200 cy of concrete.

Once the drift had been concreted, it was contact grouted in the crown to ensure complete filling of voids. A 1:1 grout mix was pumped through a series of 1-in.-diameter grout pipes at pressures as great as 150 psi. Grout nipples projected up through the holes in the precast lining at 21-ft spacings. Grout not only filled overbreak voids and cavities, but also was observed to penetrate into open joints or bedding planes a short distance into the surrounding soils. Average grout takes were on the order of two sacks of cement per foot of drift and ranged from about 0.25 sacks/ft in drift #1 to 3 sacks/ft in drift #2.

The crown or "keystone" drift #24 was irregularly shaped and therefore was excavated by hand with pneumatic spaders. Crown arch support was provided by 4W13 steel sections rolled to the proper radius and wood lagging. Hand excavation progressed at an average rate of 37 ft per day.

Drift excavation for the tunnel lining was completed in July 1985, nearly 21 1/2 months after starting. The total length of excavation for the 24 drifts, each about 1,332 ft long, was about 6 mi. Each of the 23 horseshoe-shaped drifts took an average of 19 days to excavate and 6 days to backfill with concrete. However, there was considerable variation in excavation rates as indicated by the working schedule shown on Figure 7.

Excavation of the soil core was started on August 5, 1985. Excavation was accomplished in five benches, each as much as 12 ft high. The top bench was excavated from both ends of the tunnel with front-end loaders. In subsequent lifts, the hard clays were ripped and stockpiled with dozers (Figure 12) and then placed in highway truck/trailer combinations driven through the tunnel from east to west. During peak excavation periods, as many as 20 trucks passed through the tunnel each hour.

Core excavation took about 8 months, including several interruptions to place access pit bracing. Approximately 170,000 cy of soil was removed and hauled as capping material to local landfills.

IMPACTS OF GEOLOGY ON CONSTRUCTION

Excavation and lining of the tunnel was completed 6 months ahead of schedule and within budget. As with

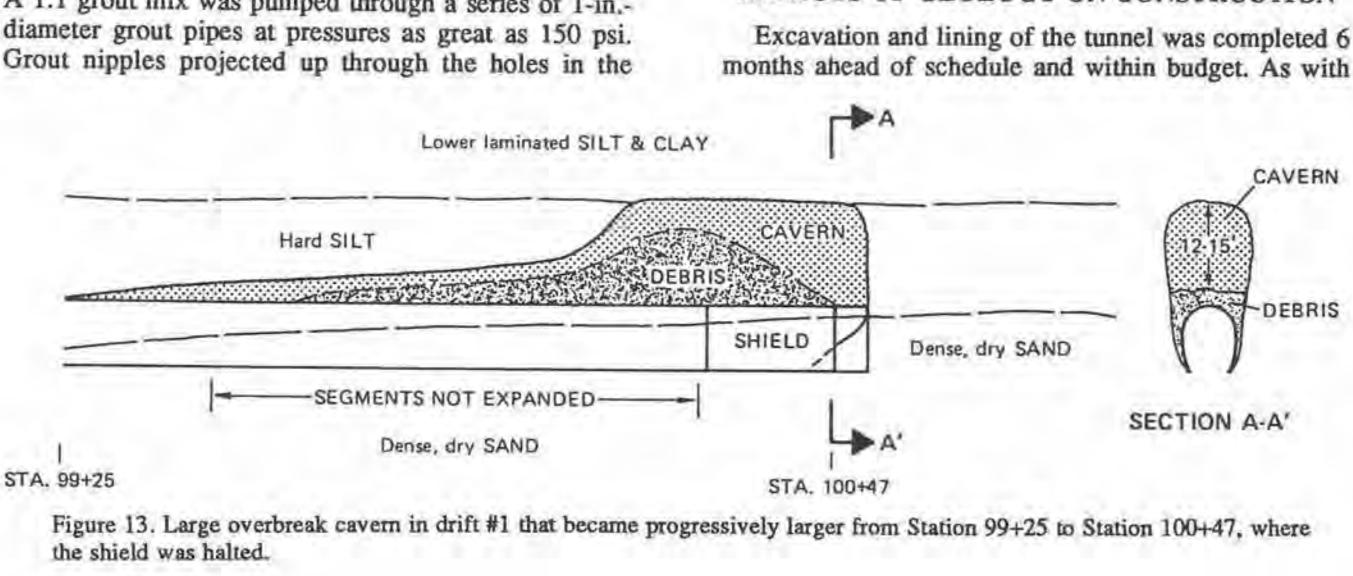

Figure 13. Large overbreak cavern in drift #1 that became progressively larger from Station 99+25 to Station 100+47, where the shield was halted.

most tunneling projects, several problems arose that temporarily delayed or hindered construction. Many of these problems required modifications to construction equipment or procedures. Nearly all of these problems, aside from normal equipment failure, related to geologic conditions that were discussed in the Design Summary Report (HNTB and Shannon & Wilson, Inc., 1982) presented as part of the bid documents.

Drift #1 Overbreak

The most noticeable variation in excavation rates occurred for drifts #1 through #3. Drift #1 experienced several problems related in part to normal startup problems and consequently took 32 working days to excavate. The drift encountered clean, fine to medium sand over its eastern 400 ft, as indicated on Figure 3. The sands were dense but nearly cohesionless and tended to run into the unsupported tunnel heading. Due to the configuration of the shield, the digger was fairly cramped, and therefore the operators generally excavated 3 to 5 ft ahead of the vertical cutting edge of the shield. As the shield encountered the sand layer, whose position rose progressively from tunnel invert to above tunnel crown, sand running from the face initially caused minor crown overbreak. Overbreak gradually extended up to 15 ft above and over the full width of the shield, as shown on Figure 13. The top of the overbreak cavity was coincident with the base of a laminated clay unit.

To correct the overbreak, the void was filled first with 19 cy of pea gravel, followed by 91 cy of grout injected from inside the tunnel. An additional 22 cy of grout was pumped into the void from ground surface through a 6-in. steel pipe.

The contractor successfully completed drift #1 by driving channel spiles ahead of the shield and by partially breasting the face using hydraulic jacks and wood lagging.

For successive drifts, a 30-in. partial hood was welded to the upper half of the shield. This helped to support the crown, gave a slight slope to the face, and also provided more working room for the digger. This hood proved sufficient to handle the sand in adjacent drifts as well as a sand pocket in drift #6. As a result, there were no additional significant overbreaks.

Alignment Control

Drift alignment was considered to be fairly critical, in that the excavation of successive drifts was likely to be accomplished by guiding the shield excavation along the exterior surface of a prior adjacent drift. Significant deviations in alignment could also lead to twisting and cracking of the concreted drifts during soil core removal and consequent loading and deflection of the main tunnel bore. Consequently, the contract plans limited the maximum horizontal and vertical deviations to 3 in. from established alignment. In drift #1, more than 50 percent of the alignment deviated by more than 3 in.; maximum vertical deviation was 8 in. and horizontal deviation was 15 in.

Alignment errors were, in part, related to normal startup problems. However, the large shield length-to-diameter ratio of 2:1, combined with the very hard, unyielding nature of the clays, made the shield cumbersome and difficult to steer. Misalignments generally required about 80 ft of tunnel length to correct, again due to the poor handling characteristics of the shield.

The worst vertical misalignment occurred as the shield encountered the dense sand in the tunnel invert in drift #1, about 400 ft from the east portal. Upon encountering the sand, the shield dove rapidly. More than 80 ft of tunnel was required for correcting the alignment.

Alignment control improved with experience with the operating characteristics of these particular shields. Periodically, shims or wedges were welded to the outside of the shield to force it to roll or move in a desired direction for improved alignment control.

Segment Damage

Poor alignment control adversely impacted the condition of the temporary lining. The lining of drifts #1 through #12 was reinforced only with polypropelene fibers; thus the segments were easily damaged by excess thrusts necessary to steer the shield. None of the liner segment damage was attributed to earth pressures. Rather, all damage was the result of handling, erection, and shield operation loads. Consequently, more than half of these unreinforced segments had throughgoing cracks and spalls, and some disintegrated. Badly damaged liners were held together with strapping or, in a few instances, were patched or replaced with shotcrete. More than 4 percent of the segments required straps or replacements.

For the upper half of the tunnel, the segments were reinforced with wire mesh. Due, in part, to better alignment control and to #4 rebar reinforcement, only 5 percent of these segments experienced throughgoing cracks due to liner erection or shield thrust. None of these liner segments was so damaged as to require strapping or replacement.

Boulders

The occurrence of scattered cobbles and boulders, some as large as 10 ft in the longest dimension, was anticipated from interpretations of the geologic history of the site and corroborated by a photograph of the old existing tunnel portal excavation, which shows a 6-ft-diameter boulder. The 48 borings encountered only 1 obstacle interpreted to be a boulder, and the two test shafts and three test adits encountered scattered boulders to about 2 ft in the largest dimension. Thus, the explorations were unable to document the occurrence of boulders more than 2 ft in diameter, but an assessment of the mode of soil deposition, that is, the transport of coarser materials by ice rafting, indicated a strong potential for widely scattered large-diameter boulders. This consideration, in part, influenced the contractor in his selection of an open-faced shield excavator, rather than a closed face wheel excavator, which might have been more efficient had the only tunneling medium been homogeneous hard clay.

In the course of excavating the 24 drifts, the contractor encountered 12 boulders with maximum dimensions of more than 3 ft; these had to be broken up by hydraulic splitters to facilitate their removal from the drift headings. Breaking up and removal of these boulders commonly required one or more shifts, during which tunnel advance was halted. In many instances, portions of the same boulder, several of which were estimated to exceed 6 ft in one dimension, were encountered in two adjacent drifts. Several boulders in excess of 6 ft long were encountered during soil core removal but were easily removed with heavy earth-moving equipment.

Ground Movements and Surface Settlement

To limit surface settlements and damage to residences and utilities and also to promote optimum tunnel liner/soil interaction, the specifications required that the contractor utilize construction techniques that limited ground deformation to 1 in. at 3 ft above, and 1/2 in. at 3 ft to either side of the advancing shield. This was one of the first tunnel projects to use ground deformations adjacent to the tunnel, rather than the resulting settlement of the ground surface, as a performance criterion. A fairly extensive instrumentation program was installed to monitor these deformations and to provide data for WSDOT and the contractor as a basis for modifying construction procedures (Robinson et al., 1987).

Deformations over drift #1, excavated in undisturbed ground, were generally less than the specified limit. Measured deformations over drift #2, excavated through ground already distressed by drift #1, exceeded 3 in. over the western half of the drift. This zone of excessive deformation in drift #2 was coincident with portions of drift #1 that had very low grout takes using a 1-1/2-in. PVC pipe. The eastern half of drift #1 had much higher grout takes using a 1-in. steel pipe. Grouting techniques were improved in drift #2 and all successive drifts by using 1-in. steel pipes, grout nipples at 21-ft spacings, grout pressures of as much as 150 psi, and four separate grout lines to service 300-ft lengths of each drift. As discussed elsewhere, the liner expansion procedures were also improved, particularly by lifting of the crown segments. Consequently, in drifts #3 through #23, ground deformations were generally less than the specified tolerance.

Ground deformations are a measure of ground or volume losses caused by soil movement into the face or around the barrel of the advancing shield and into voids behind the shield that have not been adequately filled by liner expansion or grouting. For hard clays, ground losses for individual drifts, using relatively good construction techniques, were projected to be on the order of 1 to 3 percent of the face volume and averaging about 2 percent. With only minor bulking anticipated for the hard clays, the volume of the surface settlement trough was expected to be commensurate with the average losses around individual drifts. Actual volume losses for individual drifts were determined by measuring ground displacements at several stations from 1 to 10 ft above each drift using borehole instruments (Robinson et al., 1987). Volume losses averaged about 1.9 percent for the 24 drifts and ranged from about 0.5 to 9.0 percent of the drift volumes. The greatest losses were associated with the large overbreak over drift #1, the excessive displacements associated with drift #2 and large displacements over the hand-mined rib and lagging supports in drift #24.

The surface settlement trough volumes ranged from 1.7 to 2.6 percent of the total drift volumes. The close correspondence between drift volume losses and surface settlement trough volumes indicates that within the range of accuracy of the measurements, very little bulking occurred in the hard clays. Volume losses taken as a function of the 82.5-ft-diameter bore volume averaged only 0.7 percent, indicating considerable improvement in surface settlement trough volumes relative to the 1 to 3 percent of total bore volume that would likely have occurred with full face tunneling.

Total cumulative surface settlements resulting from individual drift excavations and volume losses averaged about 2.5 in. (Figure 14). Maximum settlements of about 8 in. occurred near the west portal, near an old landslide caused by the 1938 excavation of portals for the two existing tunnels. Settlements were roughly symmetric about tunnel centerline and slightly larger above the old tunnels, where soils had been previously disturbed (Figure 15). Even the largest settlements were considerably less than the maximum 12 in. envisioned during design and resulted in only very minor damage to sidewalks and no apparent damage to utilities and buildings.

CONCLUSIONS

Mt. Baker Ridge Tunnel, the world's largest diameter soft-ground tunnel, was completed using conventionally sized tunneling equipment while significantly reducing surface settlements relative to those that might have accompanied full-face tunnels. The tunnel was constructed at a cost lower than the bid price due to the small number of claims, lower than estimated escalation of material and labor prices, and the innovative risk-sharing contracting practices adopted by WSDOT.

As with most tunneling projects, geotechnical conditions played a major role in the development of the design and the selection of appropriate construction

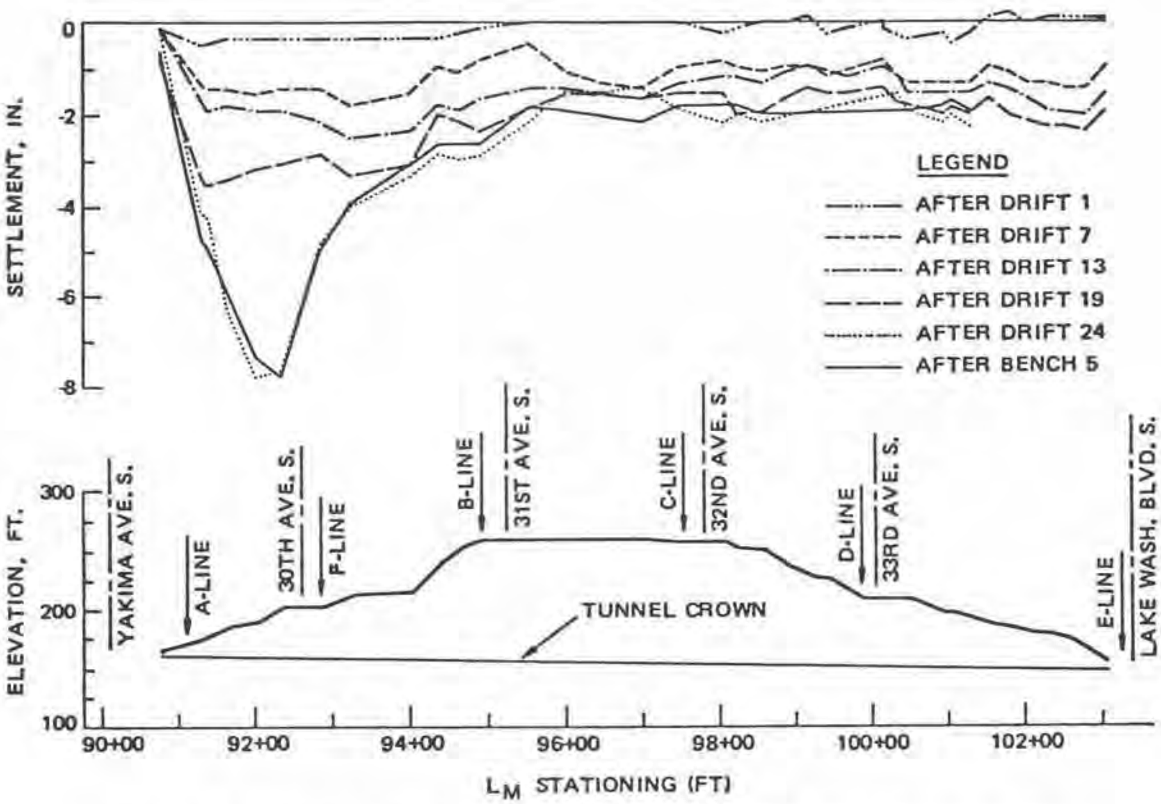

Figure 14. Plot of longitudinal surface settlements along the Mt. Baker Ridge Tunnel centerline.

methods for this innovative project. Even with well documented geotechnical conditions presented in a Design Summary Report, the contractor encountered some conditions for which he was incompletely prepared. However, through close coordination among the contractor and the design team of WSDOT, HNTB, Shannon & Wilson, Inc., and the Design Review Board, all problems were quickly resolved.

This project was selected as the 1987 winner of the National Grand Conceptor Award for Engineering Excellence by the American Consulting Engineers Council. The Mt. Baker Ridge Tunnel represents a culmination of state-of-the-art geotechnical engineering in its application to tunneling. Design and construction techniques developed on this project should prove useful for future large-diameter tunnels.

ACKNOWLEDGMENTS

Everyone involved with this project can be proud of their contributions to the successful design and construction of a state-of-the-art highway tunnel. In par-

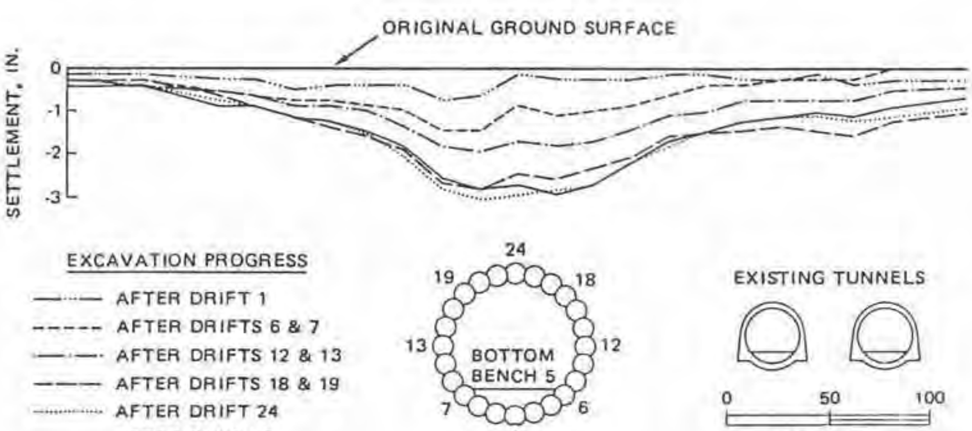

Figure 15. Plot of lateral surface settlement trough at the middle of Mt. Baker Ridge.

ticular I would like to acknowledge the contributions of R. P. Miller, H. W. Parker, and M. S. Kucker of Shannon & Wilson, Inc.; L. J. Holloway and G. Kjerbol, project manager and project engineer, respectively, of Howard Needles Tammen and Bergendoff; and O. T. Harding, project engineer for the Washington State Department of Transportation.

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Geology and Design of the Seattle Access Portion of Interstate Highway 90

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INTRODUCTION

Design and construction of the Seattle Access portion of Interstate Highway 90 (I-90) represents the culmination of 20 yr of work to bring the I-90 highway system into the city of Seattle. The section of alignment discussed in this paper starts just east of 12th Avenue South, thence passes west to 4th Avenue South where it turns north to join the existing arterials. The location of the Seattle Access project area is shown on Figure 1. The overall project involved connecting Interstate Highway 5 (I-5) and I-90 and extending I-90 westward over I-5 to downtown Seattle. The completed highway, scheduled to be finished in 1994, will include general purpose lanes, transit lanes, and high-occupancy vehicle (HOV) lanes. Much of the highway will consist of elevated structures. The general purpose and HOV lanes will descend from the elevated structures and connect to existing arterials by means of touchdown embankments. Transit lanes will descend and enter the new Metro bus tunnels.

The size and complexity of the project provided a unique opportunity to study in detail the late Pleistocene sediments along the highway alignment and add to the knowledge of the subsurface conditions for part of a major metropolitan area. This paper presents our interpretation of the sediments encountered in borings accomplished along the project alignment and discusses how these interpretations and the physical properties of the sediments affected final design of the foundation system. Field work and design efforts upon which this paper is based were completed by Hart Crowser, Inc., between December 1984 and September 1986. This paper is a compilation of findings from work accomplished for the Washington Department of Transportation.

GEOLOGY

Regional Geology

The Seattle Access project area is in the central portion of the Puget Sound lowland, a major north-south-trending structural low west of the Cascade Range. Pre-Quaternary Puget Sound lowland basement is composed of Tertiary sedimentary and volcanic rocks interpreted to consist of a mosaic of rectilinear fault blocks exhibiting various amounts of relative displacement (Danes et al., 1965; Rogers, 1970; Gower et al., 1985). A major geophysical lineament near the project area is inferred to separate shallow bedrock to the south from thick Quaternary sediments to the north beneath Seattle (Figure 2). This lineament may represent a major fault which could have a significant impact on project design.

Advances and retreats of continental glaciers, which deposited as much as 3,000 ft of sediment, dominated the history of the lowland for the past 1.6 m.y. The best studied and preserved of these sediments are those deposited by the latest glaciation (Vashon Stade of the Fraser Glaciation; Armstrong et al., 1965), which inundated the Seattle area with as much as 3,200 ft of ice 15,000 to 13,000 yr ago (Thorson, 1981). The Vashon glacier's southernmost advance was approximately 80 mi south of Seattle.

Seismicity

The Puget Sound trough, particularly the southern portion, has been identified as an area of frequent seismicity. Most of the larger events (magnitude ≥4) occur at depth and are associated with the interaction of three major crustal plates (Atwater, 1970; Davis et al., 1984). Algermissen and Perkins (1976) have suggested that a high level of seismic risk from subcrustal earthquakes can be expected in this region. The most recent large event which occurred in the area was the 1965 Richter-magnitude 6.5 earthquake whose epicenter was located approximately 10 mi south of the Seattle Access project area. The most severe damage resulting from that earthquake occurred on Harbor Island, in the former Duwamish embayment, approximately 1-3/4 mi southwest of the Seattle Access project.

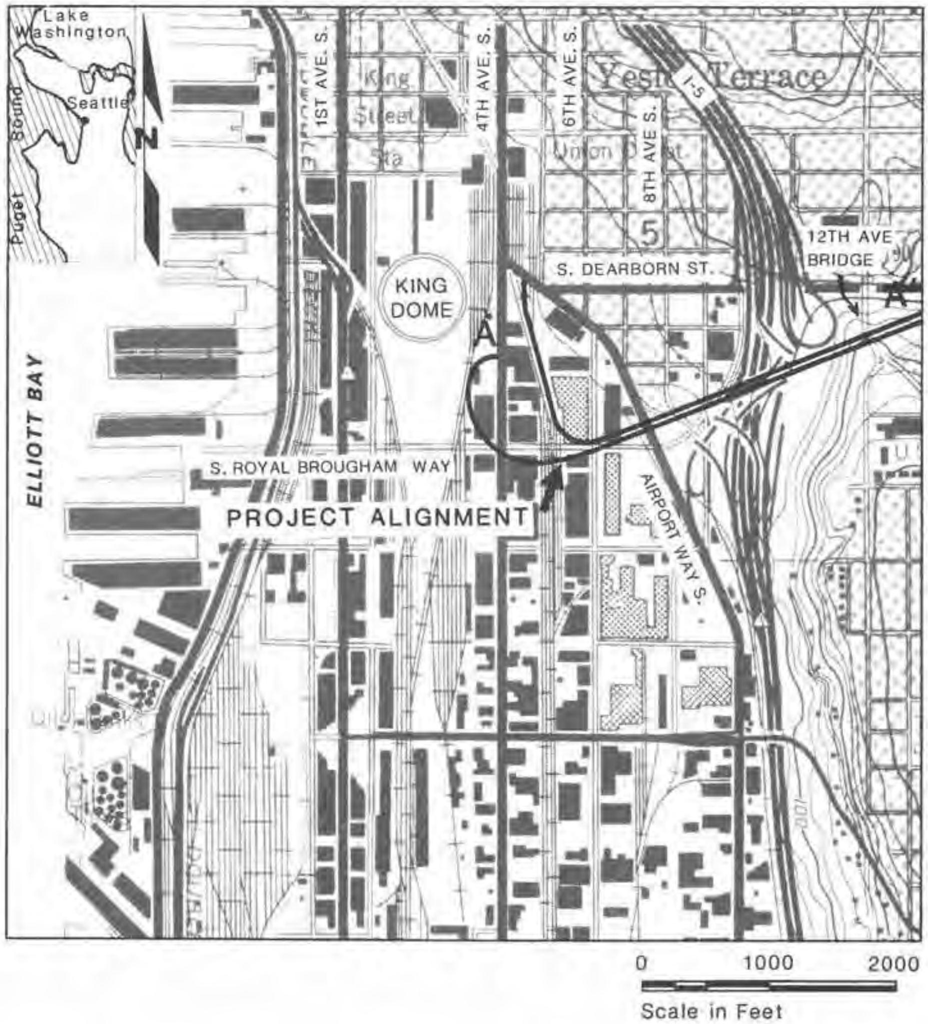

Figure 1. Location of project alignment for the Seattle Access portion of Interstate Highway 90.

For design, a Maximum Credible Earthquake (MCE) of magnitude 7.5 was assigned to the seismotectonic zone containing the project site. The major geophysical lineament near the site (Figure 2) was modeled as a fault and assigned an MCE of magnitude 6.5.

Geologic Setting

Advance of Vashon ice from Canada into the Puget lowland blocked or diverted west-flowing drainages of the Cascade Range, isolated Puget Sound from the Pacific Ocean, and formed numerous proglacial lakes. Fine-grained lacustrine sediments (Lawton Clay Member of the Vashon Drift) were deposited in proglacial lakes over a large area (Mullineaux et al., 1965; Waitt and Thorson, 1983). As the ice advanced, the lakes were filled with advance outwash sand (Esperance Sand Member of the Vashon Drift), which covered the lacustrine sediments. Lodgement till was deposited as the ice overrode and overconsolidated the advance outwash sand and lacustrine sediments. During recession of the ice, drainages were reestablished, and alluvial and lacustrine deposition ensued. Relative sea-level rise and resultant marine transgression into the Puget Sound lowland contributed glacial marine sedimentation (Domack, 1983) and created features such as the Duwamish embayment.

For the past 70 yr, the tideflats of the Duwamish embayment have been undergoing urban and industrial development. Tideflat reclamation began in the late 1800s and was completed by 1916. Material used to fill the tideflats came from the regrading of the surrounding uplands. Typically the uplands were regraded by hydraulic methods, and the spoils sluiced to the bay and used to fill the tideflats.

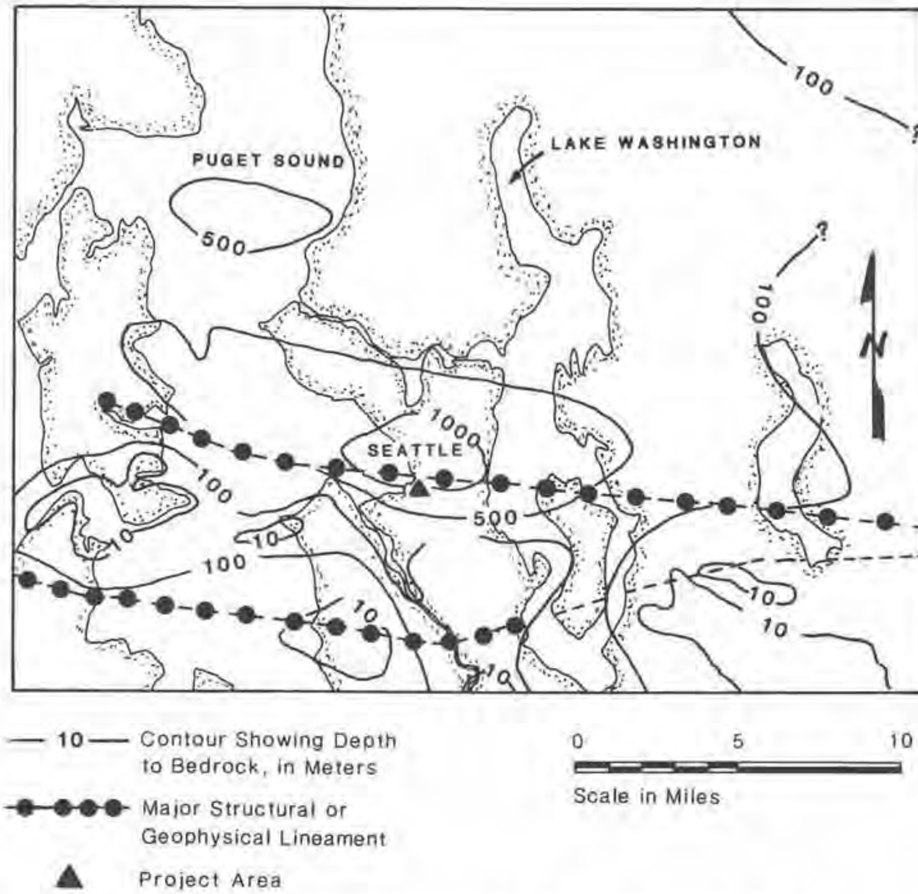

Figure 2. Major geophysical lineaments and depth to bedrock in the Seattle area.

Geology of the Project Area

Most of the project area is within the former Duwamish embayment (Figure 3). The easternmost segment of the project area descends from an upland area (Beacon Hill) underlain by pre-Vashon silt and clay, which bounds the former embayment to the east. The former eastern shoreline of the embayment is located just east of 8th Avenue South (Figure 1). Overlying marine sediments and fill deposited in the topographic low of the embayment have preserved a late Pleistocene sequence similar to that typically observed in the Seattle area (Figure 4).

A geologic cross-section of the project alignment is shown in Figure 5. The stratigraphic units shown are based upon the texture, structure, and physical properties of samples obtained from subsurface explorations and past experience with correlative sediments in the Puget Sound area. We have divided the sediments of the project area into nine units, which are discussed in detail below.

Fill

Fill encountered along the project alignment typically consists of a surficial sandy fill overlying clayey fill. The surficial fill consists of a slightly gravelly to gravelly sand and contains scattered debris from past industrial activity. This fill is thickest (20 ft) at the foot of Beacon Hill near I-5. The surficial fill postdates filling of the Duwamish embayment and was placed over the past 70 yr as this area was developed.

The underlying clayey fill consists of a heterogeneous mixture of clay, clayey silt, and silty sand with scattered pockets or lenses of sand. The texture and appearance of this unit suggests it has been hydraulically placed. Historical records indicate that, near the turn of the century, regrading of the surrounding uplands resulted in deposition of spoils in this area to reclaim the existing tideflats. This fill thins to the east toward 8th Avenue South and contains scattered rubble, wood chips, and coal fragments indicative of past industrial use of the area.

Figure 3. Major physiographic features in the Seattle area.

Recent Marine Silt/Sand

The uppermost natural sediments encountered along the alignment consist of massive, soft clayey silt to silty clay which overlie a basal gravelly sand. The silt contains shell fragments and scattered lenses of silty sand. The silt is thickest (to 30 ft) to the west of 5th Avenue South toward the center of the former embayment.

The basal gravelly sand contains little silt and abundant shell fragments. This sand thickens to the east toward the former shoreline at the base of Beacon Hill.

These deposits are interpreted to represent a fining-upward, transgressive marine sequence, reflecting postglacial marine invasion which resulted in formation of the Duwamish embayment.

Peat

A 1- to 2-ft-thick bed of compressed peat was encountered along 4th Avenue South at the west end of the alignment. Two of the five samples of peat recovered contained a 2- to 3-in.-thick layer of white to gray, silt-sized tephra. Based upon air-fall distribution (Clague, 1981), physical characteristics, and stratigraphic context (Powers and Wilcox, 1964), we interpret this to be Mazama tephra (6,600 yr B.P.). Accumulation and preservation of the peat between the Recent marine and Vashon recessional sediments indicate a time lag between retreat of the glacial ice and marine incursion into the embayment at this location.

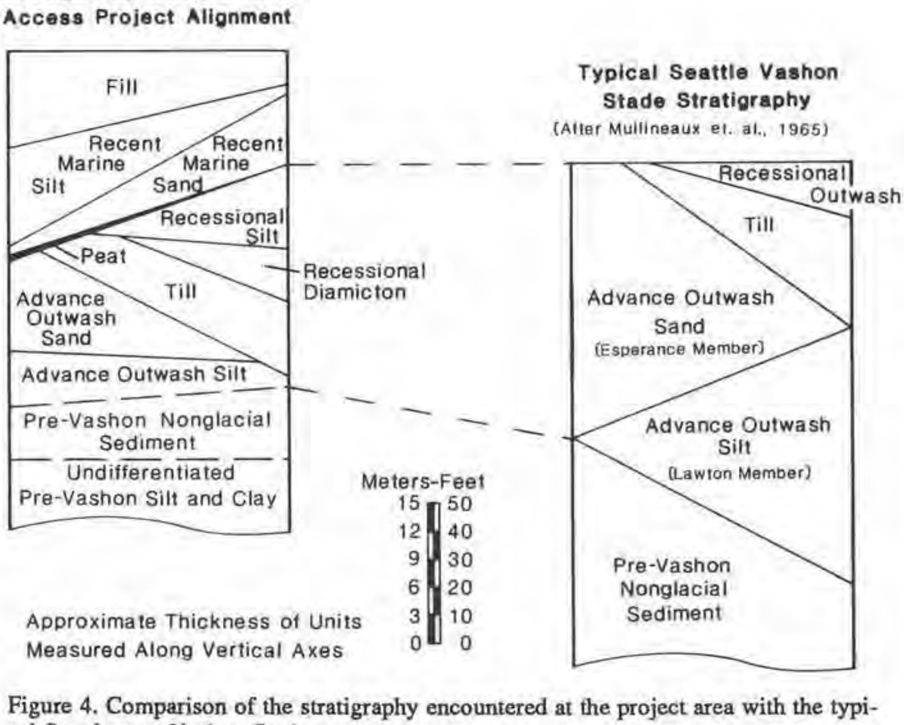

Figure 4. Comparison of the stratigraphy encountered at the project area with the typical Seattle-area Vashon Stade sequence.

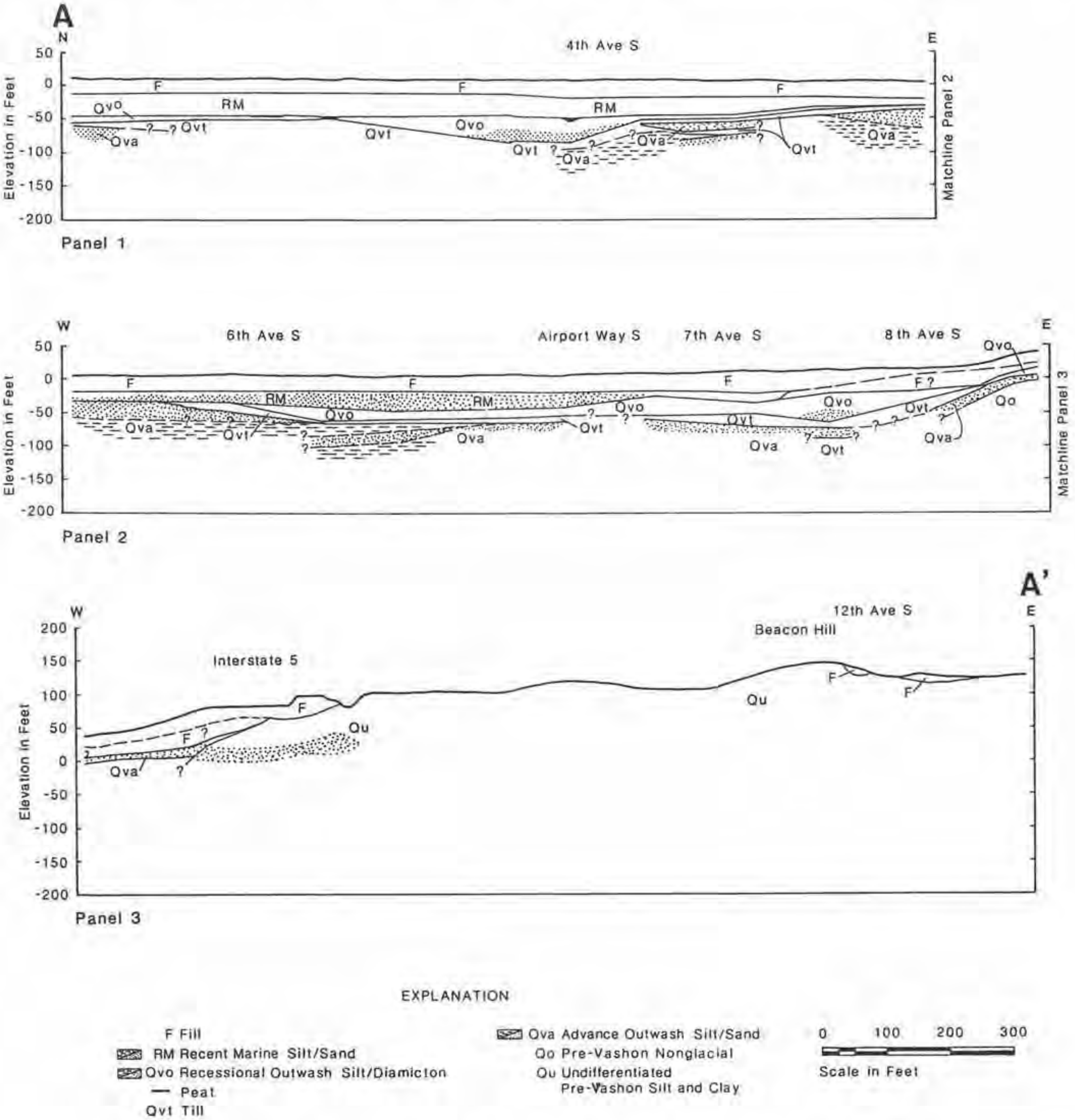

Figure 5. Geologic cross-section along the Seattle Access portion of Interstate Highway 90.

Recessional Outwash

Vashon recessional deposits encountered along the alignment consist of silt and diamicton. The silt typically consists of medium stiff to hard, wet, gray to green-gray, clayey silt to fine sandy silt with local lenses of sand. The fine texture suggests a low-energy depositional environment of either glaciolacustrine or glacial-marine origin, although no marine fossils were found. Some of the stiffer recessional silts exhibit joints, inclined bedding, and slickensides similar to those in the older, overridden silts described below. The origin of these features in a recessional environment is uncertain.

We postulate the contorted bedding and slickensides observed within portions of the recessional silt may be related to ground shaking or mass movement.

The diamicton consists of loose to dense, wet, gray, unsorted silty, gravelly sand and looks very similar to lodgement till, but it typically has a much lower density. The diamicton locally interfingers with the silt but more commonly occurs beneath the silt (Figure 4) in what may have been former topographic lows. We interpret the diamicton to have been deposited by sediment gravity flows close to the receding ice front.

Till

This unit consists of dense to very dense, wet, gray, poorly sorted, silty, gravelly sand that locally varies to gravelly, sandy silt. In places, this unit includes discontinuous beds of sand. The unit is similar in appearance to Vashon till observed regionally in outcrop and borings; that till is unweathered, unsorted, and very compact and exhibits clay skins around gravel. We interpret this unit to be Vashon Stade lodgement till, deposited directly beneath glacial ice. Till is absent towards 6th Avenue South (Figure 5), having apparently been eroded from a topographic high during glacial retreat.

Advance Outwash

Well graded sand and silt encountered beneath the lodgement till consist of coarse-grained, high-energy fluvial deposits and fine-grained lacustrine sediments. The fluvial sand generally consists of dense to very dense, wet, gray, clean to silty, gravelly sand grading locally to sandy gravel. Lacustrine sediment typically consists of hard, wet, gray, massive to laminated, thinly bedded rhythmites of clayey silt to silty clay.

The typical Vashon advance sequence (Figure 4) exhibits a uniformly graded sand (Esperance Sand Member) over laminated silt and clay (Lawton Clay Member). In places at the project area, the fluvial sand is missing and the sand and silt appear to interfinger. A complete understanding of the advance outwash stratigraphy at the project area was made difficult because few borings completed along the alignment penetrated these sediments.

Pre-Vashon Nonglacial Sediments

Sediments of a nonglacial interval were encountered in a few borings east of 8th Avenue South at the base of Beacon Hill. The sediments typically consist of very dense, brown sand and gravel with scattered interbeds of hard, brown laminated silt. These sediments are interpreted to be nonglacial because they are oxidized and contain scattered wood fragments. A decomposed log was found in these sediments near the contact with the overlying advance outwash at about 50 ft below the ground surface.

Undifferentiated Pre-Vashon Silt and Clay

The uplands east of I-5 at the north end of Beacon Hill are underlain by silt and clay of pre-Vashon age (Mullineaux, 1967). The sediments consist of hard, gray, varved to massive, clayey silt and clay with scattered silty sand and well-sorted silt. Numerous joints and slickensides within the silt and clay are attributed to adjustment of these sediments to regional stress relief and isostatic rebound of the Earth's crust upon retreat of the ice sheet.

Lacustrine and marine sections have been identified within the pre-Vashon silt and clay beneath Beacon Hill (Mullineaux, 1967). We interpret these sediments to represent a low-energy depositional environment which existed prior to the Vashon Stade glaciation.

ENGINEERING GEOLOGY

Introduction

The geologic interpretation presented above was the first step toward understanding the subsurface conditions beneath the project alignment. It was important to correctly establish the stratigraphy and contacts between units because the depositional environment and geologic history of the sediments would largely determine their engineering properties. Preliminary geotechnical investigations accomplished for the project, which did not consider the geology of the sediments, oversimplified the stratigraphy. For example, these preliminary studies grouped much of the hard recessional silt with the pre-Vashon silt and clay. The difference between the physical properties of these sediments became apparent upon completion of laboratory testing. The results of the consolidation tests shown in Figure 6 show the pre-Vashon silt and clay to be stronger and less compressible than the recessional silt.

The stratigraphy presented above became the framework for understanding the subsurface conditions and physical properties of the sediments. This understanding was applied to the design of the project's foundation system. The following section discusses our exploration methods, ground water, and the design concerns relating to the physical properties of the sediments.

Methods of Investigation

Hollow-stem auger drills and a quasi-static cone penetrometer (CPT) were used to investigate the subsurface conditions at the Seattle Access project site. A total of 63 auger borings were advanced to depths ranging from 48 to 114 ft below the ground surface. At least one boring was accomplished at the approximate location of each pile or pile group planned to support the elevated structures. Representative sediment samples were obtained using a 2-1/2-in.-O.D. split barrel (SPT) sampler for sands and gravels and a 3-in.-O.D. thin-walled steel

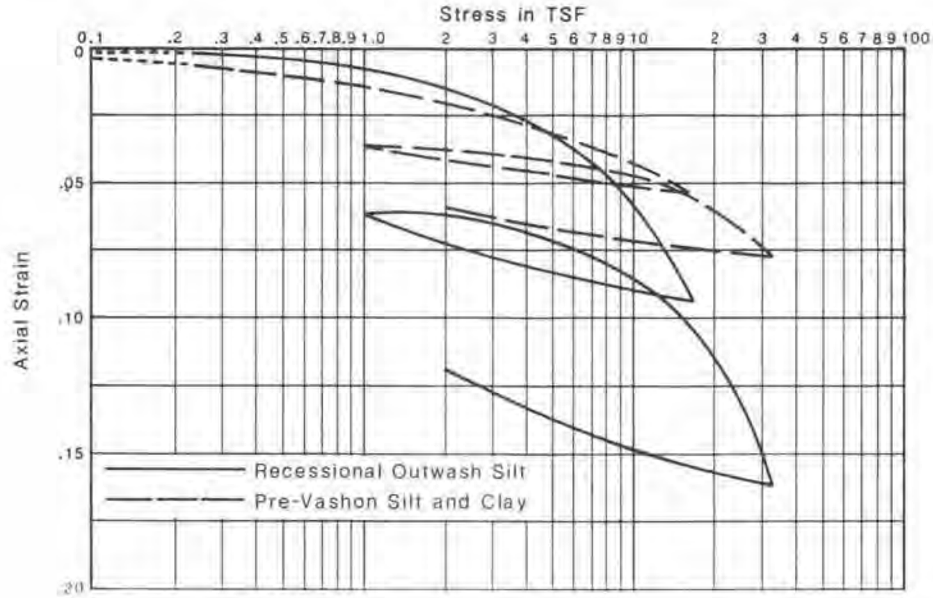

Figure 6. Comparison of consolidation test results for the recessional silt and pre-Vashon silt and clay.

tube sampler for silts and clays. Piezometers and ground-water observation wells were installed in many of the auger borings.

A cone penetrometer was used to advance 13 probes to depths ranging from 24 to 72 ft below the ground surface. Cone penetration resistance and friction data were used to classify sediments comprising the upper, normally consolidated portions of the stratigraphic section.

Ground Water

In the shallow wells west of Beacon Hill, ground-water levels typically ranged from 2 to 8 ft below the existing ground surface. The depth to near-surface ground water increases to the east along the project alignment. The depth to and occurrence of ground water within the fill varies considerably due to its inhomogeneity.

A deep, confined or partially confined ground-water table exists in the advance outwash. Borings which penetrated the till into the advance outwash sand encountered extreme heaving conditions, and piezometers installed at these depths recorded heads of as much as 80 ft.

Ground water associated with the undifferentiated silt and clay on Beacon Hill typically is perched within isolated sandy lenses at depth or in overlying, weathered soils and fill.

Design Concerns

Each stratigraphic unit contains sediments which exhibit unique physical properties and present different design concerns. Table 1 summarizes physical properties of the sediments and the resulting design concerns for each stratigraphic unit.

The near-surface fill and recent marine units are low strength, typically saturated sediments and have a combined average thickness of approximately 50 ft over much of the alignment. The fine-grained portions of these units are prone to settlement under applied loads, and the granular portions are susceptible to liquefaction during the design earthquake. Little lateral pile support is expected from these sediments.

The recessional outwash sediments are silty and highly varied in strength and thickness. These sediments contribute lateral pile support, uplift resistance, and possibly shallow pile support. However, the variation in physical properties throughout this unit will make prediction of the foundation behavior difficult.

The till and advance outwash units are high-strength sediments that provide deep foundation support for the elevated structures. High ground-water pressures encountered within the advance outwash sand influenced the type of vertical support design at certain locations.

The pre-Vashon silt and clay unit consists of high-strength sediments, which locally exhibit numerous joints and slickensides. Some deep cuts have experienced instability, part of which may relate to movement along the discontinuities.

DESIGN IMPLICATIONS

Two primary sets of engineering criteria were considered for design of the elevated structures, retaining

Table 1. Summary of physical properties and design concerns for the stratigraphic units PI, plasticity index; SU, undrained shear strength; WC, water content; LL, liquid limit

Stratigraphic unit	Thickness (ft)	SPT (blows/ft)	Physical Properties Moisture	PI	SU (psf)	Design issues
Fill						
Sand	4-34	2-20	WC > LL	1-25		Lateral pile support, liquefaction, settlement
Clay	12-25	2-15			200-600	
Recent marine						
Silt	3-36	2-9	WC = LL	15	380-850	Lateral pile support, settlement, liquefaction
Sand	3-24	2-56				
Recessional outwash						
Peat	1-2	9-20	WC LL	5-38		Lateral pile support, end bearing, pile uplift resistance, shallow pile support, differential settlement
Silt	0-30	11-50			600-4100	
Diamicton	0-20	10-70				
Till	0-26	50				Deep foundation support
Advance outwash						
Sand	4-32	30	WC < LL	14-42		High ground-water pressure
Silt	3-20	30			4500-5500	
Pre-Vashon deposits						
Silt and clay	50	15-50	WC < LL	4-34		Cut-slope stability

walls, and embankments associated with this project: (1) service loads which include structure dead load, vehicle live load, wind load, temperature-induced stresses, and stresses due to long-term creep of the post-tensioned structure, and (2) forces imposed during a design earthquake. For the service loads, structure and foundation elements are designed so stresses remain in the elastic range. For seismic design, plastic deformation of the superstructure is allowed as long as the structure remains stable when subjected to the design earthquake.

With these engineering criteria established and the characterization of the project area geology and subsurface conditions completed, design of the foundation, embankments and retaining walls considered the following conditions:

(1) The thick section of shallow, soft, and loose fill and recent marine sediments provide little lateral pile support and are highly compressible.

(2) Pockets of sand within the fill and the basal sand of the recent marine sediments existing below the water table could liquefy during the design earthquake.

(3) Due to the varied consistency of the recessional outwash silt, it might not be possible to penetrate this unit with a driven pile, and piles or piers founded in these sediments may experience differential settlement with respect to foundations founded in glacially overridden sediments.

(4) Due to the high density of the overridden sediments, it might be difficult to achieve the required pile embedment (10-15 ft) to provide sufficient uplift resistance, and the use of large drive-hammers might increase driving vibrations to unacceptable levels.

(5) High ground-water pressures in the advance outwash sand will make construction of drilled shafts difficult due to heaving conditions.

(6) Slope instability could be induced by a design earthquake in large cuts constructed in the undifferentiated pre-Vashon silt and clay.

Liquefaction

Due to the complex nature of the site stratigraphy, liquefaction analysis was performed for every SPT sample collected along the project alignment. On the basis of this analysis, stratigraphic units prone to liquefaction during the design earthquake were identified. The sediments most susceptible to liquefaction are sandy lenses within the fill and the basal sand of the recent marine deposits.

It is not feasible to reduce the effects of liquefaction with *in-situ* ground modification methods because of the scattered and varied nature of sand lenses within the fill and the large portion of the alignment underlain by the recent marine sand. To account for the potential loss of lateral pile support due to liquefaction expected during the design earthquake, the foundation elements and superstructure are designed assuming the liquefiable sediments have no strength. Some areas of wet, sandy fill below earth embankments will be overexcavated approximately 7 ft and replaced with engineered structural fill in an attempt to reduce the possible effects of liquefaction on these structures.

Embankments

Three touch-down embankments are planned in the area west of 6th Avenue South. The embankments range in maximum height from 10 to 17 ft. Owing to the soft consistency, high water content, and limited bearing capacity offered by the fill and recent marine silt (Table 1), the use of conventional cantilever retaining walls bearing on spread footings is not feasible. The chosen design uses structural earth walls (that is, reinforced earth) to support side walls of the touch-down embankments. This system exerts lower ground pressure than spread footings.

To reduce long-term settlement (estimated to be 1 1/2 to 2 ft), the area of the embankments will be preloaded. Preloading will accomplish 90 to 95 percent of the consolidation settlement within the fill and recent marine silt prior to final construction. Construction of the preloads will be staged to reduce concerns regarding the stability of embankment and preload fills. The first stage will be placed to a height of approximately 10 ft. A 3-month waiting period will ensue, during which the fill and recent marine silt will consolidate and increase in strength. The second stage will raise the fills to their design height.

Foundation

During preliminary engineering, the design team decided to design a "flexible" structural system to absorb earthquake-induced loads. The foundation system had to remain relatively rigid during service loads, but retain the ability to displace a few inches under dynamic conditions.

Spread footings will be used for a limited number of piers located on the east side of I-5. Footings at these locations will bear upon the overridden, undifferentiated pre-Vashon silt and clay.

To the west of I-5, the soft to loose fill and recent marine silt make spread footings impractical. For this portion of the alignment, a deep foundation system was designed. Critical design loads for the deep foundation system are the lateral loads induced during an earthquake. The desire for a flexible system eliminated battered piles from consideration because they would not deflect sufficiently during loading. The deep foundation system will consist entirely of vertical shafts or piles capable of as much as several inches of deflection upon lateral loading. The design of the deep foundation system had to consider the largest diameter shafts/piles possible for maximum lateral capacity, the ease of construction, and the ability to drive or construct the foundation into very dense, glacially consolidated sediments. Due to the complex geology along the alignment, three different vertical supports were designed for the deep foundation system: closed-end driven piles, drilled shafts, and composite drilled-driven piles.

Large-diameter (24-in.), closed-end pipe piles will be used at the east end of the alignment, near I-5 where the fill thins and recent marine silt is absent. The pipe piles will be driven into the Vashon till and advance outwash sediments, which will provide support for the foundation. Closed-end pipe piles were chosen over drilled shafts because they require no casing or slurry and their construction would not be hindered by high ground-water pressures expected in the advance outwash sand.

Drilled shafts were selected for foundation support only where foundation construction occurs adjacent to existing buildings. At these locations, concern over damage resulting from vibration during pile driving outweighed construction difficulties such as casing the excavation through the fill and recent marine sediments and high ground-water pressures expected in the advance outwash sand.

The majority of the elevated structures will be supported by a composite pile consisting of a 24-in.-diameter closed-end pipe driven inside a 42-in.-diameter steel casing. The casing will be driven to the top of the overridden sediments. The closed-end pipe pile will be driven into the till or advance outwash sediments to provide uplift and compressive capacity, and the large-diameter casing will be filled with reinforced concrete to provide lateral capacity in the weak, near-surface fill and recent marine sediments.

Retaining Walls

East of I-5, on-grade construction of the highway will require large cuts into the undifferentiated pre-Vashon silt and clay. Previously constructed, temporary excavations into these sediments resulted in slope instability and mass movement along existing joints and slickensided surfaces. Earth retention systems along this segment of the alignment will consist of permanent tiebacks or large-diameter cantilevered cylinder piles which do not require large temporary excavations for installation.

Pile Driving Test

A pile driving test program was accomplished to evaluate design assumptions and pile response to subsurface conditions. Two test sites were chosen: site 1 at the northwest end of the alignment, where the Vashon recessional sediments are relatively thin, and site 2 toward the east end of the alignment where a relatively thick section of Vashon recessional silt overlies till. Borings B-4 (site 1) and B-2 (site 2) were sampled and logged at each test location to document the subsurface conditions. The purposes of the pile driving test program were:

(1) To monitor vibration levels during driving.

(2) To observe ground settlement induced by pile driving.

(3) To determine if piles could be driven through the stiff to very stiff portions of the Vashon recessional silt.

(4) To determine how far driven piles could penetrate into the very dense, overridden Vashon sediments.

Measured vibrations at the test sites were less than the design levels recommended during preliminary phases of the project. Also, predrilling the piles to the bearing strata reduced the measured vibration levels within 50 ft of the pile. This information allowed driven piles to be located closer to existing buildings than recommended during preliminary design work.

Settlement of the ground surface and structures adjacent to the test sites measured during pile driving were within the range anticipated (0.01 to 0.11 ft) prior to the tests. Typically, the majority of the settlement occurred upon driving the first test pile and within 20 ft of the test group. At test site 1, measurement points within 20 ft of the test piles continued to settle as much as a week after completion of the tests.

The resistance to pile driving (in blows per foot) versus the stratigraphy at each test site for various types of piles is shown in Figure 7. Important conclusions drawn from these data are:

(1) Piles can be successfully driven through the stiff Vashon recessional silt;

(2) Closed-end piles can be embedded into overridden soils the 10 to 15 ft necessary for design uplift capacities; and

(3) Exploratory drilling and geological interpretation provided an accurate determination of bearing strata as evidenced by the good correlation between the increase in blow counts and the stratigraphically determined top of the till.

SUMMARY

Subsurface exploration and stratigraphic interpretation of sediments encountered along the Seattle Access portion of I-90 revealed complex geologic conditions and numerous stratigraphic units. The units were deposited in different environments, and these differences are reflected in the variations in sediment type and physical properties among units.

The low strength and possible liquefaction of the near-surface fill and recent marine sediments is accounted for in the foundation design by assuming these sediments have no strength and by increasing the diameter and stiffness of pile sections constructed through these sediments. Compressibility of the silt and clay portions of these units that underlie touch-down embankments is accounted for by staged construction and preloading.

Pile driving tests, accomplished through thick sections of recessional outwash sediments, indicate that piles can be driven through these sediments to bear in the underlying till and advance outwash. Supporting all piles and piers in overridden sediments alleviates concerns about differential settlement between adjacent piles.

Pile driving tests also indicate that piles can be driven into the overridden till and advance outwash the 10 to 15 ft required for design uplift capacities without having to use oversized driving hammers. The driving tests produced the anticipated vibrations and also revealed that vibrations close to the piles are reduced with predrilling.

High ground-water pressures within the advance outwash sand require the use of driven piles at most locations. The pile driving tests indicate that driven piles are feasible and drilled shafts will only be used where the foundation system must be constructed close to existing structures.

The stability of the slide-prone pre-Vashon silt and clay will be controlled by using earth retention systems which utilize permanent tiebacks or large-diameter cylinder piles which do not require large excavations for installation.

All design concerns formulated during preliminary investigations conducted for the Seattle Access project were addressed or alleviated once an accurate, relatively detailed geologic understanding of the project area sediments was developed. All large civil projects should begin with a thorough study of the geology and stratigraphy of site sediments which can be used as a framework for design of the foundation system, thus reducing construction costs and limiting the potential claims for changed or unanticipated conditions.

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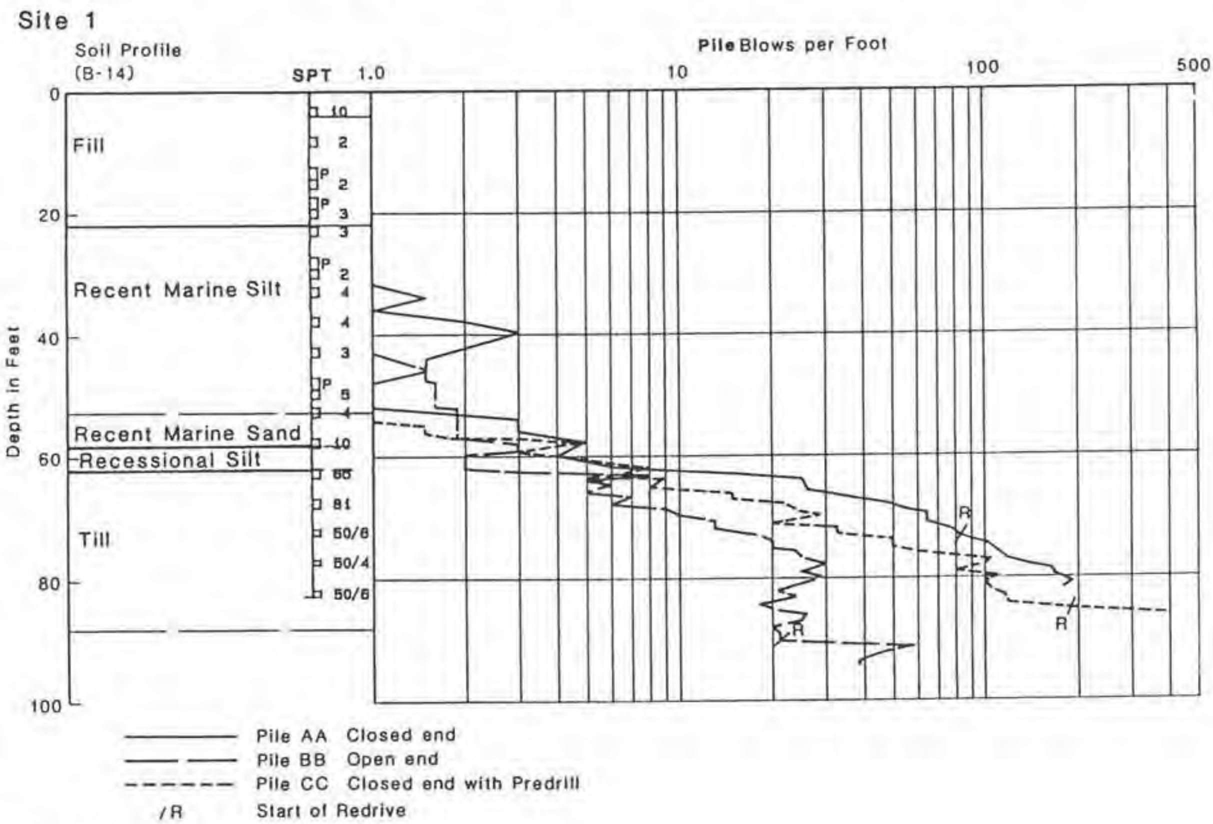

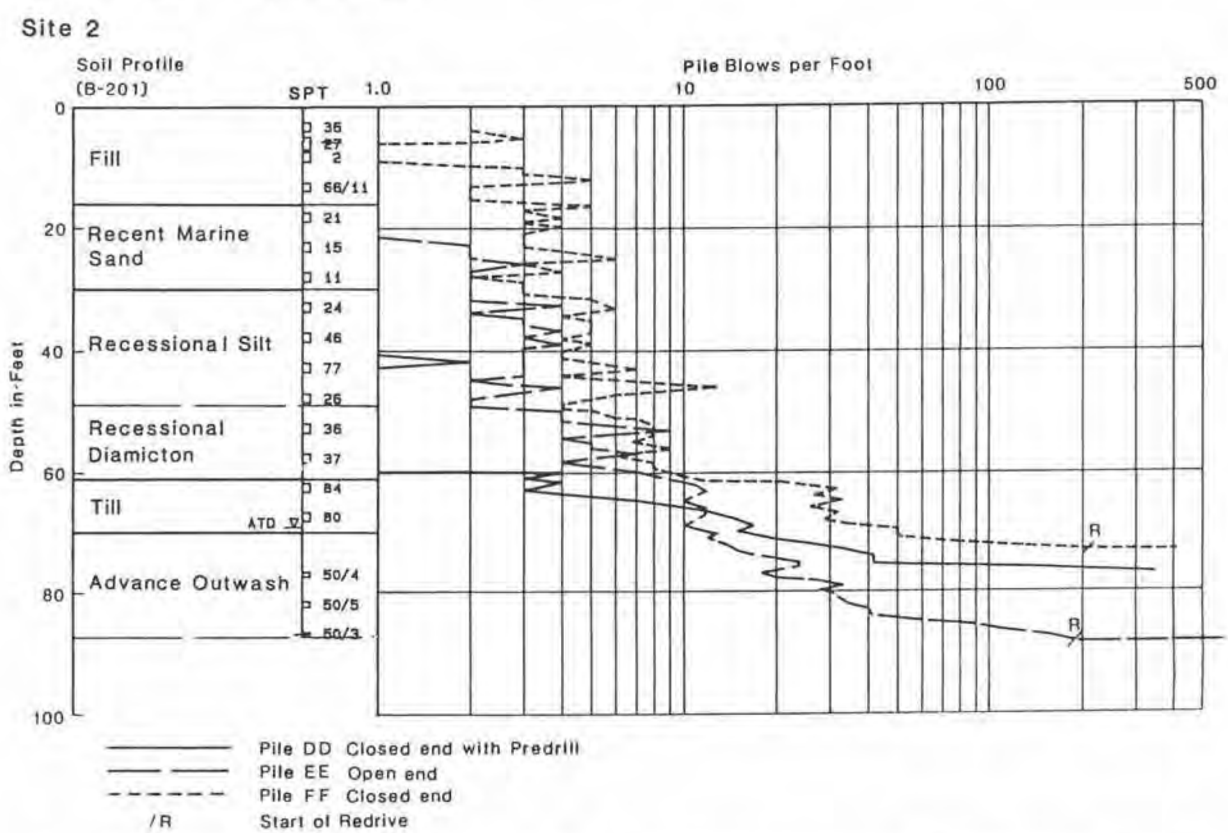

Figure 7. Pile driving resistance versus stratigraphy for different pile designs at two test driving locations. (ATD, At time of drilling; SPT, Standard Penetration Test

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Subsurface Conditions and Load Testing for the Glenn Jackson Bridge (Interstate Highway 205) across the Columbia River

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INTRODUCTION

In July 1965, approximately 1.5 million vehicles crossed the Columbia River on the twin interstate highway bridges between Vancouver, Washington, and Portland, Oregon. Average traffic for that month was 47,000 vehicles per day, and on July 2, a traffic peak of more than 55,000 vehicles was recorded (CH2M Hill, Inc., 1968). Average daily traffic (ADT) volume was increasing at an annual rate exceeding 4 percent, and by 1978 the ADT was in excess of 100,000 (Oregon State Highway Division, 1979).

These data supported the need to plan an additional river crossing and freeway to accommodate the rapidly increasing volume of commuter traffic in the Portland-Vancouver metropolitan area. The Oregon and Washington highway departments selected a route to bypass Portland and Vancouver. The new route, Interstate Highway 205 (I-205), begins at an interchange with Interstate Highway 5 (I-5) in Clark County, Washington, and passes east of Vancouver, Washington, and Portland, Oregon, rejoining I-5 about 7 mi south of Portland. Where I-205 crosses the Columbia River, 7 mi up-river from the I-5 bridges, Government Island divides the river into the North and South Channels (Figure 1). The alignment of the bridge swings to the east to provide additional clearance for the flight approach path for Portland International Airport. The bridge alignment as seen from the Washington approach is shown in Figure 2.

The 7,500-ft-long twin north channel structures were designed by Sverdrup & Parcel and Associates, Inc. The design for the twin 3,500-ft South Channel bridges was by the Oregon Department of Transportation. A continuous structure across Government Island was initially considered. However, this plan was discarded in favor of the less costly embankment across the island when it was determined that the embankment would have no effect on the upstream river elevation even during extreme high water, when Government Island is flooded. The design maximum high water level at the site was elevation 36.5 National Geodetic Vertical Datum of 1947 (NGVD).

Foundation investigations for both crossings were carried out by CH2M Hill, Inc. The initial investigations were done in 1971 and 1974, followed by pile load tests in 1974. A supplemental foundation investigation for piers 12 and 13, which support the main 600-ft span, was carried out in 1976.

This paper discusses the subsurface conditions, design, and field testing of the deep foundations for the I-205 crossing, the Glenn Jackson Bridge, of the Columbia River.

GEOLOGY

Regional Geology

The geologic history of the Portland-Vancouver area that is pertinent to this investigation began with the development of a large lake basin in Pliocene time (2-5 Ma). This basin was filled by lacustrine clays, silts, and very fine sands. These became the Sandy River mudstone, which is as much as 725 ft thick. As the basin either filled or its outlet was breached, the lacustrine conditions changed to an environment of stream deposition, and coarse gravel and sand of the Troutdale Formation were deposited over the Sandy River mudstone. The Troutdale Formation has a maximum thickness of about 1,000 ft, is cemented to various degrees, and is locally very hard (Trimble, 1963).

The coarse granular unit of the Troutdale Formation consists of material derived from the Columbia River basalt, the Hood River Conglomerate, the Dalles Formation, and other materials, including quartzite and other metamorphic rock types from more distant sources. In

Figure 1. Location of the Glenn Jackson Bridge.

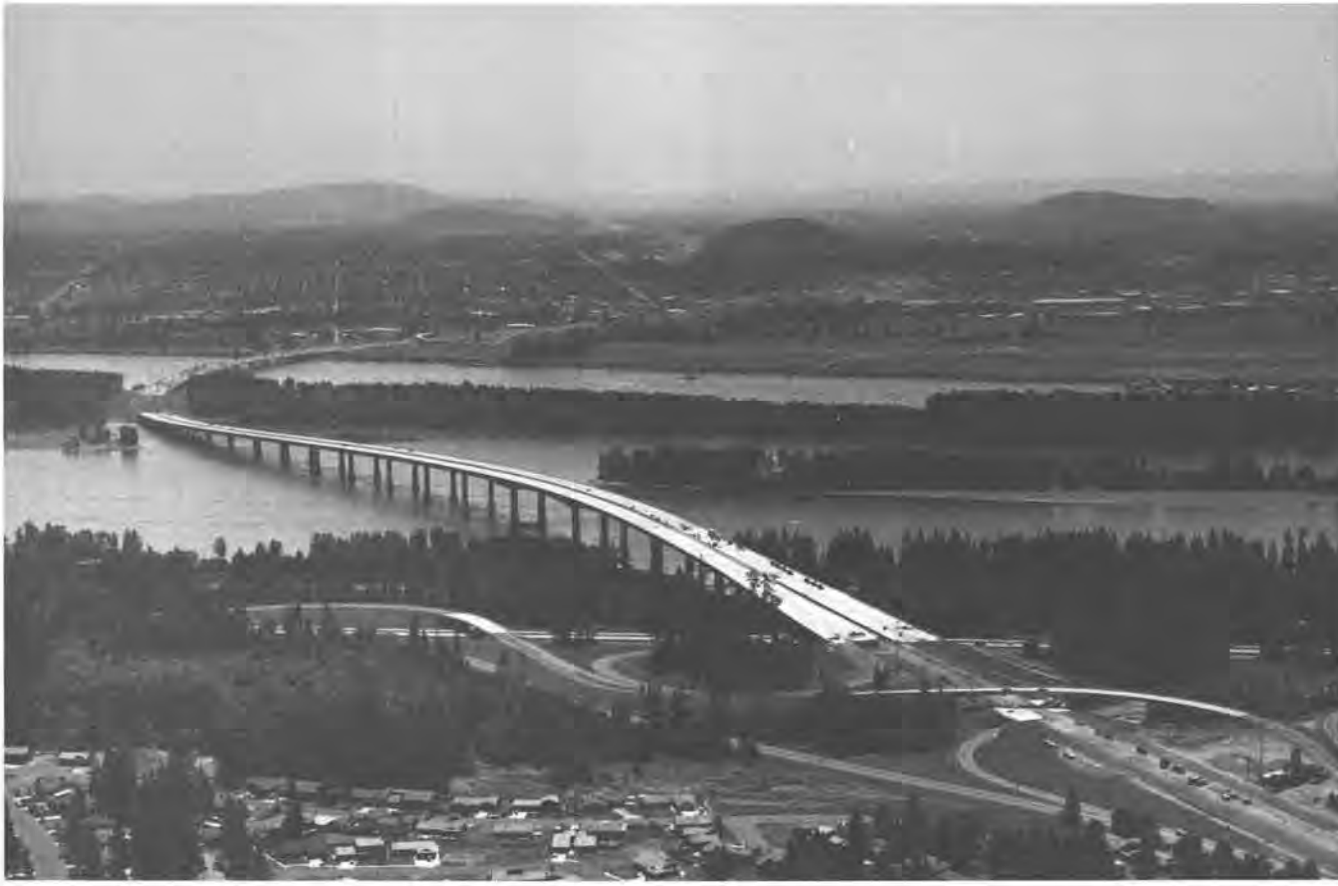


Figure 2. The Glenn Jackson Bridge as seen from the Washington (north) side of the Columbia River. The bridge crosses the North Channel to an embankment on Government Island and then crosses the South Channel into Oregon.

the coarse granular unit of the Troutdale Formation is a 25- to 50-ft-thick zone of very dense stratified siltstone, claystone, and very fine grained sandstone.

In conjunction with local volcanic activity, the Portland-Vancouver basin, after deposition of the Troutdale Formation, was uplifted, and the deposits were warped and gently folded, and several faults were produced. The Troutdale Formation was then subjected to weathering and erosion over a relatively long period of time, during which the ancestral Columbia River cut a broad valley in the formation.

At the close of the Pleistocene Epoch, the glaciers began to recede, and major ice dams in western Montana and northern Idaho were breached, releasing massive volumes of water over very short periods. The Columbia River carried large quantities of glacial debris (sand, gravel, and boulders) through the Columbia River Gorge and deposited these materials in an extensive delta at the mouth of the gorge near Washougal, Washington. This delta deposit, the Portland Delta Gravels, completely filled the broad valley downstream from the gorge and spread over adjacent lowlands. The present topography is the result of the erosion of parts of the Portland Delta Gravels by the Columbia River. The river currently occupies its ancient valley in the Troutdale Formation. The overlying Portland Delta Gravels are at higher elevations on the slopes of the Washington side of the river, and the underlying, more resistant Troutdale Formation controls erosion along the escarpment.

Site Conditions

Locations of test borings in relation to the bridge alignment are shown in Figure 3, and Figure 4 shows a generalized subsurface profile of the structure. Borings B-1 through B-37 were drilled in 1970 and 1971 using both truck- and barge-mounted rotary drill rigs (CH2M Hill, Inc., 1971). Borings B-38 through B-54 were drilled between August 8 and October 10, 1973 (CH2M Hill, Inc., 1974a).

The Washington escarpment (borings B-1 through B-8) consists of highly varied deposits of weathered, cemented gravels and cobbles, unconsolidated sand, and silt layers overlying the unweathered surface of the

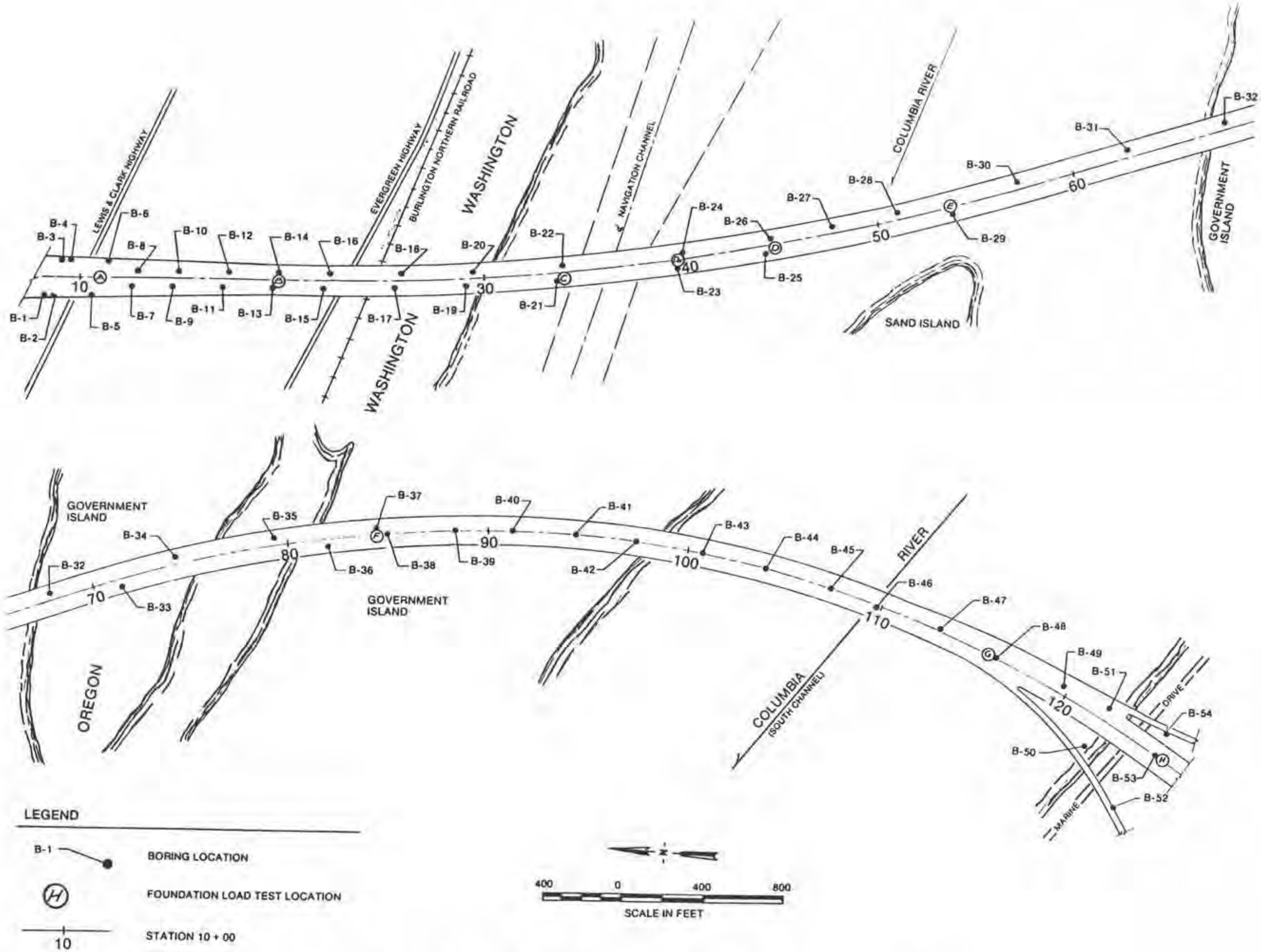
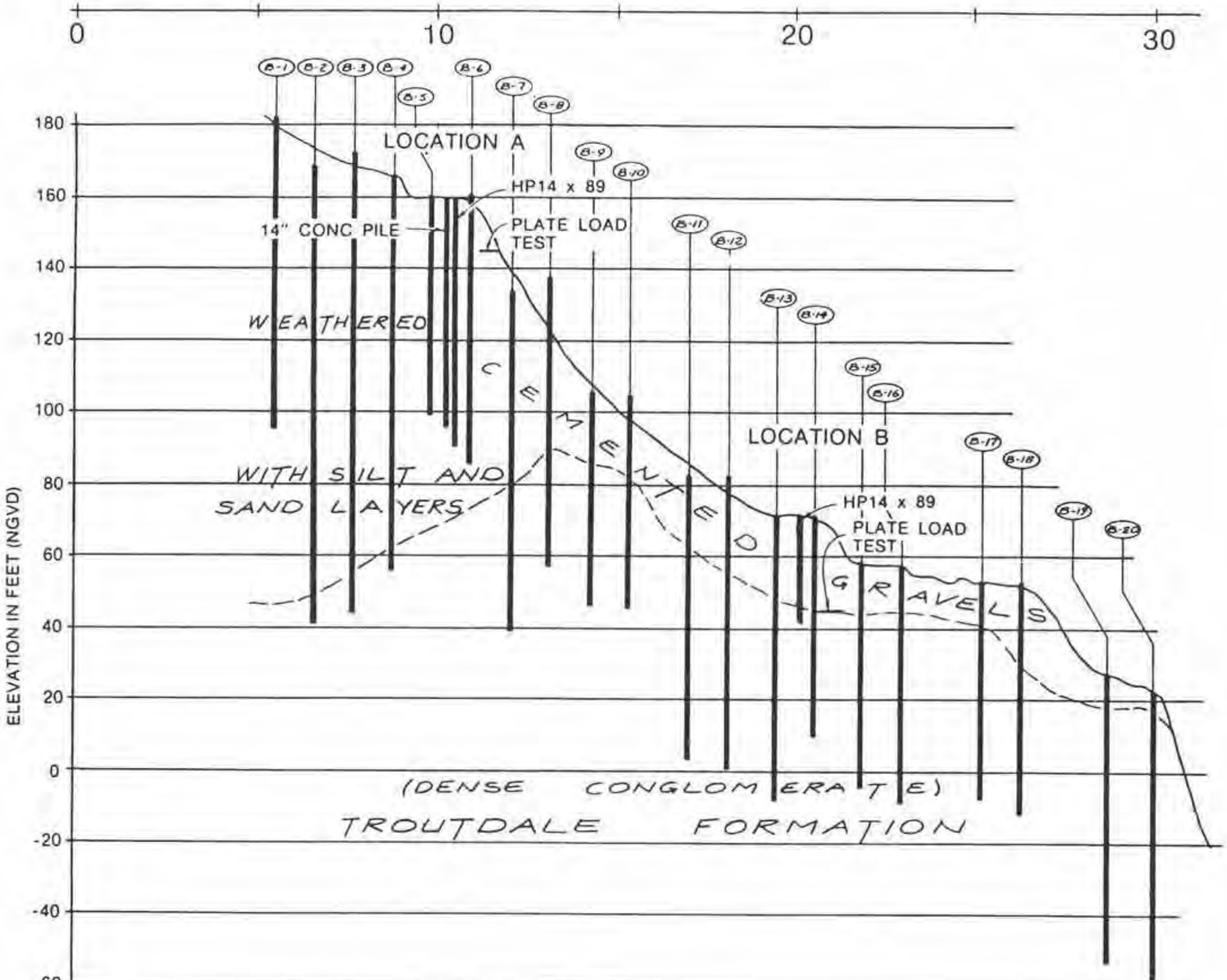


Figure 3. Site plan and boring and load test locations, Glenn Jackson Bridge.



LEGEND

LOCATION A FOUNDATION TEST LOCATION

24 IN. DIA. CASING AROUND PILE

RIVER LEVEL

Figure 4. Subsurface profile, north end of the Glenn Jackson Bridge. See Figure 3 for station locations.

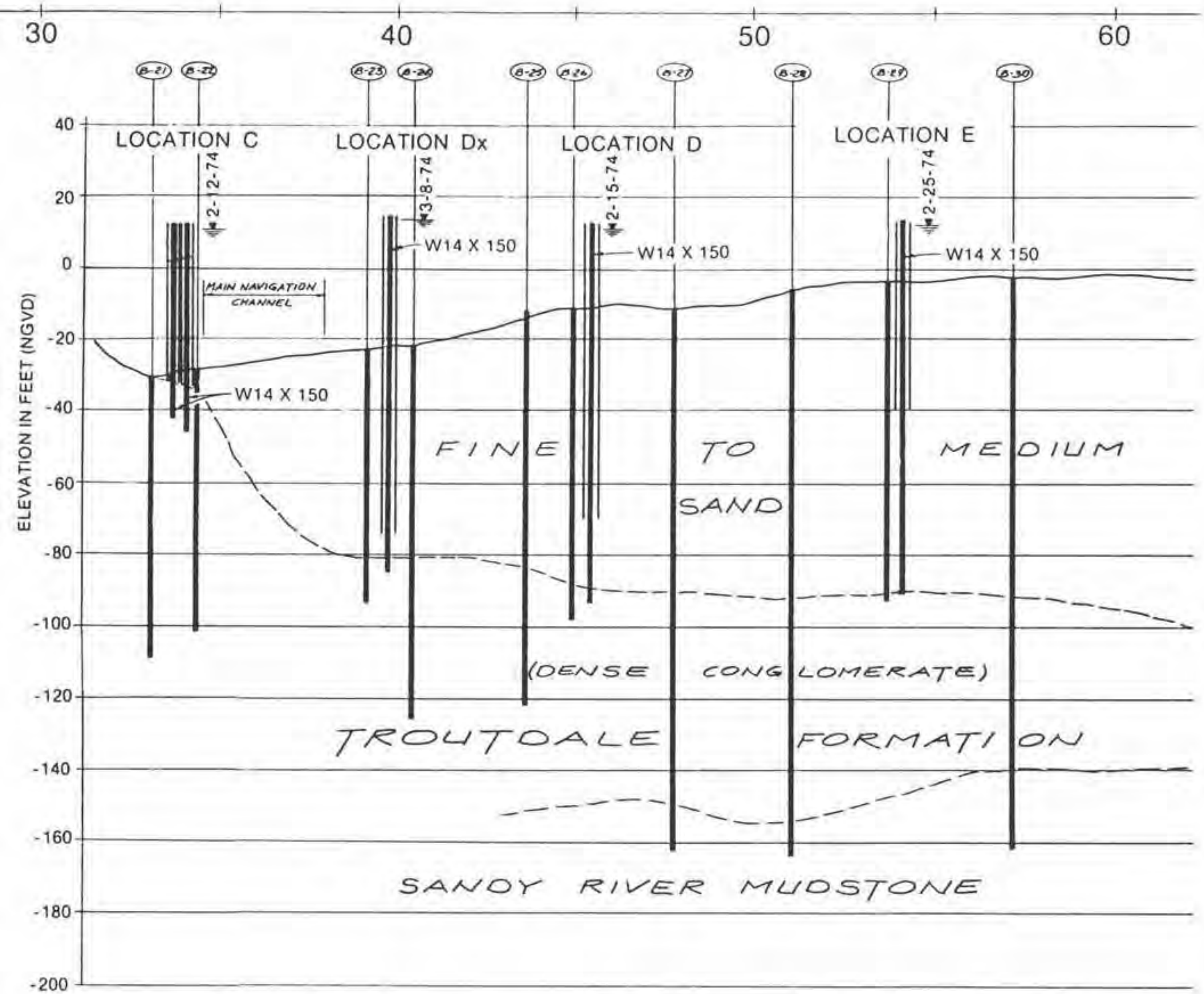


Figure 4. (continued) Subsurface profile, main Columbia River channel. See Figure 3 for station locations and the first page of this figure for explanation of symbols.

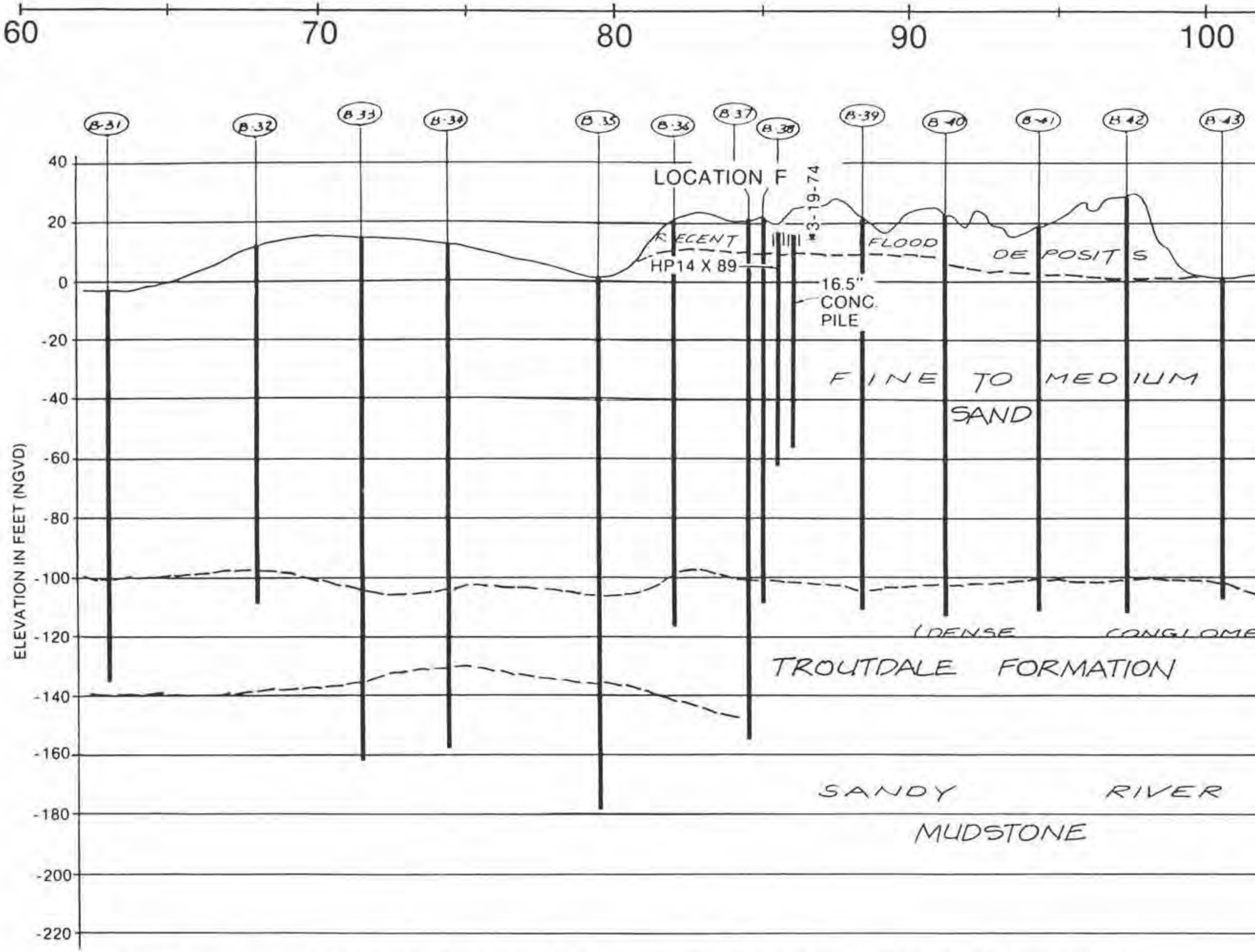


Figure 4. (continued) Subsurface profile, South Channel and Oregon shore. See Figure 3 for station locations and the first page of this figure for explanation of symbols.

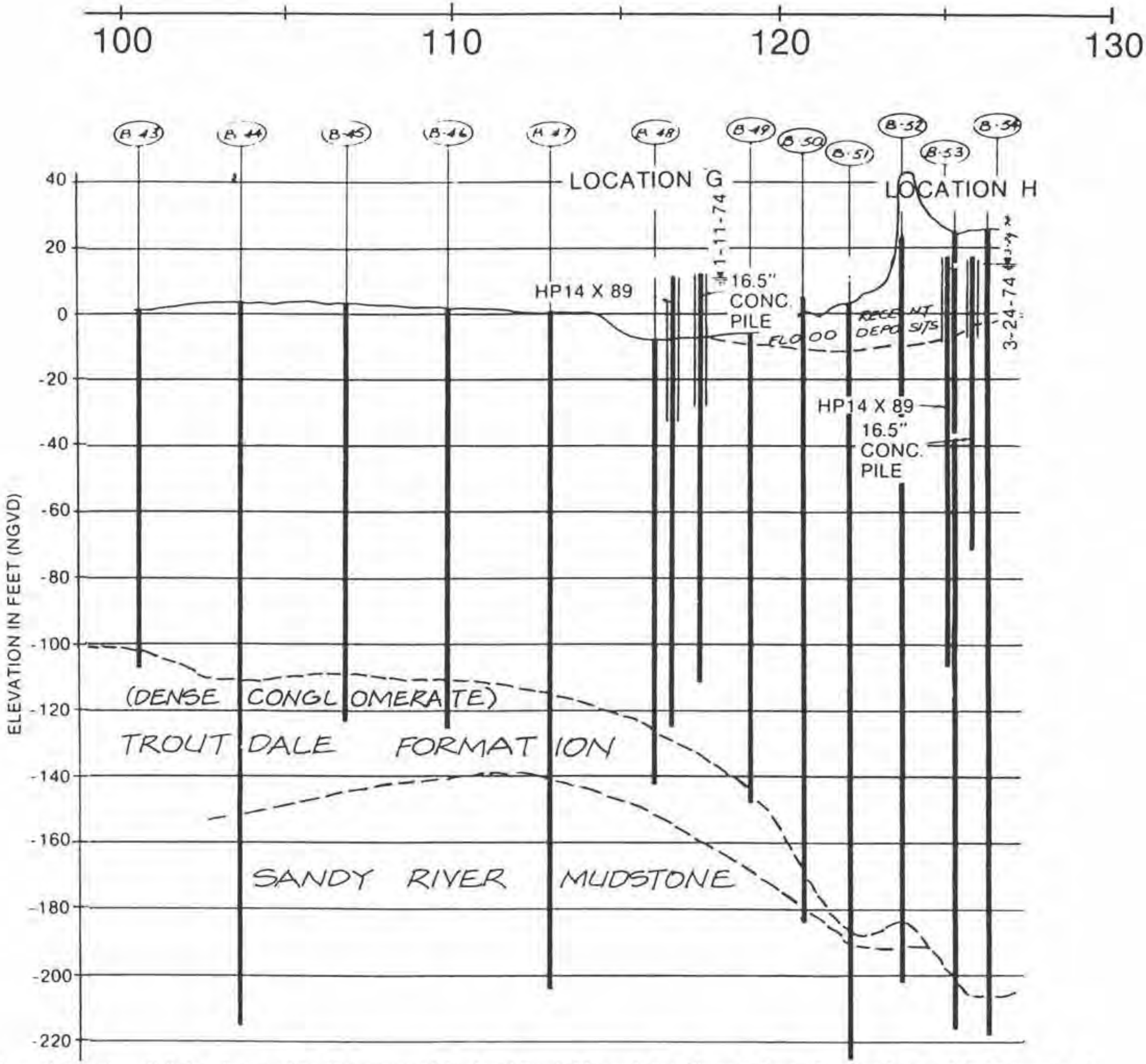


Figure 4. (continued) Subsurface profile, Government Island and South Channel. See Figure 3 for station locations and the first page of this figure for explanation of symbols.

dense Troutdale conglomerate. The unweathered Troutdale Formation is encountered at elevations ranging from 50 to 90 ft through this reach. Ground-water conditions are extremely varied. Fine silty-clay layers and cemented layers are impermeable barriers which create local perched ground-water conditions.

The lower Washington slope (borings B-9 through B-20) consists of 15 to 25 ft of fine sand and silty sand of medium density overlying the Troutdale Formation. Ground-water conditions are similar to those for the Washington escarpment.

The main river channel (borings B-21 through B-36) is underlain by a fine to medium sand, as much as 100 ft thick, over the dense conglomerate of the Troutdale Formation, which is approximately 25 to 60 ft thick. The Troutdale Formation is, in turn, underlain by the Sandy River mudstone. CH2M Hill's supplemental investigation of the foundations for piers 12 and 13 (CH2M Hill, Inc., 1976) revealed that the contact between the Troutdale Formation and the Sandy River mudstone is at about elevation -150 ft. The thickness of the Sandy River mudstone is not accurately known (according to available literature); however, borings were still in the material at elevation -215 ft.

Government Island (borings B-37 through B-42) has a subsurface profile similar to that of the main river channel, with about 10 to 20 ft of recent flood deposits of silt, clay, and silty sand overlying the river-deposited fine to medium sand. The ground water through this reach is directly influenced by the river stage. Because of the high permeability of the river sands, water-table elevations in the interior of the island are no more than about a foot above river levels.

The south river channel (borings B-43 through B-49) has a subsurface profile similar to that for the main channel reach. The dense conglomerate of the Troutdale Formation decreases in thickness and slopes to a depth of about 140 ft below river bottom at the south side of the south channel.

The Oregon flood plain (borings B-50 through B-54) consists of fine to medium river sand overlain by strata of silt, clay, and fine sand that total 35 to 45 ft in thickness. These surficial layers contain zones of highly compressible sediment. Beneath the river sand, the Troutdale Formation has been completely eroded. The Sandy River mudstone (the first rock unit encountered) is at depths approaching 220 ft below the flood-plain surface.

FOUNDATION TEST AND ANALYSIS

Preliminary Foundation Types and Locations

The geologic conditions described above required the use of several different foundation types to respond to the site conditions and construction circumstances. The foundation types considered were spread footings, end-bearing piles, and friction piles. The foundation types planned for the different subsurface conditions are shown in Table 1.

In order to confirm the foundation types being considered and determine design criteria, field testing was undertaken.

Foundation design criteria established in 1971 and 1974 were verified and modified (where applicable) by the foundation test program and analysis (CH2M Hill, Inc., 1974b). Design criteria were evaluated for spread footings, and design pile capacities for both friction and end-bearing piles were established. The load test program contract, administered by CH2M Hill, was awarded by the Oregon State Highway Division to Peter Kiewit Sons Co.

The test locations A through H (Figure 3) were selected to provide design information for the foundation types for the various reaches of the bridge. Table 2 summarizes load test types and locations.

Table 1. Foundation type and location

Subsurface Type	Foundation Type
Washington escarpment	Spread footing primarily, Some friction piles
Washington lower slope	Spread footings
Main river channel	Spread footings
Government Island	Friction or end-bearing piles
South river channel	Friction or end-bearing piles
Oregon flood plain	Friction piles

Table 2. Foundation load test program, test types and locations
(C), compression test; (T), tension test

Location	Station	Subsurface type (See text)	Load test
A	11+00	Washington escarpment	Plate load Pile load o Steel (C)* o Concrete (C)
B	20+00	Washington lower slope	Plate load Pile load o Steel (C)
C	33+50	North pier, main channel	Pile driving test o Steel
Dx	40+100	South pier, main channel	Pile driving test o Steel
D	44+50	South of main channel	Pile load o Steel (C, T)
E	54+00	South of main channel	Pile load o Steel (C)
F	85+00	Government Island	Pile load o Steel (C) o Concrete (C)
G	116+00	South channel	Pile load o Steel (C, T) o Concrete (C, T)
H	125+50	Oregon flood plain	Pile load o Steel (C) o Concrete (C)

Plate Load Tests

Plate load tests were conducted to determine criteria for the spread footings. At locations A and B, the plate load tests consisted of three separate tests at each location on 30-in. x 30-in. x 2-in. plates located approximately 10 ft apart. At location A, the plates were placed at elevation 145 ft on a dense, weathered, slightly cemented gravel. At location B, the plates were placed at approximately elevation 51 ft on the dense conglomerate of the Troutdale Formation.

The reaction load for each test location consisted of six 10-ft x 40-ft x 5-ft flexifloat barge units filled with water and weighing approximately 420 tons. The load was supported on earth benches approximately 18 ft from the center of the loading plate. Test loads were applied using three 100-ton calibrated jacks, and instrumentation consisted of four dial extensometers clamped to a wood reference frame bearing on the four corners of the jacking plate. The bearing plates were placed on hand-excavated pads that were leveled with a thin layer of fine sand and grout.

The plate test compression loads were applied in 15-ton increments until a bearing failure was noted. Loads were then reduced in 15-ton increments to zero. Each load increment was maintained constant until the rate of total plate settlement was 0.01 in./hr or less. Curves for total plate settlement versus time for loads in the range of allowable bearing pressures were developed, and loads producing general bearing capacity failure were determined. An example of a typical test showing plate settlement versus plate load at location A is shown in Figure 5.

The natural ground-water table was at or above the elevation of the plates, and the surface tested was exposed for 10 ft in each direction from the plate, thus eliminating any surcharge effect. With the conditions met of submerged footing and surcharge equal to zero, the settlement versus load curves for the plate load tests could be used to evaluate performance of full-size pier spread footings placed on the tested materials.

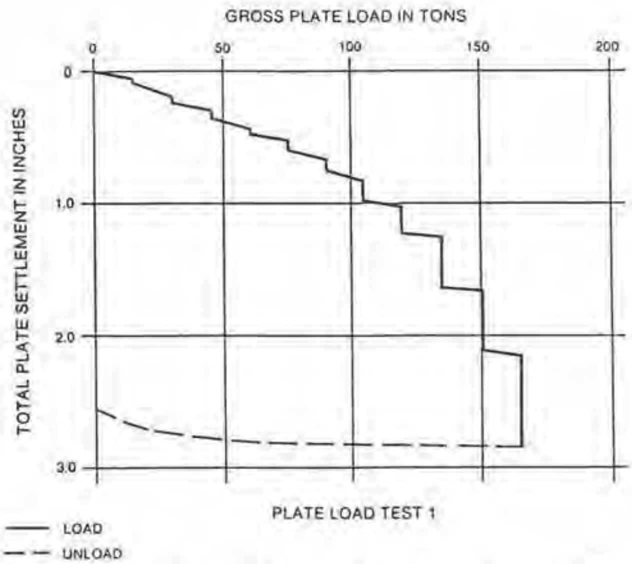


Figure 5. Pile load test at location A on Figure 4, north end of the Glenn Jackson Bridge.

Spread Footing Analysis

The following criteria were established for analytical purposes:

- Test pits and borings indicated that spread footings would be founded on granular material with some cementation. (However, cohesion was considered to be negligible.)
- Spread footings would be founded at or near the ground surface.

It was determined that settlement limitations for prototype footings (on the basis of field plate load tests) would dictate the range of allowable soil pressures for design. Because of the relationships of the prototype footing width to the ultimate bearing capacity of the foundation and the estimated settlement, each footing would develop a unique factor of safety against failure. Footing widths were in the range of 40 to 60 ft. The factor of safety for each footing can be computed only after the actual soil pressure and ultimate bearing capacity are determined. In CH2M Hill's 1971 report, a factor of safety against a bearing capacity failure of 3 was recommended.

Soil conditions within the zones of stress-increase should be similar for both the plate load test and the prototype footing. For example, the zones of stress increase for the plate load tests at location A (at the Washington escarpment) generally remained entirely within the stratum of weathered gravels. Settlement analysis for prototype footings at the Washington escarpment was made for a probable range of footing loads and size ranging from 1,750 to 3,500 tons at bearing pressures ranging from 2 to 10 tsf.

Pile Load Tests

Both friction and end-bearing piles were driven and tested during the field testing program. Friction piles tested included 14-in. octagonal prestressed concrete, 16.5-in. octagonal prestressed concrete, and HP14x89 steel. End-bearing piles tested included W14x150 steel and HP14x89 steel.

Pile Driving

All piling was driven with either a Vulcan 010 single-acting hammer (10,000-lb ram, 39-in. normal stroke) or a Vulcan 020 single-acting hammer (20,000-lb ram, 36-in. normal stroke). The cushion element for the hammer consisted of a steel cap block filled with six coils of 3/4-in. wire rope. Wood cushion blocks were provided between the top of the concrete piles and the driving bonnet. A summary of the test pile driving is shown in Table 3.

Pile Load Testing

Pile compression tests were conducted on piling driven at locations A, B, D, E, F, G, and H. Piles driven at locations C and Dx were not tested. Pile tension tests were conducted on both the steel and concrete piles driven at location G and on the steel pile driven at location D. Tests at locations D, E, and G were conducted over water; those at locations A, B, F, and H were on land.

Pile compression tests at the land locations were set up in a manner similar to that described for the plate load tests. Reaction loads ranged from 350 to 450 tons. Loads were applied using either four or six 100-ton jacks. Instrumentation was similar to that used on the plate load tests.

The reaction load for pile compression tests conducted over water was approximately 600 tons, made up of flexifloat barge units locked together as in the plate load tests. The contractor elected to support the reaction load on four 18-in.- or 20-in.-diameter, closed-end pipe piles. The loads were applied using six 100-ton calibrated jacks acting between a 30-in. x 30-in. x 2-in. jacking plate fastened to the pile and a jacking beam consisting of two stiffened W14x127 beams.

Instrumentation of the pile tests over water consisted of four dial extensometers bearing on the jacking plates and fastened to a reference beam. The reference beam consisted of two 3-in. x 12-in. timbers fastened to separate timber piles driven through casing to prevent vibrations from the river current. These timber piles were located approximately 13 ft from the center of the test pile.

The pipe pile supports for the 600-ton reaction load were used as the reactions for the tension tests. The instrumentation and reference beam was set up in the same manner as for the compression tests.

Table 3. Summary of driving data for test piles

Location	Pile Type	Length of embedment (ft.)	Hammer (Vulcan)	Hammer stroke (in.)	Hammer energy (ft.-lb)	Pile cushion	Terminal driving resistance (blows/ft)
A	14" Oct. Concrete	52	010	39	32,500	6" oak	103
	HP14X89	57	010	39	32,500	none	23
B	HP14X89	16	010	39	32,500	none	160
C	W14X150	10 13	020	36	60,000	none	180 230
Dx	W14X150	13	020	36	60,000	none	240
D	W14X150	50	020	36	60,000	none	250
E	W14X150	50	020	36	60,000	none	140
F	16.5" Oct. Concrete	66	020	24	40,000	4" oak	87
	HP14X89	73	010	39	32,500	none	43
G	16.5" Oct. Concrete	77	010	39	32,500	8" plywood	287(a)
	HP14X89	90	010	39	32,500	none	90
H	16.5" Oct. Concrete	54	010	39	32,500	6" oak	68
	HP14X89	61 71 91	010 010 010	39 39 39	32,500 32,500 32,500	none none none	62 57 63

(a) High terminal driving resistance is due to large number of cushion changes. The projestes terminal driving resistance without cushion interference is approximately 200 blows/ft

Loads for all pile tests (friction and end-bearing, and compression and tension) were applied in a similar manner, in increments varying from 15 to 25 tons. The load was reduced to zero after application of two successive load increments to determine the net movement or set of the pile after application of the sustained load. Each load increment was maintained until the rate of total pile head movement was 0.01 in./hr or less.

The test failure loads were determined from load versus pile head movement curves. An example of the pile head movement versus load at location D is shown in Figure 6.

Friction Pile Analysis

Preliminary design criteria for friction piles had been based on the assumption that pile capacities would be developed through side friction, neglecting the end-bearing resistance. However, the load test program revealed that a substantial amount of pile capacity was developed by end-bearing. For the 1974 investigation, it was assumed that, as the failure condition was approached, load transfer occurred simultaneously along the pile surface as well as across the bearing area of the pile tip. The ultimate capacity of an individual pile was determined by the sum of two separate components—end-bearing resistance and side friction resistance. The results of compression and tension tests were used to estimate the ratio of side friction to ultimate pile capacity. A summary of the ultimate capacities from the compression and tension tests is shown in Table 4.

Allowable friction pile capacities were obtained by dividing the computed pile capacities by an appropriate factor of safety. In the CH2M Hill 1971 report, a factor of safety of 3 was recommended. However, because failure modes of friction piles were observed in the field, a more liberal factor of safety (as low as 2.5) was considered, so long as pier loads were known within reasonable limits.

Friction piles used for foundation support within the river channel were designed with considerations for scour. The piles were considered to derive no load-carrying capacity from the presence of any soil within the zone of scour.

Friction piles installed in the Oregon flood plain, prior to embankment settlement, would be subject to

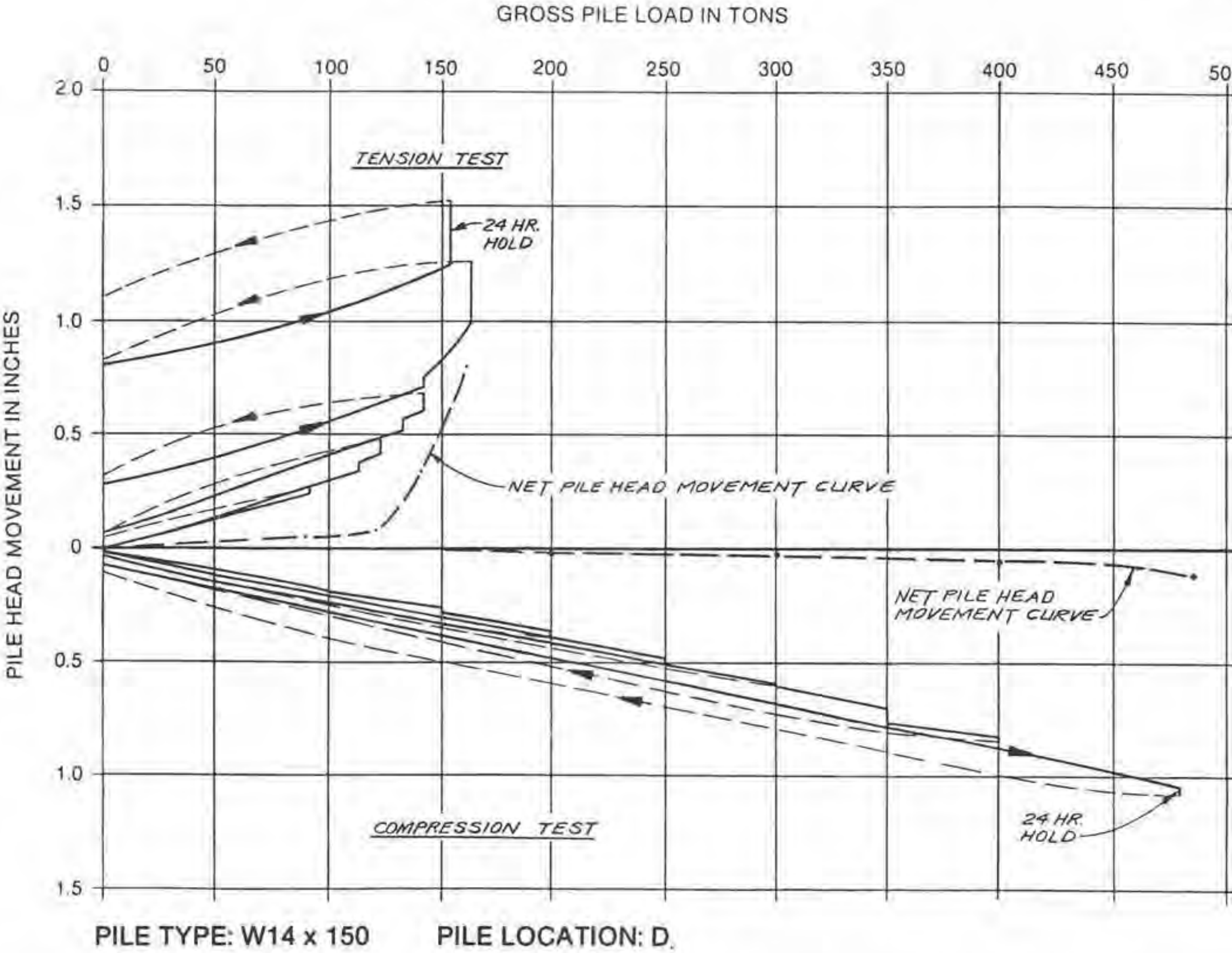


Figure 6. Pile head movement versus load at location D, south of the main navigation channel, Columbia River. (See Figure 4 for location.)

significant amounts of downdrag loads as the compressible layers of the flood-plain material consolidated and settled. These piles could not be considered to derive any load-carrying capacity from the presence of the embankment soils or compressible layers of the flood plain. It was also determined that piles should be driven through prebored holes in the approach embankment or with large enough hammers to ensure that the required penetration into the dense river sands could be achieved.

End-Bearing Pile Analysis

Compression test results for end-bearing piles driven to refusal on the Troutdale Formation indicated that ultimate pile capacity would be the structural capacity of the pile itself. Failure conditions of the soil-pile system for the end-bearing compression tests were not reached even as applied loads approached the yield point of the steel pile. Compression tests verified that end-bearing piles driven to refusal on the Troutdale Formation could be designed using the full structural capacity of the pile determined from allowable pile stresses. Allowable steel pile stresses of 12,000 psi (A36 steel) over the effective pile cross-section were recommended for design. (This was the maximum permitted by American Associations of State Highway Officials (AASHTO) for end-bearing piles for bridges at the time.)

The settlements of compression test end-bearing piles properly seated on the Troutdale Formation were due primarily to elastic shortening of the piles.

The test results indicated that production piles must be driven to refusal on the dense conglomerate of the Troutdale Formation in order to develop the intended capacities. The contract documents stated minimum requirements for the driving equipment which ensured the refusal would be met. Pile tip reinforcement was specified to reduce the possibility of driving damage and provide for uneven distribution of loading on the pile tip.

Table 4. Pile failure loads

Location	Pile type	Length of embedment (ft)	1/4 in. net pile head movement (tons)(a)	Intersection of net tangents (tons)	Inspection of load-pile head movement curves (tons)(b)
		Compression Tests			
A	14" Oct. concrete pile	52	Pile did not fail		
	HP14X89	57	205	200	190
B	HP14X89	16	Total Load 375 tons - Pile did not fail		
D	W14X150	50	Total Load 480 tons - Pile did not fail		
E	W14X150	50	Total Load 575 tons - Pile did not fail (FHWA dynamic prediction - 500 tons ultimate capacity)		
F	16.5" Oct. concrete pile	66	200	190	178
	HP14X89	73	222	163	150
G	16.5" Oct. concrete pile	77	250	233	224
	HP14X89	90	300	245	235
H	16.5" Oct. concrete pile	54	146	130	145
	HP14X89	61	230	210	176
	o Retest 1	71	290	185	176
	o Retest 2	91	240	200	235
		Tension Tests			
D	W14X150	50	140	125	125
G	16.5" Oct. concrete pile	77	65	75	75
	HP14X89	90	80	75	70

(a) 1/4-inch net pile head movement is in addition to the theoretical elastic shortening of the pile.

(b) Failure loads determined from inspection and net load pile head movement curves were used in the analysis.

FINAL FOUNDATION TYPES AND LOCATIONS

Washington Escarpment and Lower Slope

For the Washington escarpment and the lower Washington slope, the plate bearing tests provided data that supported the use of spread footings as the best foundation for this portion of the bridge. The footings were founded on a dense, weathered, slightly cemented gravel layer (borings B-1 through B-8) and on the dense conglomerate member of the Troutdale Formation (borings B-9 through B-20).

Main River Channel

The foundation support for the piers on the north side of the main river channel also consists of spread footings founded on the dense conglomerate of the Troutdale Formation (borings B-21 and B-22). At the north main channel pier no. 12, the conglomerate is exposed at the stream bed at elevation -35 ft, about 40 ft below the surface of the river (borings B-23 and B-24). At the south main channel pier no. 13, the Troutdale Formation is at about elevation -80 ft. Scour considerations required that the base of this pier be at least at elevation -63 ft; hence, spread footings were also used for this pier.

The depth of the Troutdale Formation south of pier no. 13 (borings B-25 through B-36) made spread footings uneconomical. Between pier no. 13 and Government Island, the piers are supported by high-capacity, end-bearing piles. The piles were driven through fine to coarse, medium-dense sand and founded on the dense conglomerate of the Troutdale Formation.

Government Island

At Government Island the abutments are carried on piers supported by steel friction piles. Borings B-37 through B-42 revealed a subsurface profile similar to that of the main river channel, with about 10 to 20 ft of recent flood deposits of silt, clay, and silty sand overlying the river sand. The ground water through this reach is directly affected by the river. A 1,100-ft-long embankment of compacted river sand was constructed across the island. Embankment settlement consisted of only a few inches, most of which occurred during construction. The abutment foundations were not installed until after embankment-induced settlements of the compressible foundation materials were complete to eliminate downdrag loads associated with embankment settlement.

South River Channel

The south river channel piers are supported by friction piles driven into the dense river sands. End-bearing piles were ruled out here because the dense Troutdale conglomerate slopes downward from about elevation -110 ft at Government Island to an abrupt drop to about elevation -200 ft beneath Marine Drive on the Oregon shore (borings B-43 through B-49).

Oregon Flood Plain

The foundations on the Oregon flood plain are steel friction piles driven into the dense river sands. As at Government Island and the south river channel, the great depth of the Troutdale conglomerate (borings B-50 through B-54) made friction piles the logical choice. Approach embankments were constructed of compacted granular material, underlain by a 35- to 40-ft-thick layer of soft, compressible sand, silt, and clay. As recommended, the south approach embankments on Government Island were constructed 6 to 12 months in advance of the abutment and adjacent pier construction to consolidate the compressible underlying layer and reduce pile downdrag.

CONCLUSION

The value of the extensive subsurface investigation was demonstrated during construction by the lack of unexpected subsurface conditions. In the performance of the $50 million substructure work for the Glenn Jackson Bridge, the only surprise was the lack of surprises. Length of piles driven to achieve required bearing and the foundation conditions were as predicted from information collected during the subsurface investigation. Foundation work was completed with no significant cost overruns, and only two contract claims were submitted to the state. Neither of the claims was the result of unexpected conditions and both were satisfactorily resolved without litigation.

ACKNOWLEDGMENTS

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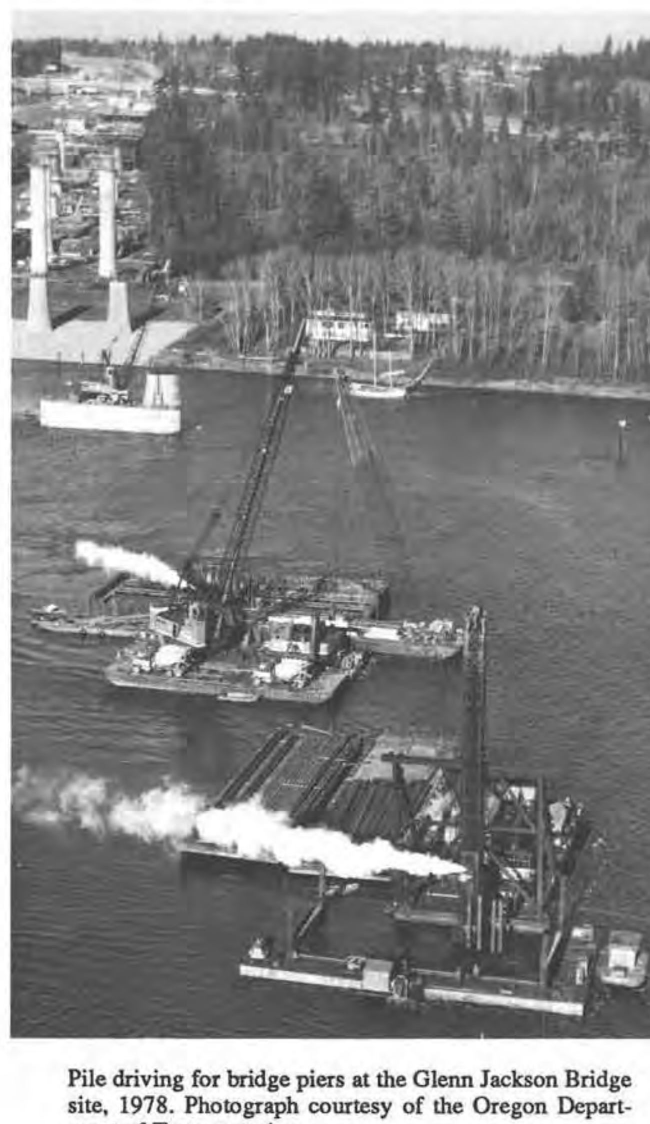
Bridge pier construction on the north (Washington) shore of the Columbia River, Glenn Jackson Bridge site in July 1987. Photograph courtesy of the Oregon Department of Transportation.



Pile driving for bridge piers at the Glenn Jackson Bridge site, 1978. Photograph courtesy of the Oregon Department of Transportation.

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Pile Foundation for the West Seattle Freeway Bridge Replacement, Seattle, Washington

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Shannon & Wilson, Inc.

INTRODUCTION

The City of Seattle recently constructed a replacement bridge and related roadway system across the West Waterway of the Duwamish River at the S.W. Spokane Street corridor of the West Seattle Freeway. The previously used north bascule bridge was made inoperable following a ship collision in 1979. The corridor is one of the busiest in Washington; it has a projected traffic count of 80,000 vehicles per day by year 2000. The bridge replacement consisted of elevated structures and interchanges located at the east and west ends of the project, as shown on Figure 1.

The project was divided into four major parts for design and contract award purposes: (1) main span structure, (2) Harbor Island structure, (3) east interchange structures, and (4) west interchange structures. Construction began on the main span structure in November 1980, and the total project was completed by 1984.

Because of the thick deposits of alluvial sands and silts along the project alignment, the heavy structural loads, and the potential of soil liquefaction during a strong earthquake, deep pile foundations were selected for support.

This paper describes the selection process for the piles and capacities, verification by the pile load test program, and the incorporation of the pile test results into the contract specifications.

SITE DESCRIPTION AND GEOLOGY

The site of the bridge replacement is near the mouth of the Duwamish River where it empties into Puget Sound on the south shore of Elliott Bay. A mile before the Duwamish reaches Elliott Bay, it divides into two channels which flow on either side of Harbor Island. Both waterways are about 350 ft wide. The West Waterway is a navigation channel for large vessels.

The Duwamish River valley is situated in the central part of the Puget Lowland, a structural and topographic trough. It is underlain chiefly by thick deposits of Quaternary sediments that overlie interbedded Tertiary volcanic and sedimentary bedrock. Most of the major topographic features of the lowland are the result of glacial deposition or erosion.

The original valley floodplain has been raised about 10 to 15 ft by hydraulic fill. The alluvial deposits are as much as 270 ft thick and consist of loose to dense sand with scattered layers of silt that have some plasticity. A generalized subsurface profile is shown on Figure 1.

FOUNDATION SELECTION AND TESTING

Timber, steel pipe of various diameters and wall thicknesses, and prestressed concrete piles of various diameters were considered during the preliminary design stage in order to select a cost-effective pile foundation for this project. In selecting the pile type and capacity, foundation settlement and lateral resistance and bending of piles, with and without soil liquefaction under the design earthquake, were considered, among other structural factors.

The preliminary pile lengths and capacities were estimated using several static capacity analysis methods and results of pile load tests performed near the project site. Based on these studies, structural requirements, and economic considerations, the following piles were selected for design:

Structure	Pile Type
Main span	36-in.-diameter, 3/4-in. wall steel pipe pile, driven open-ended; 600-ton design capacity.
Main line	24-in. octagonal prestressed concrete piles with a 15-in.-diameter hole in the center and the bottom end plugged; 200-ton design capacity.

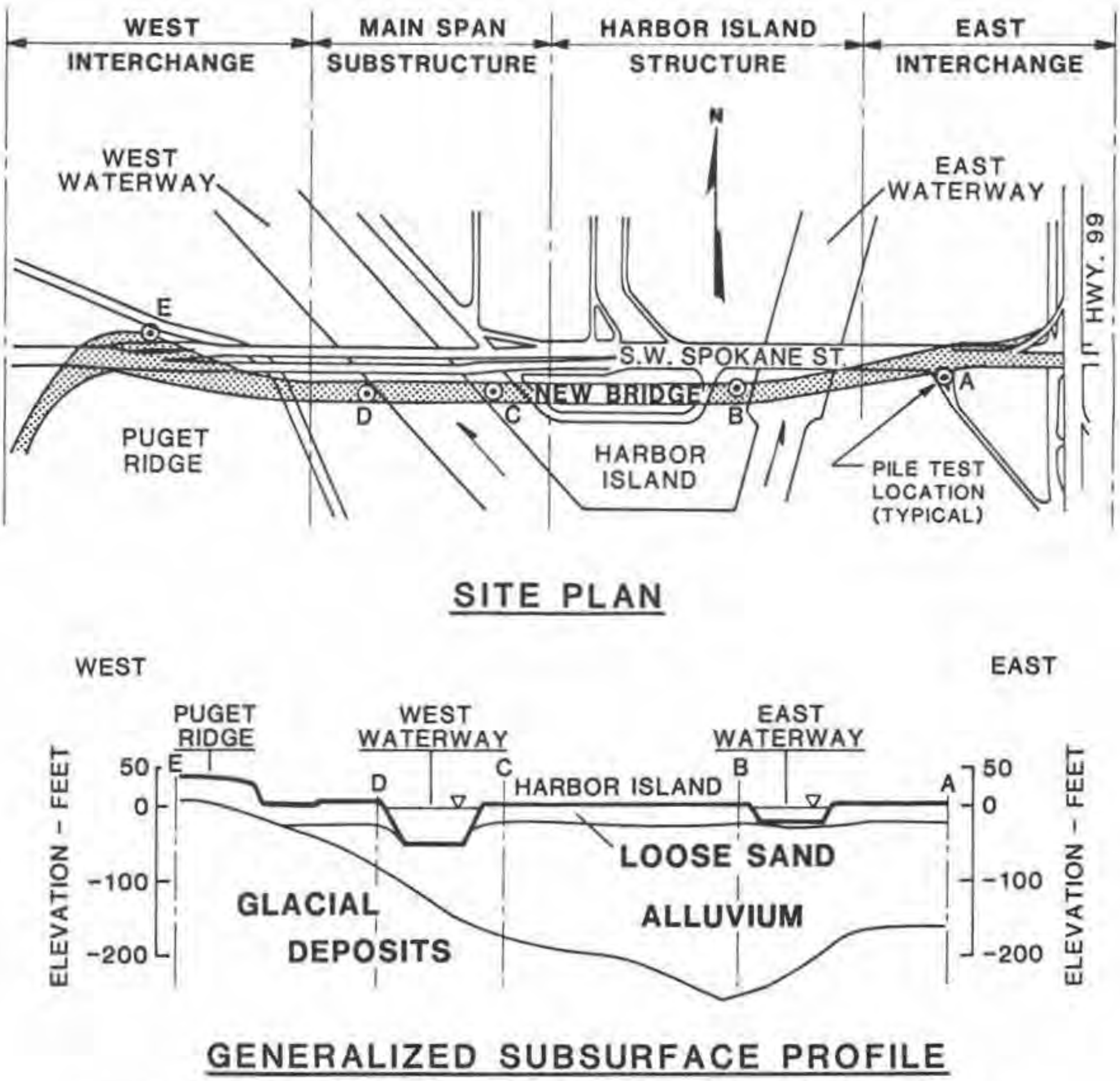

Figure 1. Site plan and generalized subsurface profile, West Seattle Freeway Bridge.

The estimated pile lengths were about 140 to 220 ft for the steel pipe piles and about 70 to 120 ft for the concrete piles.

Field explorations consisted of test borings, Dutch cone probes, and *in-situ* pressuremeter tests. Laboratory shear tests were performed to evaluate the frictional resistance between concrete or steel and soil. Triaxial compression and consolidation tests were also performed.

TEST PILE PROGRAM

During the initial design stage, a pile driving and pile loading test program was initiated at five selected locations (sites A through E), as shown on Figure 1, along the proposed alignment to determine and evaluate the driveability and vertical load carrying capacities of concrete and steel piles.

Driving test piles consisted of 16.5- and 24-in. octagonal prestressed concrete piles and 24- and 36-in.-diameter steel pipe piles. The wall thicknesses of steel piles varied from 3/4-in. to 1-1/4-in., and they were driven either closed-ended (fitted with an AP&F conical tip) or open-ended (tip reinforced). Dynamic measurements were made by Goble & Associates, Inc. on selected piles during test pile driving. The load test piles were instrumented to determine the load distribution along the pile.

Pile load tests were made on 24-in. prestressed hollow concrete piles at three locations, sites A, B, and E. Pile load tests on 24-in.-diameter x 1.25-in. wall steel pipe piles were made at each of the two main pier locations designated as sites C and D (Figure 1). A summary of the pile load testing program is presented in Table 1.

Table 1. Pile load testing program, West Seattle Freeway Bridge

	Concrete Piles		
	Site A	Site B	Site E
Load test pile diameter	24 in.	24 in.	24 in.
Anchor piles (concrete)	two 16.5 in. two 24 in.	four 24 in.	four 16.5 in.
Tests performed:			
Dynamic measurements	2	5	4
Compression load test	1	1	1
	Steel Pipe Piles		
	Site C		Site D
Load test pile (open-end)	24 in. x 1.25 in.		24 in. x 1.25 in.
Anchor piles	36 in. x 0.75 in. open end 24 in. x 1.25 in. closed end 24 in. x 0.75 in. open end		36 in. x 0.75 in. open end 24 in. x 1.25 in. open end 24 in. x 0.75 in. closed end
Tests performed:			
Dynamic measurements	3		3
Compression load test	1		1
Tension load test	1		1

Concrete Piles - Sites A, B, and E

Pile load test results and interpretations at sites A and B are presented separately from those for site E because of differences in subsurface conditions. The soil conditions encountered at sites A, B, and E are shown on Figures 2, 3, and 4, respectively.

Hammer Tests

Air-, steam-, or diesel-operated hammers with rated energies ranging from 60,000 to 90,000 ft-lb/blow were specified for the test program.

Preliminary wave equation analysis indicated that diesel hammers may be more effective in obtaining the desired pile penetration due to the higher hammer stroke for the long piles. The contractor selected a single-acting Kobe K45 diesel hammer because of its lighter weight. The stroke (height of drop) of the hammer was determined by several identifiable features on the ram along with a Saximeter developed by Pile Dynamics, Inc. The observed hammer energy for the last 5 ft of driving ranged from 71,420 to 79,400 ft-lb/blow, averaging around 74,400 ft-lb.

Pile Driving Resistances

Generally, the pile driving resistance of the 24-in. concrete piles within the alluvium was about equal to the N-values, as shown on Figures 2 and 3. N-value or the standard penetration resistance value is the number of blows required to drive a 2-in. O.D. split spoon soil sampler the last 12 in. of an 18-in. drive. It is an indicator of the consistency of cohesive soil and the relative density of granular soils. Hard driving conditions were encountered when the pile was driven through very dense sand with N-values greater than 50 blows per foot (bpf). At site E the driving resistance of the 24-in. concrete piles was somewhat less than the N-values until the pile tips penetrated glacial clay, after which the driving resistance increased with pile penetration, as shown on Figure 4.

Dynamic measurements were taken at the end of continuous driving and at redrive after 12 or more hours have elapsed.

Pile Load Tests

One compression load test each was performed at sites A, B, and E. The piles were tested to plunging failure, or to the proposed maximum test load of 600 tons. Load versus pile settlement results are presented on Figures 2, 3, and 4 for sites A, B, and E, respectively.

Two independent measuring systems were used to measure the strain at various points along the load test piles to estimate the load distribution: the SINCO vibrating-wire strain gauges, and stainless steel telltale rods, 1/4-in. in diameter. The strain gauges and the base mounts for the telltale rods were welded to a 2-1/2-in.-

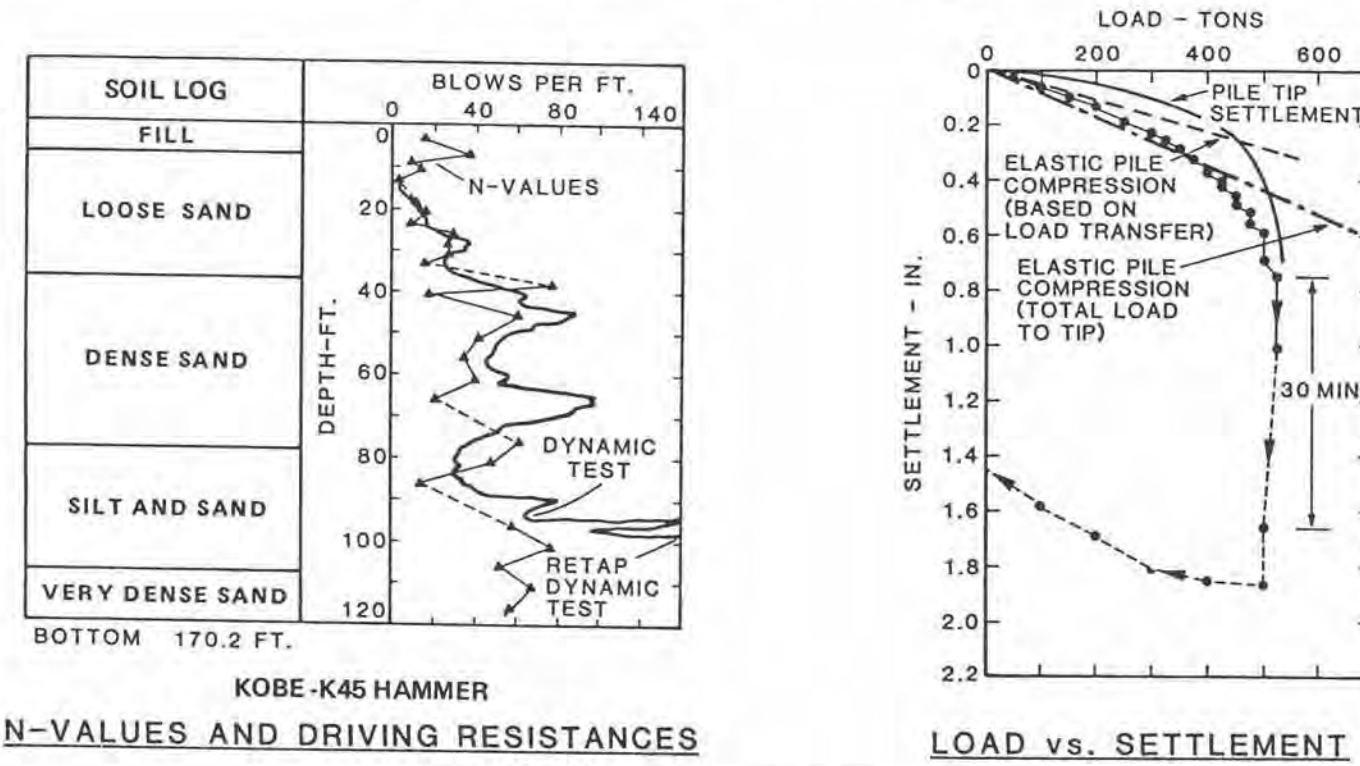

Figure 2. Site A 24-in. concrete load test pile (A-LTP).

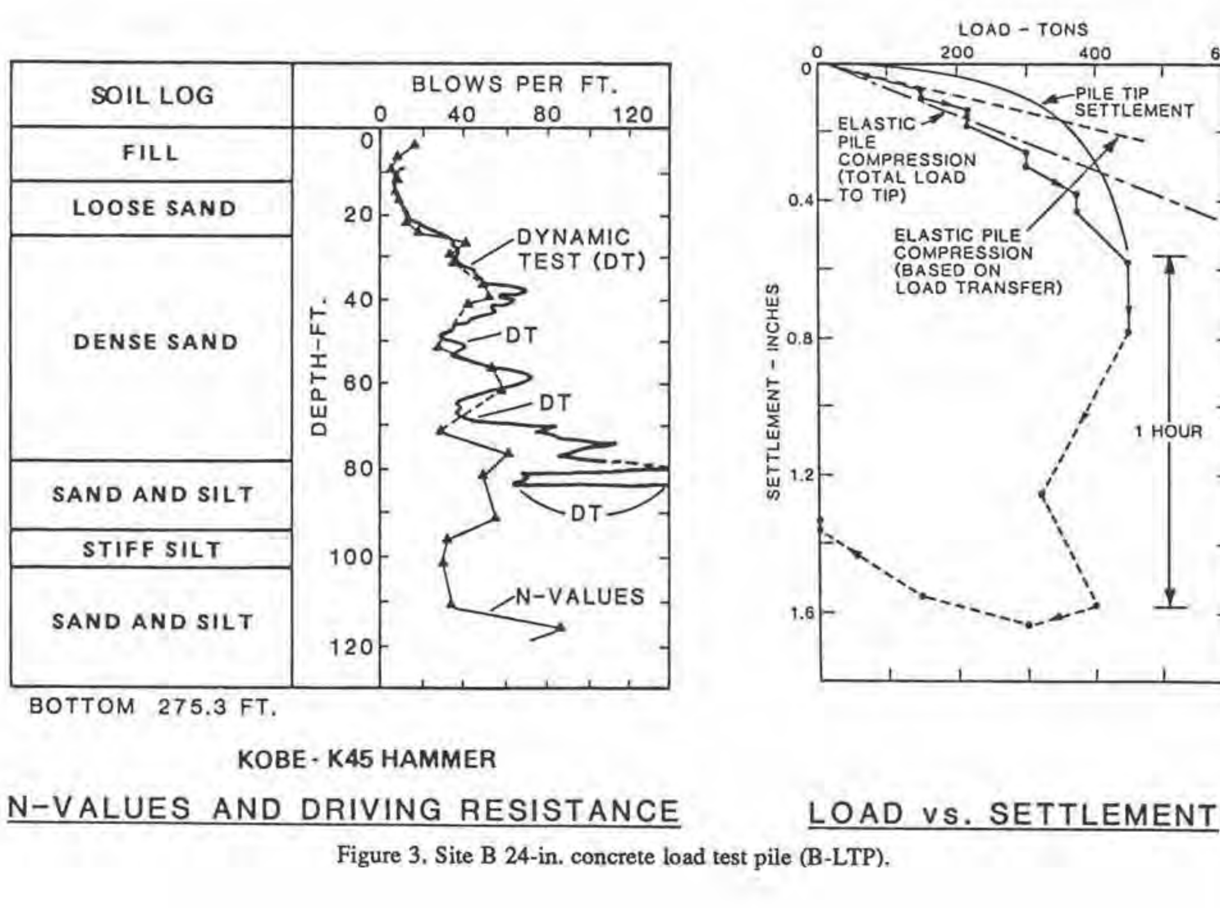

Figure 3. Site B 24-in. concrete load test pile (B-LTP).

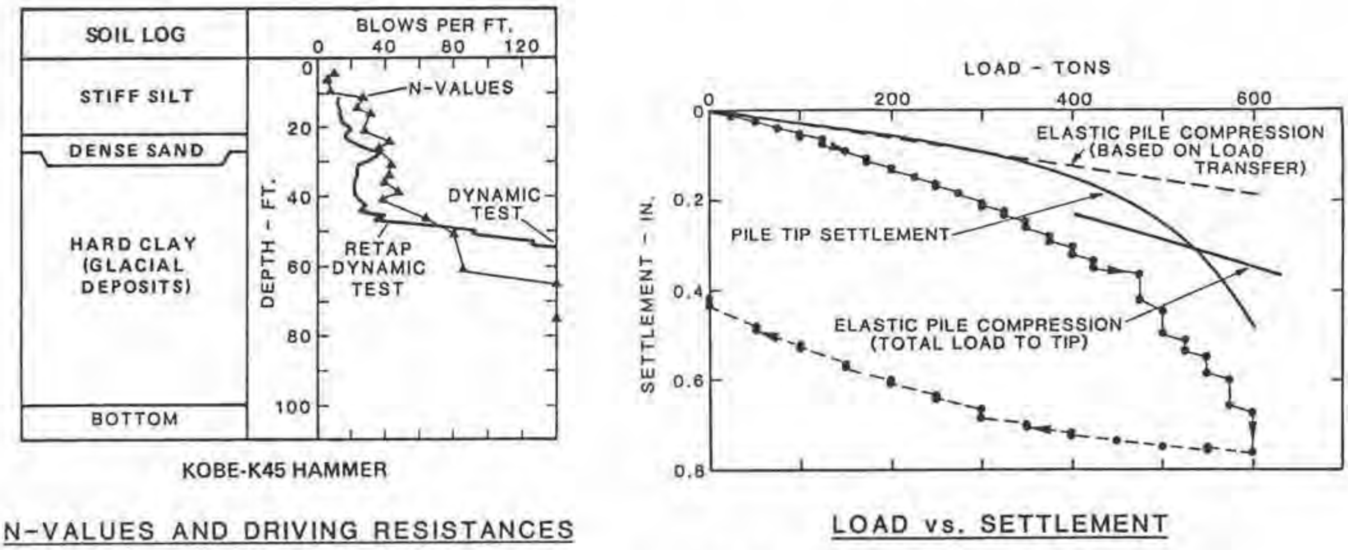

Figure 4. Site E 24-in. concrete load test pile (E-LTP).

square steel casing with a 1/8-in. wall thickness. The instruments were installed in the central hole of the pile after pile driving. Following installation of the instruments, the pile was backfilled with high-early strength sand-cement grout.

Load Transfer Measurements

Loads at the strain gauge level were computed using the formula:

$$P = E_c \times A_p \times q_e$$

where: P = applied load,

E_c = composite elastic modulus,

A_p = cross-sectional area of pile, and

q_e = change in strain of the pile between zero load and the applied load.

The deflection between the top of the telltale rod and the top of the pile was measured with a linear potentiometer. The average load between two telltale tips was determined by the following equation:

$$P = E_c \times A_p \times \Delta L/L$$

where: L = distance between the two telltale tips, and

ΔL = measured deflection difference of the two telltales.

The possible residual strain of the backfilled grout was not measured.

The composite elastic modulus of concrete was computed by assuming that the load at the top level instruments was equal to the applied load on top of the pile using the above equations.

The load distributions along the load test piles are shown on Figures 5, 6, and 7 for sites A, B, and E, respectively. Loads computed from strain gauges are shown at the instrumentation level. The tips of the telltale rods were located at the strain gauge level. Therefore, the average loads from the telltale rod data are plotted midway between strain gauge levels.

Ultimate Pile Capacities

The ultimate pile capacities as obtained from static pile tests and dynamic measurements at sites A, B, and E are summarized in Table 2.

The results indicate that the ultimate capacities as determined by dynamic measurements during restrike of piles A-LTP and B-LTP corresponded closely to the static test loads at failure. Restrike dynamic measurements were not made on the 24-in. concrete pile at site E due to scheduling.

Pile freeze capacity (or setup) ranged from 70 to 200 tons for 24-in. concrete piles depending on the soil conditions, pile penetration, and setup time allowed.

Piles A-LTP and E-16.5 (corrected to simulate E-LTP 24-in. pile) were analyzed using the CAPWAP (case pile wave analysis program) method to provide an estimate of ultimate frictional resistance along the pile during a selected hammer blow. These results are shown on Figures 5 and 7. The plots indicate a shape similar to that of the measured static load transfer data, but the magnitude in loads is slightly greater.

Load Test Results and Interpretation

The elastic pile deflections based on the measured load transfer data were calculated using the formula developed by Vesic (1977) and are shown on the load-settlement plots, Figures 2, 3, and 4. The elastic pile compression, assuming that the total applied load was transmitted to the tip, is also shown.

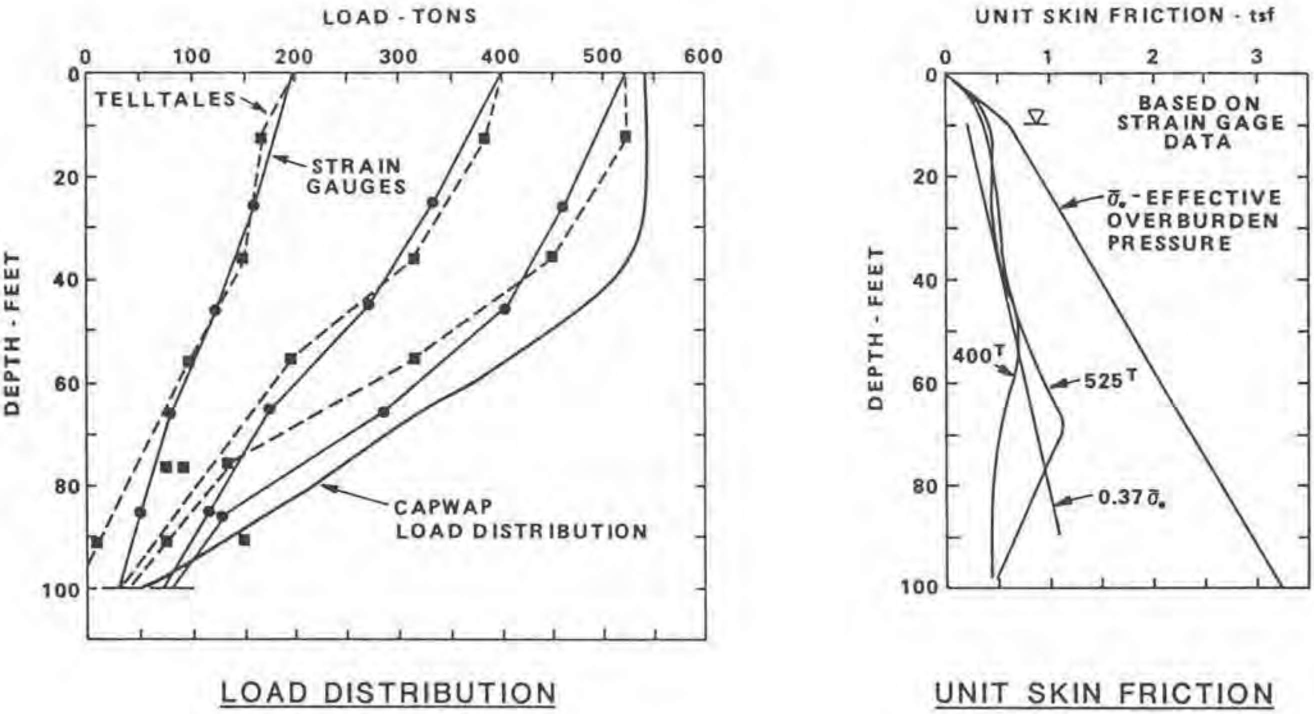

Figure 5. Site A 24-in. concrete load test pile (A-LTP).

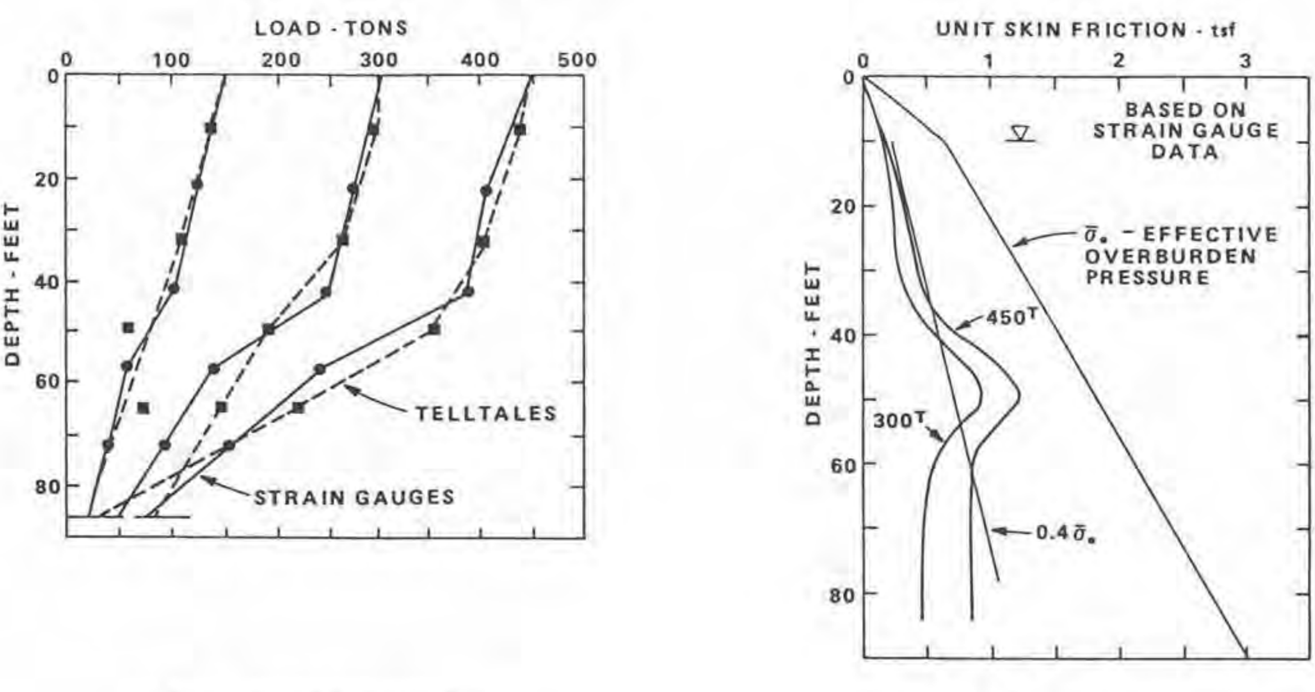

Figure 6. Site B 24-in. concrete load test pile (B-LTP).

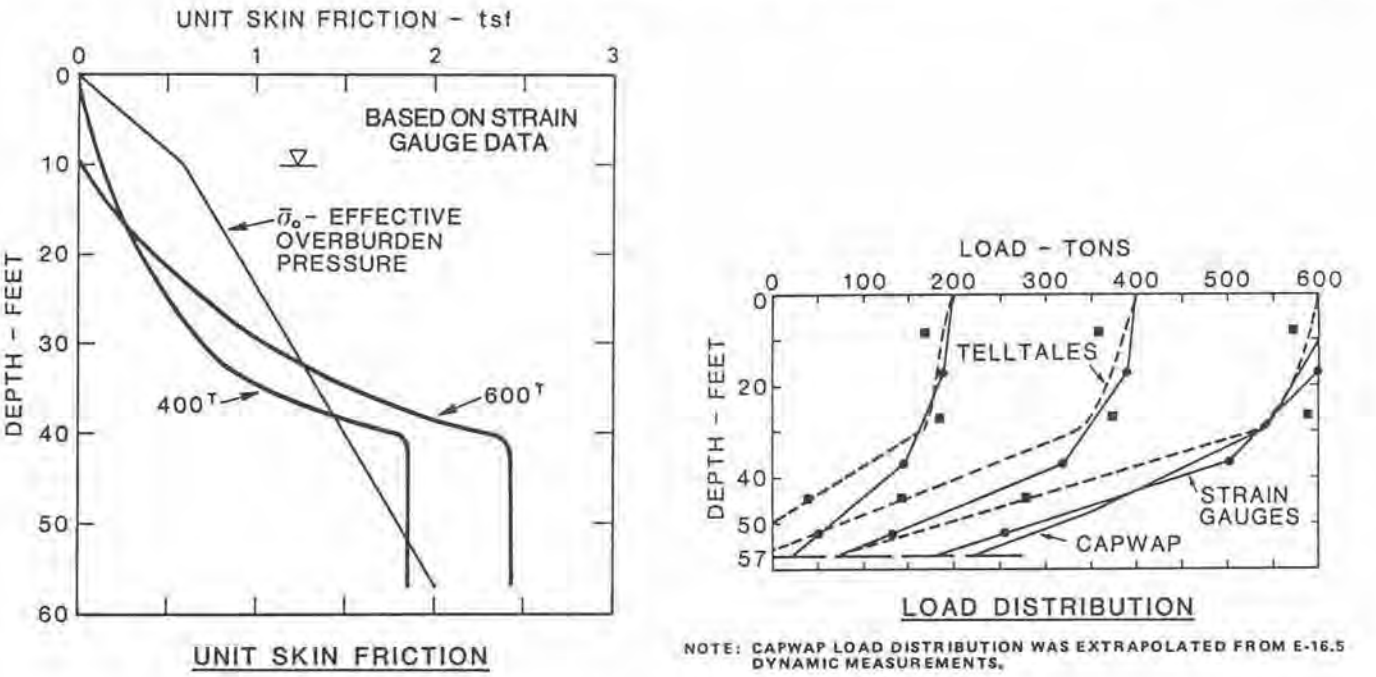

Figure 7. Site E 24-in. concrete load test pile (E-LTP).

Sites A and B - Alluvial Deposits

The magnitude and distribution of apparent skin friction, Q_s, were developed using the measured load transfer data and the following equation:

$$Q_s = p \times f_s$$

where: p = surface area of pile,

f_s = unit skin friction = $\bar{\sigma}_o K \tan\delta$,

$\bar{\sigma}_o$ = average effective overburden pressure along the pile shaft,

K = lateral earth pressure coefficient, and

$\tan\delta$ = coefficient of friction between pile and soil.

Estimated average Ktan values were about 0.37 at site A and 0.40 at site B. The ultimate apparent unit skin friction was about 1 tsf, which corresponds well with values recommended by Meyerhof (1976) and Reese and Wright (1977) for N = 40 to 50 bpf cohesionless soils.

The frictional angle, δ, between sand and a concrete block was about 31°, using direct shear equipment. Thus, $\tan\delta = 0.6$. Using this value, the K values at sites A and B would be about 0.62 and 0.67, respectively.

The measured point loads at failure were about 84 and 77 tons at sites A and B, respectively. The apparent end-bearing pressures would then be 25.7 and 23.6 tsf. These apparent end-bearing pressures are less than those calculated from the ultimate end-bearing capacity formula for deep foundations.

Test data obtained by Bozozuk et al. (1979) indicate that ultimate skin friction is fully mobilized when the pile tip settlement is about 0.16 to 0.2 in. Once the ultimate skin friction is fully mobilized, the frictional resistance may remain constant or may decrease, but it would not increase under additional applied load. Thus, the additional load will be transmitted to the pile tip. The amount of strain or deflection of the pile tip to develop end-bearing is believed to be greater than that required to develop the ultimate skin friction. In addition, Vesic (1977) and Hunter and Davisson (1969) found that after pile driving, residual stresses may remain in the soil around the pile and below the pile tip. We believe that the end-bearing capacity would be greater than that measured if residual stresses are considered.

Site E - Glacial Deposits

A plot of measured pile top and tip settlement versus applied load for a 24-in. concrete pile driven into glacially-overridden clay at site E is presented on Figure 7. Under the maximum test load of 600 tons the net settlement, after rebound, was 0.44 in. The ultimate load is estimated to be about 650 tons.

Table 2. Summary of pile capacities, West Seattle Freeway Bridge.

EOD, end of continuous driving; J, case damping value; CM, case method; R, restrike; CAPWAP, case pile wave analysis program; Setup (pile freeze) determined from EOD and R data; LTP, load test pile; A-16.5, 16.5-in.-diameter concrete pile at site A; B-NE, northeast anchor piles at site B. Piles other than load test piles (LTP) were anchor piles for the load frame.

Site	Pile	Penetration (feet)	Ultimate capacity (tons)	Method	Failure load (tons)
A	A-LTP	98.5	542(R) Setup=200t	CAPWAP	525
A	A-16.5	96	321(R) Setup=65t	CAPWAP	
B	B-LTP	83.5	352(EOD) 422	CAPWAP (assuming setup of 70 tons)	425
B	B-NE	72	423(R)	CM J=.15	
B	B-NE	77	357(EOD)	CM J=.15 (assuming setup of 70 tons)	
E	E-LTP	45	235(R)	CM J=.5	
E	E-LTP	57	No Data		>600
E	E-S(24) (Driving Test Pile)	50	376(R)	CAPWAP	
E	E-16.5	63	448(R)	CAPWAP	

The apparent skin friction between a pile and clay was determined by the following equation:

$$Q_s = p \times f_s$$

where: Q_s = shaft friction load,

p = surface area of pile,

f_s = unit skin friction = $c \times \alpha$,

c = undrained shear strength, and

α = reduction factor.

The apparent unit skin friction was about 2.43 tsf at an applied load of 600 tons below a depth of 40 ft, as shown on Figure 7. The apparent skin friction appears to be relatively uniform. The N-values were generally greater than 50 bpf in this zone.

The shear strength of the undrained glacially consolidated clays was determined by first consolidating undisturbed specimens under an effective confining pressure of 400 psi, the estimated preconsolidation pressure. The confining pressures were then reduced to three selected pressures and each specimen allowed to swell. The specimens were then sheared in an undrained condition. These tests resulted in an undrained shear strength of 6.5 tsf at an overburden pressure of 3 tsf.

The apparent unit skin friction, f_s, under the maximum test load of 600 tons, is about 2.43 tsf. Assuming c = 6.5 tsf, the reduction factor, α, is 0.38. Considering that the skin friction was almost fully mobilized under a test load of 650 tons, the reduction factor, α, would be 0.4.

As shown on the load transfer plot (Figure 7), the point bearing load, Qp, increased from 22 to 73 to 178 tons at applied loads of 200, 400, and 600 tons, respectively. These correspond to end-bearing pressures of 6.6, 22, and 54 tsf. The pile tip settlements were about 0.06, 0.15, and 0.49 in., respectively, indicating that the rate of pile tip settlement increased after the 400-ton load.

The ultimate end-bearing of driven piles in clay may be determined from the following equation:

$$Q_p = 9 \times c \times A$$

where: Q_p = end-bearing,

9 = bearing capacity factor for deep foundations in clay,

c = undrained shear strength, and

A = area of pile tip.

At an estimated load of 650 tons, the end-bearing pressure from load transfer plot is about 60 tsf. Using the above equation, the calculated c would be about 6.7 tsf, which is near the ultimate c value of 6.5 tsf as obtained from the high-pressure triaxial compression test.

Steel Pipe Piles - Sites C and D

The main span substructure piers are supported by 36-in.-diameter x 3/4-in. wall steel pipe piles, driven open-ended. These piles are designed for a 600-ton static dead plus live load, plus an additional 600-ton seismic load. It was considered not feasible to conduct a pile load test on a 36-in.-diameter pipe pile. Therefore, a 24-in.-diameter x 1-1/2-in. wall steel pipe pile driven open-ended was tested and the results used to analyze the 36-in. pile. The ultimate capacity of the 36-in. steel pipe pile which was used as an anchor pile for the load frame was also estimated by dynamic measurements.

As shown on Figures 8 and 9, sites C and D are underlain by fill and alluvial deposits consisting of sand, silt, and clay, below which are very dense sand and silt and glacially overridden deposits of interbedded hard clay.

Hammer Tests

Several hammers, including steam-powered Vulcan 060 (rated energy = 180,000 ft-lb/blow) and Commaco 300/5 (rated energy = 150,000 ft-lb/blow) and a Delmag D62-12 diesel hammer, were initially considered for driving the steel pipe piles. Wave equation analyses using a revised WEAP (Wave Equation Analysis of Pile Driving) program for the three hammers along with pile and subsoil combinations were performed by Goble & Associates, Inc. The results revealed that under the same rated energy, the hammer with a longer stroke would drive the long steel piles (150 ft) more efficiently. Thus, a single-acting Delmag D62-12 diesel hammer was selected. Based on the hammer strokes measured from a Saximeter, the range of average hammer energy for the last 5 ft of penetration was 112,000 to 156,300 ft-lb/blow.

Driving Penetrations and Resistances

All steel pipe piles were driven into the glacially overridden deposits. At site C, the piles penetrated 30 to 34 ft into the glacial bearing layer, while at site D the piles penetrated 52 to 63 ft (excluding closed-end piles) into the glacial bearing layer. The 24-in. closed-end pile at site D penetrated only 31 ft into the glacial bearing layer.

The driving resistances above the glacial deposits were on the order of 10 to 40 bpf at site C (Figure 8), and about 10 bpf at site D (Figure 9). These resistances increased with depth to more than 120 bpf when the piles were penetrating into the glacially overridden deposits.

At sites C and D the driving resistances of the 24-in. open-ended and closed-end piles were about the same magnitude above the glacially overridden deposits but the closed-end piles drove harder in the glacially overridden deposits.

Pile Load Tests

Pile load tests were performed on 24-in.-diameter open-ended steel pipe piles to form the design criteria for the open-ended 36-in. steel pipe piles.

The instrumentation system for steel pipe piles was the same as for the concrete piles. The instruments were installed after removing the soils inside the pile with a churn drill.

At site C, the compression test was terminated at an applied load of 750 tons when a shop weld on the 36-in.-diameter anchor pile (pile C-N) failed. The pile was then tested in tension to the proposed maximum test load of 800 tons.

At site D, the pile was loaded in 100-ton increments to 940 tons which was held for 12 hr. After unloading, the pile was reloaded to 1,050 tons when the load reaction frame tilted enough so that additional loads could not be added.

The results of the vertical pile load tests are presented on Figure 9 for site D. The measured pile deflections were less than the elastic pile compression, assuming that the total load is transmitted to the pile tip. Therefore, applied loads are probably resisted in skin friction, and end-bearing has probably not been mobilized under the maximum applied loads of 750 and 1,050 tons at sites C and D, respectively.

Ultimate Pile Capacities

The ultimate pile capacities as predicted from dynamic measurements and as determined from pile load tests are summarized on Table 3.

The piles at sites C and D did not fail under the maximum test loads. Pile D-LTP was loaded to 1,050 tons in compression and the predicted ultimate capacity, based on the dynamic measurements, was 1,125 tons, which we believe is reasonable. For pile C-LTP the predicted ultimate capacity of 826 tons may be low. Since the 24-in. load test pile and the 36-in. pile were driven to about the same depth, and since the surface area of a 36-in.-diameter pile is 1.5 times that of a 24-in.-diameter pile, the capacity of a 36-in.-diameter pile should also be about 1.5 times that of a 24-in.-diameter pile. On this basis, the ultimate capacities of the driven 36-in.-diameter test piles at sites C and D would be about 1,200 and 1,700 tons, respectively.

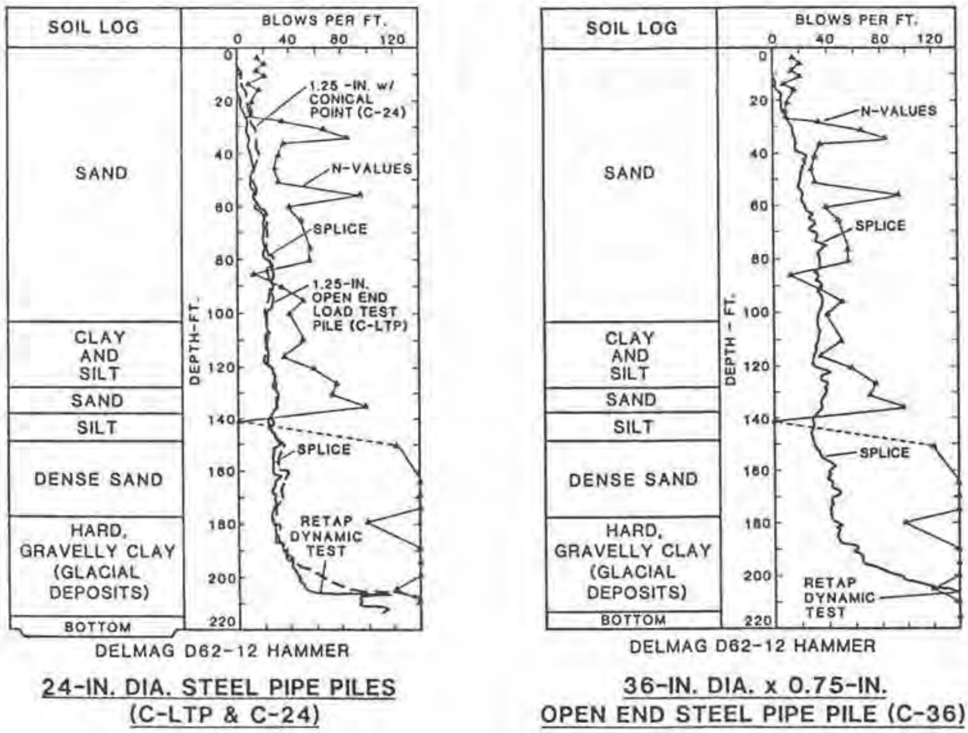

Figure 8. Site C pile driving resistances.

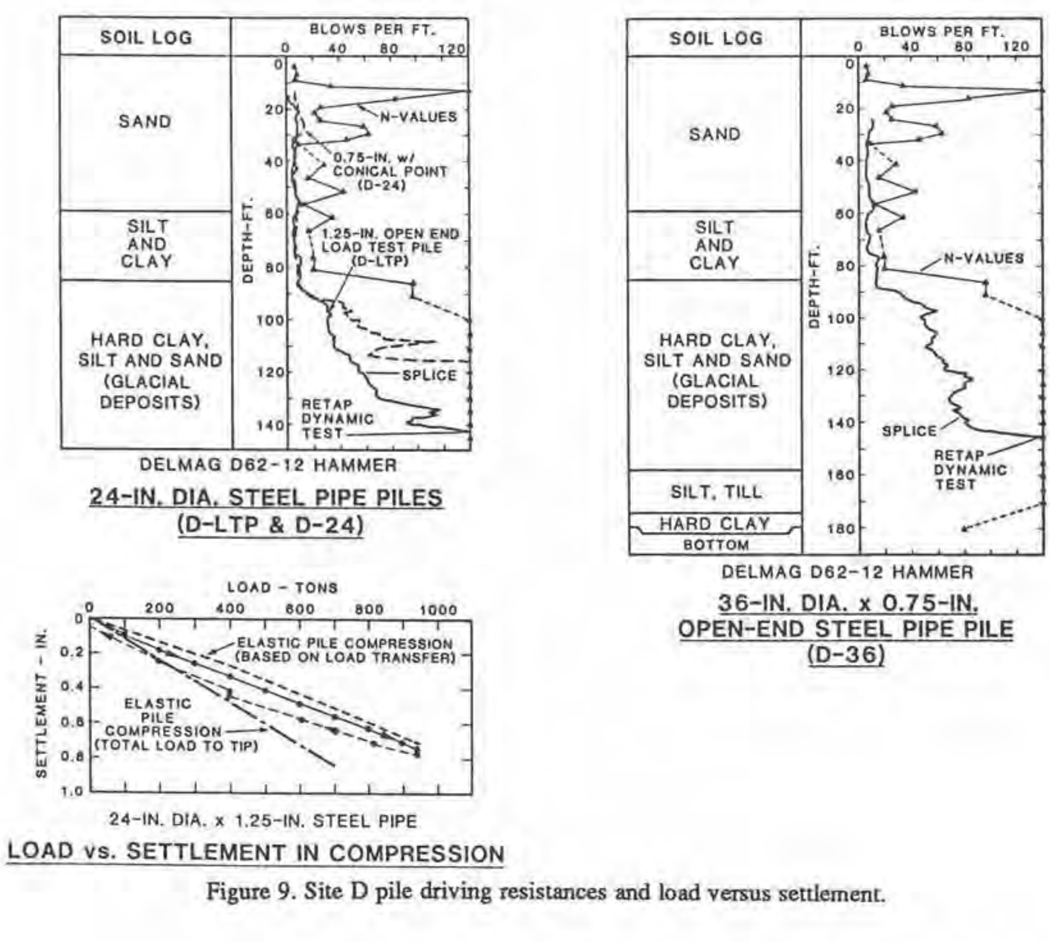

Figure 9. Site D pile driving resistances and load versus settlement.

Table 3. Predicted ultimate pile capacities from pile load tests, West Seattle Freeway Bridge; see Table 2 for explanation of abbreviations.

Site	Pile no.	Predicted capacity Dynamic measurements (tons)	Method	Applied Maximum test load Compression (tons)	Tension (tons)
C	C-LTP (24-in. dia.)	826	CAPWAP	750	800
	C-N (36-in. dia.)	1,150	CM	---	---
D	D-LTP (24-in. dia.)	1,125	CAPWAP	1,050	1,000
	D-E (36-in. dia.)	1,200	CAPWAP	---	---

Load Test Results and Interpretation

At site C, the average Ktanδ value for the alluvium was about 0.18 during compression and 0.15 during tension. At site D, the average Ktanδ value was about 0.16 during compression and about 0.10 under tension. Since these piles were not load tested to failure, the calculated Ktanδ values may not be the ultimate values. The load transfer data indicate that the majority of the applied loads was apparently taken up in skin friction only.

Distribution of unit skin friction for various applied loads is presented on Figures 10 and 11 for sites C and D, respectively, on the basis of our interpretation of the load transfer data.

A CAPWAP analysis was made on the load test pile at site D by Goble & Associates, Inc. The results indicate the ultimate compression capacity to be 1,125 tons, with an estimated end-bearing of 155 tons, indicating that almost all of the load was carried by skin friction.

Pile Installation Criteria

Specifications for pile installations were developed based on the results of the pile test program and subsurface data. In general, there were two main considerations. One was to estimate the necessary penetration to develop the design ultimate capacity, and the other was to develop a driving resistance to satisfy the dynamic pile driving formula which, in this case, was the WEAP program.

The pile penetration was based on the exploration data and the pile load test results. The estimated pile tip elevations were selected to have the tips in sand with a minimum of about 15 ft of sand below the pile tip to reduce the pile group settlement.

The driving criteria were based on the continuous driving resistance considering freeze factor. Tests and previous experience in the area indicated that the freeze factor for concrete piles ranged from about 2 to more than 5.

For the steel pipe piles, the specified pile penetrations were about 30 and 50 ft into the glacial deposits for the East Channel (site C) and West Channel (site D) piers, respectively.

Hammers other than those used in the pile test program were permitted. However, it was required that those hammers be calibrated using the dynamic pile analyzer.

CONSTRUCTION PILE INSTALLATION

The test pile program was accomplished after the conceptual design of the West Seattle Freeway Bridge Replacement project was developed, but prior to the final design. On the basis of the results obtained from the test pile program, the final pile penetrations for the concrete and steel piles were reduced by about 10 to 25 percent from the preliminary estimated lengths prior to the load test program. This resulted in a significant cost reduction for the project. During the bidding process, the geotechnical report, including the pile test data and results, was made available to all prospective contractors. The pile test data eliminated many questions concerning pile hammer selections and pile driveability. This may have contributed, in part, to the fact that the bid prices were less than the engineer's estimated costs for every project contract. The total savings in foundation costs were about $10 million.

CONCLUSIONS

A comprehensive pile load test program was accomplished during the initial design stage of this very large project. The information gained was beneficial in final design and preparation of contract specifications. It is also believed to have had an impact on the four contract bid prices, all of which were below the engineer's estimate.

Static empirical methods supplemented with local pile experience proved satisfactory for estimating pile

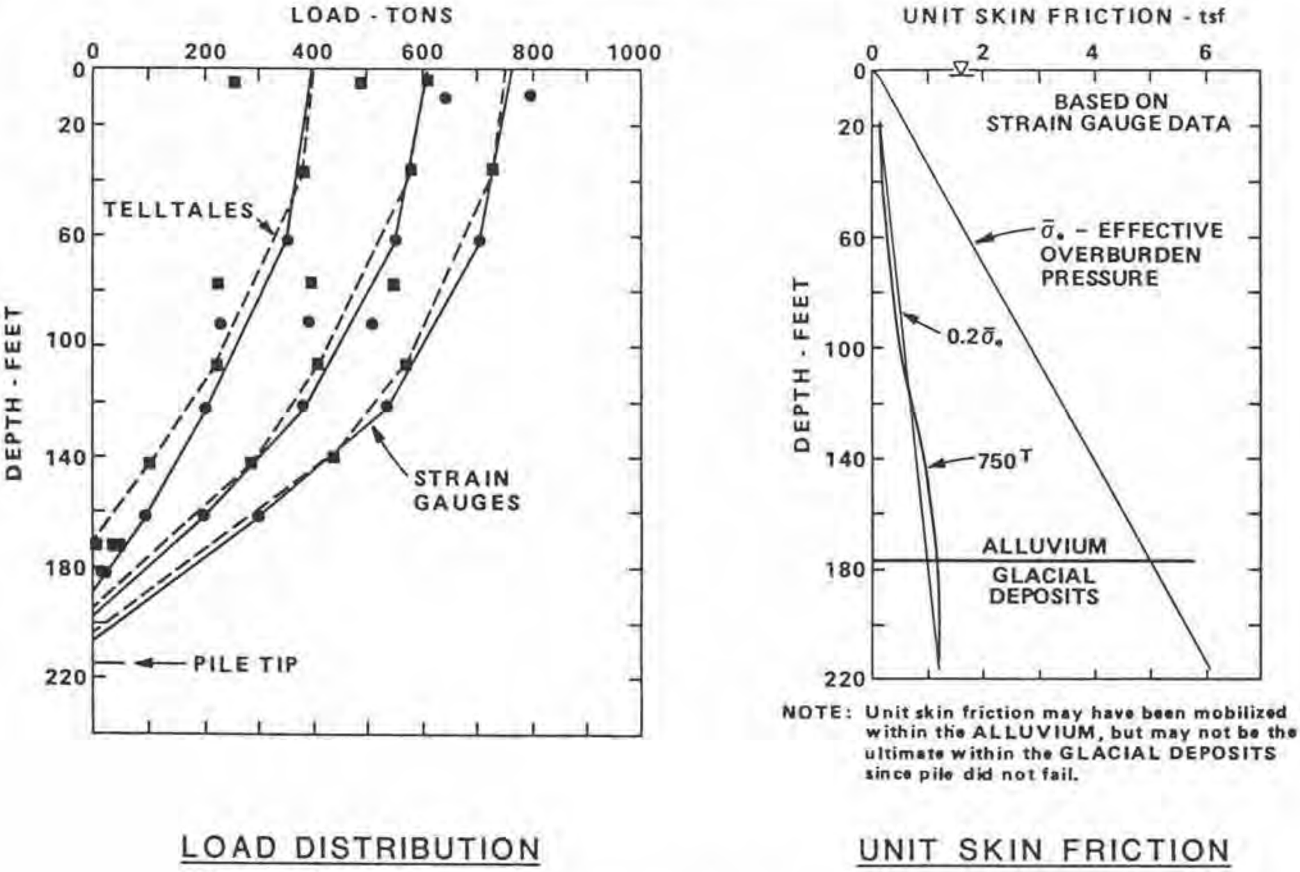

Figure 10. Site C 24-in. x 1.25-in. steel load test pile (C-LTP).

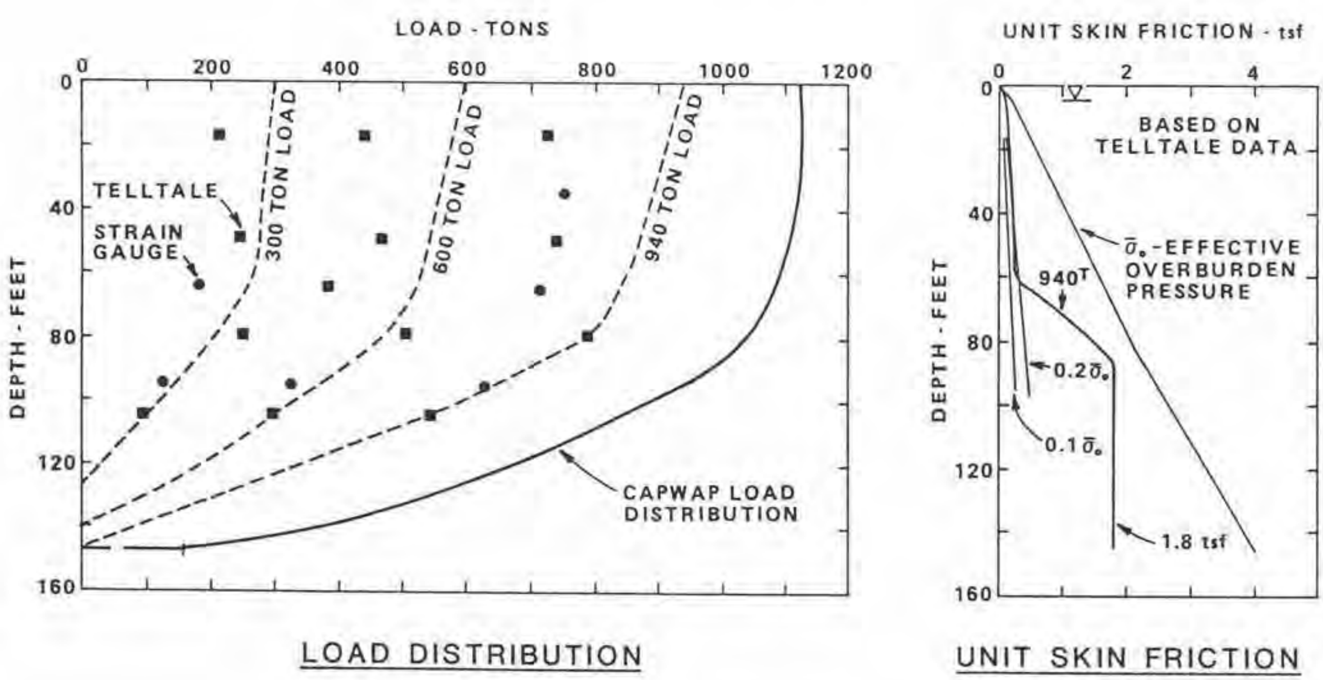

Figure 11. Site D 24-in. x 1.25-in. steel load test pile (D-LTP) load.

penetrations and pile capacities. However, instrumentation data indicated that the apparent frictional resistances and end-bearing capacities are different from those calculated from empirical formulas.

The WEAP program was very useful in selecting pile driving hammers that successfully drove the 24-in. concrete and 24- and 36-in. steel piles to the required penetration and desired capacities with the driving stresses below the ultimate stress of the pile material.

The pile analyzer was used extensively and was useful in the overall understanding of pile driving. It was used to evaluate hammer performance and energy delivered to the pile as well as to estimate capacity and provide soil constants for wave equation analysis for that particular pile and hammer. The pile analyzer was particularly suited to evaluate performance of diesel hammers.

ACKNOWLEDGMENTS

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West Seattle Bridge under construction across Harbor Island in 1986. Photograph courtesy of Shannon & Wilson, Inc.

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Engineering Geology of Loess in Southeastern Washington

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INTRODUCTION

Loess is a wind-deposited soil composed primarily of silt-size particles. While the most extensive deposits of this type in the United States are located in the Midwest, loess blankets the majority of southeastern Washington and extends into northeastern Oregon and northern Idaho. Traditionally, this part of the country has been a fertile farming area and has thick, rich soils.

Loess is characterized by its loose structure and is composed of silt and fine sand particles coated by a clay binder. In some places, this structure allows vertical or near-vertical cuts exceeding 50 ft in height to perform exceptionally well, provided the water content remains low. Conversely, upon wetting, loess becomes relatively unstable, and slump failures can occur in slopes as flat as 2H to 1V (Missouri State Highway Department, 1978).

Erosion in loess is a common problem resulting from its structure and therefore has been the subject of some study by the agricultural and engineering communities, especially in the midwestern United States. Some of the most severe erosion in the United States is in southeastern Washington where losses of 1.0 to 1.4 psf/yr are common and 4.5 to 9.0 psf/yr occur on some steep slopes. In addition to the loss of fertile soil for agriculture, the sediment load added to streams is extremely high.

Recent emphasis on soil conservation in southeastern Washington has prompted farmers and government agencies to examine their routine practices. Because of this, the Washington State Department of Transportation (WSDOT) is concerned with the performance of their cut slopes along the highways and county roads that cross the loess area. Some of these cut slopes have been rapidly deteriorating due to erosion and/or slope failures.

In the summer of 1984, the WSDOT and the Federal Highway Administration began a research program to investigate the design and performance of cut slopes in loessial soils. This paper is based on the first phase of the research (Higgins et al., 1985).

The objectives of this paper are twofold: (1) to summarize the physical properties of southeastern Washington loess, and (2) to summarize the failure mechanisms of cut slopes in this loess deposit.

The conclusions of this paper are based on a reconnaissance study of roadcuts which included observations of slope degradation processes and the analysis of 46 soil samples to determine physical properties of the deposit. Sample locations are shown in Figure 1. An effort was made to pick sampling sites evenly spaced throughout the study area to enable the authors to estimate trends in the physical properties of the deposit and to compare these properties with observed engineering behavior, for example, erosion and slope failure. Samples were collected from road cuts by hand augering horizontally into the face of the cut. Vertical samples were taken at the road cuts to verify the lack of grain-size variation in the upper part of the deposit; however, deep sampling from boreholes was not performed.

INDEX PROPERTIES OF SOUTHEASTERN WASHINGTON LOESS

Deposit Boundaries and Origin

As noted, a loessial deposit blankets the majority of southeastern Washington and extends into northern Idaho and northeastern Oregon. Traditionally the deposit has been subdivided into four units: Palouse, Nez Perce, Ritzville, and Walla Walla. The earliest engineering reference to these subdivisions was by Eske (1959). The unit boundaries are shown in Figure 1. These units evidently are based on pedological classification; therefore their value with respect to engineering properties is questionable.

The most likely source materials for the southeastern Washington loess deposit are the Ringold Formation and the fine waterlaid sediments of the Touchet beds lo-

Figure 1. Areal distribution of southeastern Washington loess (Eske, 1959) and sample locations for the field study.

cated west of the deposit. A discussion of the various thoughts on origin is included in Foley (1982).

Grain-size Variation

The range in grain-size distribution of the 46 samples examined is shown in Figure 2 and corresponds closely with the boundaries established by Holtz and Gibbs (1951) for Missouri River Valley loess deposits (Figure 3). Using Holtz and Gibbs' classification (Figure 3), 10 percent of the samples were classified as sandy loess, 68 percent as silty loess, and 22 percent as clayey loess.

In a loess deposit, clay-size content, water content, and density all tend to increase with distance from the source. The change in grain size-distribution is a primary indicator of variation in engineering properties and behavior with location for loessial soils (Holtz and Gibbs, 1951; Missouri State Highway Department,

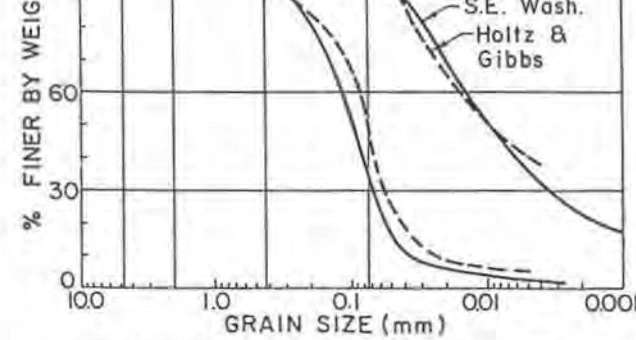

Figure 2. Range in grain-size distribution for 46 samples of southeastern Washington loess. Compare this range with the range shown in Figure 3.

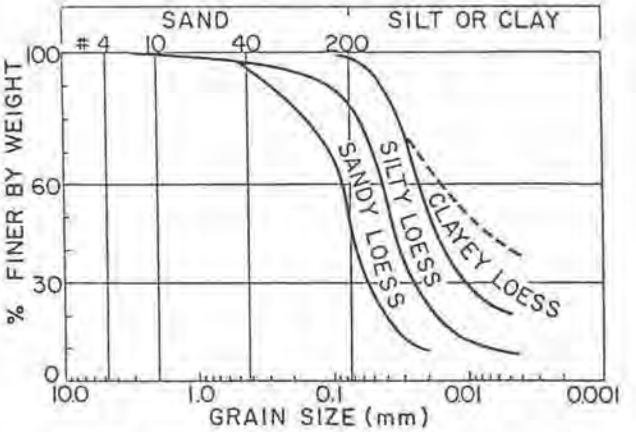

Figure 3. Range in grain-size distribution and classification of Missouri River Valley loess. After Holtz and Gibbs (1951).

1978). Therefore, definition of directional variation within the deposit becomes extremely important.

The 46 samples tested supply sufficient data to establish general trends within the deposit that should be helpful to engineering geologists and geotechnical engineers. Figure 4 shows the locations of three cross-sections constructed through the deposits. In Figure 5, the east-west cross-section A-A′, a general increase in clay-size content and a decrease in sand to the east are shown. In Figures 6 and 7, the north-south cross-sections B-B′ and C-C′, fairly constant clay-silt-sand ratios are revealed; there are only local fluctuations. However, cross-section C-C′ shows a higher clay-size content and lower sand content than B-B′.

Clay-size content (<0.002 mm) was contoured for the study area in Figure 8. Although anomalies are present, a definite trend of increasing content of clay-size material to the east is apparent. As established in Figures 5, 6 and 7, sand content decreases from west to east. As shown later, the higher clay-size content along the Washington and Idaho border affects the engineering performance of cut slopes.

None of the data collected supports the use of unit boundaries (shown in Figure 1) as engineering units. Although the subdivisions may be useful from an agricultural or pedological standpoint, the data clearly demonstrate that with regard to the properties mentioned earlier, variations in engineering properties can be expected to show east-west trends independent of unit boundaries. Figure 9 shows approximate boundaries of loess types (using Holtz and Gibbs' classification scheme, Figure 3) for southeastern Washington loess based on the 46 samples.

Figure 10 shows a typical vertical profile at a sampling site determined from five evenly spaced sampling points. Little variation in the relative percentages of the sand, silt, and clay fractions was detected. Although the percentage of sand-size particles varies by less than 1 percent, the amount of clay-size material (<0.002 mm) varies by as much as 5 percent, which is still relatively minor for a natural soil.

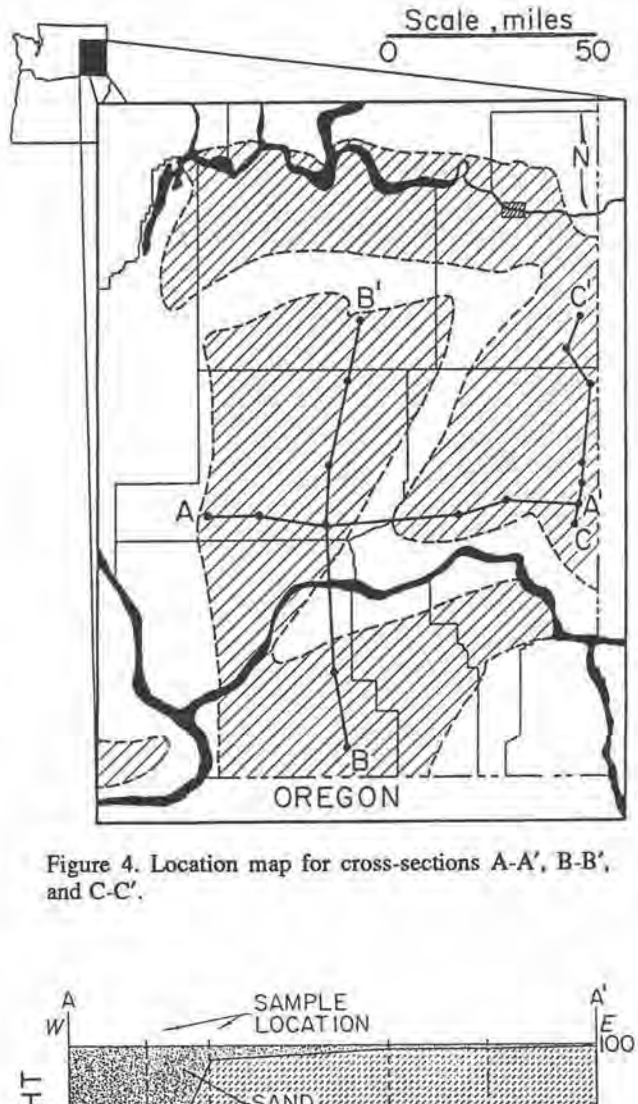

Figure 4. Location map for cross-sections A-A′, B-B′, and C-C′.

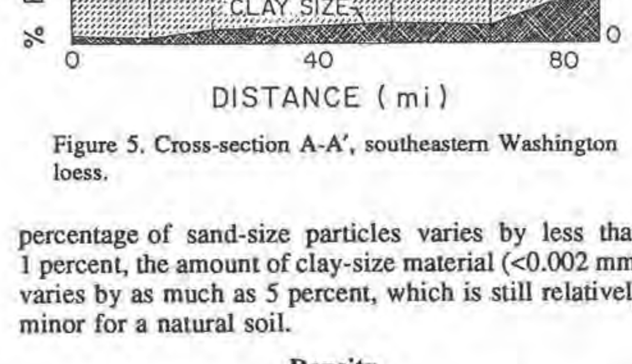

Figure 5. Cross-section A-A′, southeastern Washington loess.

Density

Although *in situ* densities were not measured, limited data are available from a previous investigation. Lobdell (1981) reported dry densities ranging between 95 and

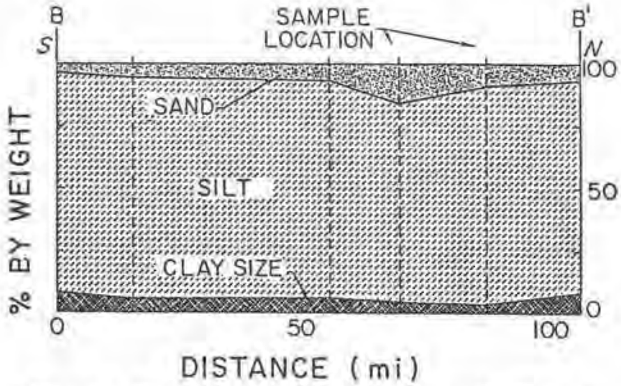

Figure 6. Cross-section B-B', southeastern Washington loess.

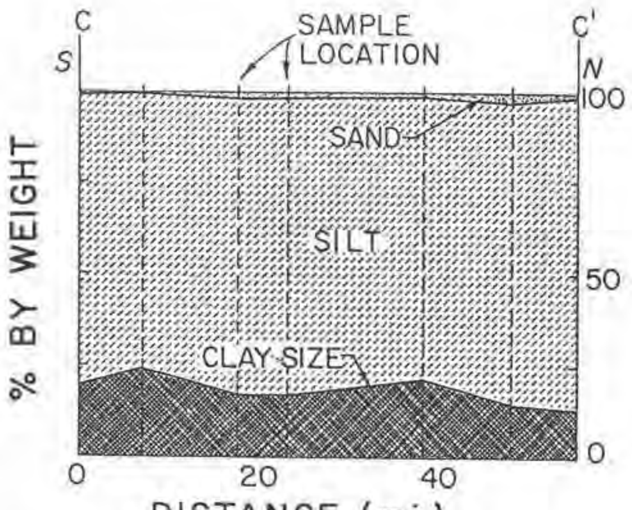

Figure 7. Cross-section C-C', southeastern Washington loess.

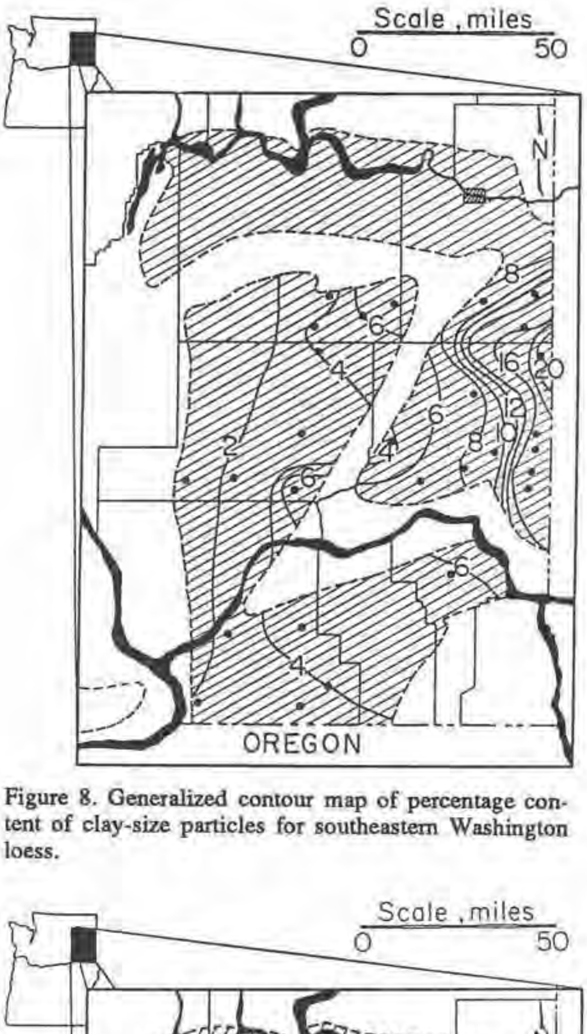

Figure 8. Generalized contour map of percentage content of clay-size particles for southeastern Washington loess.

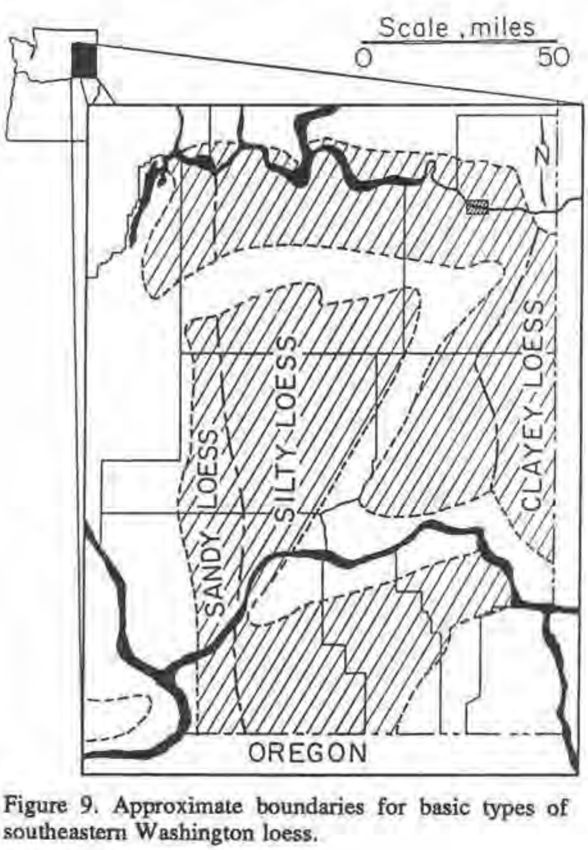

Figure 9. Approximate boundaries for basic types of southeastern Washington loess.

98 pcf in Palouse loess. These densities were determined from 6-in. block samples from an excavation site at Washington State University. Thus, the values Lobdell reported should be an accurate indication of *in situ* dry density for the eastern extreme of the deposit. Since this portion of the deposit has a higher clay content, it has a somewhat higher *in situ* density than the silty loess to the west. Thus, it is assumed that the density data reported by Lobdell (1981) provide an upper bound for the deposit.

Water Content

Data for natural water content were obtained for 22 sites throughout the study area. Water content ranged from 4.5 to 27.7 percent. As might be expected, water content shows a high degree of variation from location to location. Even so, water content generally tends to increase from west to east. The directional variation in water content may be attributed to two factors. First,

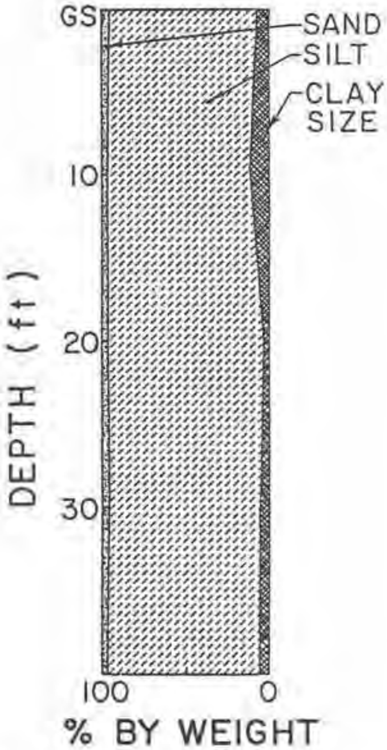

Figure 10. Typical variation in textural composition with depth for southeastern Washington loess.

content of clay-size material increases from west to east, as determined by the grain-size analysis. Water content should be expected to increase with increasing clay-size content. Second, mean annual precipitation tends to increase from west to east by as much as 100 percent.

Plasticity

Plasticity characteristics were evaluated by the Atterberg limit tests. A representative sample of the plastic and liquid limit tests are plotted in Figure 11. Examination of the plot and the grain-size distributions reveals two groupings of the data. Silty loess tends to have liquid limits ranging from 24 to 32 and plasticity indexes of O (nonplastic) to 11. Clayey loess has liquid limits that range between 33 and 49 and plasticity indexes ranging from 11 to 27. The two sandy loesses plotted on Figure 11 are nonplastic.

Calcium Carbonate

During field sampling, various forms of caliche (calcium carbonate) were encountered. In most places caliche was present as either root fillings or nodules. To a lesser extent caliche was found as indurated sheets lying parallel to and 6 in. to 1 ft behind the surface of a cut face. This appears to be a phenomenon related to evaporation and is similar to calcium carbonate occurrences noted in Mississippi (Krinitzsky and Turnbull,

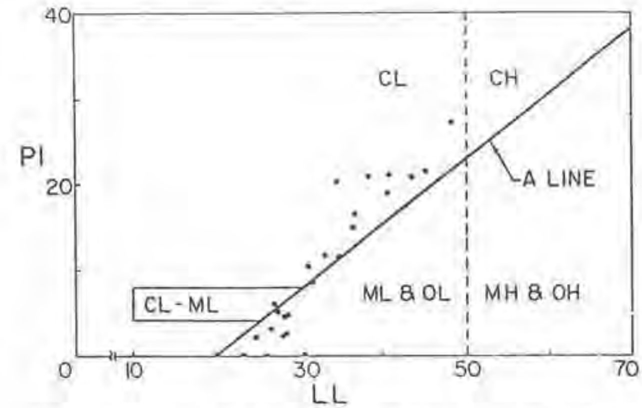

Figure 11. Plot of plastic index (PI) vs. liquid limit (LL) for southeastern Washington loess.

1967). Because of the discrete nature of most caliche deposits, the absence of caliche in an area where it might be expected does not necessarily imply that it is absent from the area as a whole.

The presence of caliche appears to be related to mean annual precipitation. In areas averaging more than 20 in. of precipitation per year, caliche was not present in the samples collected. Conversely, a large majority of samples collected in the area where mean annual precipitation is less than 15 in. contained significant quantities of caliche.

FAILURE MECHANISMS ASSOCIATED WITH SOUTHEASTERN WASHINGTON LOESS

Between July 1984 and July 1985, roadcuts in loess along the major highways crossing southeastern Washington were examined for deterioration. Although this was a short-term project including only one spring season, a sufficient number of slope failures or deteriorations was observed to define the most common problems with cut slopes in the study area. Deterioration of these cut slopes can be attributed to three factors: erosion, shallow slides or flows, and rotational slides.

Erosion Damage in Silty Loess

Erosion (including piping) is by far the most common form of slope degradation noted by the authors in silty loess of the study area. In many places large erosion gullies are initiated by small piping failures originating 5 to 10 ft behind the top of the cut face. As the pipe enlarges, the surface tends to cave, forming a gully. Once a gully forms, running water enlarges it rapidly.

In virtually all instances examined, major erosional features were found to be the result of insufficient drainage around the cut slope or insufficient channel protection. The vast majority of serious erosion problems was observed in long cuts transecting small drainage areas where no provisions were made for conveying excess runoff away from the slope face. The highly erosive nature of silty loess requires the diversion of runoff from what would normally be considered insignificant drainage areas.

Figure 12 illustrates the result of truncating a small drainage area without providing a means of conveying excess runoff from the slope or providing any erosion protection on the slope face. Although this is a rather small cut, the principle is well illustrated. During the late spring of 1985 numerous examples of this type of failure were observed in all sizes of cuts and in both silty and clayey loess.

Figure 13 illustrates silty loess near Walla Walla and demonstrates the progressive nature of erosion in loessial soils. Erosion was initiated where this small side drainage was truncated by the highway cut to the right and has progressed rapidly up-gradient. Erosion will continue until the overall gradient is decreased below that causing channel scour. Although this photograph illustrates the erosion of a natural channel, it also serves as an example of the high erodibility of silty loess where flow is concentrated; therefore, the need for channel protection for a surface drainage system is obvious.

In the cut shown in Figure 14, the majority of the erosion in the upper portion of the cut slope is due to piping. Below the pipes, gully erosion is beginning to develop. When the pipe enlarged to the point where the overlying material could no longer be supported, caving occurred, and the overlying vegetation was deposited in

Figure 12. Erosion of loess by flow over the face of a cut (near La Crosse).

Figure 13. Gully erosion in a side ditch due to insufficient channel protection (U.S. Highway 12 near Walla Walla).

Figure 14. Erosion pipes developing into gullies on a 30-ft-high cut along U.S. Highway 12 near Walla Walla.

the depression. Piping was found to be a very common factor in the formation of erosion gullies in silty loess.

Figure 15 shows the rather small, gently sloping drainage area above the erosion illustrated in Figure 14. Extensive erosional problems may develop from extremely small drainage areas due to the fact that silty loess is highly susceptible to rapid erosion.

The above observations indicate slope erosion can become severe where runoff is allowed to flow over the cut face or concentrate near the top of the face. In these places piping and gullying of the cut face begin rapidly. Erosion may be extensive where flow is concentrated. From this experience, it appears that maintaining the natural slope of the land and allowing land use (farming) close to the top of the cut is not a good practice. Instead, due to the high erosion potential on the cut face, surface drainage should be an integral part of the cut slope design. Diversion of flow away from the cut face should prevent rapid deterioration by piping and gullying. The diversion channels should be protected from gullying. In the same area as that shown in Figure 14 were several large, near-vertical cuts where the natural drainage diverted water away from the cut face. These cuts showed little erosion even though they were of the same age and material as the adjacent eroded slopes.

One of the major problems in designing runoff drainage systems in loess is determining the degree of channel protection necessary for a given gradient. This choice is usually based on experience.

Figure 15. Low area at the center of the photograph is the runoff source causing the erosion shown in Figure 14. Cut slope is on the left.

Another serious erosional problem in loessial soils, more specifically on flattened slopes, is surface erosion during construction and until the establishment of a good vegetative cover. Extensive and rapid damage in the form of deep rills was observed on new slopes cut in silty and clayey loess. Even with adequate drainage, extensive damage may result from raindrop impact and rill erosion. Although these observations were made with respect to cut slopes, the general principles should be applicable to any earth construction project in loess.

Shallow Slides and Flows

Shallow slides and flows were observed in the eastern part of the study area primarily within the clayey loess zone where most of the larger cut slopes have been cut at approximately 2H to 1V. This type of failure is largely due to late winter and early spring climatic conditions. It is common to get precipitation or snowmelt at this time of year, which results in a thin layer of thawed, saturated soil overlying either frozen or unsaturated soil. The layer of thawed, saturated soil along with any overlying vegetation tends to slide (or flow) downslope.

The form and frequency of this type of failure in the study area are primarily due to the amount of precipitation, clay-size content, and slope angle. In the western two-thirds of the study area, where precipitation is less than 15 in. annually, shallow slides or flows were rarely observed. Clay content increases from west to east, as does precipitation. Increases in clay content (which result in lower permeability and lower soil strength) combined with increased precipitation, raise the likelihood of the near-surface saturated conditions required for failure.

Two forms of shallow slope failure were observed and appear to be related to soil type. In silty loess it is not uncommon to observe sheets of vegetative cover and soil, 2 to 6 in. thick with an arcuate upper boundary, move downslope. In this form, only minor damage was observed; no cases of extensive degradation were noted. In contrast, in clayey loess shallow slides and flows are common and result in major slope degradation. Movement appears to be a mudflow phenomenon, and both large- and small-scale failures are observed. At least minor damage was observed in the majority of clayey loess slopes more than 10 ft high, and numerous cuts experienced major loss of vegetative cover during the spring of 1985. Small-scale mudflows in clayey loess, such as in Figure 16, are by far the most common form of failure. Failures are typically 1 to 10 ft wide, and the depth of failure ranges between inches and 4 ft. In most places the initial failure is followed by increased erosion due to the loss of vegetative cover. Severe gully erosion is a common result.

Although not as common, larger slides and/or flows have been observed. The failure mechanism is thought to be the same as for smaller scale failures. There is a low depth-to-width ratio for the failure surface. Figure 17 illustrates a larger failure.

Figure 16. Small flow of soil and vegetation in a 15-ft-high cut slope along U.S. Highway 195 near Pullman.

In all places where extensive damage from slides or flows was observed, slopes were 2H to 1V or steeper. No instances of failure were observed for slopes flatter than 2.5H to 1V. These observations agree with those of past investigations of similar deposits, which found slopes greater than 2.5H to 1V too steep to maintain good vegetative cover in loessial soils (Missouri State Highway Department, 1978; Royster and Rowan, 1968).

Exposure has an influence on these shallow failures. It is not uncommon to find that two opposing cuts with similar slopes and drainage, one facing north and the other south, exhibit extremely different performance. The north-facing slope will invariably demonstrate a greater degree of degradation due to shallow slides and flows than the south-facing slope. This is not surprising, as slopes facing north typically have higher average water contents (due to the lack of exposure to sunlight and the resultant evaporatuion) than those having any

Figure 17. Failure of cut slope in clayey loess near Pullman. White areas are snow. Cut is about 30 ft high.

other orientation. Figures 18 and 19 show south- and north-facing slopes (respectively) cut in clayey loess at 2H to 1V. Note that Figure 18 shows that some erosion has occurred in the area where small patches of vegetation and soil have slipped away. However, the north-facing slope (Figure 19) has experienced numerous shallow failures which have stripped the vegetation. Obscured by the snow drift are scarps from larger failures, which have occurred over the past several years, that have significantly steepened the crest of the slope. The result of the shallow slides is exposure of bare soil and rapid formation of erosion gullies, some of which range up to 1.5 ft in depth in the slope shown in Figure 19.

The practice of constructing drainage ditches immediately below the toe of slopes appears to contribute to shallow slope failure also. Periodic highway maintenance requires removal of sediment from the ditches, which may result in undercutting the toe. Additionally, drainage ditches located directly adjacent to the toe can raise the water content (during wet periods), further encouraging failure. This problem may be reduced by placement of drainage ditches 10 to 15 ft away from the toe of the slope.

Rotational Failures

Although only one rotational failure was observed during the field study, other investigators have reported such failures (Hardcastle; 1984; McCool, 1984). These failures occurred in clayey loess in the extreme eastern section of the study area. Failures of this type have taken place primarily during the spring when soils were at or near saturation. Rotational failure surfaces may be rare because loessial soils in the study area generally do not reach saturation at depths much greater than 3 ft. Although complete saturation is not a requirement for producing a rotational failure in loess, it is likely that near-saturated conditions are necessary to reduce strength sufficiently to produce a deep-seated failure of any geometry.

The rotational failure observed during the spring of the field study was a slump/earth flow in a natural slope in clayey loess soils. The slope was inclined approximately 2H to 1V, the head scarp was approximately 15 ft high, and the failed material flowed about 50 ft from the toe of the scarp (Figure 20). The failure occurred where runoff and ground-water seepage from the surrounding field could be expected to collect and saturate the soils.

Figure 18. South-facing slope near Pullman shows little degradation. Cut is about 60 ft high.

Figure 19. North-facing slope (counterpart of slope in Figure 18) exhibits slide scar and severe gully erosion.

Figure 20. A slump/earthflow in clayey loess near Pullman.

CONCLUSIONS

Preliminary analysis indicates that the index properties of southeastern Washington loess are similar to those of midwestern loess. The range in grain-size distribution is essentially the same as for Missouri River Valley loess. Laboratory test results suggest some trends in index properties for southeastern Washington loess. These are: (1) a decrease in mean grain size from west to east, which is reflected in a decrease in sand and an increase in clay and water content; (2) constituent percentages of sand, silt, and clay remain relatively constant from north to south.

Most of the degradation of roadcuts in southeastern Washington loess was due to two problems. Where the deposit is classified as silty loess, inadequate erosion control was the primary cause of failure or deterioration. In the eastern extreme of the study area, oversteepened cuts in the clayey loess have resulted in severe loss of vegetative cover for many slopes due to shallow slides and flows. This has resulted in accelerated gully erosion on bare slopes.

DESIGN RECOMMENDATIONS

On the basis of the field observations, near-vertical cut slopes (1/4H to 1V) should perform well in silty loess if surface drainage is diverted away from the slope face and water content remains low. In clayey loess, near-vertical cuts have ravelled rapidly, and many cuts of 2H to 1V have been too steep to maintain a protective vegetative cover. The ability of the slope to maintain a vegetative cover is greatly increased when flattened to 2.5H to 1V.

The keystone of successful erosion control is providing adequate drainage above the cut. This should be accomplished by constructing a drainage ditch 10 to 15 ft behind the top of the cut, preferably before the cut is opened. The purpose of this ditch is to prevent flow over the slope and to drain water away from the head of the slope in order to prevent saturation and piping failure. Due to the highly erosive nature of loess, erosion control within the drainage ditch requires, at a minimum, heavy seeding. Synthetic liners or asphalt are necessary at even moderate gradients. In most instances where long lateral cuts are employed, side ditches must be constructed to convey excess runoff away from the slope. Side ditches are generally steeper than lateral drainage ditches (above the crest of the cut) and will usually require a protective liner.

Where flattened cuts are utilized, slope protection must be initiated immediately following construction. The most widely used method is providing a heavy straw mulch cover as soon as seeding has been completed, but synthetic mats may work well also.

ACKNOWLEDGMENTS

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Rural Development and Land-Use Planning

Thomas E. Koler and William D. Evans, Jr., Chapter Editors

Rural Development and Land-Use Planning: Introduction

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During the first century of Washington's statehood, the practice of engineering geology has been an integral part of rural development and land-use planning. In Washington State's first 50 years, our branch of earth science was primarily used in the mining and railroad industries. In the last 50 years, further diversification of engineering geology has occurred in our state with the increased use of our specialty in the forestry and agricultural industries. The practice of engineering geology in these rural industries and associated land-use planning has been crucial in our state's rural development. Although rural engineering geology in Washington has a wide range of applications, several are common: slope stability analysis, locating and developing geologic materials for road or railroad construction, ground-water location, and water-supply development. The papers presented in this chapter, therefore, have a common thread and at the same time demonstrate the diversification of the practice of engineering geology in Washington.

Engineering geologists from the private sector and state and federal agencies in Washington have written the six papers presented in this chapter. The first five papers present case histories that show how some geologic conditions can affect rural development and land-use planning. The last paper describes a novel way of using geological data for planning purposes in national and private forest lands, data which can later be incorporated into a geographic information system. Since forestry and agriculture are the dominant rural activities in this state, much of this chapter is devoted to the practice of engineering geology in these two industries.

In the first paper, Robert L. Logan of the Division of Geology and Earth Resources, Washington Department of Natural Resources, discusses the implications of geologic hazards in rural development through a case history of the Marblemount landslide in northwestern Washington. Logan gives an interpretation of how the cause and effect of the movement of a large mass of slope material can be influenced by forest practices and the litigation that could follow, even under the best land-use planning.

Robert L. Schuster, Alan F. Chleborad, and William H. Hays, the authors of the second paper, are with the U.S. Geological Survey in Denver, Colorado. Their work deals with the evaluation of a series of large landslides along the Columbia River in south-central Washington, which they refer to collectively as the White Bluffs landslides. The authors' initial concern was that landslides in this area would block the Columbia River, and they felt that the movement of these large slope features could adversely affect nuclear facilities on the river's west bank, as well as a future dam and its reservoir. After their evaluation, however, the authors found that these potential risks were low. Their investigation showed that the landslide activity was directly related to agricultural irrigation wetting previously dry slopes. A caveat is therefore given by the authors to planners of future irrigation projects to avoid or mitigate wetting of dry slopes.

Richard D. Eckerlin, author of the third paper, is an engineering geologist with the U.S. Army Corps of Engineers, Seattle District. His case history describes the Bridgeport landslide along the Columbia River in north-central Washington. As with the White Bluffs landslides, the Bridgeport landslide's movement is influenced by soil wetting through the use of irrigation water. However, since the Chief Joseph Dam and Reservoir are just downstream of the landslide, reservoir level fluctuation is an added potential cause of slope instability. Although Eckerlin does not predict any risk to the dam and reservoir as the result of continued movement of the Bridgeport landslide mass, he does give attention to the possible danger (albeit low) of a large wave in the reservoir generated after a rapid mass movement of this large landslide.

Stanley H. Duncan, environmental geologist for Weyerhaeuser Company, is the author of this chapter's fourth paper. Duncan gives a case history of a timber harvest planning project in southwestern Washington. For this project he used a cost-effective slope stability assessment that he has developed. The major concern for this timber sale planning project was slope stability in an area adjacent to a sensitive fisheries resource. This paper shows how an interdisciplinary team composed of a geologist, hydrologist, and logging engineer utilized a terrain evaluation map in conjunction with Duncan's empirically developed slope stability index to produce a landscape unit map. With this map the team was able to classify each landscape unit by its relative risk of slope failure from the construction of roads and landings. The result of this analysis provided management with the information to determine what level of engineering was required to minimize landslides within this environmentally sensitive area.

The fifth paper, co-authored by Koler and Kenneth G. Neal, engineering geologists assigned to the Olympic National Forest in northwestern Washington, provides case histories of work on two overlapping timber sale areas in the Quinault Ranger District. The purposes of this paper are to show how engineering geology is practiced in the Olympic National Forest and to give a history of how the transition has been made from the "hand-cranked" to the computer-assisted method of evaluating slopes for stability.

Thomas K. Reilly, who has been an engineering geologist on the Olympic and Gifford Pinchot National Forests and is presently the zone engineer for the Siskiyou National Forest, wrote the last paper in this chapter. Reilly describes how an engineering geologist can provide geologic data in a "user-friendly" way to non-geologists who need the data to complete their tasks for timber sale planning. In this paper are examples of how this tool was used in the Gifford Pinchot National Forest in southwestern Washington. As this method evolved during the early 1980s, Reilly guided it so that it can be used for a geographic information system once one is implemented by the U.S. Forest Service.

Carefully applied engineering geology methods allow for proper design of forest roads and timber harvest units. Note clear-cut area in background and the established plantation in the foreground.

Slope Stability and Rural Land Management: The Marblemount Landslide Case History

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INTRODUCTION

Geology, physiography, and climate vary widely in Washington and strongly influence rural land management within the state. Agriculture is the major type of rural land management and includes timber production and other kinds of crop farming. The timber industry in Washington generally uses areas of rugged terrain, while food-production agriculture is more commonly associated with areas of relatively subdued topography. Mass wasting manifests itself differently in these environments. Erosion by sheetwash and gullying is more commonly associated with general farming practices, whereas earthflow-slumps and debris torrents are typical of steeper, forested terrain. Conditions such as soil fertility and availability of ground water are important to agriculture, but slope stability is critical to management of the steeper terrain typical to large tracts of forest land throughout Washington.

Road construction and maintenance and tree harvesting are integral parts of timberland management. These activities influence slope stability by altering natural conditions, such as mass balance and drainage. Consequently, risks associated with slope stability are of great concern to rural land managers, especially those managing commercial timberlands. Because improper forest practices can contribute to slope failures on timberlands, the question of liability in rural land management is becoming a major issue. This is especially true when landslides originating in timberlands impact adjacent populated areas.

Proper management of forested slopes can reduce the number of slope failures, but this requires a thorough knowledge of slope processes. Progress toward improving our understanding of slope processes has been slow in Washington. Even a systematic inventory of data related to slope process is still in its infancy. Some of these inadequacies can be attributed to lack of professionals with the appropriate technical background in forest slope stability on the staffs of involved agencies or companies. Many timberland managers have only recently become aware of the effect that their industry has on slope stability, and there seems to be some confusion about what type of professionals to hire to help identify and mitigate the problem.

From an historical viewpoint, it is easy to understand why the problem of unstable slopes has not been addressed until recently. It was not until the Forest Practices Act of 1974 (Chapter 76.09 Revised Code of Washington [RCW] and Washington Administrative Code [WAC] 222) were implemented that a comprehensive forest practices plan began to evolve for Washington's non-federal lands. This law and its revisions have increased accountability for landslide hazards among timberland managers, but sound mitigation programs are only in beginning stages.

Prior to the act, many miles of roads were built, used, and abandoned. These roads are often referred to as orphaned roads. Orphaned roads, although state-of-the-art at the time of construction, were rarely built or abandoned to modern standards. To properly abandon a road to current standards is to leave it in a condition suitable to control erosion. This may involve outsloping, water barring; blocking to prevent access by four-wheel-drive vehicles; removal of bridges, culverts, and fills; and final approval of abandonment by the Department of Natural Resources (DNR) (WAC 222).

Many of these orphaned roads add greatly to the difficulty of reducing slope stability hazards. Not only are poorly constructed and/or unmaintained roads inherently susceptible to failure, but their existence and/or location are often unknown. Even if their locations are known, hazard identification requires trained professionals to inspect each mile of each road.

Road inspection and mitigation of road-related slope hazards are usually costly. Effective inspection requires time and experienced geotechnical personnel. Further, thick stands of young trees and brush on the right-of-way commonly hinder access for both personnel and equipment. In many instances, a road will need to be cleared and rebuilt to provide passage for equipment necessary for proper abandonment. Given these circumstances, prioritization of road inspections is not a simple task.

Although timberland management has improved greatly since passage of the Forest Practices Act, formidable tasks lie ahead. One major problem facing the logging industry and DNR now is locating and inspecting orphaned roads. Inspection by itself does not ensure that all hazards will be recognized. For example, the inspectors do not usually have any knowledge of subsurface conditions of old road fills, nor can they easily obtain the information without costly subsurface exploration. Once inventoried, priorities must be established and steps must to taken to mitigate hazards due to slope failure.

The following case history illustrates how management practices on one property can affect an adjacent area and how failure to recognize or mitigate hazards created by previous management activities can lead to disaster. Much of the following information about the Marblemount slide was taken from Orme (1985).

THE MARBLEMOUNT LANDSLIDE CASE HISTORY

On the evening of November 1, 1985, a landslide occurred near the town of Marblemount in Skagit County, Washington, about 4 mi east of the confluence of the Cascade and Skagit rivers (Figure 1). At the time of the slide, Georgia-Pacific Corporation, a timber products company, owned the property upon which the slide occurred. DNR owned the land immediately upslope and also regulated forest practices on private forest lands in the state.

Cascade River Park, a private development, was struck by the slide, which destroyed a mobile home and killed its four occupants (Figures 2 and 3).

Location and Extent of the Landslide

Marblemount is located in the western foothills of the Cascade Range, on the North Cascades Highway. The area receives an estimated average annual rainfall of about 75 in. Relief is moderate to extreme in the immediate vicinity of the slide.

At about 5,000 ft elevation, Lookout Mountain is the drainage divide to the north of the area and contributes runoff over a slope distance of 8,700 ft. The headwall of the landslide is in a switchback of an abandonded logging road. Failure of the road prism of the switchback initiated the slide, which occurred on a slope of 25° or steeper and eventually reached the county road, 1,000 ft below the headwall. As depicted in Figures 4 and 5, a 700-ft-long zone of transition from erosion and transport to runout and deposition is located just below the county road on glaciofluvial terrace deposits and the floodplain of the Cascade River.

The landslide traveled down a slope distance of approximately 1,700 ft for a total vertical distance of approximately 600 ft (Figure 4), scouring out a path from 75 to 100 ft wide (Easterbrook, 1987). The failure was described as having the combined characteristics of a slide, flow, and avalanche. It was triggered (at least in part) by heavy rainfall; local geology and accidental drainage diversion contributed to the slope failure.

Figure 1. Location of the Marblemount landslide.

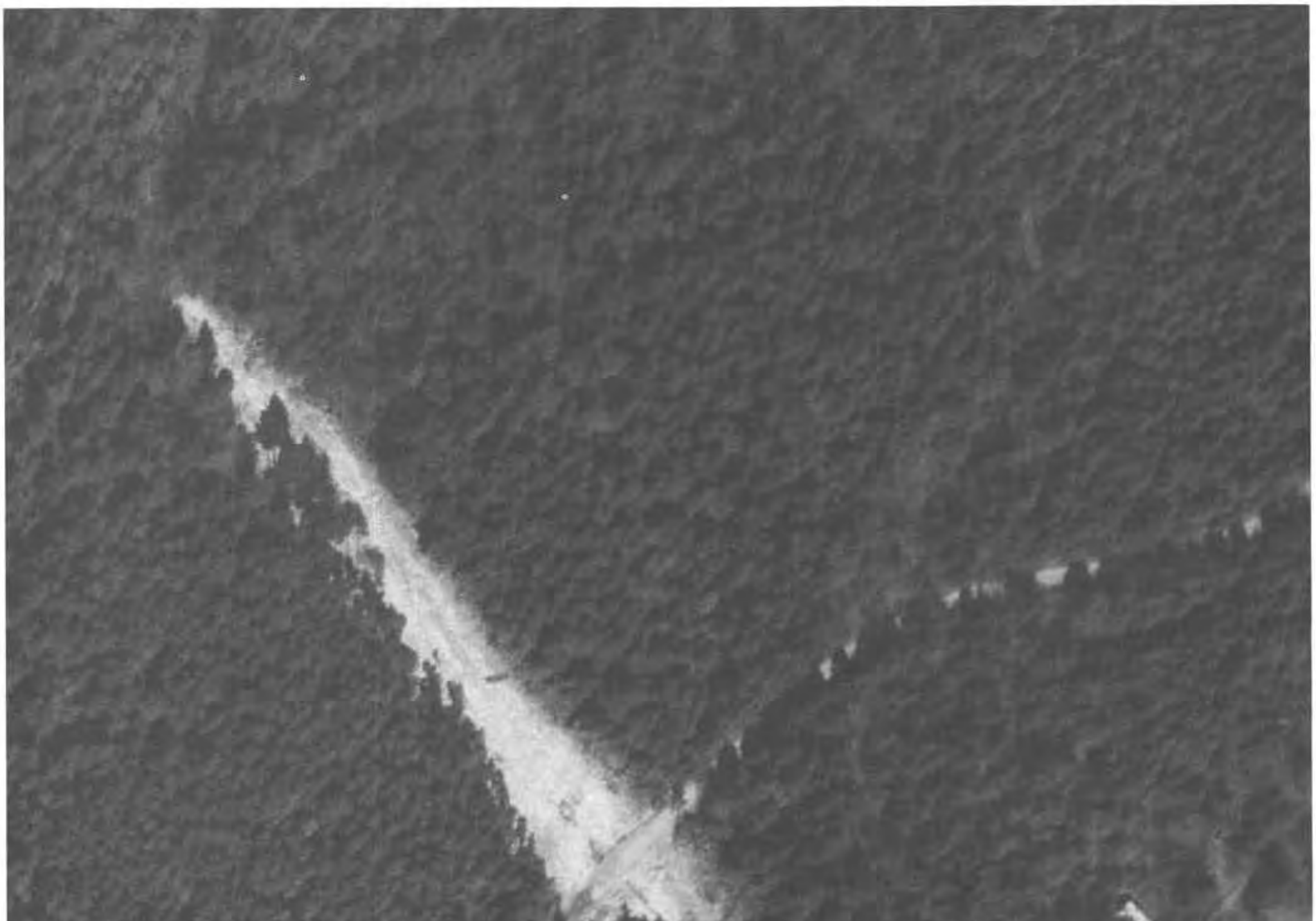

Figure 2. Scar caused by landslide. The Cascade River is located in the lower right corner of the photograph, the switchback in the upper center, and the county road just to the right of center, about one-third of the way up the scar. Photograph was taken after the roads were cleared of slide debris. Photographs by Walker and Associates for DNR.

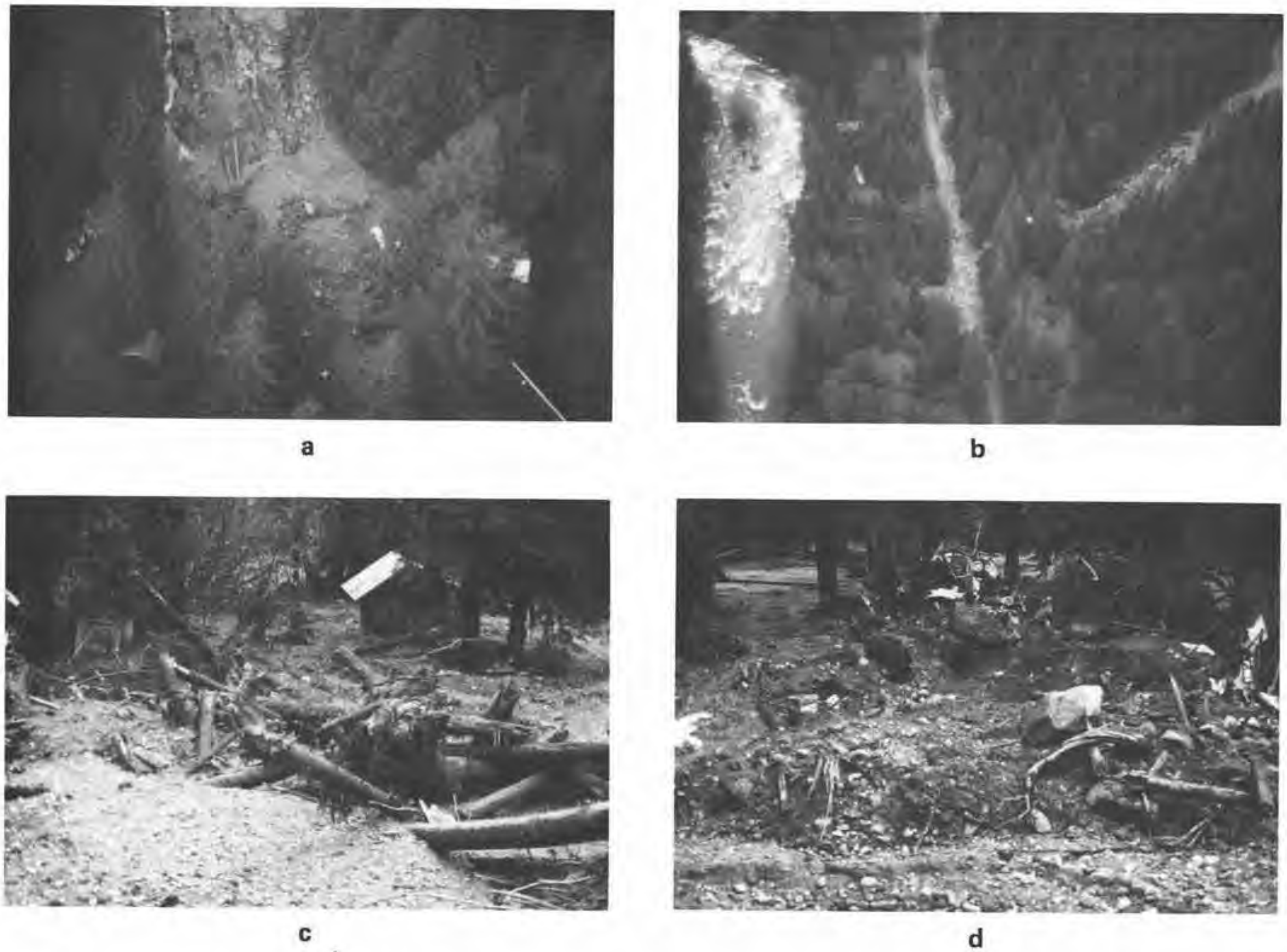

Figure 3. Aerial photographs of (a) the Cascade River development and damage caused by the slide and (b) transition and runout areas in the Cascade River development where the landslide crosses the access road below the county road; view to the west. Views (c and d) of landslide deposits and damage within the development. Photographs by G. W. Thorsen, 1985.

Site Conditions

Weather

Not only was October 1985 wetter than normal, but unusually heavy rainfall occurred in the 2 days immediately preceding the slope failure. Records from five rain-gaging stations indicate that at those stations precipitation ranged from 15.74 in. to 17.59 in. after October 10 and before November 1, 1985. This represented record or near-record totals for the period. At one of the stations, between 4:00 p.m. October 31 and 6:00 p.m. November 1, the rate of rainfall increased every hour to reach a total for that period of 3.63 in. Another nearby station recorded 1.60 in. from noon to 6:00 p.m. on the day of the slide. Compounding the effects of such heavy precipitation was the fact that the rain fell on the snowpack of the October storms.

Geology

Metamorphic bedrock is overlain by till and glaciofluvial terrace deposits at the slide site (Figures 5 and 6a). The bedrock, a massive to gneissic meta-quartz diorite of the Skagit Metamorphic Suite of Misch (1966), is exposed within the head of the slide and in the roadcut just above the head of the slide. Foliation in the bedrock exposed by the slide is at right angles to the regional foliation, which strikes northwest and dips steeply east or west. This anomalous foliation indicates that there could be a local bedrock structure, such as a fault, that could influence slope stability by acting as a ground-water barrier that could channel water toward the slide.

The meta-quartz diorite is exposed along the county road and near the head scarp, suggesting that the over-

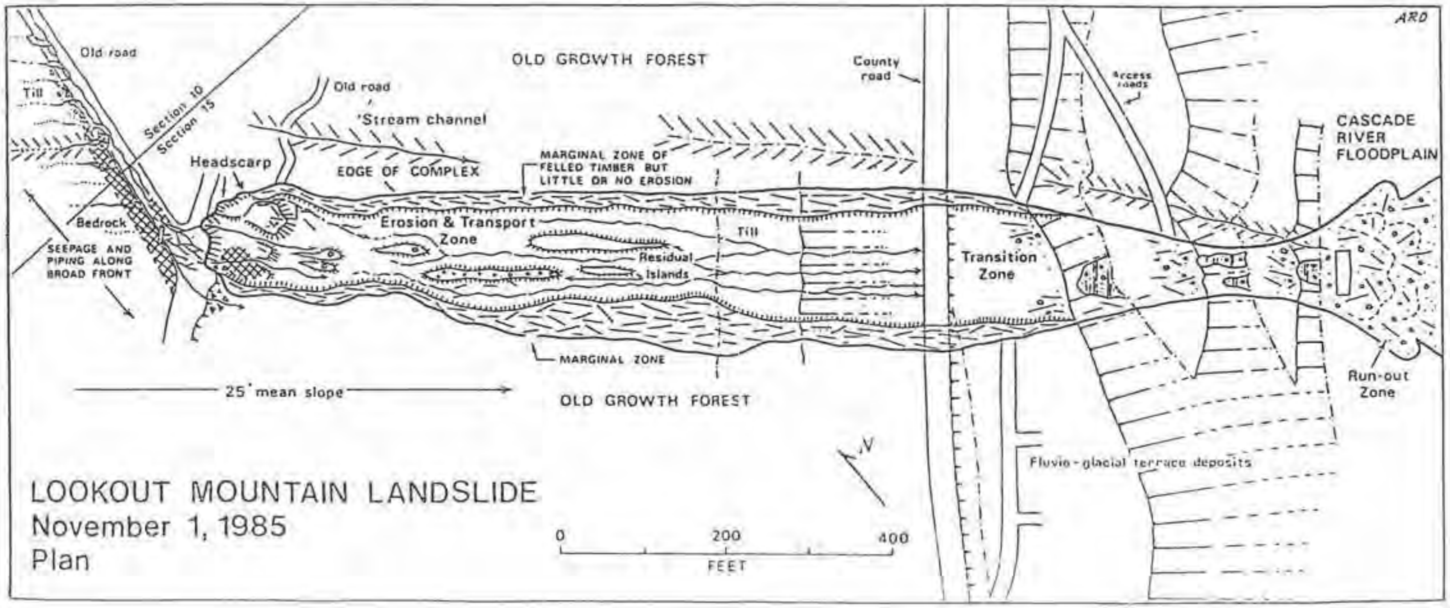

Figure 4. Features of the Marblemount landslide in November of 1985. From Orme (1985).

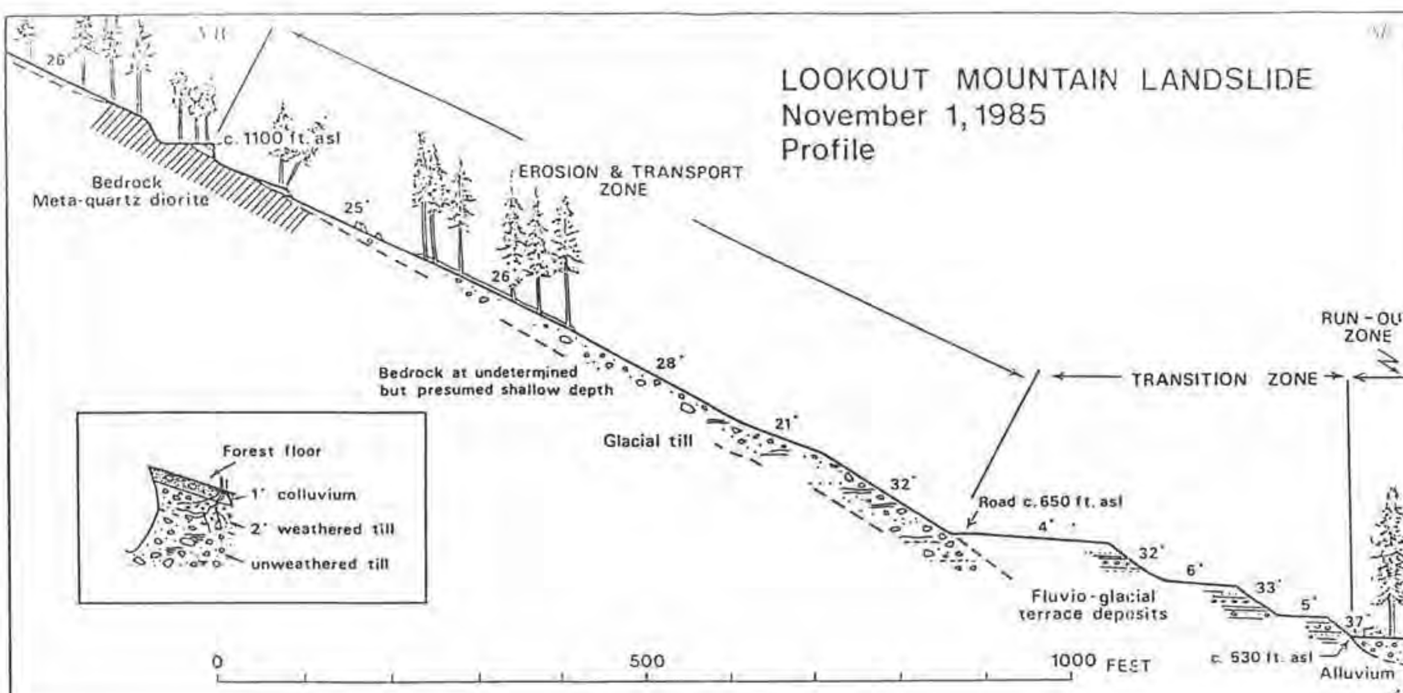

Figure 5. Cross-section of the Marblemount landslide showing geology, morphology, and zones of slide movement. From Orme (1985).

lying till is not thick (Figure 6b). The compact till consists of poorly sorted mixture of boulders and cobbles in a pebble, sand, or locally clay-rich matrix. This relatively impermeable till is lodged against the bedrock and underlies most of the slide scarp. About 2 ft of weathered till and 1 ft of colluvium overlie the fresh till and allow root penetration and ground-water infiltration. This configuration is common throughout the erosion and transport (depletion) zone of the slide. Strength and permeability differences between the fresh and weathered till, although not measured, apparently exist because the failure plane is coincident with the interface.

Glaciofluvial terrace deposits lap onto the till and bedrock, causing a major break in slope near the county road. The transition zone between the erosion and transport zone (head and mid-slope) and the run-out zone (toe) coincides with the limits of the glaciofluvial deposit. Cascade River alluvium laps onto the terrace deposits and underlies the run-out zone.

Ground-Water and Surface Drainage

Foliation and fracture planes in the bedrock and the probable low permeability of the till permit very little deep ground-water infiltration in the slide area. Hence, most ground water is concentrated in the overlying weathered till and associated colluvium.

Local surface drainage conditions also contribute to the saturation of slope materials. The slope above the switchback is broadly convergent and tends to concentrate surface and shallow subsurface runoff toward the site.

Slope failure was estimated to have taken place at the contact between the weathered and unweathered till.

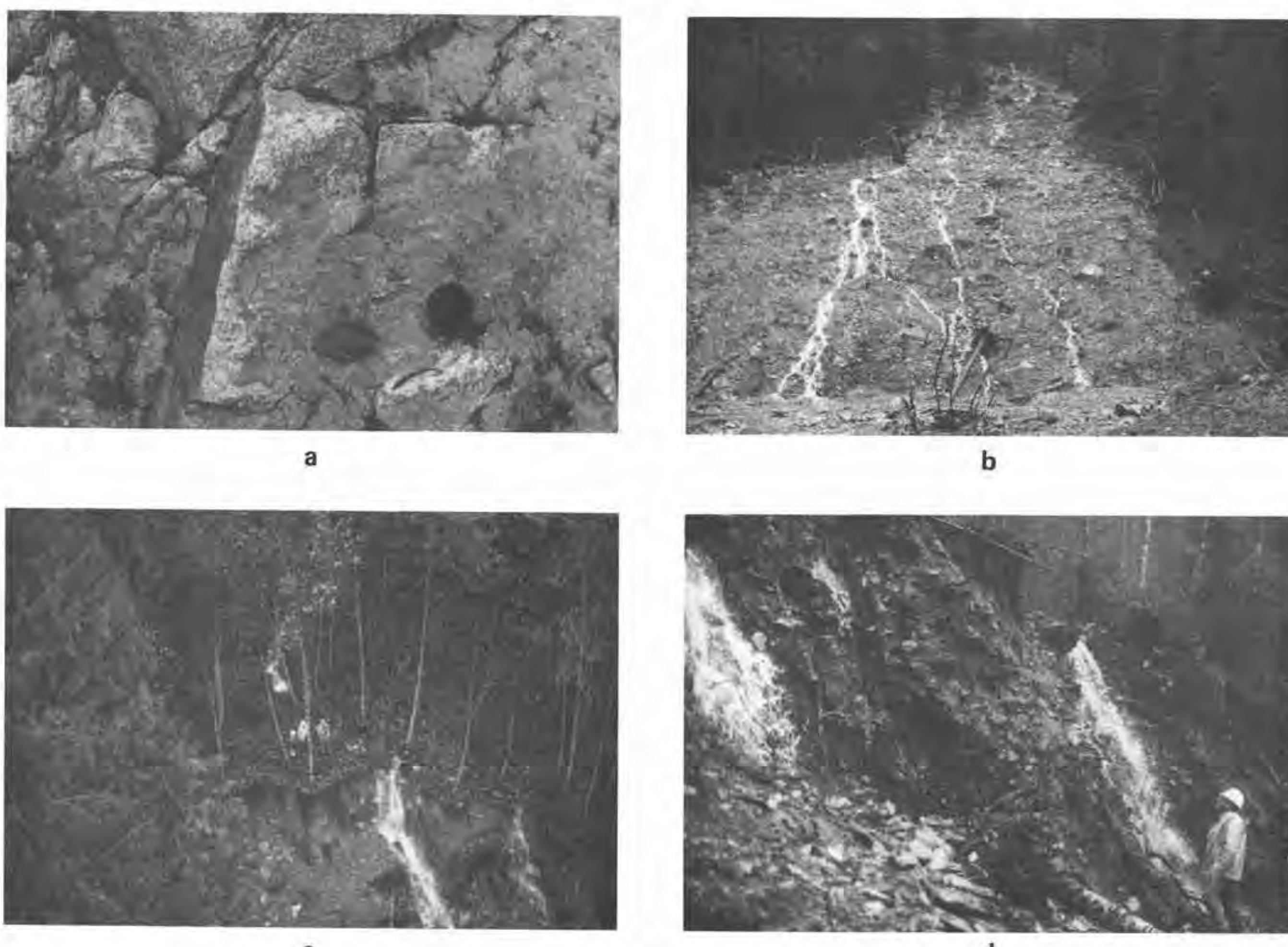

Figure 6. (a) Till lodged on meta-quartz diorite; (b) view from near the county road upslope toward the headscarp area. Note the shallow nature of the slide and the abundant surface runoff; (c) surface drainage at the head of the slide; (d) contact between the base of the road fill and pre-road ground surface located near center of photograph. Note the buried log in the right center of the photograph. Photographs by G. W. Thorsen, 1985.

The till and bedrock forced water from the large upland drainage area to the surface in the area of the head of the slide and along the roadcut above the switchback (Figure 6c).

Seepage and piping directed water onto the road at various locations above the switchback. Normally, the runoff traveled down the inside of the road and across the outside of the switchback turn and to the slope below. An alluvial fan, deposited on the upper part of the road above the switchback by drainage from a minor channel, and a log that had fallen onto the road above the switchback diverted runoff from the inside toward the center of the road and across the switchback and saturated the fill.

Road Characteristics

Weight of the saturated fill on the underlying thin soil materials apparently caused the failure (Easterbrook, 1987). The maximum thickness at the outside berm of the road was 13 ft. Only 6 ft of fill was initially exposed in the slide scarp (Figure 6d), suggesting that the original failure occurred mostly in the soil materials located below the fill. Easterbrook (1987) calculated that 430 cy of fill material were involved in the slide, and he maintained that the added weight of the fill material "was the principal cause of the instability of the slope." Orme estimated the total volume of inorganic debris involved in the slide to be 7,134 cy.

Tree ring evidence from red alders growing on the logging road indicated that it had not been used for about 25 to 30 yr; the road had been built in the 1940s and abandoned in the 1960s (Estate of Wilson v. Georgia Pacific Corporation, 1987). It had survived the nearly 30-yr period with apparently little or no maintenance and failed only after its original configuration had been sufficiently altered by natural processes.

LEGAL CONSEQUENCES OF THE LANDSLIDE

A case was brought to trial in Washington Superior Court as *Wilson* v. *Georgia-Pacific and the State of Washington*. It was alleged that, as the landowner, "defendant Georgia-Pacific was negligent with regard to certain forest practices and that defendant Natural Resources was negligent in failing to regulate defendant Georgia-Pacific in those practices" (Estate of Wilson v. Georgia Pacific Corporation, 1987). This claim was rejected by the trial court because the road was built, used, and abandoned prior to the Forest Practices Act and Forest Practices Regulations of 1974 (Silver, 1988). Instead, the defendants were charged as landowners with the responsibility for certain conditions on their land and not as regulators or regulated entities under the Act.

These conditions, it was alleged, should have been recognized, on the basis of the state of knowledge of the geology, hydrology, and weather patterns of the area. In other, earlier lawsuits against landowners (including the defendants), similar slide and debris torrent conditions were involved. Also, the plaintiffs presented evidence that DNR had been aware of a slide on the other side of the Cascade River that had impacted the same park development about 10 yr before. Finally, evidence was also presented that a request had been made the summer prior to the slide from the DNR to its local units to inspect roads for potential slide problems. The road involved in the case was not inspected. It was believed that due to the passage of time since abandonment the road had become stable through natural processes. It was claimed that if the road had been inspected, the dangerous state of the switchback would have been recognized.

The jury ruled against the State, and the plaintiff was awarded \$2.6 million of which the State's share was \$2.3 million. Georgia-Pacific settled out of court just prior to the trial date for \$285,000. Apportionment of the damages was appealed.

DISCUSSION

While it is futile to speculate about what could or should have been done, the implications of the outcome of this case seem clear. As urban and other types of development continue to spread into rural areas in Washington, the cost of doing business on forest lands will increase. Both mitigation costs and tort liability will become greater financial burdens on the timberland owner. Under the Revised Code of Washington (RCW) 76.09.300, the state has a no-action alternative based on costs and benefits (Washington State Forest Practices Board, 1988). As Olshansky and Rogers (1987) have pointed out, tort liability may not be the most desirable method of reducing landslide hazards, but it is currently the *de facto* policy in many states.

Progress toward well-integrated planning and development may well be slow, considering the enormity of the task. Road construction techniques, stream cleanout practices, slope stability mapping, and orphaned roads inventory are just a few of the areas that will need improvement and research. If improvements do not keep pace with development, then risks to life and property will increase and may be translated into larger losses of revenue for forest managers and regulators.

With increasing population pressures and a general lack of public awareness of geologic hazards, development of high-risk areas continues to increase. This problem has been recognized and addressed in urban areas of Washington for a number of years (Tubbs, 1974; King County, 1980). Rural areas, however, are more difficult to administer. The regulating agencies have an intimidating task of inventorying and mitigating geologic hazards. Land zoning can discourage development in hazardous areas, but questions persist: To what extent can zoning restrict development on private land? Can the role of geology in planning be made sufficiently widely understood and accepted? Can geologists and engineers predict, in a timely and inexpensive manner, where slope failures will occur and where they are unlikely to occur on thousands of miles of overgrown logging roads? Will cleaning and rebuilding such roads so that heavy equipment can be brought in to remove some hazardous fill cause more problems than are solved?

Forest practices regulations and management practices are currently being extensively revised. DNR is presently preparing to inventory geologic hazards associated with their lands in much greater detail than ever before. The Timber/Fish/Wildlife agreement (Waldo, 1987) holds promise for effective cooperation among rural timberland owners. However, the issues of the responsibility and potential liability that come with land ownership are not easily legislated.

The State and private timber companies have thousands of miles of forest roads to manage. Parts of this road system have potential for failures and damages. Large tracts of timberlands may become uneconomical to develop if encroachment by private developers renders the risks of logging or other development greater than the possible benefits.

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The White Bluffs Landslides, South-Central Washington

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INTRODUCTION

Since the late 1800s, agricultural irrigation has caused landslides in semiarid to arid valleys of the interior plateaus of western North America. In these valleys, some slopes that have been dry or nearly dry since the end of the Pleistocene have failed when the geologic materials comprising them have been wetted by irrigation. This paper describes selected landslides from a large group of recent irrigation-induced landslides in Pliocene fluvial-lacustrine sediments in the 50-km-long White Bluffs area along the east side of the Columbia River in south-central Washington (Figure 1).

This stretch of the Columbia forms the eastern boundary of the Hanford site of the U.S. Department of Energy. Several nuclear facilities on the Hanford site near the west bank of the river are factors to be considered in assessing landslide risk in the White Bluffs area. This stretch of the river has also been proposed as the site of the Ben Franklin Dam, an earthfill structure approximately 15 m high; if constructed, the dam and its reservoir could be detrimentally affected by landslides.

South-central Washington is arid; average annual precipitation along the Columbia River in the White Bluffs area is about 180 mm. However, the groundwater regime has been significantly altered by the addition of agricultural irrigation water, which is brought south some 150 km from Grand Coulee Dam. Irrigation in the area began during the period 1953-1964. Since then, about 1,500 mm of irrigation water (eight times the annual precipitation) has been applied annually to croplands immediately east of the bluffs (Brown, 1970).

In the study area, vertical and lateral erosion by the Columbia River has left steep, 45- to 170-m-high bluffs (the White Bluffs) along its east side. The bluffs commonly are capped by Quaternary fluvial and windblown sediments. The Pliocene fluvial-lacustrine Ringold Formation underlies these sediments and forms the steep slopes of the White Bluffs. The Ringold Formation is nearly horizontal (locally the beds dip about 1° toward the river), and it consists mainly of weakly indurated claystones, siltstones, and sandstones. The formation overlies and conceals rocks of the Columbia River Basalt Group.

DISTRIBUTION AND GENERAL CHARACTER OF RINGOLD FORMATION LANDSLIDES IN THE WHITE BLUFFS AREA

Newcomb and others (1972) observed that landslides are an ever-present hazard along parts of the Columbia River in south-central Washington. General distribution of landslides (most of which are prehistoric) has been indicated on medium-scale (1:62,500) geologic maps by Grolier and Bingham (1971), Newcomb et al., (1972), Myers et al., (1979), and the Washington Public Power Supply System (1981). Additional detail on landslides along this stretch of the Columbia River has been presented by Schuster and Hays (1984), Hays and Schuster (1987), and Schuster et al, (1987). Figure 1, based on our field study, shows the distribution of active and inactive landslides in the White Bluffs area.

The soft rocks of the Ringold Formation are relatively strong when dry, but they lose much of their strength when wetted. The many prehistoric landslides along the bluffs suggest wetting in the past, but the present climate probably is too dry to sustain natural wetting. However, the irrigation water being applied to croplands east of the Columbia River has raised the water table in the White Bluffs area significantly and has formed perched water tables. Thus, local saturation of the Ringold Formation has increased pore pressures, reduced shear strengths, and lowered the overall stability of the slopes comprising the White Bluffs (Brown, 1972).

According to Varnes' (1978) classification, most mass movement in the White Bluffs is complex, in that individual landslides involve more than one type of gravitational movement. They commonly start as earth slumps or slides, disaggregating downslope into earth avalanches and earth flows.

The total area of landslides in the White Bluffs is approximately 6.8 sq km. About 80 percent of this consists of inactive prehistoric landslides; the other 20 percent is made up of landslides that have moved in the

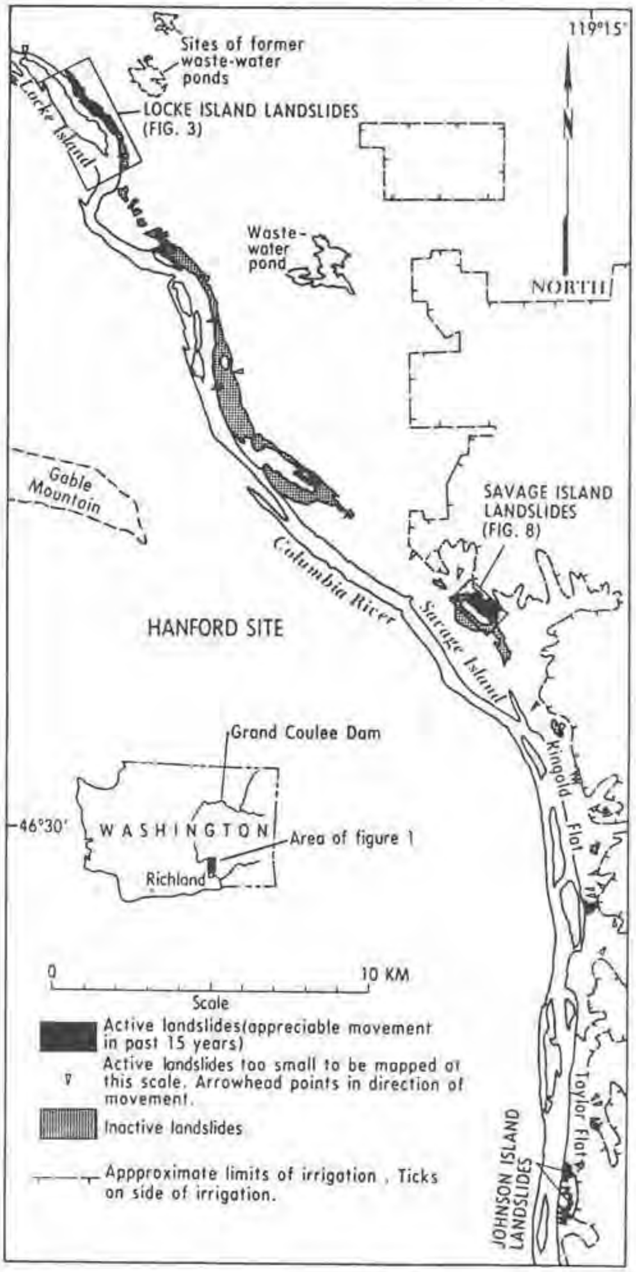

Figure 1. White Bluffs landslide areas, south-central Washington.

past 15 yr. We have identified about 50 individual active landslides along the east shore of the river. Fourteen of these have surface areas of more than 1 ha each.

Three major landslide areas in the White Bluffs are currently of the greatest importance because of their size and recent or potential activity. We have named these landslide areas (Figure 1) after nearby islands in the Columbia River. The two largest landslide areas of the three are the Locke Island landslides, near the north end of the White Bluffs, and the Savage Island landslides, near the middle of the bluffs. The Johnson Island landslides, at the southern end of the bluffs, currently do not have the volume of active landsliding displayed by the Locke Island or Savage Island landslides; however, the Johnson Island area shows signs of potential for major landslide activity.

LOCKE ISLAND LANDSLIDES

At their northern end, in the vicinity of Locke Island, the White Bluffs rise about 100 m above the Columbia River. The bluffs consist of 50 to 70 m of Ringold Formation overlain by 15 to 50 m of Quaternary silt and sand. In the early 1980s, the Locke Island landslides (Figures 1 through 6), which occurred in these materials, were extremely active. The setting for these landslides differs from that of other active White Bluffs landslides to the south in two significant respects:

(1) The bluffs at Locke Island are on an outside bend of the river; thus, the base of the bluffs and the toes of the landslides are subject to erosion by the river.

(2) In an attempt to lessen the landslide hazard in this part of the bluffs, the area within about 8 km east of the river has not been opened to irrigation. In contrast, the Savage Island and Johnson Island landslide areas are immediately adjacent to irrigated croplands. In the Locke Island area, however, irrigation waste-water ponds have been established within 2 km of the river in order to enhance wildfowl habitat. During the late 1970s and early 1980s, these ponds were the main source of seepage water to the bluffs, and thus were a major cause of failure of slopes above the river. In recognition of the landslide danger, water levels in these ponds have been lowered considerably, and landslide activity along the Locke Island stretch of the bluffs has declined dramatically.

The total area of landsliding in the Locke Island area is about 68 ha; of this, 59 ha consists of active landslides (less than 15 yr old). The total volume of these active landslides is estimated at 12 million cu m. The slides generally started as deep-seated slump blocks; in descending the slope toward the river, they have been transformed into thick trains of earth-flow debris. The toes of individual landslides have progressed as much as one-third of the way (150 m) across the northeast channel of the river (Figures 2, 3, and 4).

River erosion of the toe of the slope probably was the major cause of landsliding here in the past; these landslides, in turn, were partly eroded away by the river. Since about 1970, a more important factor has been the irrigation waste water that seeps to the bluffs from the waste-water ponds either over the relatively impermeable surface of the Ringold Formation or through per-

Figure 2. Locke Island landslides and northeast channel of Columbia River; view downstream. See Figure 3 for scale. Photograph by H. S. Holmes, U.S. Bureau of Reclamation, April 1985.

vious layers within the formation. This seepage emerges from the bluffs as springs flowing from the landslide head scarps (Figure 5).

Economic losses from these landslides have been minor because there is no development along the northeast bank of the Columbia River in the Locke Island area. There has been some loss of potential agricultural land, but the primary direct loss has consisted of siltation of spawning beds of anadromous fish in this last free-flowing stretch of the Columbia River in central and eastern Washington. In addition, there is a possibility of damage to nuclear-reactor pumps and other equipment on the Hanford site if sediment from landslides enters cooling-water intakes (Waltors and Brown, 1979).

Harty (1979) and Waltors and Brown (1979) have noted the possibility that an extremely large landslide in the Locke Island area could block or divert the Columbia River and flood deactivated reactor sites near the west bank. In our opinion, a blockage due to gravita-

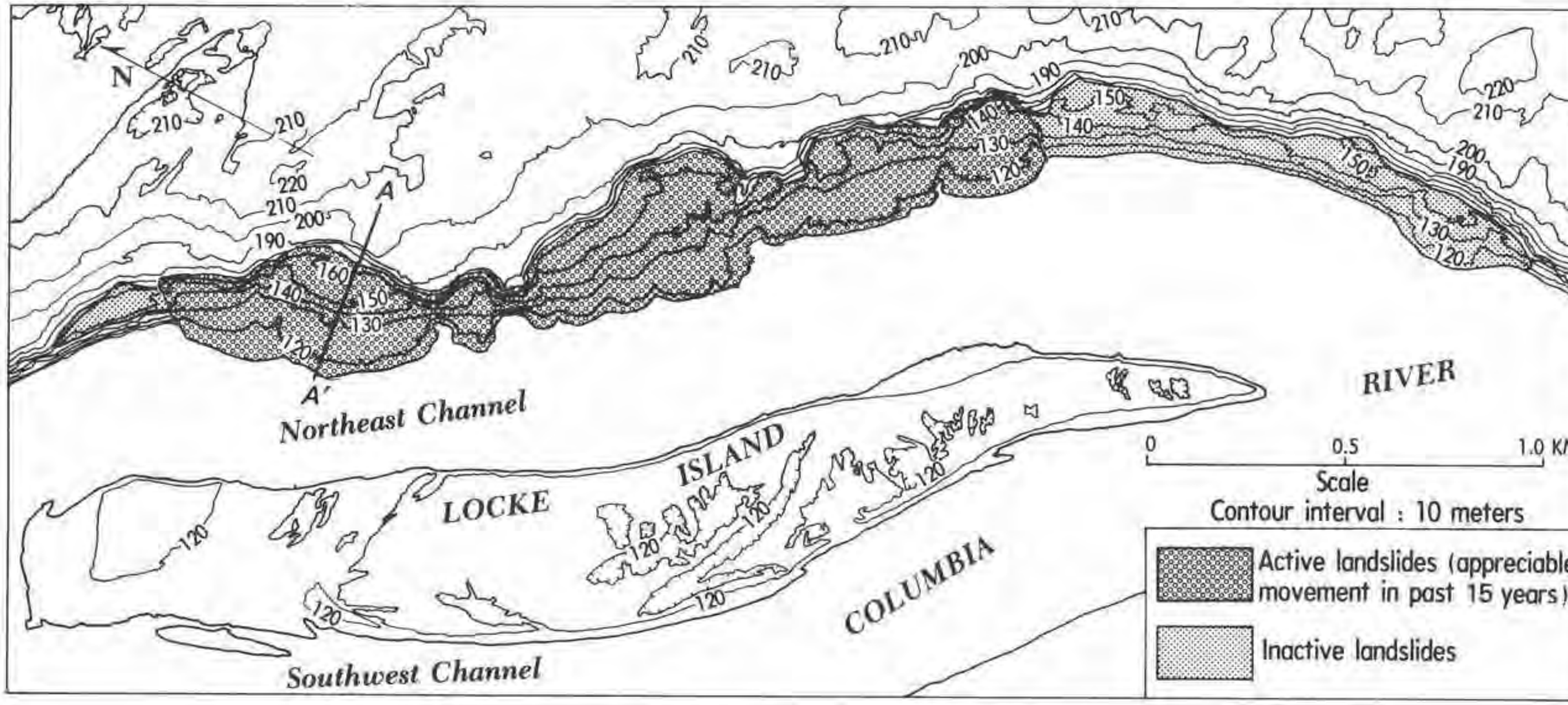

Figure 3. Locke Island landslide area.

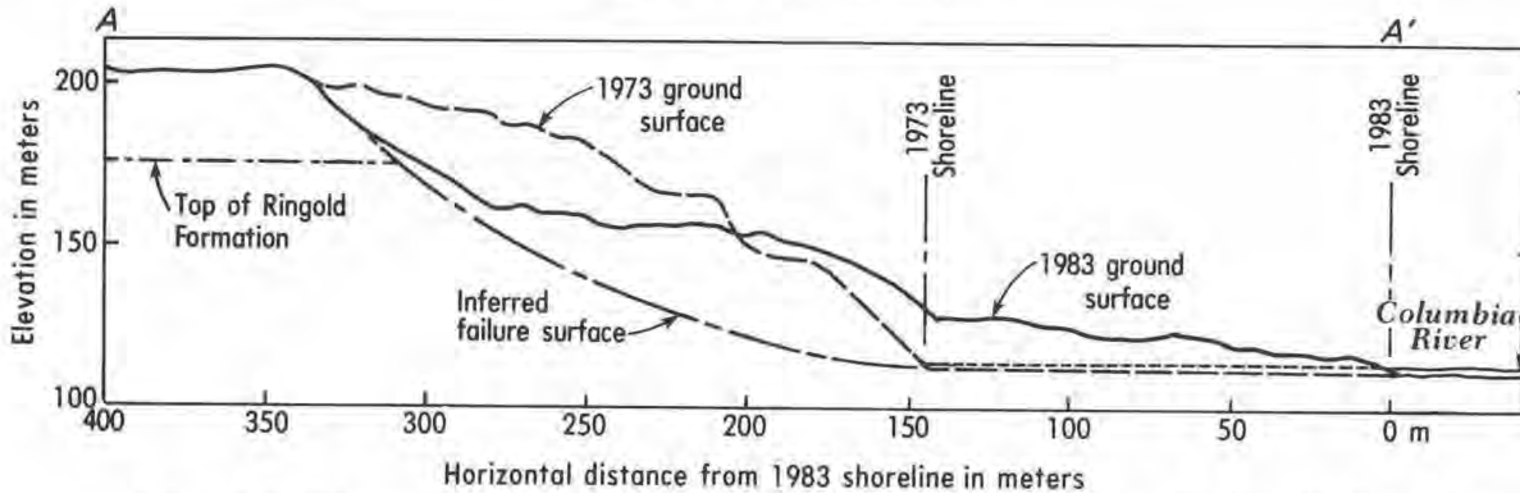

Figure 4. Cross section A-A' (Figure 3) through one of the largest landslides in the Locke Island area. This landslide, which underwent its greatest activity in 1980, has extended one-third of the distance across the northeast channel of the Columbia River.

Figure 5. Seepage of irrigation waste water from the face of a 40-m-high head scarp of an active landslide in the Locke Island landslide area. Seepage (dark horizontal band on face of scarp) is concentrated at contact (arrows) between the Ringold Formation and overlying, more pervious Quaternary sediments.

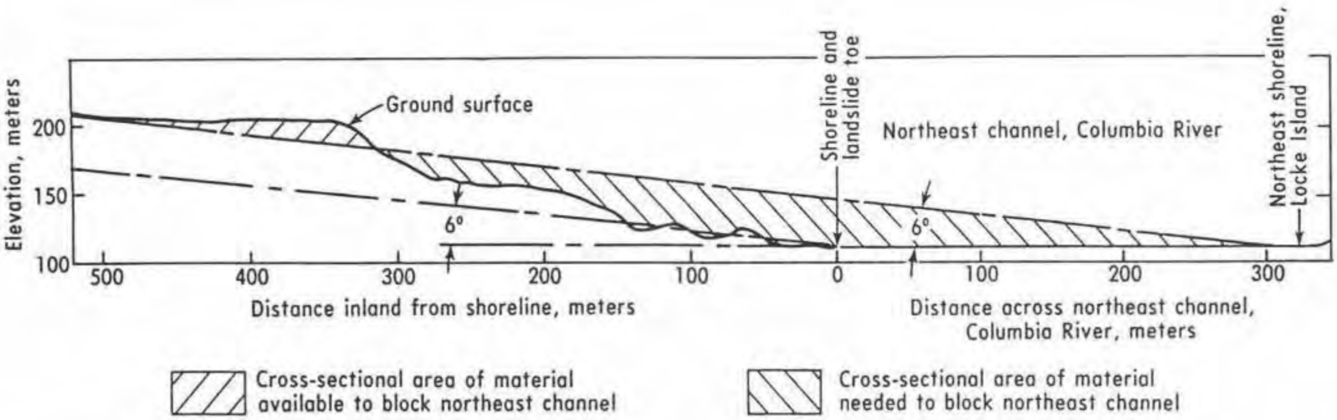

Figure 6. Balance-of-materials approach to illustrating that, for a "worst-case" example, there is no possibility that a landslide in the Locke Island area (along section A-A′ of Figure 3) can dam the northeast channel of the Columbia River. The 6° slope chosen for the hypothetical worst-case landslide that would just reach the near shore of Locke Island is based on measured flattest slopes of the toes of existing earth flows shown in Figures 2, 3, and 4. Events that might produce depositional slopes flatter than 6° are very unlikely.

tional slope failure is very unlikely. There simply is insufficient material available in the bluffs to produce damming even under the worst possible landslide conditions. Figure 6, a material-balance analysis of damming potential, illustrates that even the northeast channel of the river cannot be dammed in the vicinity of Locke Island in this manner.

SAVAGE ISLAND LANDSLIDES

For the past several years, the Savage Island landslides (Figures 7 through 10) have been second only to the Locke Island landslides in size and amount of landslide movement along the Columbia River in south-central Washington. They include about 120 ha of active and inactive landslides, of which about 47 ha consists of a continuous landslide mass, with a volume of about 10 million cu m that became active after 1968 and has enlarged dramatically since 1980 (Figure 8).

The active landslide mass consists of many coalescing complex landslides. It is bounded upslope by a nearly vertical head scarp 1,400 m long and up to 28 m high. In 1981, the head scarp retreated into adjacent irrigated fields; large blocks, some of whose surfaces still maintained crops of alfalfa, separated from the head scarp along vertical joints and slid slowly downward on the landslide mass. At first, the blocks maintained both physical integrity and attitude (Figure 9); however, they slowly degenerated, and in about a year individual blocks had deteriorated to jumbled masses that were indistinguishable from the older earth-flow mass. Unlike the case for the Locke Island and Johnson Island landslides, the Savage Island landslide is located about 1 km from the Columbia River and does not flow into the river. As shown in Figure 10, advance of the toe of the landslide has been blocked at its distal margin by a 50-m-high ridge of Ringold Formation, the base of which is about 450 m from the head scarp of the landslide.

We attribute landslide activity here to irrigation of croplands east of the bluffs, both immediately adjacent to the retreating head scarp and farther east. Landslide activity has been greatest in the spring of the year when the combination of irrigation water and residual moisture from winter precipitation creates a critical soil-moisture relationship. Seepage from the landslide and the head scarp forms small creeks that pond on the landslide and at its toe. In April 1983, flow from springs on the landslide was estimated at 70 cu m/hr.

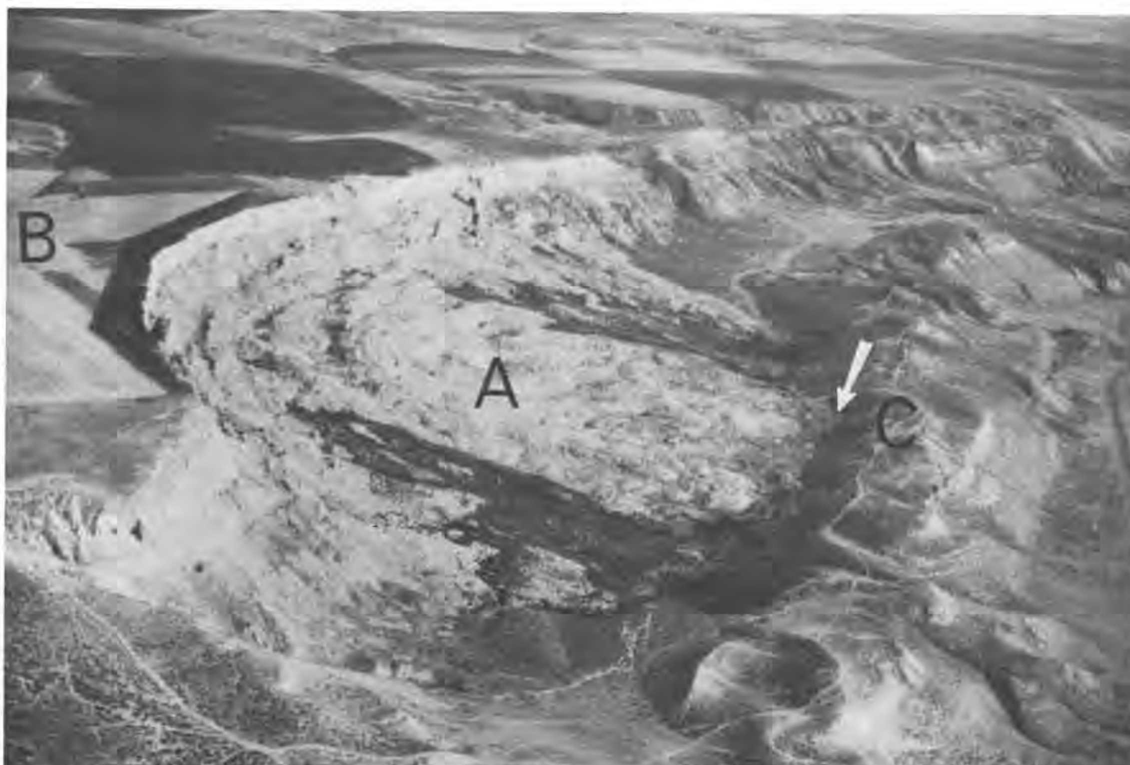

Figure 7. Aerial view (south) of the Savage Island landslide area, September 1982. A, center of active landslide; B, irrigated cropland immediately east of landslide; C, ridge of Ringold Formation that is blocking additional movement of the landslide. In the spring of 1982, the toe of the landslide (indicated by white arrow) moved 15 m (measured vertically) up the east side of the ridge at C. See Figure 8 for scale.

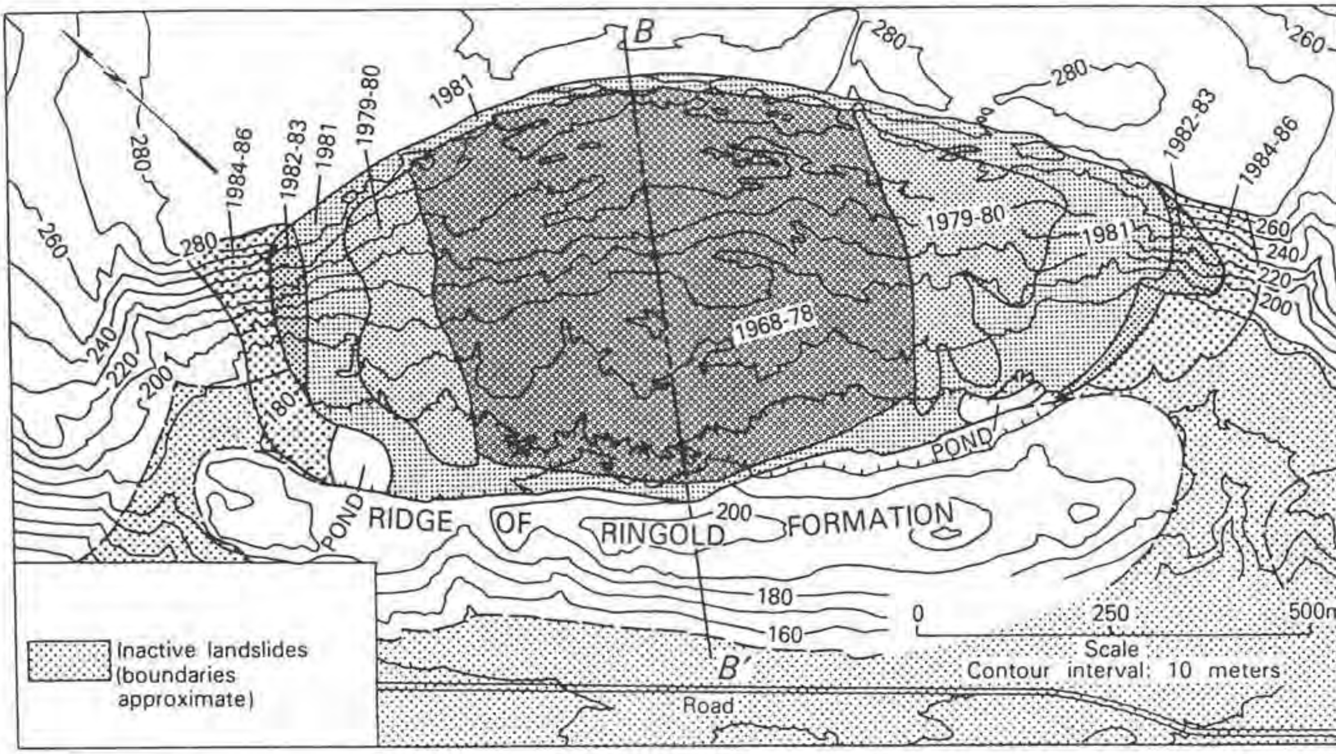

Figure 8. Active (since 1968) and inactive landslides in the Savage Island landslide area. Numbers on active parts of the landslide indicate years in which areas first showed landslide activity.

Figure 9. View to the south along the 20-m-high head scarp of the active Savage Island landslide, spring 1981. Note nearly intact Ringold Formation blocks that recently had calved from the head scarp. Older blocks have moved downslope and broken up. Note dark-colored alfalfa still growing on top of one block (arrow).

The only economic loss from the landslide has been the destruction of about 4 ha of prime cropland by encroachment of the head scarp. If irrigation continues on the upland adjacent to the landslide, landslide encroachment can also be expected to continue.

JOHNSON ISLAND LANDSLIDE AREA

At their southern extremity, the bluffs along the eastern bank of the Columbia River overlook Johnson Island (Figure 1) and, on the west side of the river, an important nuclear facility just north of Richland. The

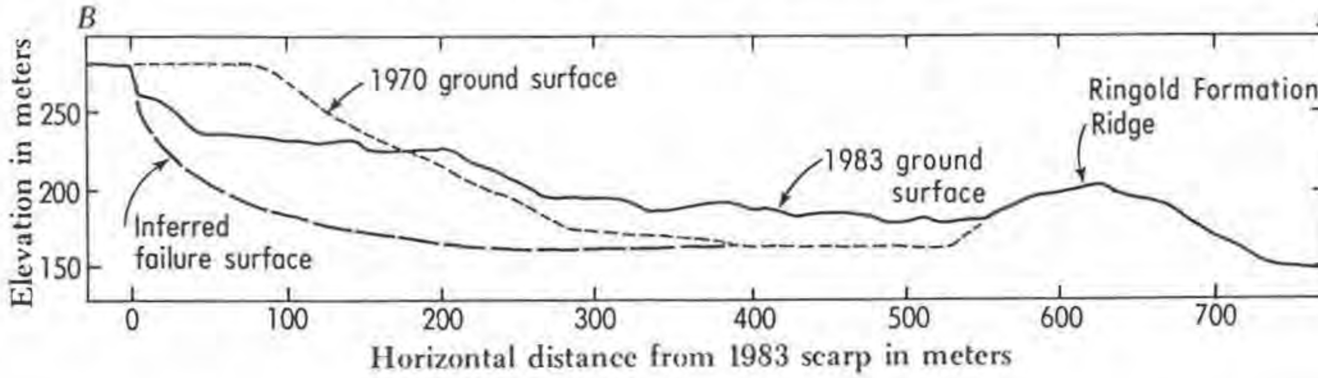

Figure 10. Cross section B-B′ (Figure 8) through the active Savage Island landslide. The position of failure surface has been inferred from field evidence. Note "runup" of toe of landslide onto outlying Ringold Formation ridge.

bluffs rise to an average height of 120 m above the river; their slope toward the river averages about 20°.

The Johnson Island landslide area differs from the areas previously described in that only a small percentage of the bluffs has been subjected to recent major landslide activity (Figure 1). However, a few recent slope failures have resulted from irrigation of croplands and fruit orchards immediately to the east of the bluffs, and we anticipate more such failures. Irrigation, plus greater-than-average precipitation from 1977 to 1984 (Figure 11) has raised the water table near the bluffs about 12 m since 1968 (Figure 12). The increased water level in the Ringold Formation forming the bluffs caused significant landslide activity along the bluffs in 1979 and 1981 (Figure 13) and again in 1985.

A 0.5-km-long shear crack (Figure 14) formed along the south end of the bluffs in 1981 on a part of the slope that had previously been inactive. The upper face of this separation forms the head scarp (as much as 3 m high) for a large, slowly creeping landslide that is evident lower in the slope mainly by a slight bulging of the downslope profile. Differential movement between the upper and lower sides of the crack has averaged 0.25 m/yr (slope measurement) in the 6 yr since the crack occurred.

The presence of this crack and other smaller shear cracks, an abundance of water in the gullies on the slope, the outward bulging of the surface, and the activation of several small landslides in the past few years lead us to expect major landslide activity on these slopes

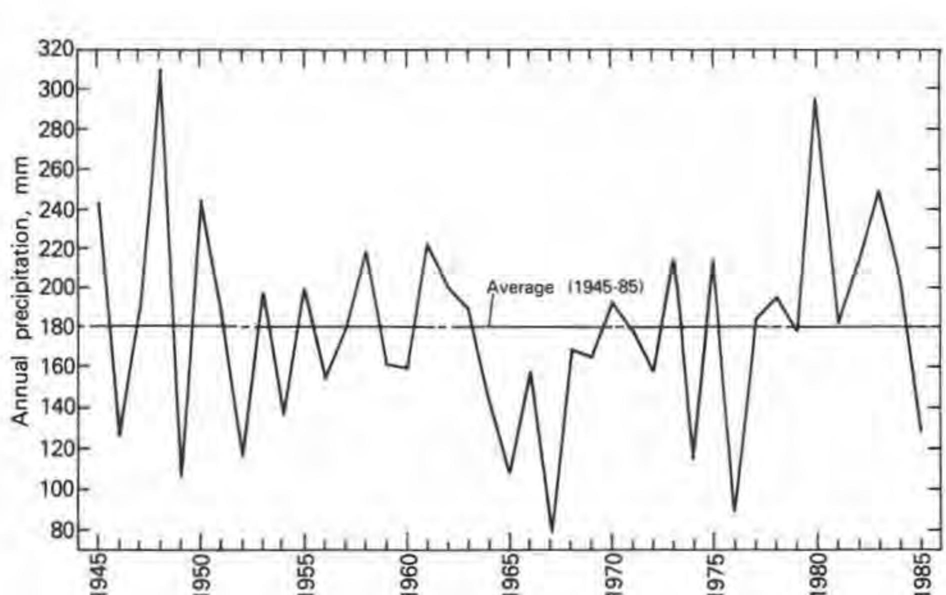

Figure 11. Annual precipitation from 1945 to 1985 at Richland on the west bank of the Columbia River approximately 10 km south of the south end of the White Bluffs landslide area.

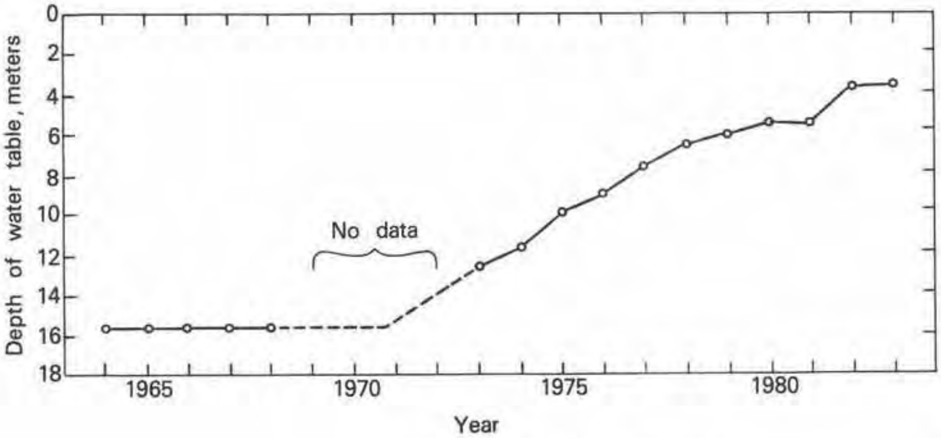

Figure 12. Variations in depth of ground-water table beneath the ground surface at a well located 1 km east of the top of the White Bluffs in the Johnson Island area, 1964-1983. Data from U.S. Bureau of Reclamation.

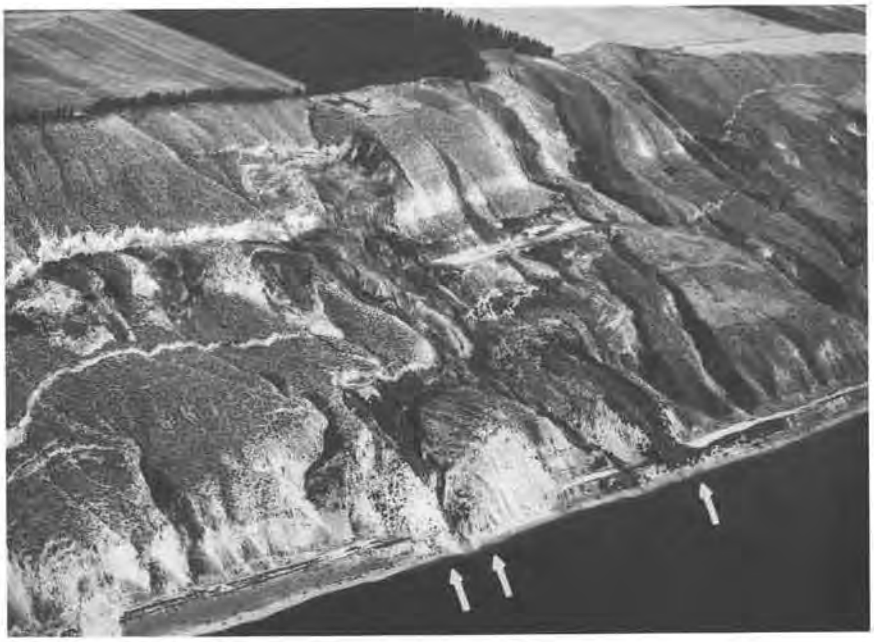

Figure 13. Landslides that blocked a county road along the Columbia River in 1979 (double arrow) and 1981 (single arrow) at the north end of the Johnson Island landslide area. Bluffs in the photo average 160 m high. Photograph was taken in 1983; view is to the southeast.

Figure 14. South end of the Johnson Island landslide area, showing the 0.5-km-long shear crack (white arrows) that formed along the upper face of the bluffs in 1981.

within the next few years if irrigation is continued as at present. As is the case for the Locke Island and Savage Island landslide areas, major slope failures on the bluffs in the Johnson Island area probably will not have significant economic impact. The county road along the river has already been destroyed by landslides (Figure 13). We doubt that a landslide can occur that is large enough to block or significantly divert the Columbia River. However, new landslides could regress to the point where croplands on the edge of the irrigated upland above the slope would be lost.

CONCLUSIONS

The 50-km stretch of the White Bluffs that forms the eastern shore of the Columbia River north of Pasco in south-central Washington is an outstanding example of landsliding primarily due to waste-water seepage from irrigation of adjacent croplands. Economic losses from these landslides have not been large because of a sparse population and the type of development in the affected areas, and there is little chance that the landslides will block or significantly divert the Columbia River. However, the occurrence of these major landslides on slopes that, because of the arid climate, were nearly free of historic active landsliding until irrigation was introduced, provides a lesson that should be remembered in planning future irrigation projects.

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