ENGINEERING GEOLOGY IN WASHINGTON

Volume II

RICHARD W. GALSTER, Chairman
Centennial Volume Committee
Washington State Section, Association of Engineering Geologists

WASHINGTON DIVISION OF GEOLOGY AND EARTH RESOURCES

BULLETIN 78

1989

Prepared in cooperation with the Washington State Section of the Association of Engineering Geologists

WASHINGTON STATE DEPARTMENT OF
Natural Resources
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This report is for sale (as the set of two volumes only) by:

Publications
Washington Department of Natural Resources
Division of Geology and Earth Resources
Mail Stop PY-12
Olympia, WA 98504

Price	$ 27.83
Tax	2.17
Total	$ 30.00

Mail orders must be prepaid; please add $1.00 to each order for postage and handling.
Make checks or money orders payable to the Department of Natural Resources.

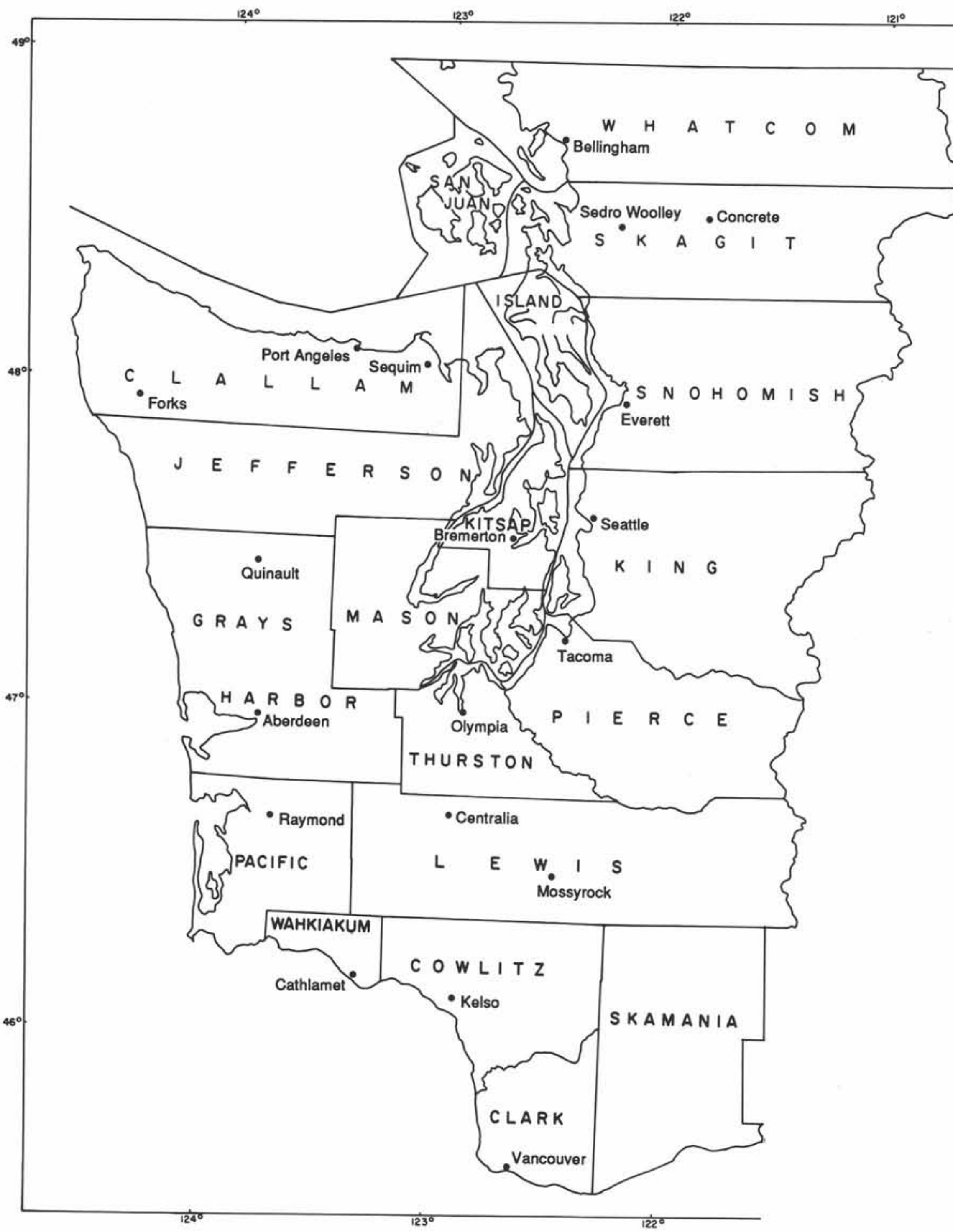
This publication is printed on acid-free paper.

Printed in the United States of America.

VOLUME I

PART II: ENGINEERING GEOLOGY CASE HISTORIES

VOLUME II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Part II: Engineering Geology Case Histories

(Continued from Volume I)

Engineering Geology in Urban Areas

William T. Laprade and William D. Evans, Jr., Chapter Editors

Engineering Geology in Urban Areas: Introduction

WILLIAM T. LAPRADE
Shannon & Wilson, Inc.

Geologic conditions are constraints to urban development, especially as pressure on the land becomes heavy. With low population density, geologic hazards such as landsliding, subsurface cavities, flooding, seismic shaking and subsiding or compressible ground are merely avoided so that few natural events are of consequence to people. However, as the less suitable areas are developed and urban population density increases, geologic conditions in these places are commonly ignored, flouted, or not recognized. The problems created by inattention to geologic conditions lead to conflict between adjoining property owners or property owners and governments.

Therefore, the need has arisen for governmental planning agencies to delineate geologic hazards, to promulgate rules concerning development in high-risk zones, and to require owners to obtain professional advice when developing or modifying their property or buildings.

The papers in this chapter discuss a broad spectrum of problems and situations encountered in urban and suburban settings in the Seattle-King County area. As the largest metropolitan area in the state, the greater Seattle area has experienced significant urban pressure and thus may serve as a valuable example for other urban areas.

Booth's paper describes the effects of suburban development on drainage basin hydrology. It chronicles reasons for thoughtful regulations for drainage basin planning.

The paper by Gurtowski and Boirum presents for the first time a comprehensive review of foundation and excavation conditions and related construction techniques in Washington's largest city, Seattle. Included is a table of the soils, foundation types, and pressures and shoring design criteria for most of the high-rise buildings of the downtown district.

Laprade and Thompson review the geologic conditions and construction procedures involved in Seattle's largest public works project, the Downtown Seattle Transit Project, a bus tunnel beneath the central business district. The twin 20-ft-diameter tubes were constructed in complex glacial soils and in the midst of existing structures such as tieback anchors, a railroad tunnel, and building foundations.

In glacial soils, natural hazards may simultaneously impact municipal property, utilities, and residential property. Miller's contribution reviews an illustrative case history from the Fauntleroy District of West Seattle.

One of the more challenging series of civil works projects in the Pacific Northwest was the regrading of the downtown Seattle area during the early years of the 20th century, which Morse records in his chapter. Morse brings a personal viewpoint to the discussion, as his father, William Chester Morse, was a project engineer for one of the regrade contractors, and both he and his father served as Seattle City Engineer.

Walsh and Bailey describe a recent example of problems involved in unwitting urban development above an abandoned coal mine in Renton. Renton, Issaquah, Black Diamond, Bellingham, and Roslyn have developed some areas above coal mine shafts and drifts. Hardly a year goes by without an incident involving a soil collapse associated with these early mines.

Buechel and Yamane's contribution illustrates the effectiveness of drainage as the primary remedial measure for landslide control in a residential area. As an engineering alternative to an expensive soldier pile wall, this approach won the American Society of Civil Engineers Small Project Excellence Award in 1986.

In the last paper in this chapter, the geologic conditions and remedial measures for a landslide along a major thoroughfare in a fast-growing suburban area, the Green River valley, are described by Evans.

As urban development spreads and increases in density, the engineering geologist's responsibility becomes more demanding and complex. Thorough research and knowledge of local history and geologic conditions are prime requisites for providing services to the public in an urban environment.

West Seattle residence damaged by a landslide. Photograph by William Evans, 1986.

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Washington Division of Geology and Earth Resources Bulletin 78

Runoff and Stream-Channel Changes Following Urbanization in King County, Washington

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INTRODUCTION

Changes in storm-water runoff rates and in the physical characteristics of stream channels typically accompany changes in land use. Such changes have been documented in a variety of studies (for example, Wilson, 1967; Seaburn, 1969; Hammer, 1972; Leopold, 1973), particularly during conversion of the drainage basin from forestry or agriculture to urban or suburban uses. The changes in runoff and stream channels affect not only in-stream conditions such as fisheries resources but also adjacent land uses by flooding, channel expansion and encroachment, and more widespread downstream sedimentation.

The west part of King County is undergoing both rapid urban development and substantial consequent changes to stream channels. Population had grown to 1.35 million residents in 1985; an additional 300,000 people are anticipated by the year 2000, suggesting that such impacts are likely to increase.

Outside of the main urban areas and even within many of them, storm-water runoff is collected and piped only locally. The discharge of each storm-sewer system is then released to the natural streams, which thus serve as the trunk storm-drainage network. It is along these natural components of the storm-drainage system that the hydrologic effects of urbanization are most noticeable.

King County is located in the eastern and central Puget Lowland (Figure 1), an area that has undergone multiple glaciations during the last 2 million years. Most of the developing parts of the county are underlain by moderately consolidated to unconsolidated ice-sheet deposits of the last glaciation, the Vashon Stade of the Fraser Glaciation (Armstrong et al., 1965), which dates between about 14,000 and 20,000 yr B.P. in this area (Waitt and Thorson, 1983). Rates of erosion and stream-channel incision are highly dependent on the variety of sediment locally forming the substrate. In addition, all but the largest rivers flow over a landscape whose gradients largely reflect glacial processes and not the long-term adjustment of topography to the existing drainage network. Thus stream channels of a given drainage area can vary greatly in their competence to transport sediment, in the type and amount of sediment that is delivered to them, and in their sensitivity to land-use changes.

HYDROLOGIC BACKGROUND

A hydrologic system is characterized by its predominant runoff process. Under the *overland flow regime*, the rate of precipitation typically exceeds the infiltration capacity of the soil. Water at and near the surface that cannot be absorbed flows above unsaturated ground as Horton overland flow (HOF). Where rainfall intensities are low relative to soil infiltration rates, a *subsurface flow regime* generates surface flows only near the base of slopes, if at all. Water is delivered to these areas by subsurface pathways, either through the surficial soil and duff layer ("shallow subsurface flow") or through deeper substrata. Surface water flows only downslope of where the water table intersects the ground surface; thus it differs from HOF by flowing over saturated ground.

Storm runoff generally arrives at a stream or other conduit within a few hours to perhaps a day or so. Where HOF dominates, the speed of surface-water flow (typically a few tenths of a meter per second) allows all parts of a basin as large as many tens to hundreds of square kilometers to contribute a portion of their precipitation to storm flow at the outlet. In contrast, subsurface flow rates may be as much as several orders of magnitude slower for most storms; the greater part of such basins will contribute only small volumes of runoff. Only the re-emergent ground water at the footslopes ("saturated return flow") and rain that falls on this saturated ground ("direct precipitation") will contribute to storm flow. As the storm continues, the ground-water table can rise or perched water tables can develop, and so the amount of ground contributing surface flow will also increase.

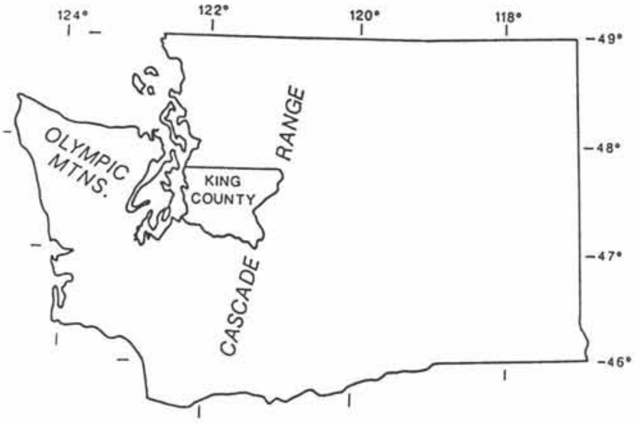


Figure 1. Location of King County

The magnitudes of storm flows are determined not only by the dominant runoff process but also by the sheer volume of water that reaches the stream channel. Reduction of this volume can occur in several ways:

(1) interception, storage, and re-evaporation of precipitation on the vegetation canopy;

(2) transpiration by plants;

(3) detention in or on the soil;

(4) evaporation from the soil surface; and

(5) percolation to deep ground water.

The first avenue of loss is particularly well correlated with hydrologic regime. Where the yearly precipitation falls mainly as a few intense storms, a fixed amount (representing a small percentage) of each storm's rainfall will be trapped in the canopy, while most of the remainder passes through to run off as overland flow. Where rainfall intensities are low enough to permit subsurface flow to dominate, that same fixed amount of intercepted precipitation per storm may substantially reduce the amount of water reaching the ground over many such low-intensity events. For example, data from small basins in New Zealand before and after clearcutting (Forest Research Institute, 1980) show interception to account for 450 mm per year, 24 percent of the total annual rainfall. Transpiration removed an additional 435 mm. Allowing for changes in soil evaporation, clearcutting increased the total annual water yield by 76 percent, from 945 mm to 1,665 mm per year. This change may raise the ground-water table, and thus increase the area of saturated ground, contributing to individual storm peaks in a basin. In the Piedmont region of the American Southeast, summary data on 80 small (less than 1 sq km or 250 acres) catchments (Trimble et al., 1987) yielded a relationship between annual streamflow change, y (in mm), and percentage of forest cover change, x:

$$y = 3.4x$$

Thus total forest-canopy removal there increases runoff by 340 mm per year.

Where subsurface flow previously dominated, the entire process of runoff generation changes with development. Parts of the basin that never before generated any surface runoff now deliver a percentage of their precipitation directly to the outlet. Water velocities increase by up to several orders of magnitude. Conversely, less water may be available for recharging deeper ground water because more is intercepted and removed at the ground surface. The result is a flashy stream on both storm and seasonal time scales, higher during storms and lower or dry altogether between storms.

Western Washington typifies conditions under the saturated flow regime. The region is underlain primarily by low-permeability till, lacustrine deposits, and bedrock, on which a more permeable soil has developed in the forest root zone over the last 14,000 yr since deglaciation. Particularly in these areas, therefore, the distinction between "shallow" and "deep" ground water is based on the physical characteristics of the substrata. Much more permeable glacial-outwash deposits cover most of the remaining area in a discontinuous layer one to many tens of meters thick. Precipitation falls primarily during low-intensity, frequent events; thus canopy interception figures significantly in the hydrologic balance, and infiltration capacities of undisturbed soil are rarely exceeded. The largest runoff peaks are produced by multi-day storms, which continue long enough to raise hillslope ground-water tables and thereby expand the area of runoff-producing saturated ground surrounding streams and swales.

SEDIMENT TRANSPORT AND STREAM-CHANNEL EROSION

Stream channels are commonly developed in sediment that the flow has transported in the past (and presumably will again in the future). Thus the channel reflects the entire suite of discharges that occur over time. Typically, these "alluvial" rivers fill the channel with their 1.5- to 2-yr flood. Yet this is not the only flow that moves appreciable sediment, nor does a channel necessarily maintain only that same capacity immediately after a higher magnitude event (Pickup and Rieger, 1979).

Although most sediment discharges from drainage basins in suspension, the slower, intermittent movement of sediment on the bed determines channel form. Hillslope sediment sources constrain the distribution of grain sizes in the channel, which typically allow rivers to be classified as either sand-bedded or gravel-bedded. The sediment moves in response to the shear stress applied by the moving water on the bed, once that stress has exceeded a critical minimum value that increases with increasing grain size. Although the mechanics of entrainment and transport should be equivalent for all sizes of bed sediments, the predominant particle size (sand or gravel) strongly influences channel behavior.

Sand-bedded streams form where coarser sediment is not supplied to the channel from upslope or upstream. The basal shear stress (τ_b) necessary to transport at least some bed sediment is typically attained at discharges not far in excess of base flow. The magnitude of that stress is determined by:

$$\tau_b = \rho_w \, g \, d \, S,$$

where ρ_w = density of water, g = gravitational acceleration, d = water depth, and S = energy gradient (≈ water-surface slope). At bankfull stage or greater, that critical transporting stress is greatly exceeded and sediment transport is active.

A variety of experimentally determined bedload transport equations relate the transport rate (q_B) to the applied shear stress, commonly with the basal stress (τ_b) in excess of the (low) critical transporting stress raised to the 3/2 power. Data from many rivers suggest that, at a given point along the channel, depth (d) changes with changing water discharge (Q) raised to about the 0.35 power. Thus the rate of bedload transport in sand-bedded streams increases with water discharge to a fractional power around 0.5 because

$$q_B \propto \tau_b^{1.5} \propto d^{1.5} \propto (Q^{0.35})^{1.5} \approx Q^{0.5} \quad \text{(Eq. 1)}$$

Thus big floods transport more bedload than small floods, but the proportional increase in transport rate is less than the increase in water discharge. Over the long term, the greater frequency of the 1- to 2-yr floods moves the largest net volume of sediment. This implies that these channels, which typically are filled by the 1.5-yr flood, are sized to just carry the flow that moves the most channel-forming sediment—an intuitively reasonable phenomenon.

Transport conditions for gravel-bedded rivers differ substantially. Observations and flume experiments by Parker et al. (1982) show that the bed-surface pavement, typically a single layer of clasts two to five times larger than the underlying grains, moves only at discharges that are roughly one-half or more of those at bankfull stage. This condition may be exceeded several times per year; but even during extreme floods, the applied shear stress will exceed the critical stress of the bed material by two or three times at most. Under these conditions the relationship between sediment and water discharges is highly non-linear and very sensitive because small shear-stress increases yield large changes in the *excess* of applied stress over threshold stress. A flow increase of 10 percent may increase sediment discharge by an order of magnitude. Despite this radical difference from sand-bedded streams, preliminary data suggest that the greatest long-term amount of bedload transport also occurs during the 1- to 2-yr floods.

Table 1. 1984-85 flow data, calculated depth, and predicted sediment transport per unit width of stream (Parker et al., 1982) for Upper Soos Creek gaging station

Discharge (cfs)	Depth (m)	Transport rate (m^3/m/sec) (for midpoint discharge in range)	Percent of time flow in discharge interval
10			82.9
30			8.0
50			3.2
70			2.1
90			1.0
110	0.42	0.02×10^{-4}	1.11
130	0.46	0.07×10^{-4}	0.73
150	0.49	0.2×10^{-4}	0.38
170	0.52	0.6×10^{-4}	0.21
190	0.55	1.2×10^{-4}	0.12
210	0.58	2.3×10^{-4}	0.04
230	0.61	4.1×10^{-4}	0.04

An example highlights these characteristics (Table 1). Continuous flow data and calibrated simulations using a numerical watershed model, HSPF (Hydrologic Simulation Program Fortran), are available on Soos Creek in southern King County. Discharge rates and duration data are from 1984-1985, a year with one near-bankfull flood and no extreme events. Bankfull channel dimensions and sediment sizes were determined by field measurement; average slope and drainage-basin area were derived from topographic maps. The basin as yet is only slightly urbanized. The depth-discharge relationship expressed in Table 1 (column 2) is derived from the measured bankfull depth extrapolated to other discharges by use of calibrated model results on Coal Creek (Figure 2), a slightly to highly urbanized catchment located 20 km north of Soos Creek (King County, 1987). Bedload transport (Table 1, column 3) is calculated using the equation of Parker et al. (1982).

Sediment transport can be estimated for larger flows as well. Discharge data can be extended to less frequent floods by using relationships derived from 34 yr of annual-peak records on Coal Creek and model results for ungaged subbasins (King County, 1987). These results (Table 2) also agree well with an earlier, regional com-

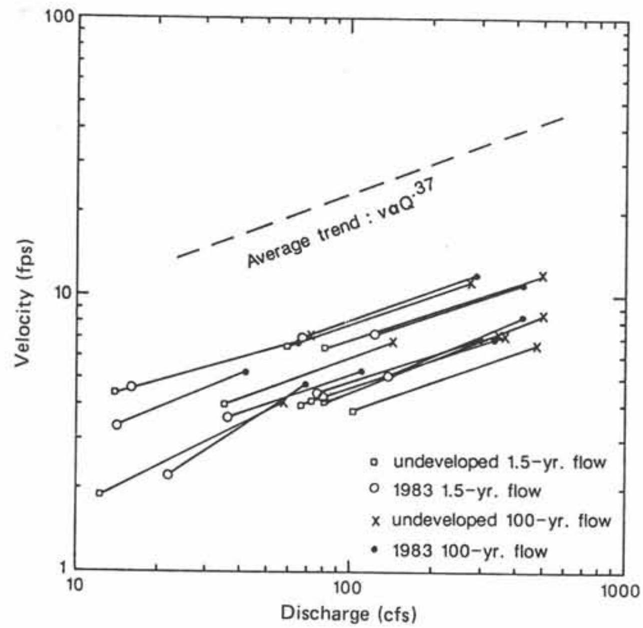


Figure 2. At-a-station changes in discharge and velocity for 1.5-yr and 100-yr flows in 18 subcatchments in Coal Creek (King County, 1987) under both developed and undeveloped conditions. Lines connect flood peaks measured at the same gaging point. Assuming channel width is proportional to $Q^{0.1}$ (Dunne and Leopold, 1978), the channel depth should vary with about $Q^{0.5}$.

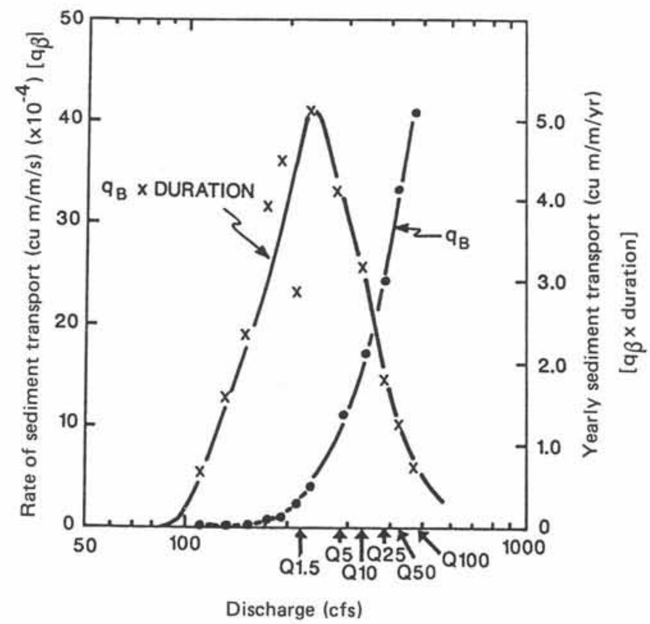


Figure 3. Bedload transport, calculated by the method of Parker and others (1982), for several flood discharges at the Upper Soos Creek gage. Both instantaneous sediment discharges (volume per unit width per unit time) and yearly cumulative sediment discharges are shown. Durations of multi-year flows are scaled proportional to the duration of $Q_{1.5}$, with assumed discharge of 220 cfs.

pilation by Frederick and Pitlick (1975), suggesting their applicability to this example as well. The sediment transport rate of course increases with increasing discharge, whereas the duration of any such flood with a longer recurrence interval is proportionally reduced (scaled relative to the duration of the 1984-1985 bankfull flood, assumed = $Q_{1.5\text{-yr}}$). The product of transport rate and flow duration yields the results graphed in Figure 3, namely the relative long-term amounts of sediment transported by various flows. Even though instantaneous transport rates increase dramatically with increasing water discharge in gravel-bedded streams such as this, bankfull stage remains the primary sediment-transporting flow for such streams as well.

Table 2. $Q_{flood}/Q_{1.5}$ for three flood levels, calculated for the 24 individual subcatchments of Coal Creek (King County, 1987). Depth ratios assumed $\alpha\ Q^{0.5}$

	$Q_{1.5}$	$\frac{Q_{10}}{Q_{1.5}}$	$\frac{d_{10}}{d_{1.5}}$	$\frac{Q_{25}}{Q_{1.5}}$	$\frac{d_{25}}{d_{1.5}}$	$\frac{Q_{100}}{Q_{1.5}}$	$\frac{d_{100}}{d_{1.5}}$
Average		2.2	1.5	3.0	1.7	4.5	2.1
Range of values	2.1-110 cfs	1.9-2.5	1.4-1.6	2.6-3.4	1.6-1.8	3.9-5.2	2.0-2.3

EFFECTS OF URBANIZATION

Basin Changes and Hydrologic Responses

Development is accompanied by activities having irreversible effects on drainage-basin hydrology, particularly in those areas previously dominated by subsurface flow. Roads are installed, collecting surface and shallow subsurface water in continuous channels with discrete surface-water crossings. Vegetation is cleared and the soil compacted and partially stripped. Regrading eliminates previously undrained depressions. Subsurface utilities, together with any other trenches specifically installed for drainage, intercept yet deeper subsurface water and rapidly pipe it out of the basin to a surface-water discharge. Building construction adds to the impervious surface cover over the basin, further reducing infiltration and speeding water flow to the basin outlet. Yet this final aspect of development only results in the last, and commonly not the most significant, impact to an area's hydrology.

The first such impact is the increase in total volume of surface-water runoff. As described previously, this may add several tens of centimeters to the annual runoff budget.

The second major impact is the concentration of water into surface channels. This effect is poorly quantified. In regions where HOF flow dominates, its impact may be negligible. Where most water moves in the subsurface, however, its interception and discharge into a nearby surface swale may subject surface soils to erosive processes that had not occurred in the undisturbed state, with dramatic and sometimes catastrophic results (such as the case histories that follow).

The third, most widely recognized impact of drainage-basin development is the increase in peak discharge at the basin outlet, commonly expressed by the increase in discharge for floods of a given return frequency. The change is well demonstrated by Coal Creek model output for the (simulated) undeveloped state and the (gaged and simulated) 1983, partly developed basin (King County, 1987). In Figure 4, the 1983 1.5-yr and 100-yr peak discharges from subbasins ranging from 0.3 to 18 sq km (80 to 4,500 acres), expressed as a proportion of their predevelopment peaks, are plotted against the basin area covered by impervious surface area. The correlation between development and increased discharge is irregular but nonetheless obvious.

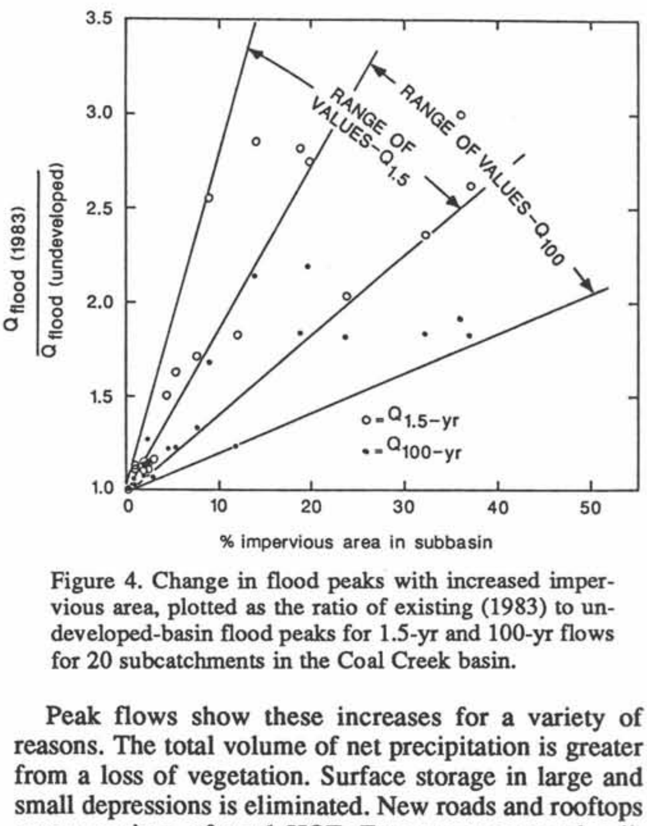


Figure 4. Change in flood peaks with increased impervious area, plotted as the ratio of existing (1983) to undeveloped-basin flood peaks for 1.5-yr and 100-yr flows for 20 subcatchments in the Coal Creek basin.

Peak flows show these increases for a variety of reasons. The total volume of net precipitation is greater from a loss of vegetation. Surface storage in large and small depressions is eliminated. New roads and rooftops assure regions of total HOF. Even on uncovered soil, runoff increases because construction or logging equipment can reduce the surface permeability by more than 90 percent. Nearly all of this reduction occurs in a single pass by such vehicles (Moore et al., 1986). Disseminated subsurface flow is intercepted by ditches and trenches, and small surface-water rivulets are combined into larger channels and pipes. These effects can dramatically increase the rate at which water is delivered to the basin outlet. The entire basin can respond to shorter and more intense rainfall peaks; it also must discharge a greater total volume of water as stormflow, typically in a much shorter period of time. During larger storms, the undeveloped basin generates most of its runoff from direct precipitation and return flow from the (maximally expanded) saturated zones surrounding swales and streams. Thus the developed state shows less increase during larger storms because constructed impervious surfaces do not represent as radical a change under these conditions.

Stream channels respond rapidly to basin urbanization. Measured increases in channel cross-sections are documented in various studies (for example, Hammer, 1972; Leopold, 1968). The initial response may be almost immediate, and the channel itself may re-equilibrate with the increased flows within a few years. Yet the hillslope failures resulting from increased bank undercutting may not achieve an equivalent, pre-development rate for decades or perhaps centuries following even a single episode of channel enlargement. Under some circumstances channel incision may continue (see below); an increased rate of slope failures may then become a permanent regime.

The progressive effects of development may be partly quantified. Although no discrete threshold is suggested by Figure 4 (in contrast to Morisawa and LaFlure, 1979, figure 11), subbasins where the impervious areas exceed 20 percent experience nearly doubled 100-yr flows, and 1.5-yr flows average perhaps 2 1/2 times greater, than in the predeveloped condition. (Typical suburban development has about 35 percent impervious area.) This 2 1/2-fold discharge increase may transport up to ten times more sediment than in the predeveloped state (refer to Figure 3). The 100-yr flow (and presumably intermediate events) will transport proportionally less sediment over time, so the dominance of the more frequent flood stages on long-term sediment transport will be enhanced. Finally, because 1- to 2-yr floods typically correspond with bankfull stage, the new channel should eventually adjust to accommodate about twice or more the undeveloped discharge.

Where well-graded sediment is delivered to a stream channel, an increased water discharge will yield an increase not only in the rate of bedload transport but also in the size of sediment flushed out as suspended load. As a result, the average bed-sediment size increases. Bed slope will tend to decrease, because any increased hillslope delivery of sediment lags (and ultimately is driven by) this accelerated rate of channel transport. Thus the bed undergoes a net degradation. The end product will be increased water discharge that is less capable at moving a less mobile (coarser) bed. Bedload transport does increase but is feedback-limited, because the physical changes in the channel limit the effectiveness of increased discharge due to development.

In sand-bedded streams, the bedload cannot coarsen appreciably. Thus bed-slope reduction will be qualitatively more important in equilibrating sediment flux with water discharge; this setting more closely achieves Mackin's (1948) concept of a graded river, where only channel gradient adjusts to balance sediment and water fluxes. The end product in such cases is likely to involve a greater degree of channel downcutting than in their coarse-bedded counterparts.

Mitigation of Development-Related Impacts

Increased flooding and stream-channel erosion have long been recognized as undesirable but virtually unavoidable byproducts of development. Mitigation of these drainage-related impacts generally has occurred only where explicitly required by the governing municipality; thus the status of storm-water control in a basin is well represented by the history of applicable regulation during its development. In King County, present (1987) requirements for most basins require that post-development floods up to the 10-yr event may discharge at no more than the predicted pre-development 10-yr rate. Storms below this level may discharge up to this level; storms above this level may overflow without further control. Where downstream flooding is a known problem, control up to the 25-yr storm, with discharge at a pre-development 5-yr rate, can be imposed.

Historically, these controls have evolved to address the most immediate drainage problems, typically related to flooding. Standards therefore emphasize large storm peaks, with the maximum requirement representing an uneasy compromise between existing downstream property rights and developers' reluctance to invest in onsite drainage systems that benefit only off-site owners. Smaller storm peaks receive little attention because their flows remain contained (or nearly so) by the existing channel.

In many parts of King County, these controls have proven inadequate to control runoff problems for a variety of reasons. First, in many instances the initial prediction of flows to be matched is unreliable. Second, the structures built to control discharges may not function as designed. For example, efforts to achieve infiltration of storm water suffer from overly optimistic recharge rates derived from shallow percolation tests, careless construction practices that destroy the permeability of the soil, and rapid clogging from inadequate erosion control in the surrounding basin (Jensen, 1987). Third, storms larger than the design event may still occur too frequently to be tolerated. For example, a 10-yr flow (by definition, a flow with a 1/10 chance of occurring in any year) has a 99.5 percent chance of being equalled or exceeded at least once in a 50-yr period. Even if flows up to the 25-yr event are controlled, an 87 percent chance exists that this level too will be exceeded during any 50-yr period. Depending on the nature and density of downstream habitation, these may not be acceptable flooding probabilities.

Finally, these controls scarcely affect any potential increase in stream-channel *erosion* below developed basins. In the undisturbed basin, the 1- to 2-yr flows move the most sediment over time. Even if the most restrictive 5-yr-discharge control is imposed on the development, these near-annual storms will now discharge from the developed, controlled basin at peak rates as much as 60 percent higher (the ratio of $Q_{5\text{-yr}}$ to $Q_{1.5\text{-yr}}$ in this region). In gravel-bedded rivers, bedload transport rates will increase 4- to 5-fold. (Refer to Figure 3.) The greater volume of storm water, necessitating a longer duration of discharge, will further increase the total transport of material.

Special restrictions on larger storms, such as restricting large post-development storms (for example, the 100-yr event) to even smaller pre-development discharges (for example, the 10-yr flow), are no more effective. Referring to Figure 4 (ratio of pre- and post-development 100-yr storms) and Table 2 (ratio of 100-yr to 10-yr flows), a 100-yr post-development storm from an uncontrolled basin discharges at about four times the rate of the pre-development 10-yr storm (that is, the design standard). Releasing this 100-yr storm at the controlled 10-yr rate is a vast improvement over no control because the lowered peak will limit the size of the coarsest bedload that can be moved. Yet sediment transport is still likely to be more vigorous than during the *pre*-development 100-yr event because the necessarily increased duration moves a greater total volume of sediment.

In sand-bedded rivers, where problems of channel scour and bank erosion can be dramatically worse, impacts are even more difficult to control. By equation 1, sediment discharge per unit time increases as $Q^{0.5}$. Yet for a given volume of storm water, such as collected in a retention structure awaiting restricted release, the duration of flow varies inversely with Q. So a 100-yr storm released from a developed basin at the pre-development 10-yr rate will move only 60 percent of the sediment per unit time relative to the predeveloped condition—but it will do so for as much as four times longer. Paradoxically, more restrictive release rates with correspondingly longer durations actually make the problem worse. This condition persists until basal shear stresses are low enough that sediment transport no longer varies with $\tau_b^{1.5}$, implying that a release rate that will not augment erosion on sand-bedded streams must be kept little greater than base flow for most storms.

CASE HISTORIES

In King County, a number of localities illustrate the activity of stream-channel erosion following upstream development. In none of the following instances were established regulations ignored or misapplied; instead, these examples illustrate the difficulty of mitigating the effects of drainage-basin disturbance and the remarkable speed at which the channels may respond to those changes.

Snoqualmie Valley Sidewalls

Along the west wall of the Snoqualmie River valley (Figure 5), post-glacial incision and undercutting by the Snoqualmie River has left a 150-m-high slope with an average gradient of more than 25 percent. The upper several meters are low-permeability lodgment till of Vashon age (15,000 yr); below the till lies between 50 and 100 m of Vashon-age advance outwash, an uncemented fluvial sand deposit with some gravel primarily near the top. Below this deposit lies a silt and clay layer, mapped as transitional beds of early to pre-Vashon age by Minard and Booth (1988). Postglacial runoff from the nearly flat till uplands is collected in a dendritic set of ravines deeply incised into the advance outwash. Over the intervening millennia since deglaciation, these channels have lowered their gradient relative to the prevailing slope of the valley wall to balance the influx of sideslope sediment, the transport of sediment down the channel, and the water discharge received primarily by shallow subsurface flow from above.

Recent development in this area began with logging of second-growth timber on the upland till surface in the late 1970s. Shortly thereafter, the road system was improved to a paved surface, located a few tens up to about a hundred meters from the edge of the slope, with side ditches and culverts at low points (about one per 200 m). The land was subdivided into multi-acre residential lots; about half of the twenty or so parcels in the area are presently built upon. Because of the lot size and procedure by which the land was subdivided, no stormwater detention was required, nor was any provided.

A variety of changes to downstream water courses has occurred since this activity began. Probably the most dramatic change has occurred downstream of a single 12-in. culvert; the drainage area of 0.3 sq km (250 acres) presently contains four houses and 300 lineal meters of two-lane road. Downstream of the culvert, stormwater flows in a small surface channel, presently (summer 1987) incised between 10 and 100 cm into the upland surface with a gradient of about 15 percent. It then drops over a vertical 3-m headwall of a 2-m-wide gully paved with cobbles and boulders to 40 cm diameter (Figures 6a). After an additional 30 m of transit, the water plunges down a nearly vertical 20-m precipice eroded into the advance outwash, forming the rear wall of a sinuous slot averaging no more than 1 m in width (Figures 6b, 6c). The base of this slot lies less than 5 m above the contact of the sandy outwash with the underlying transitional beds; emergent ground water joins the surface flow here and the ravine widens considerably, with hillslope sediment up to 10 m above the channel bottom involved in active mass movement.

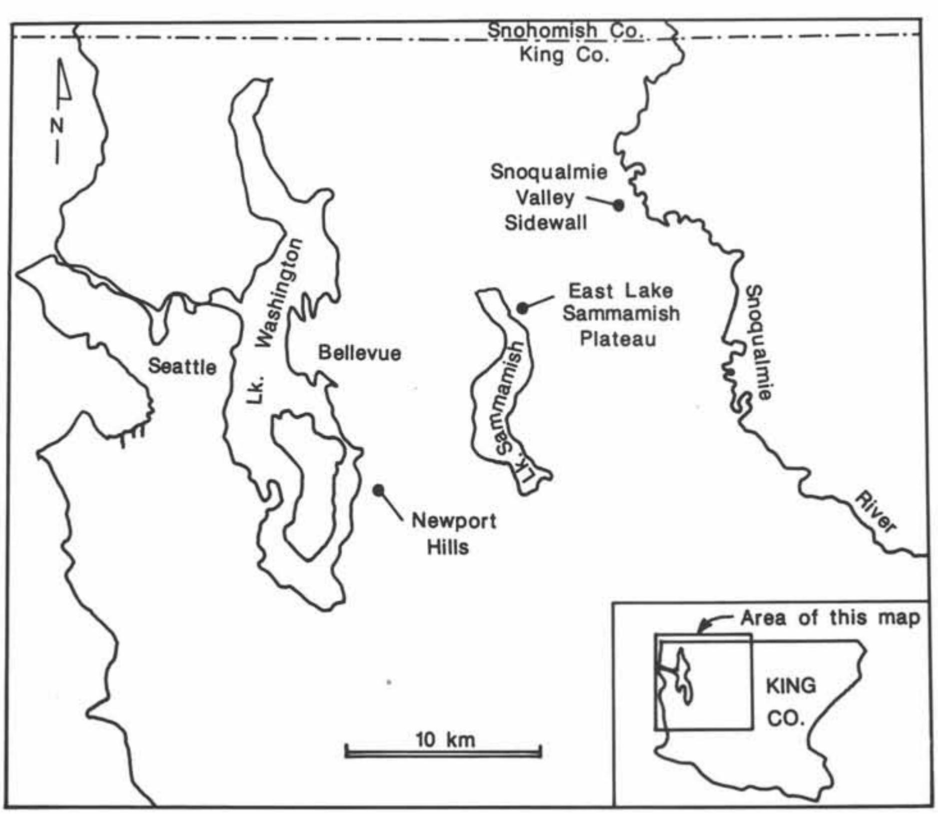


Figure 5. Location of examples discussed in text.

6a

6b

6c

Figure 6. Stream incision in the Snoqualmie River valley sidewalls. 6a, Upper channel. Low-gradient upstream channel terminates abruptly at headwall. Flow is directly towards camera. 6b, Headwall of lower channel reach. Only the upper one-third of the headwall is included in this picture. 6c, View vertically down and downstream from the top of the headwall (viewed from the location of the hat in Figure 6b).

Rates of erosion can be estimated in this setting. The road and culvert system was probably installed in 1979 or 1980. Judging from other swales on this hillside, significant surface-water flow and consequent incision was probably absent in the upper channel before this time. The incision was first noted in summer 1986 and erosion markers placed soon thereafter. Following a single winter, the upper and lower headwalls have each retreated an additional 30 to 50 cm. Landsliding on the lower canyon sidewalls has caused at least 2 m of sideslope retreat. There is no indication that these processes are presently decelerating, although vertical incision of the lower channel reach has probably reached near-equilibrium because of a base level set by a nearby trunk stream.

This locality exemplifies the hazards of uncontrolled discharge of surface water into previously unutilized topographic swales, which may owe their formation to enhanced mass-wasting rates in more saturated soils but which do not normally carry surface runoff even during extreme events. The water is not only misplaced, it is augmented by tree clearing, soil compaction, (limited) hard surfaces, and interception of shallow subsurface water. This system is particularly responsive because the advance outwash is highly erodible, lacks adequate armoring gravel in the parent deposit, and is exposed in this area on high-gradient slopes. The channel and surrounding sideslopes will seek and eventually attain a new equilibrium configuration, as they did following deglaciation. Although substantially less transport of sediment will be needed to reach this new stability than occurred during postglacial time, surrounding land use and land users are currently much less tolerant of such changes than during the early Holocene.

Newport Hills Area

As the intensity of uncontrolled development increases, the rate and magnitude of stream-channel impacts may also increase. Although nowhere in King County are erosive sand deposits exposed as thickly and steeply as above the Snoqualmie River, elsewhere in the county these deposits are located where more intensive urbanization has already occurred during an era when no drainage controls were required for development. One such locality is the "Newport Hills Tributary" to Coal Creek, a stream that descends from till-covered uplands into the Lake Washington trough (Figure 5). Along this particular tributary, years of uncontrolled discharge eroded a canyon that not only mobilized very large volumes of sediment but also had initiated severe sideslope landslides (Figure 7). Incision of as much as 10 m has occurred over about 20 yr, involving 1,000 m of channel length and threatening more than 50 houses above. Repairing the impacts of inadequately controlled runoff here has required a partly implemented, $3 million program to armor and pipe the stream.

Figure 7. Gully incision along the Newport Hills tributary to Coal Creek.

East Lake Sammamish Plateau

Between the troughs of Lake Washington and the Snoqualmie River lies yet a third trough, Lake Sammamish, with its own sideslopes of thick, steeply exposed sandy advance outwash underlying upland till. Development here is proceeding at a rapid pace on the upland surface. Because of severe flooding at the toe of the slopes above Lake Sammamish, all developments draining toward the lake's east side must limit flows up to the 25-yr storm to no more than the 5-yr, pre-development release rate. Because the soils are known to be easily eroded, some care has recently been taken to dissipate flow from storm-system outfalls. Despite these controls, severe erosion is occurring in each of the sideslope channels that serve to convey storm water from developed areas.

The best documented example (Figure 5) involves a small swale, previously dry in its upper reaches, that now receives storm water via a detention vault from about thirty 1/8-acre residential lots. Flow is discharged into the head of the swale through a 20-m-long dispersion pipe that began functioning early in 1986. During the summer of 1986, faint signs of intermittent surface flow in the form of displaced leaves, sorted sand pockets behind obstructions, and local rivulets as much as 5 cm deep were visible in the upper 150 m of channel on a 15 percent gradient. Below this reach, the overall gradient steepens, with the flows ultimately merging with a perennial stream of indeterminant surface drainage area fed by a 200-sq-m boggy area on the hillside. Over the next 2 km down to Lake Sammamish, stream channel incision of 50 cm was typical; local incision of about 1 m was the extreme amount observed.

By the summer of 1987, conditions had changed. The lower one-half of the upper 150-m reach now was incised up to 1.5 m. Immediately upstream of this gully's headwall, a shallow intermittent surface channel ran between vertical pipes or burrow holes that led about 80 cm deep to a probably continuous horizontal piping channel, presumably representing an intermediate subsurface stage in gully formation (Dunne, 1980). The runoff had incised an entirely new route down to the main, spring-fed stream.

Incision on the main stream now averaged 1 m below the junction and locally exceeded 2 m (Figure 8a). An 18-in. culvert clogged and washed out in late 1986 near the base of the slope, where eroded sediment begins to drop out as the slope gradient decreases (Figure 8b). Two additional private culverts and a state highway remain in the path of the runoff and its sediment load.


a

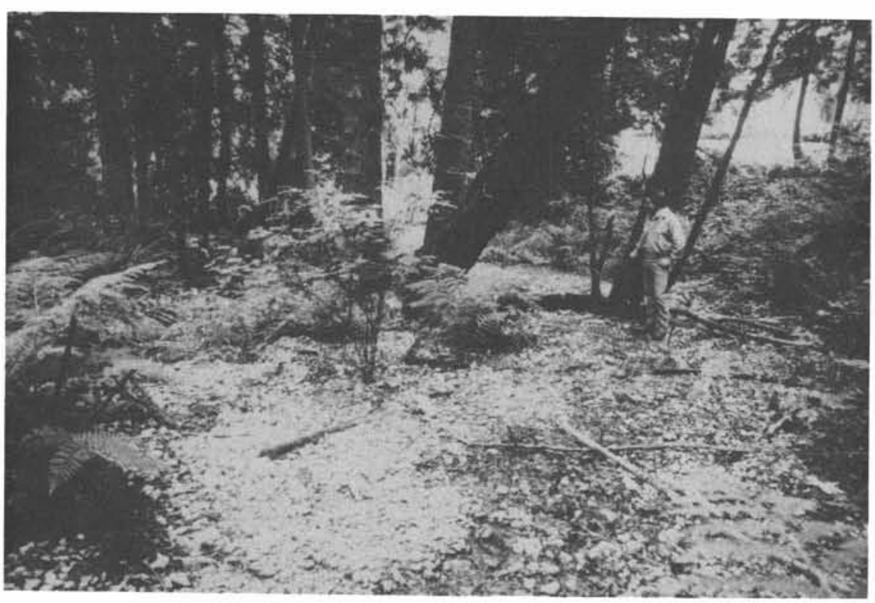

b

Figure 8. Stream incision off the East Lake Sammamish plateau.

8a, Recent incision into hillslope colluvium. Flow is from left to right; average stream channel gradient about 15 percent.

8b, Depositional fan just upstream of washed-out culvert. Aggradation here is less than 1 yr old.

Although efforts at storm-water control in urbanizing areas surely provide a substantial level of mitigation over what would result without them, sediment-transport analyses demonstrate that these efforts do not re-attain undeveloped drainage conditions. Adjustment of stream channels will result wherever urbanization occurs; what makes these (and other geologically similar) areas distinctive is the degree to which the physical channel must change in response to changing flow conditions and the speed at which those channel changes can progress. If development is to occur without substantial and sometimes catastrophic impact to downstream stream channels, in-stream inhabitants, and sideslope property owners, these highly sensitive areas must be recognized and a set of genuinely effective drainage controls accepted by municipality and developer alike.

SUMMARY

Climate, soils, and vegetation together determine a region's dominant process of hydrologic response to storm events. In regions such as western Washington, runoff follows slow, subsurface flow paths over most of the basin, a condition that is radically altered by typical urban storm-water management.

Fundamental differences in bedload transport exist between sand-bedded and gravel-bedded stream channels. Although both types of channels are typically filled to capacity by the 1- to 2-yr flood in the undisturbed state, the rate of sediment transport in gravel-bedded streams is much more sensitive to changes in water discharge over the range of typical flood flows.

Urbanization imposes several changes on the drainage basin. Vegetation removal, soil compaction, new impervious surfaces, and piping increase both the rate and total volume of storm runoff. Where this runoff is discharged into existing gravel-bedded channels, the likely results are incision and bed coarsening. Where discharged into sand-bedded channels, only channel incision (with its consequent reduction of water-surface gradient) can accommodate the effects of enhanced sediment transport. As a result, incision is likely to be much greater in such channels. If the discharge point is a swale not previously occupied by surface water, the impacts will be even more dramatic. Typical storm-water management techniques, although perhaps adequate to control some flooding problems downstream of development, are generally ineffective in addressing impacts to stream channels from enhanced sediment transport.

Three examples from western King County demonstrate the potential severity of stream-channel erosion problems. Not coincidentally, all involve incision through the sand-dominated advance outwash deposits, commonly exposed on moderate to steep side-slopes adjacent to major topographic troughs. The size, density, and drainage controls of the individual developments are at most secondary determinants of the rate and magnitude of channel changes. The ineffectiveness of efforts to date, and the expense necessary to correct problems once they have occurred, argue for better recognition of susceptible channels before development begins and for the establishment of effective strategies for control or avoidance.

REFERENCES

Armstrong, J. E.; Crandell, D. R.; Easterbrook, D. J.; and Noble, J. B., 1965, Late Pleistocene stratigraphy and chronology in southwestern British Columbia and northwestern Washington: *Geologic Society of America Bulletin*, Vol. 76, pp. 321-330.

Dunne, T., 1980, Formation and controls of channel networks: *Progress in Physical Geography*, Vol. 4, pp. 211-239.

Dunne, T. and Leopold, L. B., 1978, *Water in Environmental Planning*: W. H. Freeman, San Francisco, CA, 818 p.

Forest Research Institute, 1980, Streamflow from forests: *What's New in Forest Research*, Rotorua, New Zealand, No. 92, 4 p.

Frederick, J. E. and Pitlick, J., 1975, *A dimensionless rating curve for the rivers of the Puget Lowland*: Department of Geological Sciences, University of Washington, Seattle, WA, unpublished report.

Hammer, T. R., 1972, Stream and channel enlargement due to urbanization: *Water Resources Research*, Vol. 8, pp. 1530-1540.

Jensen, B., 1987, *An investigation of storm water management using infiltration basins in King County, Washington* {M.S.thesis]: Department of Civil Engineering, University of Washington, Seattle, WA, 46 p.

King County, 1987, *Coal Creek Basin Plan and Final Environmental Impact Statement*: King County, Seattle, WA, 230 p.

Leopold, L. B., 1968, *Hydrology for Urban Land Planning–Guidebook on the Hydrologic Effects of Urban Land Use*: U.S. Geological Survey Circular 554, 18 p.

Leopold, L. B., 1973, River channel change with time–An example: *Geological Society of America Bulletin*, Vol. 84, pp. 1845-1860.

Mackin, J. H., 1948, Concept of the graded river: *Geological Society of America Bulletin*, Vol. 59, pp. 463-512.

Minard, J. P. and Booth, D. B., 1988, *Geologic Map of the Redmond Quadrangle, King County, Washington*: U.S. Geological Survey Miscellaneous Field Investigation Map MF-2016, 1 sheet, scale 1:24,000.

Moore, I. D.; Burch, G. J.; and Wallbrink, P. J., 1986, Preferential flow and hydraulic conductivity of forest soils: *Soil Science Society of America Journal*, Vol. 50, pp. 876-881.

Morisawa, M. and LaFlure, E., 1979, Hydraulic geometry, stream equilibrium and urbanizations. In Rhodes, D. D., and Williams, G. P. (editors), *Adjustments of the Fluvial System*; Geomorphology Symposium, Tenth Annual Meeting, Binghamton, NY, pp. 333-350.

Parker, G.; Klingeman, P. C.; and McLean, D. G., 1982, Bedload and size distribution in paved gravel-bed streams: *Journal of the Hydraulics Division*, ASCE, Vol. 108, pp. 544-571.

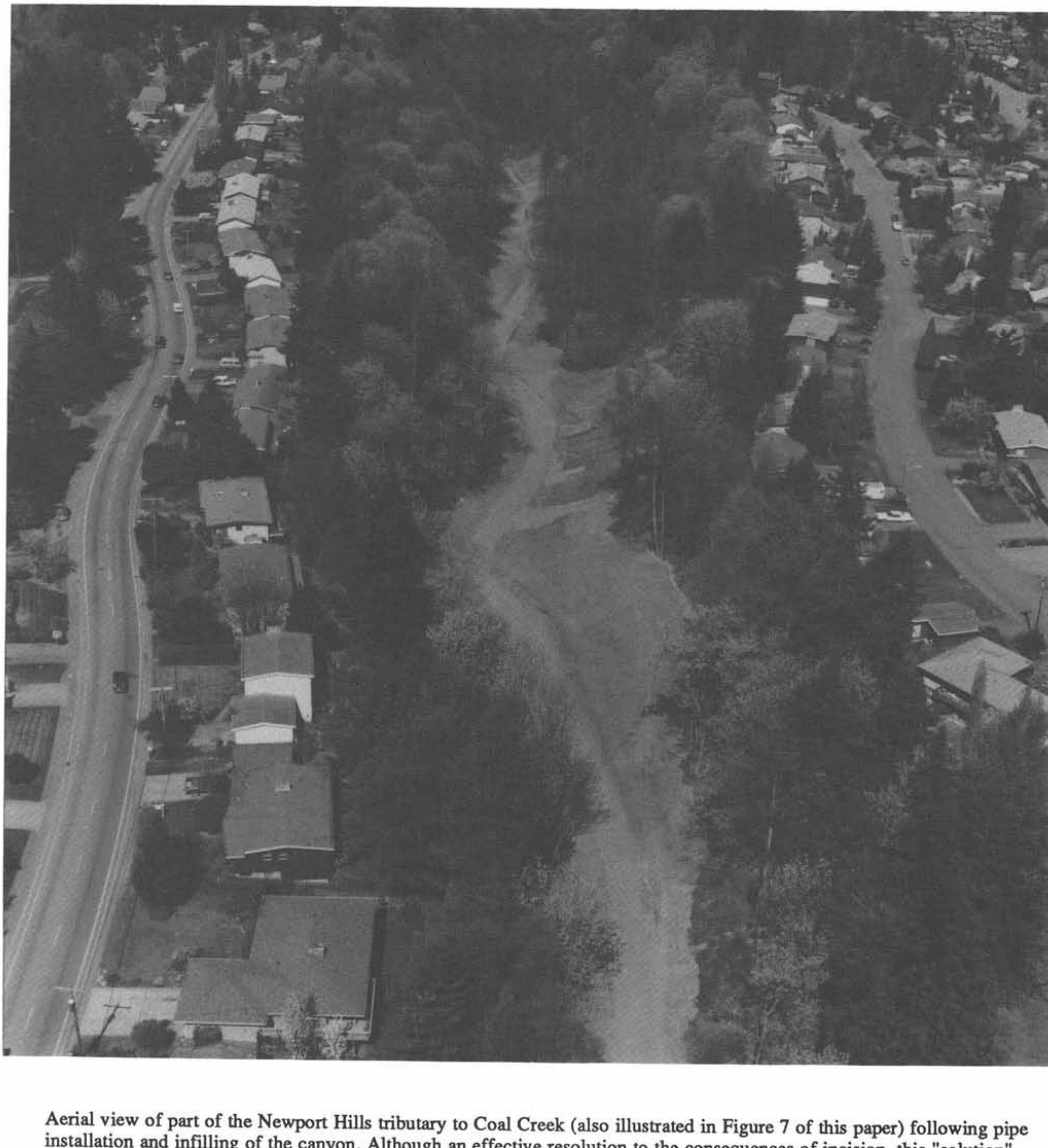
Pickup, G. and Rieger, W. A., 1979, A conceptual model of the relationship between channel characteristics and discharge: *Earth Surface Processes*, Vol. 4, pp. 37-42.

Seaburn, G. S., 1969, *Effects of Urban Development on Direct Runoff to East Meadow Brook, Nassau County, Long Island, New York*: U.S. Geological Survey Professional Paper 627-B, 14 p.

Trimble, S. W.; Weirich, F. H.; and Hoad, B. L., 1987, Reforestation and the reduction of water yield on the southern piedmont since ca. 1940: *Water Resources Research*, Vol. 23, pp. 425-437.

Waitt, R. B., Jr. and Thorson, R. M., 1983, The Cordilleran ice sheet in Washington, Idaho, and Montana. In Porter, S. C. and Wright, H. E., Jr. (editors), *Late-Quaternary Environments of the United States*: University of Minnesota Press, Minneapolis, MN, pp. 53-70.

Wilson, K. V., 1967, *A Preliminary Study of the Effects of Urbanization on Floods in Jackson, Mississippi*: U.S. Geological Survey Professional Paper 575-D, pp. D259-D261.



Aerial view of part of the Newport Hills tributary to Coal Creek (also illustrated in Figure 7 of this paper) following pipe installation and infilling of the canyon. Although an effective resolution to the consequences of incision, this "solution" engenders such cost and environmental damage itself that early recognition and mitigation represent the only rational long-term approach to these problems.

Engineering Geology in Washington, Volume II
Washington Division of Geology and Earth Resources Bulletin 78

Foundations and Excavations for High-Rise Structures in Downtown Seattle

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INTRODUCTION

This paper presents a summary of foundation, wall, and shoring design criteria used for high-rise structures in downtown Seattle, Washington. It is based on geotechnical reports and records of more than 35 building projects. Most of the structures have deep basements for parking, which required vertical excavations in excess of 30 ft. The following pages present a brief description of the soils encountered at these sites, the various foundation types and bearing pressures, and a discussion of the lateral earth pressures used for the design of permanent and temporary walls, including a description of shoring system elements. The accumulated data are summarized in Table 1, and the locations of the buildings are shown in Figure 1.

SEATTLE AREA SOILS

The Puget Sound area was subjected to several major glaciations during the Pleistocene Epoch. Glacial ice originated in the cordilleran and coastal mountains and the Vancouver Range of British Columbia. The maximum southward extent of ice reached the area about half way between Olympia and Centralia. As a result of these glaciations, the Puget Lowland was filled to significant depths by glacial and nonglacial sediments during the Pleistocene.

All structures in the downtown Seattle area are constructed on soils which have been glacially deposited and/or glacially overridden. The soil strata in the greater Seattle area include glaciolacustrine and interglacial clays and silts, glacial outwash sands and gravels, lodgment till, and glaciomarine drift. The distinction between lodgment till and glaciomarine drift is recent and somewhat controversial (Shannon & Wilson, 1986). Much of what is now considered to be glaciomarine drift was in the past classified as till where it possesses till-like texture and composition. Drift deposits were usually classified as glaciolacustrine clay or silt where they did not contain sufficient sand and gravel to be considered till. Soil descriptions for glaciomarine drift listed in Table 1 are based upon geotechnical engineering reports and our experience in downtown Seattle.

Clays and Silts

Excavations for many large high-rise structures in Seattle have encountered very stiff to hard, gray, clays and silts. Most of these fine-grained soils were deposited in glacial meltwater lakes and were subsequently covered with coarser outwash before being overridden and densely consolidated by glacial ice. Clay and silt deposits of interglacial periods which have been glacially overridden have also been encountered, but these are generally found south of James Street. Interglacial deposits are distinguished from glacial deposits by having a greater organic content and lenses of fine to medium sand that contain red andesite particles.

The clays and silts range from massive to laminated and in places are blocky or fractured and slickensided. Locally they are distorted or sheared and have a "diced" appearance, thereby having a lower mass strength than the surrounding soil.

Sands and Gravels

Most sands and gravels at Seattle high-rise construction sites are glaciofluvial outwash deposits that range in composition from silty fine sand to clean gravel. They are generally stratified, but changes in grain size occur over short distances vertically and horizontally. The deposits are generally water-bearing, but the amount of recharge varies widely. Ground water in these deposits is generally perched on underlying glaciomarine drift or lacustrine deposits. Probably the most widespread of these outwash deposits is the Esperance Sand Member of the Vashon Drift, which in most places is a clean to slightly silty, locally gravelly, fine to medium sand.

Lodgment Till

Lodgment till consists of gray, gravelly sand with scattered cobbles and boulders in a clay/silt matrix. It usually contains from 15 to 35 percent fines; the fine materials are generally of low to medium plasticity. It is very dense and often referred to as "hardpan". Till has considerable cohesion and high frictional shear strength and is generally considered one of the more competent bearing soils in the area. This unit is generally present

Figure 1. Locations of downtown Seattle buildings discussed in text and Table 1.

Table 1. Foundation and excavation data for high-rise structures in downtown Seattle; see Figure 1 for building locations. Soil types are described in the text; (B), belled drilled pier; (S), straight shaft

Bldg. no.	Building	No. of stories	Approx. compl. date	Excavation depth (ft)	Excavated soils	Temporary lateral earth pressure (psf)	Permanent lateral earth pressure	Foundation type	Foundation bearing soils	Foundation bearing pressures (ksf)			
												Drilled piers	
										Footings	Mats	Shaft friction	End bearing
1	Washington Building		1958	25				Footings	Glacial outwash	8			
2	Plaza 600	20	1969	20				Footings	Glacio-lacustrine	10			
3	4th & Blanchard Bldg	25	1978	30	Hard silt and clay	32H		Footings	Glacio-lacustrine	6 12			
									Glacial outwash				
4	Westin Bldg	34	1981	15				Footings	Glacio-lacustrine (clay)	6-8			
5	Columbia Center	76	1983	120	Hard clay over dense sand	27H	40 pcfl	Footings	Glacial outwash	15-20			
6	IBM Bldg	20	1963	40	Hard clay/ dense silt	40 pcf	50 pcf	Mat	Glacio-lacustrine (clay)		9		
7	Westin Hotel	40	1968	24	Hard clay	25H	35H psf	Mat	Glacio-lacustrine (clay)		12-20		
								Footings	Glacio-lacustrine (clay)	10			
8	Park Place	21	1970	30				Mat	Glacio-lacustrine (clay)		5		
								Footings	Glacio-lacustrine (clay)	8			
9	1600 Bell Plaza	33	1974	50	Dense sand	22H		Mat	Glacial outwash		10		
								Footings	Glacial outwash	8-10			

Table 1. Foundation and excavation data for high-rise structures in downtown Seattle (continued)

Bldg. no.	Building	No. of stories	Approx. compl. date	Excavation depth (ft)	Excavated soils	Temporary lateral earth pressure (psf)	Permanent lateral earth pressure	Foundation type	Foundation bearing soils	Foundation bearing pressures (ksf)			
												Drilled piers	
										Footings	Mats	Shaft friction	End bearing
10	Rainier Tower	40	1975	50	Hard clay	40H	50 pcf	Mat	Glacio-lacustrine		16		
								Footings	Glacio-lacustrine (clay)	8			
11	Seattle Hilton	25	1978	55	Very stiff to hard clay and silt	30H		Mat	Glacio-lacustrine (silt)		12		
								Footings	Glacio-lacustrine (silt)	12			
12	1111 3rd Ave	34	1979	50-65	Dense sand over hard clay	22H	22H psf	Mat	Glacial outwash		8		
						36H	36H psf	Footings	Glacial outwash	10			
13	Marsh & McLennan, 720 Olive Way	19	1980	40	Loose to medium dense sand	30H	30H psf	Mat	Glacio-lacustrine (silt)		5-6		
								Footings	Glacio-lacustrine (silt)	8			
14	Continental Place	33	1980	50	Medium dense sand over hard clay/silt	48H	48H psf	Mat	Glacial outwash		6-8		
								Footings	Glacial outwash	16			
15	First Interstate Center	48	1982	80	Hard clay and silt over dense sand	30H	30H psf	Mat	Glacio-marine drift		8		
								Footings	Glacio-marine drift	10-12			
16	Block 5 Development	55	1988	80	Dense silt/sand and hard clay/silt	20H	20H psf	Mat	Glacial outwash		10		
						26H	26H psf	Footings	Glacial outwash	14			

Table 1. Foundation and excavation data for high-rise structures in downtown Seattle (continued)

Bldg. no.	Building	No. of stories	Approx. compl. date	Excavation depth (ft)	Excavated soils	Temporary lateral earth pressure (psf)	Permanent lateral earth pressure	Foundation type	Foundation bearing soils	Foundation bearing pressures (ksf)			
										Footings	Mats	Drilled piers	
												Shaft friction	End bearing
17	Westlake Mall	20	1988	40-50	Fill over dense silt/ sand	26H	26H psf	Mat	Glacio-lacustrine (silt)		10		
								Footings	Glacio-lacustrine (silt)	14			
18	Gateway Tower	55	1989	60	Hard clay and silt	25H	35 pcf	Mat	Glacio-lacustrine (clay)		16-20		
								Footings	Glacio-lacustrine (clay)	10			
19	Seattle 1st Nat Bank Bldg	50	1967	75	Hard clay over dense silt and sand	48H	48H psf	Drilled Piers(B)	Glacio-marine drift			N/A	30
20	Unigard Financial Center	27	1970	60	Dense sand	21H	21H psf	Drilled Piers(B)	Glacial outwash			N/A	30
								Footings	Glacial outwash	12			
21	Bank of California	42	1970	64	Hard clay	48H		Drilled Piers(B)	Glacio-marine drift			N/A	40
								Footings	Glacio-lacustrine (clay)	8			
22	Pacific Bldg	23	1970	20				Drilled Piers(B)	Glacio-marine drift			N/A	50
23	Henry M. Jackson Building	37	1972	36	Dense sand over hard clay	29H	29H psf	Drilled Piers(B)	Glacio-lacustrine (clay)			N/A	20
24	Peoples Nat Bank	19	1972	40	Sand fill	22H		Drilled Piers(B)	Glacio-marine drift			N/A	40
					Hard clay	40H		Footings	Glacio-lacustrine (clay)	6			
					Dense sandy silt	20H							

Table 1. Foundation and excavation data for high-rise structures in downtown Seattle (continued)

Bldg. no.	Building	No. of stories	Approx. compl. date	Excavation depth (ft)	Excavated soils	Temporary lateral earth pressure (psf)	Permanent lateral earth pressure	Foundation type	Foundation bearing soils	Foundation bearing pressures (ksf)			
										Footings	Mats	Drilled piers: Shaft friction	Drilled piers: End bearing
25	One Union Square	36	1978	30	Hard clay	33H		Drilled Piers(B)	Glacio-lacustrine (silt)			N/A	30
								Footings	Glacio-lacustrine (clay)	6			
26	Seattle Sheraton Hotel	34	1978	40	Outwash sand	22H	22H psf	Drilled	Glacio-marine drift			N/A	30
					Glacial till	15H	15H psf	Piers(B)					
					Hard clay	40H	40H psf						
27	Seafirst 5th Ave Plaza	42	1980	75	Hard clay and silt	32H	40H psf	Drilled Piers(B)	Glacial outwash			N/A	30
								Footings	Glacio-lacustrine (clay)	10			
28	Madison Hotel	27	1981	70	Hard clay and silt	27H	27H psf	Drilled Piers(B)	Glacio-lacustrine (clay)			1	20
29	King County Correction Facility	19	1982	30	Dense sand over stiff clay	28H		Drilled Piers(S&B)	Glacio-marine drift			1	30(B) 40(S)
30	6th & Pike Building	29	1983	29	Granular fill	30H		Drilled Piers(B)	Glacio-marine drift			N/A	40
31	Century Square	27	1985	40	Dense silt and sand	27H	27H psf	Drilled Piers(B&S)	Glacio-lacustrine (silt)			1	40
32	Two Union Square	58	1989	60	Hard clay/ dense silt	30H	30H psf	Drilled Piers(B)	Glacio-marine drift			2	40
								Footings	Glacio-lacustrine (silt)	10			

Table 1. Foundation and excavation data for high-rise structures in downtown Seattle (continued)

Bldg. no.	Building	No. of stories	Approx. compl. date	Excavation depth (ft)	Excavated soils	Temporary lateral earth pressure (psf)	Permanent lateral earth pressure	Foundation type	Foundation bearing soils	Foundation bearing pressures (ksf)			
												Drilled piers	
										Footings	Mats	Shaft friction	End bearing
33	Washington State Convention & Trade Center		1988	30	Fill over dense silt/ sand over glaciomarine drift	30H	35 pcf	Drilled Piers(B&S)	Glacio-lacustrine			1-1.5	20
								Drilled Piers(B&S)	Glacial outwash			2	30
								Drilled Piers(B&S)	Glacio-marine drift			4	40
								Footings	Glacio-lacustrine	10			
34	Pacific First Center	45	1989	70-80	Hard clay	30H	30H psf	Drilled Piers(B&S)	Glacio-lacustrine			1	30
								Drilled Piers(B&S)	Glacio-outwash			2.5	30
								Drilled Piers(B&S)	Glacio-marine drift			2.5	50
								Footings	Glacio-lacustrine	12			
35	Marathon Block 2	35		60	Dense silty sand over hard silt/clay	25H	35 pcf	Drilled Piers(B)	Glacio-marine drift			1	30
								Footings	Glacial outwash	15-20			
36	Crowne Plaza Hotel	33	1978	55				Footings and Mats	Glacio-lacustrine	12	12		
								Drilled Piers(B)	Glacio-lacustrine			N/A	15
37	Seattle Trust Tower	35	1985	30	Medium dense	30H	30H psf	Drilled Piers(S)an	Glacio-marine drift			1	40
					sand over dense/silt sand				Glacio-lacústrine (clay)	10			

at elevations higher than the level of downtown Seattle, and no high-rise building listed in Table 1 is believed to be founded in lodgment till.

Glaciomarine Drift

The composition of glaciomarine drift is varied, and it generally contains more clay than lodgment till. It can be distinguished from lodgment till by a higher concentration of exchangeable sodium cations determined by chemical testing.

According to Shannon & Wilson (1986), this drift is believed to have been deposited in a marine environment from material carried by floating ice. Texture can vary radically by soil type over short distances and consist of clay or silt with scattered coarse-grained particles. In general, it is a mixture of coarse particles in a finer grained matrix and has the appearance of lodgment till. The clay component has generally low to moderate plasticity. Clam shells have been observed in these deposits in the drilled piers at the Washington State Convention and Trade Center (WSCTC) and in borings drilled for the Downtown Seattle Transit Project.

Locally glaciomarine drift contains zones of sand, gravel, and cobbles, as well as boulders, which are generally near the surface of the unit. The granular particles are hard, abrasive, granitic and metamorphic materials. Commonly interbedded within glaciomarine drift are layers of hard, gray, silty clay and clayey silt.

Except for sand and gravel zones, the glaciomarine drift is relatively impervious, and ground water can be perched on top of it. However, sand and gravel zones within the drift can yield significant volumes of water, but flow rates usually diminish in a few hours to several days.

Fill

Fill was placed in parts of downtown Seattle to thicknesses of 30 ft or more during regrading efforts early in the city's history, and many high-rise building excavations extend through and laterally support fill soils. Most fills are composed of glacial soils from higher elevations. Engineers consider fill soils to be unsatisfactory bearing materials for large structures, and all of the buildings included in Table 1 were founded below any manmade fills.

Soils as Foundation Materials

All of the naturally occurring soils discussed above are competent foundation bearing material, although some soils are stiffer than others. The glaciomarine drift, lodgment till, and outwash deposits are granular soils which have been consolidated by the weight of glacial ice. As a result they have high frictional shear strengths (angles of internal friction) and very low compressibilities. Most glaciomarine drift and lodgment till have some cohesion, which further increases their shear strength. Hence, they are considered to be the most competent bearing soil in the downtown Seattle area and have historically been assigned the highest design bearing pressures. The hard clays and silts, which have also been consolidated by the weight of glacial ice, have lower shear strengths, which may even approach residual values due to the presence of fractures and slickensides. They also have a lower modulus of elasticity than granular soils and thus are more compressible.

HIGH-RISE BUILDING FOUNDATIONS

Subsurface soil conditions are not usually considered in selecting the depth of the lowest level of a structure in downtown Seattle because depth is predetermined by other factors. The lowest level of a Seattle high-rise building is in part based on zoning requirements, which consider such factors as the required number of parking spaces as a function of building height and/or floor-space area above street level. For example, the 76-story Columbia Center required more than 900 underground parking spaces. This necessitated seven floors of underground parking and, consequently, an excavation to about 120 ft below Fifth Avenue. The Westin Building is 34 stories high but required only a 15-ft-deep excavation because parking space was provided in an attached above-ground garage.

The geotechnical engineer must evaluate the suitability of various types of foundations bearing on the particular soil deposits at and below a preselected foundation level. Important variables in the selection of foundation type, that is, spread footings, mat, drilled piers, or a combination thereof, are column spacing, load and point of load application, and tolerable total and differential settlements of the structure. Foundation types are illustrated on Figure 2. For example, if highly competent soils such as glacial outwash or glaciomarine drift deposits are present at basement level, then the structure could be supported on spread footings or a mat foundation. Settlements in these combinations of foundation and substrate are usually acceptable since these deposits have low compressibilities. If heavy tower columns are to be located adjacent to an existing structure and the near-surface soils consist of glaciolacustrine clayey deposits, drilled piers would most likely be used to transfer the structure loads to deeper, less compressible soils and reduce the possibility of causing settlement of the adjacent building. A mat foundation may also be unacceptable if heavy tower or column loads are applied near the perimeter of the building because of the great stiffness requirements to even out potential differential settlements between the edges and the interior of the mat structure.

The foundation data presented in Table 1 are divided into four main categories: spread footings, mats, drilled piers, and combinations thereof. Buildings 1 through 5 in Table 1 are supported on spread footings. Buildings

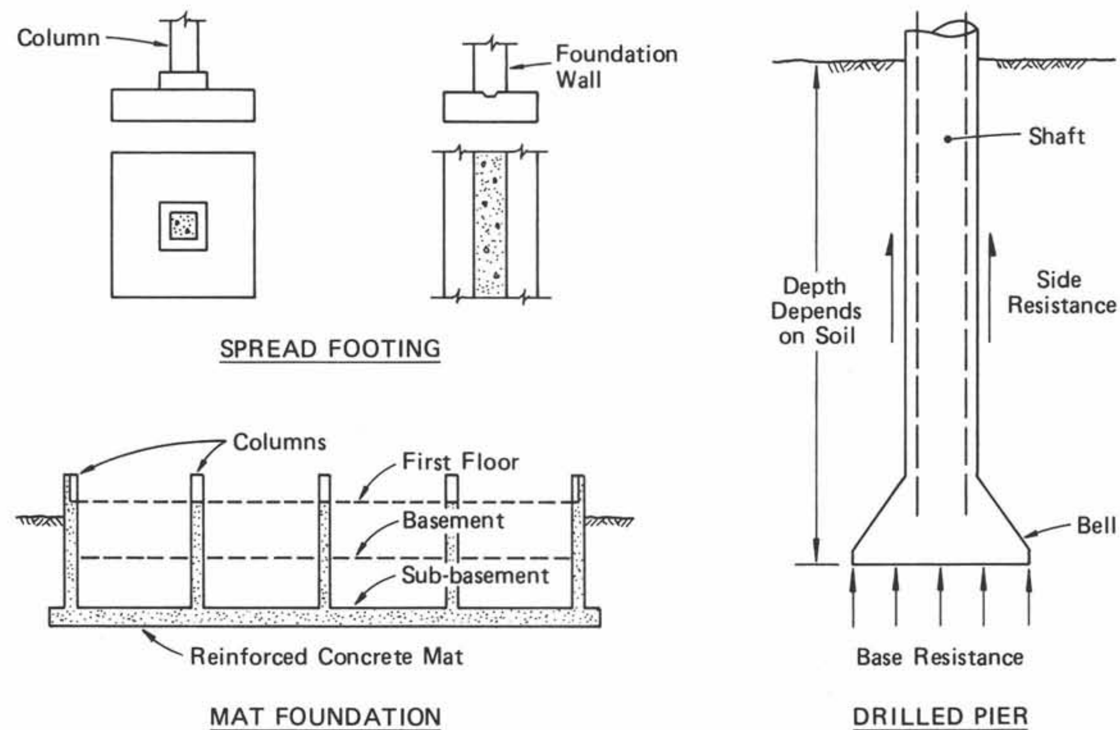

Figure 2. Building foundation types.

6 through 18 are supported on mat foundations, and structures 19 through 35 are supported on drilled piers, while buildings 36 and 37 are supported by a combination of foundation types. Also included in Table 1 are design values for spread footings supporting low-rise portions of buildings, such as plaza and concourse areas, elevators, and entrances.

Spread Footings

A spread footing is a foundation which accepts a single or small group of columns or a wall loading and distributes the load onto the soil at a design or allowable soil bearing pressure. The first five buildings listed in Table 1 have their major bearing columns or tower supported on spread footings and were designed for bearing pressures ranging from 6 to 20 kips per square foot (ksf).

Bearing pressures used for design within glaciolacustrine deposits are generally lower than for those granular soils. Higher bearing pressures can be used to design footings that bear in granular soils (outwash, glaciomarine drift or till) or where deep excavations compensate for much of the building weight, reducing the net increase in soil pressure. Footings for the 76-story Columbia Center are designed with a bearing pressure ranging from 15 to 20 ksf in glacial outwash deposits about 120 ft below Fifth Avenue. Justification for this high bearing pressure (20 ksf) as compared to the Washington and 4th and Blanchard buildings, which are less than half as tall and also founded within glacial outwash (8 to 12 ksf), is the deeper excavation. A significant fraction of spread footing settlement in a deep excavation is the recompression of elastic heave which occurs from soil removal. Recompression occurs during construction as the load of the structure is applied.

Design footing bearing pressures for the remaining buildings in Table 1 vary considerably. Within glaciolacustrine clayey deposits pressures range from 6 to 10 ksf; lower values are used in shallower excavations and in less competent clays. Bearing values used within silty glaciolacustrine deposits, which are less compressible than clayey portions, commonly range from 10 to 14 ksf. Footings at First Interstate Center are embedded in glaciomarine drift, and bearing pressures for support of low-rise tower columns at this site ranged from 10 to 12 ksf. In glacial outwash deposits bearing pressures range from 8 to 20 ksf.

Mat Foundations

Bearing pressures for design of mat foundations are listed for buildings 6 through 18 in Table 1. These pressures depend on depth of excavation, building weight, and area of the mat, among other considerations, and are determined from structural requirements such as tolerable total and differential settlement across the mat. Generally, higher pressures can be tolerated in glacial outwash and glaciomarine drift than in glaciolacustrine deposits.

An important factor in selecting a mat foundation for support of a high-rise building is the uniformity of soil conditions below the mat. If the thickness and type(s) of soil vary significantly beneath the mat, differential settlements may be intolerable. Although the mat settlement profile can be adjusted somewhat by varying the stiffness of the mat, soil uniformity becomes more critical as the plan dimensions of the mat increase, resulting in a deeper stress bulb. An example of the effective use of a mat foundation is Rainier Tower where the underlying glaciolacustrine deposits were relatively thick and uniform.

Drilled Piers

Bearing pressures used for the design of drilled piers to support Buildings 19 through 35 are presented in Table 1. Both belled piers and straight shaft piers in combination with belled piers have been used in Seattle. However, none of the high-rise buildings is supported entirely with straight shaft piers. Drilled piers have been embedded in each of the glacially overridden soil units of downtown Seattle.

The choice of a straight shaft drilled pier or belled pier depends on column location and load, soil conditions, and lateral load requirements. Straight shaft piers have been used to support heavy columns located along property lines in combination with belled piers elsewhere under the building for a number of projects including the King County Correction Facility, Century Square, and Pacific First Center. Seattle Trust Tower is supported with straight shaft piers along the property line in combination with spread footings supporting the interior tower core.

As a general rule, piers are not belled in granular and/or water-bearing soils such as glacial outwash; however, straight shafts can be constructed under these conditions. Belling is normally accomplished in clayey glaciolacustrine deposits. Such piers have been completed in laminated silt; in some places this required steel shoring to support the roof of the bell. Some of the drilled piers at the WSCTC were belled completely within glaciomarine drift because a water-bearing cobble layer was encountered between overlying hard clay and the till-like drift. Bells with diameters of as much as 22 ft have been completed on a routine basis in downtown Seattle. Ten-ft-diameter straight shaft piers were constructed at the WSCTC to resist lateral earthquake loads. This diameter was selected to meet lateral stiffness requirements so that both the vertical column load and lateral seismic loads could be accommodated.

Drilled piers are constructed by open hole methods if the soils remain stable during drilling and belling. At many sites, however, casings are required to stabilize the borehole while drilling through fill and glacial deposits, particularly where water-bearing soils are encountered. Normally casings are driven into an impermeable layer to seal out the water. At the WSCTC as many as three telescoping casings were required in drilling a single pier to maintain the borehole and bell in a dry condition. For this particular project which spans Interstate Highway 5, 94 drilled piers were required; their sizes ranged from 3-ft shafts with 6-ft bells at depths from 40 to 116 ft to 6-, 8-, and 10-ft shafts with bells as large as 22 ft in diameter at depths ranging from 60 to 106 ft.

Shaft friction and end bearing pressures for design of drilled piers are listed in Table 1. Probably the most significant variation over the past 20 yr is that shaft friction has been included into the design of drilled piers. Friction values ranging from 1 to 1.5 ksf in glaciolacustrine deposits, to 2 to 2.5 ksf in glacial outwash, and to 1 to 4 ksf in glaciomarine drift have been used for design. A friction value of 4 ksf was intended for use only with straight shaft piers at the WSCTC.

End bearing design pressures have remained fairly consistent over the years. For piers bearing in glacial outwash, a value of 30 ksf has been used consistently since 1970. Some of the sites where such values were used are the Unigard Financial Center, Seafirst 5th Avenue Place, WSCTC, and Pacific First Center. Within glaciolacustrine deposits, bearing values have ranged from 20 to 30 ksf, the higher values used in silty rather than clayey soils. Bearing pressures for piers in glaciomarine drift have ranged from 30 to 50 ksf, with 40 ksf being used most often.

Combined Foundations

The last two buildings listed in Table 1 utilize a combination of foundation types for support. Crowne Plaza Hotel is supported on spread footings, mats, and drilled piers, and all are bearing within glaciolacustrine deposits. The high-rise portion of Seattle Trust Tower is supported by drilled piers along the property line and spread footings within the interior core area. There, the piers are straight shaft, 4 ft in diameter, and spaced 6 ft on center. The central core consists of four square spread footings ranging in size from 21 to 25 ft. A unique feature of this structure is that the piers are bearing within glaciomarine drift and the core footings are embedded in glaciolacustrine (clay) soils.

PERMANENT LATERAL EARTH PRESSURES

Lateral earth pressures used for the design of permanent basement walls at a number of downtown Seattle high-rise structures are listed in Table 1. Permanent walls have been designed and constructed for either uniform lateral pressures or triangular (fluid) lateral earth pressures. Uniform pressures may be truncated at the top and/or bottom and are the same pressures as those used for temporary shoring. Triangular lateral pressures may be either in the active or at-rest conditions. Typical earth pressure distributions are shown in Figure 3.

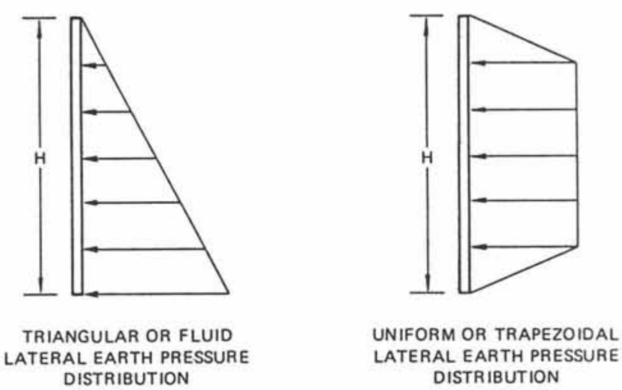

Figure 3. Typical distributions of lateral earth pressures, downtown Seattle foundations.

Excavation and construction of a shoring system alter the magnitude and distribution of earth pressures in the soil surrounding a building site. The distribution of lateral pressures on a temporary shoring system is primarily a function of the type of system and the method of installation. These distributions have been determined through instrumentation at a number of sites, and the pressure distribution on a conventional tied-back temporary shoring system can be approximated by a rectangular or truncated distribution.

What pressures remain after the permanent wall is constructed and the tiebacks have been released is not so well documented. At one extreme, if no deformation occurs when temporary shoring loads are transferred to the permanent structure, permanent wall pressures would be the same as the temporary shoring pressures and could be approximated using a rectangular or trapezoidal shape. If relatively large deflections (on the order of 0.001 times the depth of the excavation) occur when this load transfer is made, the permanent wall loading would be a triangular, active earth pressure. In reality, some deflection occurs at every load transfer, but the magnitude of such movement is probably not sufficient to cause the active condition, and the resulting pressure distribution at the end of the construction period is somewhere in between a triangular or trapezoidal shape.

The magnitude and distribution of pressures may also change with time as soils creep and are vibrated by seismic and other events. Depending on the stiffness of the wall, earth pressures may gradually tend toward a triangular, at-rest condition over many years.

To our knowledge, all of the walls for the buildings listed in Table 1 have performed satisfactorily. It should be mentioned that a triangular or fluid lateral earth pressure calculated from a weight of 35 pcf has approximately the same total force as a uniform pressure of 22H psf truncated at 0.2H and is more or less independent of wall height. The total force with a fluid weight of 50 pcf is similar to a truncated uniform pressure of approximately 33H psf.

EXCAVATIONS

Most excavations for high-rise structures in downtown Seattle are made by installing soldier piles around the perimeter of the excavation and retaining them with a series of temporary tieback anchors. Soldier pile anchor shoring wall details are illustrated in Figure 4. A discussion of design criteria including lateral earth pressures for various soil types, lagging pressures, no-load zone criteria, soldier pile embedment and kickout resistance requirements, and other developments follows.

Soldier Piles

Soldier piles, which are generally placed within the property, consist of either steel wide-flanged beams or double channels connected with steel straps. The piles are placed around the perimeter of the excavation in predrilled holes which are backfilled with concrete. Lean concrete is normally used; however, structural concrete is sometimes placed below the base of the excavation. Soldier piles are generally spaced 3 to 8 ft apart, but spacings as great as 13 ft have been used. On a few projects the bottom of the soldier pile shaft has been belled to provide additional end-bearing capacity.

Generally, belled soldier piles and those utilizing structural concrete are designed as part of the permanent wall system, and they support vertical wall loads. Another method of using soldier piles to support permanent walls is to embed them into drilled piers installed along the property line. These types of permanent wall systems have been utilized within the past 10 yr in downtown Seattle.

As mentioned, the steel members are placed into predrilled holes. Borehole diameters range from 24 to 48 in. and are generally maintained in a dry condition, but concrete is placed by tremie method if water is present. Boreholes are usually drilled deeper than the design tip elevation of the soldier pile. The pile is then lowered into the hole and braced at the surface and concrete placed.

Tieback Anchors

Tieback anchors consist of high-strength steel (dywidag) bars or cables grouted into the soil behind the shoring wall. They are installed in levels as the excavation proceeds. Most anchors are inclined about 20° from horizontal and are connected either to walers spanning the space between soldier piles or directly to piles through cutouts in the flanges or between channels. After the grout has set (normally about 3 days), the anchors are proof tested and prestressed. The excavation then continues to the next lower level of anchors. An obvious advantage to this system is that the excavation remains open and access is not hindered by internal bracing. Also, the deflection of tieback shoring walls is generally about one half that of an internally braced shoring wall having a similar height. In simple terms, the purpose of tieback anchors is to prestress the shoring wall against the excavation surface.

The design of tieback soldier pile walls always incorporates a no-load zone, which is illustrated in the section in Figure 4. This zone has the approximate shape of the Rankine active pressure wedge, except that the base of the wedge begins at a point which is horizontally offset from the base of the wall. The horizontal offset is generally expressed as a ratio of the excavation height (H), and it ranges from 1/2 to 1/4H, or to a minimum of 20 ft for relatively deep excavations. The angle of the no-load zone normally is 60° measured from the

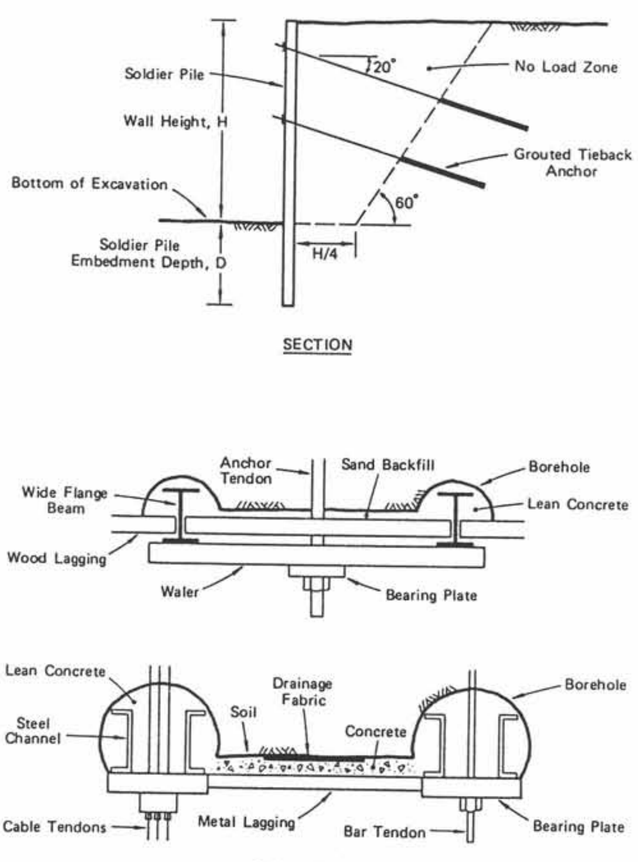

Figure 4. Detailed view of soldier pile-tieback anchor shoring walls as used in downtown Seattle high-rise building foundations.

horizontal, although a 45° angle extending for a horizontal distance equal to H/2 has also been used. Larger horizontal offset ratios are generally used in order to satisfy slope stability requirements, which is evaluated for each shored excavation. Theoretically, most tieback shoring walls are designed to deflect sufficiently to mobilize active lateral earth pressures bound by the wedge of soil between the no-load zone and wall. Shoring wall deflections should be approximately 0.05 to 0.1 percent of the wall height in order to mobilize active pressures. Consequently, anchors passing through the no-load zone remain unbonded by backfilling with sand or the tendons are covered with tape or tubing. Tieback anchor lengths in downtown Seattle have ranged from 25 to 110 ft.

Anchor tendons consist either of bars ranging in diameter from 1 to 1-3/8 in. or bundles of 0.5- to 0.6-in.-diameter seven-wire strand cables. Bar design loads have ranged from 77 to 140 kips, and single cable loads have ranged from 25 to 35 kips; a typical maximum anchor design load is approximately 200 kips. An advantage of strand tendons is that the number of cables can easily be adjusted for a variety of design and test loads within a particular shoring wall.

Grouted anchor capacities are mainly a function of soil type, depth of overburden, and quality of construction. Anchors can be installed within open holes drilled with a continuous flight auger or a shorter 3- to 5-ft-long single flight auger. Where water-bearing soils are encountered, tendons are placed into a continuous flight auger prior to drilling; the hole is then drilled to the desired depth and a sand-cement-fly ash grout is pumped through the auger as it is withdrawn at a slow, continuous rate. Pressure grouted anchors have also been installed. Pressure grouting, including single and multiple stages, has been used in clayey soils with varied success. Pressure grouting involves driving a closed or open end casing with an air-track or rotary drilling rig to the desired depth, installing the tendon (bar or cable strands), retracting the casing in short sections, and pumping a water-cement grout through the casing. Single-stage grouting pressures range from 150 to 300 psi. Post-grouting or multiple-stage grouting involves the same initial grouting stage, but the grouted zone is pressurized again after the initial grout has set through a series of valves located in the anchor zone. Post-grouting was used in clay for one project in downtown Seattle and pressures on the order of 900 psi were necessary to fracture the initial stage of grout. This method of grouting was selected because of its high probability of success, since replacing failed anchors would have been extremely difficult and costly.

Single-stage pressure grouting for horizontal anchors in hard clay has yielded load transfer rates from 1 to about 6 kips per lineal foot (klf) of grouted length. Using the post-grouting technique, ultimate friction values of 8 to 10 klf have been observed. Ultimate friction values in clay for open hole anchors normally range from 2 to 3 ksf. They may be higher if the borehole is carefully reamed or rifled after being drilled and cleaned. It has also been the authors' experience that higher friction values may be achieved if grout is placed into the borehole by tremie method from the end of the anchor rather than having grout flow by gravity from the top of the borehole. Grouting through the auger in both the bond zone and the no-load zone in clay on a recent project yielded ultimate friction values from 3.5 to 4 ksf.

In granular soils similar anchor installation methods are used. Open hole drilling is only used where soils tend not to cave, such as in a dense sand that is above the ground-water table and that has sufficient apparent cohesion. Otherwise, grout is placed through a continuous flight auger, or single-stage pressure grouting is utilized. Post-grouting has not been attempted in granular soil in downtown Seattle. Ultimate friction values for open hole and auger-grouted anchors range from 2 to 6 ksf for dense sand and till or glaciomarine drift soils. These values assume an overburden pressure of at least 1 ksf. In addition, tremie grouting tends to yield higher friction values, probably due to an increase in confining pressure. Friction values or load transfer rates using pressure grouting in dense clean sand and gravel ranged from 11 to 13 klf. Load transfer rates of 8 klf have been obtained in medium-dense, silty sand (recessional outwash).

Another type of tieback anchor recently used in downtown Seattle is the multi-helix screw anchor manufactured by the A.B. Chance Company of Centralia, Missouri. The design of the helix, including helix diameters and spacing, is proprietary and completed by the A.B. Chance Company. Basically, the square anchor shaft is connected to a dywidag bar and inserted into a long square kelly bar, which rotates the screw anchor through the no-load zone and into the bonded area. When a required torque is achieved, rotation is stopped and the kelly bar retracted. After the connection plate is welded to the soldier pile, the anchor is proof tested without the need for grouting.

For the particular project where screw anchors were used the soil consisted of a hard glaciolacustrine silt/clay. Three helix plates with diameters of 8, 10, and 12 in. at approximately 2-ft spacings were required to achieve the design capacity of 60 kips. The required torque in the anchor zone was near the maximum torque rating of the helix shaft. During proof testing, the anchor extension was generally greater than has been typically observed for grouted anchors of similar capacity. This was anticipated and was attributed to end-bearing resistance effects of the helix plates. Creep rates were within acceptable ranges, and lift-off tests conducted after the 35-ft-high wall was completed indicated less than a 5 percent load loss.

In Seattle, all tieback anchors are designed for a factor of safety of 2 with regard to soil failure. In other words, the ultimate friction values or load transfer rates given previously are divided by 2 for anchor design.

Lagging

Lagging is placed between the soldier piles to retain the soil and provide a safe working condition for personnel working in the excavation. Pressure-treated wood lagging is used for the majority of the shoring systems. Wood lagging thicknesses range from 2 to 6 in.; the selection of thickness depends on soldier pile spacing, depth of excavation, and surcharge loads. Lagging thickness increases as pile spacing, surcharges, and depth increase. For the majority of projects with a pile spacing on the order of 4 to 8 ft, depths to 80 ft, and a lateral surcharge pressure of about 80 psf, a lagging thickness of 3 in. has been satisfactory and generally independent of soil type.

On a few projects, such as the Seattle First National Bank Building and Seafirst Fifth Avenue Plaza excavations, lagging was not used. The retained soils consisted of hard silts and clays, and soldier pile spacing ranged from 3 to 6 ft. For the relatively close pile spacing the cohesive soil would tend to arch between the piles without lateral support. However, some local sloughing and slabbing-off of soil did occur, and consequently lagging has been used to satisfy government safety requirements for all soldier pile shoring systems in Seattle.

For double steel channel soldier piles, wood lagging is placed behind the flanges of the channels, as shown in Figure 4. For wide flange beam soldier piles, wood lagging is either placed behind the flanges or connected to the front of the beam with plates bolted to studs welded to the pile. (The latter method has not been used in recent years.) Voids or spaces deeper than approximately 2 in. behind the lagging are filled with pea gravel or clean sand to promote drainage. Voids larger than 2 ft in depth are generally pumped full of lean concrete.

Another lagging system recently used on soldier piles adjacent to existing buildings is illustrated in the lower of the two plan views in Figure 4. The lagging consists of galvanized steel decking which is welded directly to the piles. The space between the decking and soil is filled with concrete. Drainage fabric materials are generally placed against the soil prior to placing concrete.

Soil Nailing

Soil nailing has been recently used to retain two excavations in the greater Seattle area. These excavations were on the order of 30 ft deep. The lateral support system consisted of a series of grouted anchor bars installed in a rectangular grid pattern against a near-vertical excavation surface. The soil face was then covered with a wire mesh and shotcrete as the excavation deepened.

The retained soils for these projects generally consisted of basal lodgement till and/or fine-grained outwash deposits. Soldier piles are not utilized in this system, and anchors may or may not be prestressed.

Design Lateral Earth Pressures For Temporary Shoring

Temporary lateral earth pressures used for the design of shoring systems in downtown Seattle are listed in Table 1. The most interesting feature of the data is how much the lateral pressures for hard glaciolacustrine clay have changed over the years while values for dense (glaciolacustrine) silt and outwash sand have not changed. For example, in the late 1960s and early 1970s, shoring systems for the Seattle First National Bank, Bank of California, IBM Building, Rainier Tower, and Peoples National Bank (all retaining hard clay) were designed for 40H to 48H psf, whereas more recent projects, such as Two Union Square, Pacific First Center, Seattle Hilton, Madison Hotel, and 4th and Blanchard Building, have used values ranging from 27H to 32H psf in hard clay. These lower values have evolved through analysis of data from instrumentation indicating wall displacements and ground settlements lower than anticipated and through a better understanding of the locked-in lateral stress phenomenon of preconsolidated glaciolacustrine clay deposits.

Numerous landslides occurred after excavating in hard clay during construction of Interstate Highway 5 in the early 1960s. A result of analyzing these landslides was recognition that low levels of strain would release high lateral stresses in the hard clay soils. The deformations associated with the excavations reduced shear strengths to relatively low residual strengths, and additional failures occurred. Consequently, shoring walls built during and shortly after this period were designed for relatively large lateral earth pressures. Shoring walls retaining preconsolidated clays, which may be slickensided, will deflect sufficiently to release the locked-in lateral stresses, thus reducing the lateral earth pressure coefficient. However, experience has shown that sufficient lateral restraint can be provided by a soldier pile tieback anchor shoring system so that low residual shear strengths do not develop. Thus, lower lateral earth pressures can be justified. Lower design lateral earth pressures and higher clay shear strengths also result from three-dimensional arching effects around property corners as compared to the freeway landslides.

The data in Table 1 indicate that tieback anchor shoring walls designed to retain dense sand have not changed significantly over the years. Early examples of these walls are the Unigard Financial Center (1970), Peoples National Bank (1972), and 1600 Bell Plaza (1974); they have design lateral earth pressures of 22H psf. More recent examples include the Block 5 Development (1988) and 1111 Third Avenue Building (1979), where design values are 20H and 22H psf, respectively. Walls retaining loose granular soils or fill have been designed for approximately 30H psf; examples are the Marsh & McLennan Building (1980), 6th and Pike Building (1983), and WSCTC (1988).

Shoring walls retaining a variety of soils such as dense silt and sand and hard clay have been designed for values ranging from 25H psf (Gateway Tower) to 30H psf (First Interstate Center and Seattle Trust Tower). Other examples include King County Correction Facility (28H), Columbia Center (27H), Century Square (27H), Henry M. Jackson Building (29H), and Westlake Mall (26H). Generally, a weighted averaging method is used for estimating pressures where combinations of soil types are retained.

Very few shoring walls in downtown Seattle have been designed to retain mainly till-like soils (lodgment till and glaciomarine drift). However, in downtown Bellevue, the First Hill area of Seattle, and in the University of Washington area, tieback shoring walls in till-like soil have been designed for values ranging from 15 to 20H psf.

The slope or pressure distribution curves for braced excavations in dense sand and silt have not changed significantly over the years in downtown Seattle. They are generally truncated at the top and base of the wall at heights of approximately 0.2H ft. For clay, uniform pressure distributions were used in the late 1960s and early 1970s, but more recently the truncated or trapezoidal distributions similar to those for dense sand have been used successfully.

Lagging Pressures

The pressure distribution for the design of lagging has remained relatively consistent over the years and is generally assumed to be uniform between soldier piles. The magnitude of pressure expressed as a percentage of the lateral earth pressure has also been consistent. Generally, 50 percent of the lateral earth pressure was used to design lagging and resulted in thicknesses on the order of 3 in. However, as excavation depths increased, and in some instances pile spacing increased, lagging thicknesses appeared to become unreasonably thick, on the order of 8 in. or more. For these types of shoring walls, the percentage factor has now been reduced to values on the order of 20 to 30 percent of the lateral earth pressure.

Lagging deflection measurements completed for the Columbia Center excavation (Grant et al., 1984) and Two Union Square excavation indicate that a percentage of approximately 5 to 8 of the design lateral earth pressure could be used. These deflection results have been used to design lagging as a function of the clear spacing between soldier-pile-drilled holes defined as "s". A value of 35s in psf has been used successfully on a few recent projects.

Soldier Pile Design Criteria

In addition to being required to resist bending and shear due to lateral earth pressures, soldier piles must also be designed to resist vertical loads from tieback anchors or permanent vertical walls, and they must have adequate embedment below the base of the excavation to resist kickout. Vertical forces are normally assumed to be resisted by skin friction and end-bearing pressures of the soil below the base of the excavation. These values are a function of soil type, ground-water conditions, and the quality and care of construction, that is, amount of disturbed soil left in the bottom of the drilled hole. For hard clay and dense silt soils, allowable friction values range from 1 to 2 ksf, and allowable end-bearing values range from 10 to 20 ksf. In granular soils, allowable friction values range from 2 to 4 ksf and allowable end-bearing values range from 20 to 40 ksf. Data presented by Grant et al. (1984) indicate that vertical forces are also resisted by shear forces between the pile and soil above the base of the excavation. These results have been incorporated into soldier pile design on some recent projects to resist vertical forces from tiebacks and permanent wall loads.

In order to resist pile kickout below the base of the excavation, typical sheet pile wall design criteria are generally used. In other words, the shoring lateral earth pressure and active lateral pressure behind the pile are resisted by the passive soil pressure in front of the pile below the bottom of the excavation. Active pressures are assumed to be a function of the pile width below the bottom of the excavation, and the shoring pressures are assumed to be a function of the pile spacing above the base of the excavation. Passive pressures are normally assumed to be distributed across a space 2 to 3 times the width of the soldier pile drilled hole or the pile spacing, whichever is less. Passive pressures are generally calculated from equivalent fluid weights ranging from 250 to 350 pcf for clay and silt to 450 pcf or more for granular or till-like soils. The embedment is estimated by calculating either moment or horizontal equilibrium below the lowest tieback anchor level. Recently, the active pressure behind the wall has been neglected in estimating pile embedment to resist kickout. The reason for eliminating the active pressure is that if the pile is designed for this additional driving force, it may become so stiff that it would not deflect into the excavation and thereby mobilize the available passive soil resistance.

Past experience has indicated that if pile embedded is at least 8 to 12 ft below the bottom of a horizontal excavation surface and within glacially overridden soils, it would be adequate to resist kickout. Providing kickout resistance by leaving a soil berm against the bottom of the piles, rather than embedding the pile the full 8 to 12 ft below the bottom of the excavation, has not been entirely successful, particularly in clays.

ACKNOWLEDGMENTS

The majority of the data presented in Table 1 was obtained from the files of Shannon & Wilson, Inc. These data originated either through direct involvement with site development and design or through consulting assignments with the Seattle Engineering Department and contractors. Some of the information was provided in personal communications with geotechnical and structural engineers of other local consulting firms and personnel of the Seattle Engineering Department. The assistance of all these people is greatly appreciated.

REFERENCES

Grant, W. P.; Yamane, G.; and Miller, R. P., 1984, Design and Performance of Columbia Center Shoring Wall, Seattle, Washington: *Proceedings, International Conference on Tall Buildings*, Singapore, The Institution of Engineers, Singapore, pp. 651-661.

Shannon & Wilson, Inc., 1986, *Geotechnical Report, Stations and Pine Street Line Structure*: Prepared as subconsultant to Parsons Brinkerhoff Quade & Douglas, Inc., for Municipality of Metropolitan Seattle, Seattle, WA, DSTP-PB Catalog No. B06.26, April, 1986.

Completed excavation for the Columbia/Sea First Center, downtown Seattle, in 1983. This is the deepest shored excavation on the West Coast. Photograph courtesy of Shannon & Wilson, Inc.

Engineering Geology in Washington, Volume II
Washington Division of Geology and Earth Resources Bulletin 78

Engineering Geology of the Downtown Seattle Transit Project

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PROJECT DESCRIPTION

The Downtown Seattle Transit Project (DSTP) is a public transportation system tunnel that extends from the south edge of the central business district near Union Station to the northeast edge of the downtown area near the Interstate Highway 5, or I-5 Freeway, as indicated on Figure 1. The tunnel, consisting of twin 20-ft-diameter tubes (Figure 2), is designed to carry diesel-powered buses which will be converted to electric power in the tunnel. The total length of the tunnel is 6,300 ft of which 5,100 ft is tunneled and 1,200 ft is cut-and-cover. The tunneled portion lies underneath Prefontaine Place and Third Avenue and then turns 90 degrees eastward in an arc to Pine Street, where it changes to cut-and-cover construction (Figure 2). Depths to the top of the tunnel range from 10 to 50 ft. The pillar between the tunnels is 16 ft wide along the portion of the alignment south of the Pioneer Square Station and 20 ft wide between Pioneer Square Station and Westlake Station, except where it narrows at the approaches to the station structures.

The reason for constructing the tunnel for buses beneath downtown Seattle is that surface bus traffic had nearly "gridlocked" the transportation corridors during rush hour, and studies indicated that the problem would only become worse as high-rise office building construction continued at a fast pace. Therefore, a "new street" needed to be built, and a tunnel was considered the best method of achieving that goal.

The tunnel along Third Avenue southward from Pine Street was excavated by tunneling machine below ground to minimize disruption of surface traffic. The tunnel along Pine Street was constructed by excavating an open, braced, or tied-back hole, forming the tunnel tubes and backfilling (cut-and-cover).

Three underground, in-line stations, known as the Westlake, University Street, and Pioneer Square Stations, allow pedestrian access to the tunnels. The University Street and Pioneer Square Stations are each approximately 570 ft long, about 70 ft wide, and 60 to 70 ft deep. The Westlake Station is approximately 580 ft long, 70 ft wide, and 50 to 60 ft deep. The stations were constructed by cut-and-cover methods with internally braced and/or tied-back shoring systems. Shoring systems included vertical reinforced concrete diaphragm walls, which were incorporated into the permanent station walls.

The Convention Center Station (North Staging Area) occupies the two blocks bounded by Pine Street and Olive Way and Ninth and Boren avenues (Figure 1). Finished grade in the Convention Center Station is lower than the surrounding streets, requiring retaining walls up to approximately 50 ft in height.

The Pine Street line structure extends from the Ninth and Pine Station westward beneath Pine Street to Westlake Station. It is a rectangular reinforced concrete box constructed in a braced excavation. Roadbed grade along this structure ranges from approximately 30 to 50 ft below existing street grade. After construction of the structure the excavation was backfilled to re-establish street grades.

The International District Station (South Staging Area) is located between Fourth and Fifth Avenues and Jackson Street and Airport Way South, in the old Union Pacific Railroad yard. Like the Convention Center Station, this station consists primarily of pavement and ramps below the surrounding street levels.

Some major concerns considered in the design of the DSTP were the existing Burlington Northern Railroad tunnel near the south portal, ground-water conditions and dewatering provisions in the Yesler and Jefferson streets area, shoring of excavations adjacent to existing high rise structures, tunneling-induced settlement along the entire alignment and its effects on existing structures

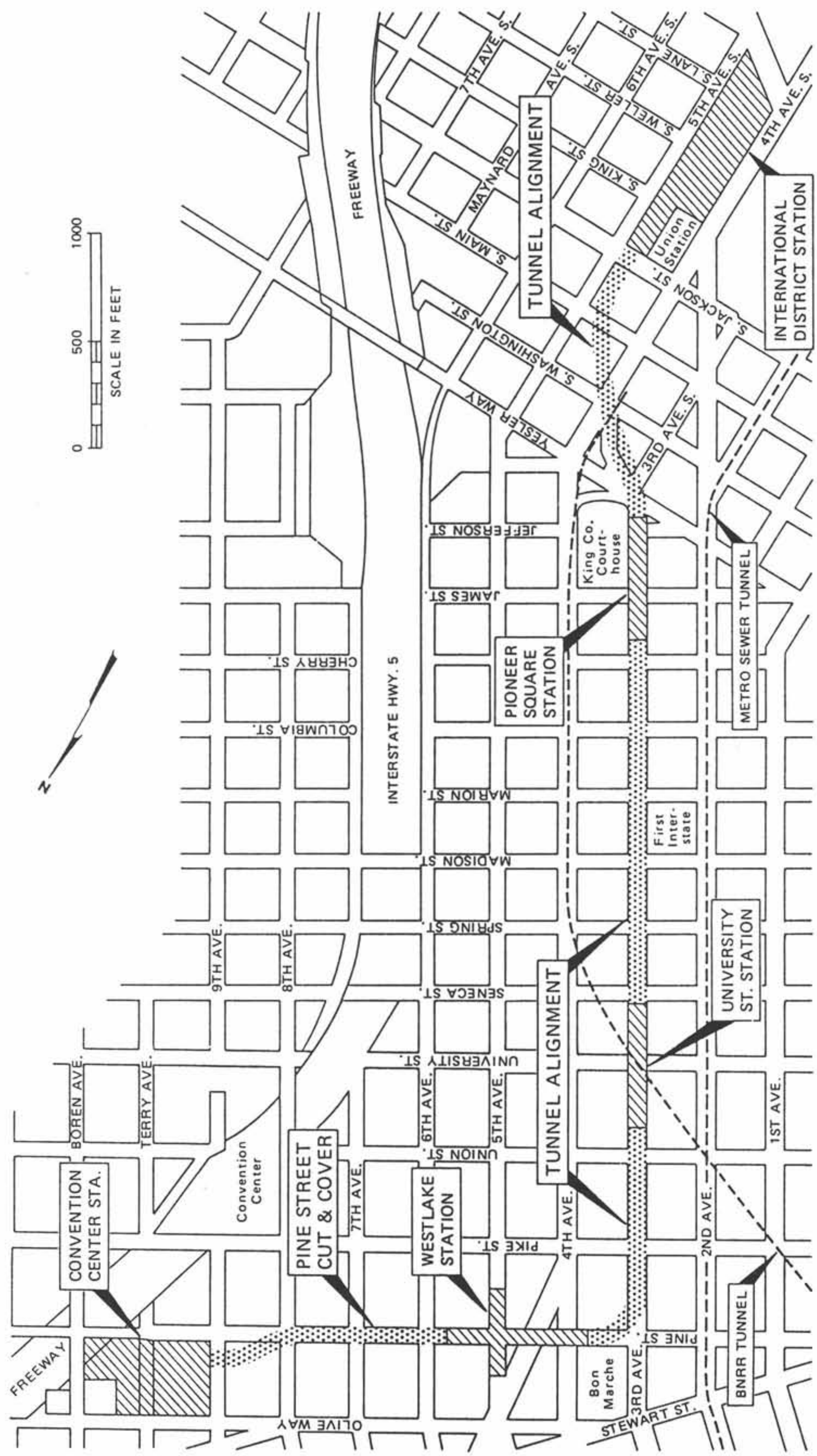

Figure 1. Plan of the Downtown Seattle Transit Project showing major structures in the area.

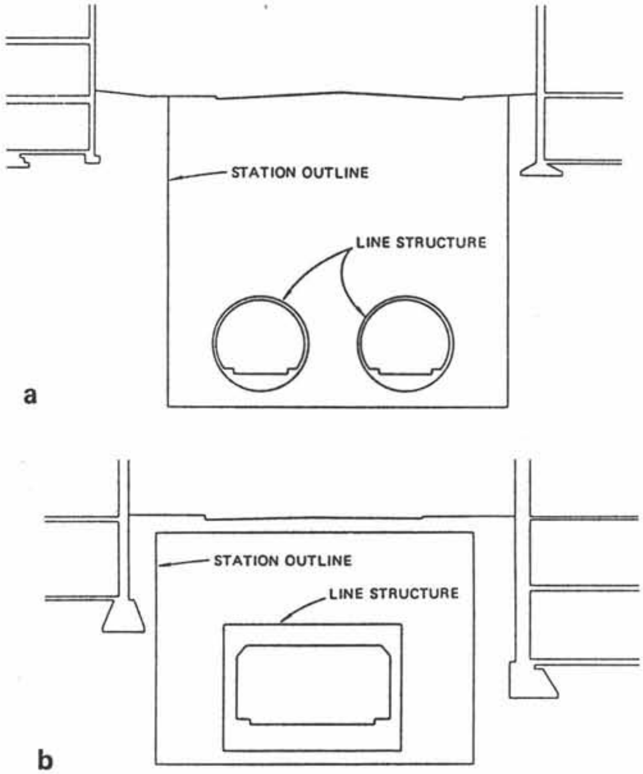

Figure 2. Typical sections of tunneled (a) and cut-and-cover (b) sections. Tunnneled section is on Third Avenue; cut-and-cover section on Pine Street.

and utilities, and construction in old landslide debris at the Convention Center Station.

SEQUENCE OF STUDIES

Geotechnical studies for the DSTP paralleled the overall phased design schedule. During feasibility studies by the Municipality of Metropolitan Seattle (Metro) in 1984, a Fatal Flaw Analysis was made. An approximate north-south corridor consisting of Second, Third, and Fourth avenues had been chosen by others. Second Avenue was eliminated because a major sewer tunnel had been built in 1968 beneath that street. Fourth Avenue was not desirable because the Burlington Northern Railroad (BNRR) tunnel ran for a significant distance beneath that street. As a consequence of the location of these facilities, Third Avenue was the chosen route. The Fatal Flaw Analysis was primarily an office study, but one boring was drilled in the area where the BNRR tunnel crossed beneath Third Avenue. This crossing was considered one of the major engineering considerations of the Third Avenue route.

During the preliminary design phase in 1985, Shannon & Wilson drilled and sampled 25 borings (TB-1 through TB-25) (Figure 3). Borings along the tunnel lines were spaced approximately every two blocks, with additional explorations at the proposed station locations. In 1986, 75 more borings (Figure 3) were drilled and sampled, filling in between the preliminary study explorations and concentrating on station locations. Other testing included *in situ* pressuremeter tests, coring of the existing BNRR concrete tunnel liner, three groundwater pump tests, and air track drilling for location of buried foundations. Observations were also made during the relocation of the Westlake Sewer, which was a small-diameter pipe-jacked tunnel along Fourth Avenue required to be moved for the bus tunnel.

GEOLOGIC CONDITIONS

Geologic History

Seattle is located in the central portion of the Puget Lowland, an elongated topographic and structural depression bordered by the Cascade Range on the east and the Olympic Mountains on the west. The lowland is characterized by low rolling relief with some deeply cut ravines. In general, the ground surface elevation is within 500 ft of sea level.

The Puget Lowland was filled to significant depths by glacial and nonglacial sediments during the Pleistocene Epoch, although bedrock does crop out in scattered locations throughout the area. Within the City of Seattle, bedrock outcrops in a few locations: at Alki Point in West Seattle, along the western slope of the Duwamish River valley near Boeing Field, in the southern portion of Rainier Valley, and at Seward Park in southeastern Seattle. Elsewhere the rock is deeply buried by Pleistocene and Holocene sediments. Based on deep drill holes and seismic profiling by others, the depth to bedrock in downtown Seattle may well be more than 3,000 ft (Yount et al., 1985).

Geologists generally agree that the central Puget Lowland was subjected to at least four major glaciations during the Pleistocene Epoch. The glacial ice originated in the Coastal Mountains and the Vancouver Range of British Columbia. The maximum southward advance of ice was about halfway between Olympia and Centralia.

The Pleistocene stratigraphic record in the central portion of the Puget Lowland is a complex sequence of glacial and interglacial sediments. Local erosion and deposition of sediments further complicate the geologic setting.

An interpretation of the geologic history of the downtown Seattle area and specifically along the project corridor is presented on Figure 3. This was prepared on the basis of the information recovered from the borings for the transit tunnel along Third Avenue and Pine Street and additional boring logs from nearby projects. The interpretation is also related to the general glacial history of the Puget Lowland. The soil strata encountered in the subsurface explorations have been geologically grouped into lithogenetic units. A

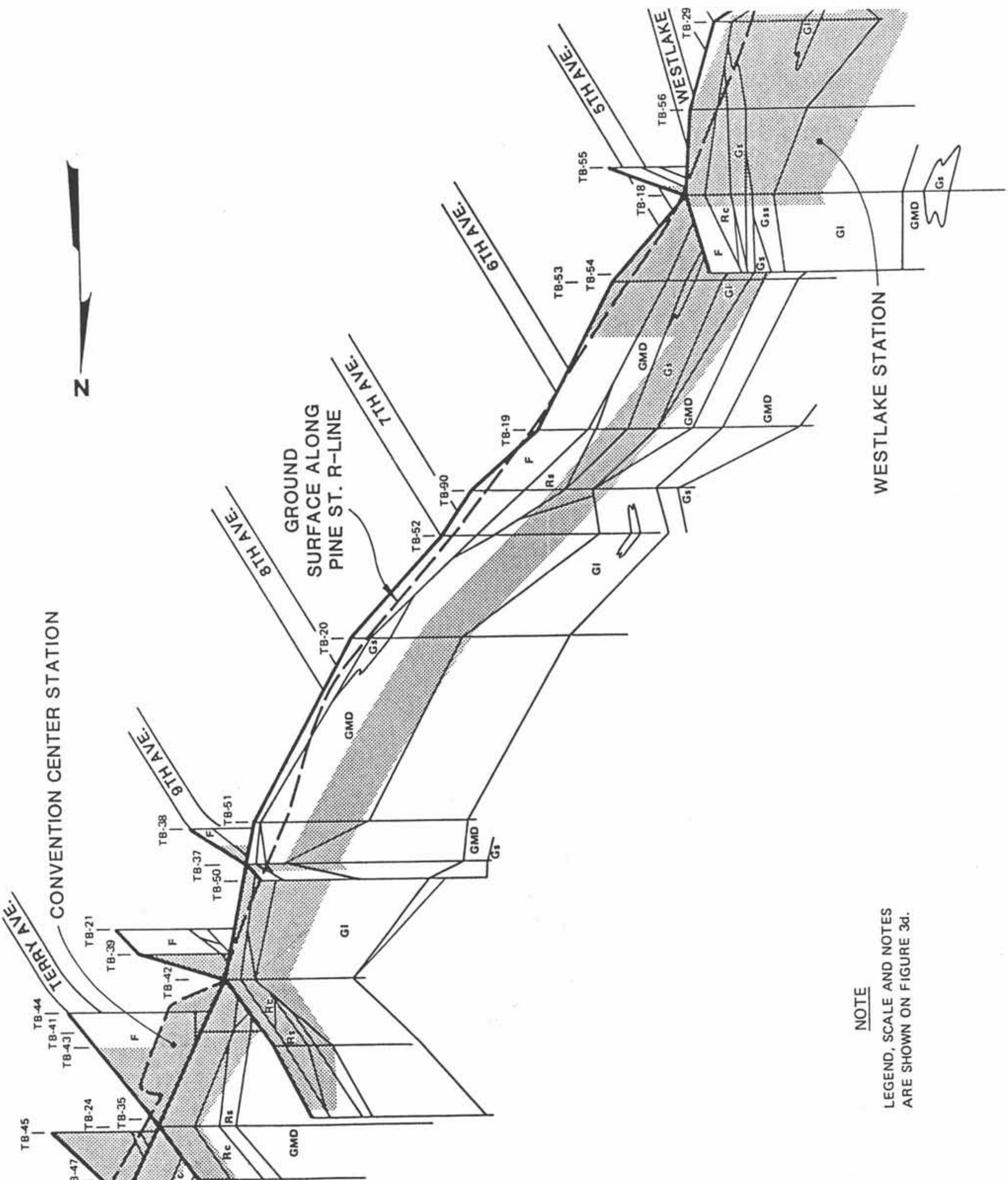

Figure 3a. Generalized geologic conditions in downtown Seattle.

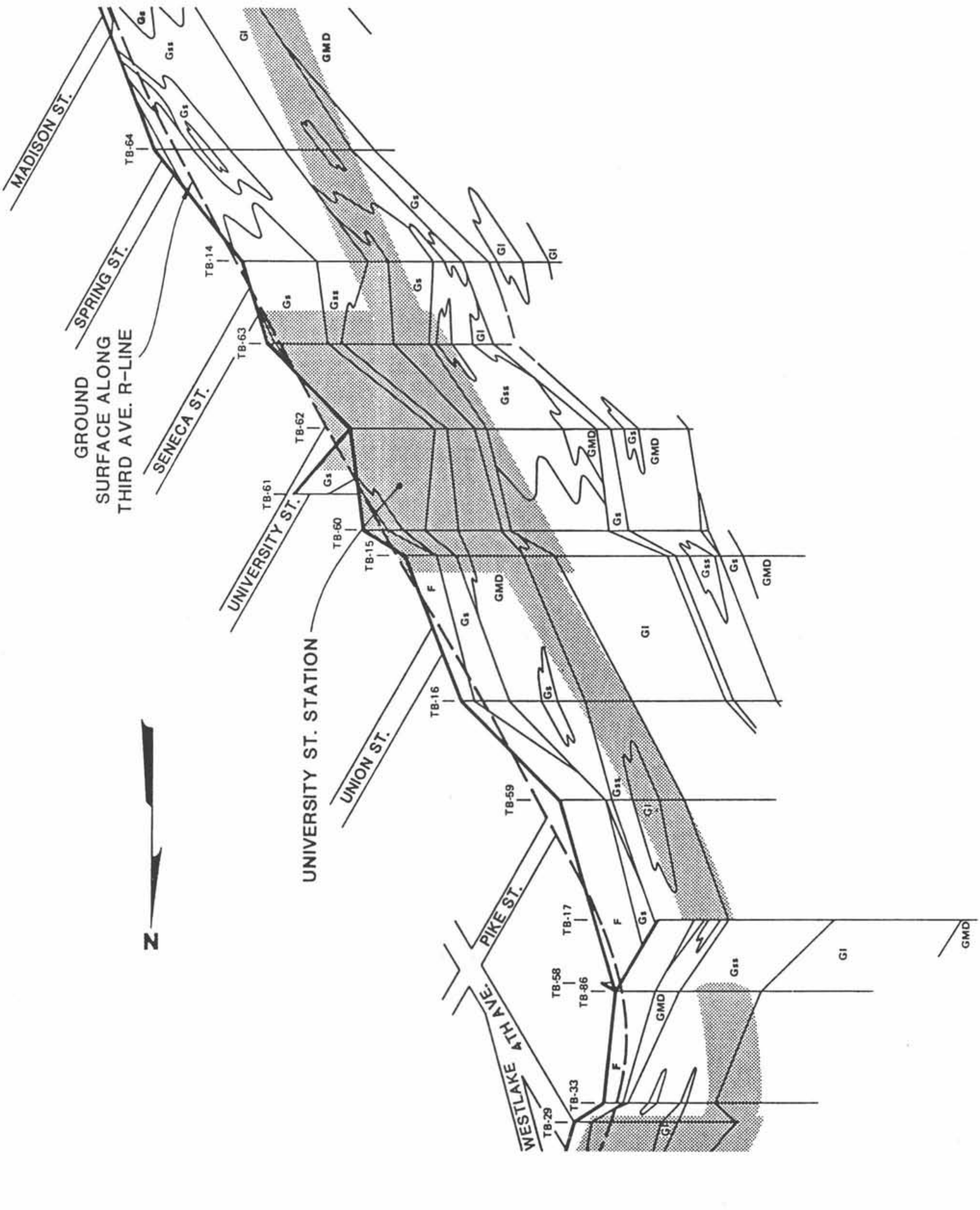

Figure 3b. Generalized geologic conditions in downtown Seattle.

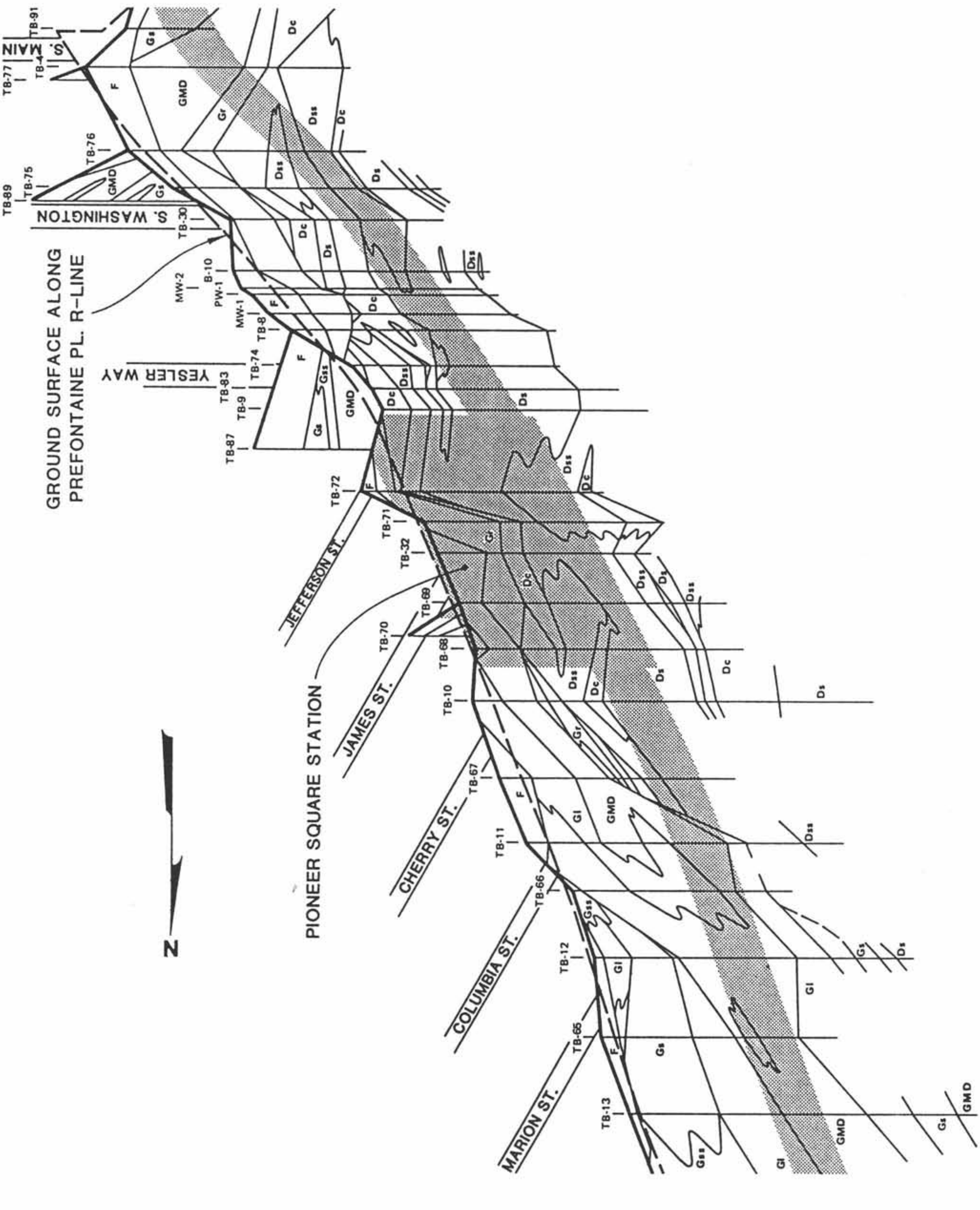

Figure 3c. Generalized geologic conditions in downtown Seattle.

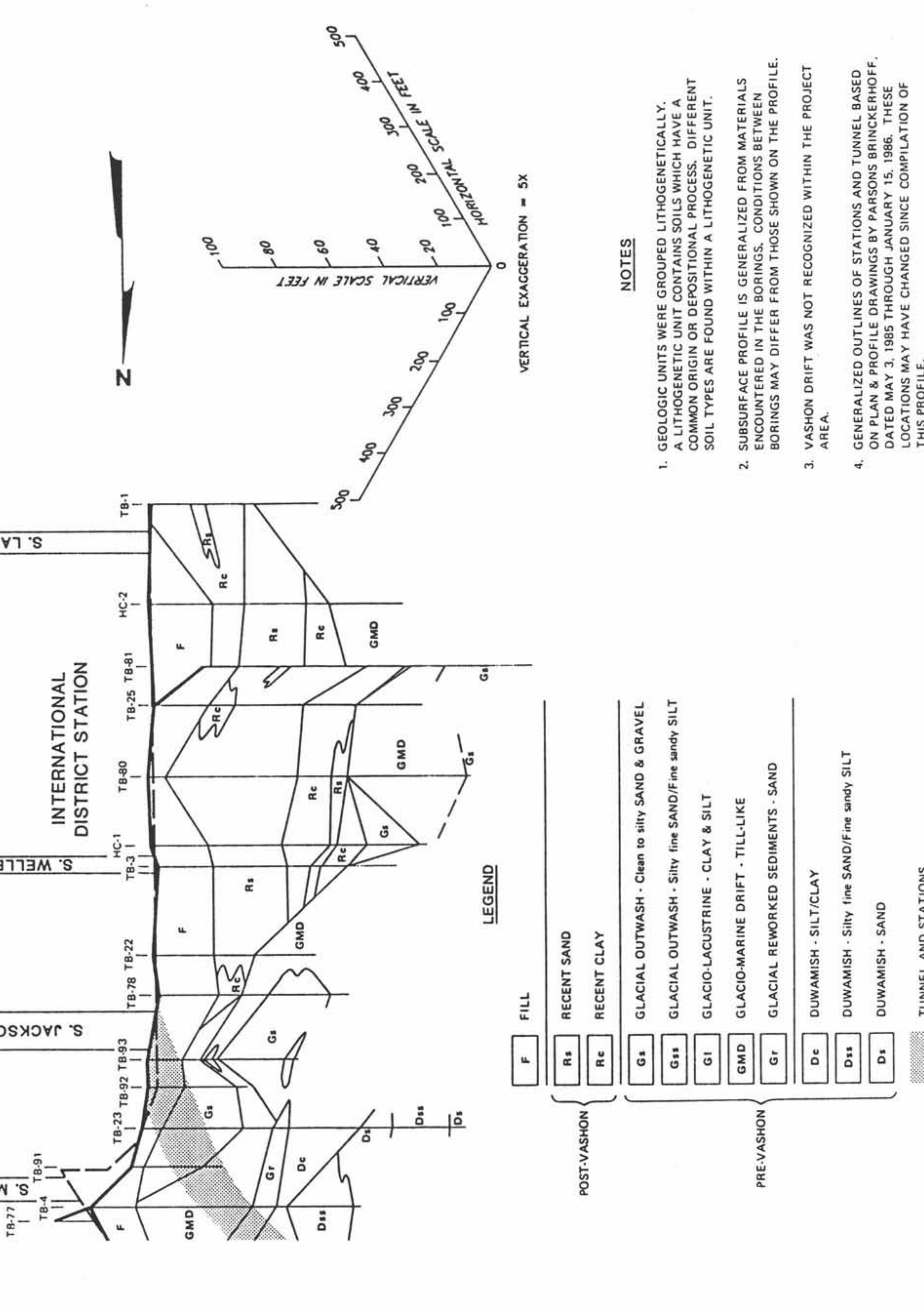

Figure 3d. Generalized geologic conditions in downtown Seattle. (Note: post-Vashon sediments areof Holocenc age.)

lithogenetic unit contains soils which are interpreted to have a common origin or process of deposition. Different soil types may be found within each lithogenetic unit.

Based on the soils encountered during explorations for the DSTP alignment and previous geologic interpretations for downtown Seattle soils, the following is a stratigraphic outline for the late Pleistocene and Holocene geologic history of downtown Seattle:

Stage	Deposits
Post-Vashon (interglacial)	- Artificial fill, Holocene deposits
Vashon (glacial)	- Not identified in the project area
Olympia (interglacial)	- Not identified in the project area
Pre-Vashon (glacial)	- Advance and/or recessional outwash
	- Glaciomarine drift
	- Glaciolacustrine deposit
	- Beach or alluvially reworked sediments
Duwamish formation (interglacial)	- Lacustrine/estuarine deposit
	- Fluvial deposit

Our studies indicate that Vashon and Olympia soils may not be present in the project area, with the possible exception of some outwash deposits along Pine Street.

The oldest geologic unit encountered was the Duwamish Formation. These interglacial sediments were deposited in a delta and offshore embayment as the ancestral Duwamish River mouth shifted. The estuarine/lacustrine clays and silts of this unit were deposited in mudflats on the delta, as overbank sediments along the river course, or offshore of the river mouth. Organic materials were incorporated as both finely divided particles and peat layers in swamp and mudflat environments. Sand and silt were deposited in the main channels of the river contemporaneously. Volcanic ash accumulated in topographically low areas during and after eruptions of Cascade Range volcanoes. Pumice, ash, and red andesitic particles were transported by the river and mixed with the dark gray Duwamish sands and also deposited as distinct layers. These red particles effectively mark the interglacial sands. Interfingering or overlapping of these alluvial and estuarine sediments is common.

At the end of the interglacial period, existing interglacial and glacial soils were reworked by streams and by rising waters of a glacial lake. This latter process tended to remove any existing vegetative cover and create a beach-like environment in which proglacial and previously deposited interglacial material became mixed and washed. Glacial ice subsequently moved further southward into Puget Sound from British Columbia and blocked the Strait of Juan de Fuca, creating a large lake throughout the Puget Lowland. The resultant thick sequence of glacial sediments is complexly interbedded and interfingered due to repeated fluctuations of the ice front. Glaciolacustrine soils consist of fine particles laid down in a quiet lake as the glacial ice stood north of Seattle. Glaciomarine drift strata record either subaqueous deposition during the advance of the ice terminus or ice-rafted sediments. As the ice front moved closer to Seattle, coarser sediments (sands and gravels) were deposited as outwash over the clays and glaciomarine drift.

An interglacial period (believed to be the Olympia) followed; it is recorded in peat deposits and mammal fossil remains on the west side of First Hill (uphill to the east of the project).

Following this interglacial interval, a large lake formed as the ice (Vashon Stade) once more blocked the Strait of Juan de Fuca. Clays and silts of this event have not been identified in the project area but are exposed on the slopes of Queen Anne and Magnolia hills. In some areas of the project site, gullies or ravines were eroded into the surficial soils and the channels subsequently filled with pervious outwash materials. As the ice advanced south, coarse granular materials were deposited throughout the Seattle area by outwash streams. The project area was overridden and the soils were densely compacted by the weight of the ice. At higher elevations (but not downtown) lodgment till was deposited and remains today.

In the 12,000 to 13,000 yr since the last glacial episode, sediment has accumulated in topographically low areas, such as the Elliott Bay embayment and the Pine Street swales. In Elliott Bay, sands deposited by the Duwamish River and by creeks entering the bay from surrounding hills are interbedded with clays and silts during overbank floods of the Duwamish River. Clays were deposited in Elliott Bay and peat accumulated in swampy areas in the past several thousand years.

Erosion by surface water and modification of steep slopes by sliding have also altered the landscape. In the past 100 yr, parts of downtown Seattle have been significantly regraded. Some of the upper layers have been removed and used to fill topographic lows, or were sluiced into Puget Sound. Imported fill or backfill has also been placed.

Engineering Characteristics of Geologic Units

Duwamish Formation

The Duwamish formation (Mackin et al., 1950) is the oldest soil unit encountered in this project. It was recognized in excavations for the Pioneer Square Station and the tunnel in this area, and it is likely present at greater depths beneath much, if not all, of the project corridor.

This formation consists of two members: the fluvial member and the lacustrine/estuarine member. The fluvial member is a very dense, gray to dark gray (black), silty to clean, locally gravelly, fine to coarse sand. It is commonly characterized by fragments of pumice and sand-size red andesite grains and lenses or layers of ash. The pumice fragments range from sand to gravel size and are white. Thin layers or lenses of silty sand are scattered throughout the deposit. This member is generally very permeable, and excess piezometric head was measured within it near the proposed south tunnel portal. Due to the thickness and widespread distribution of this member, significant ground-water recharge was expected. The material is easily excavated, but slopes do not stand where water flow occurs. This type of soil was expected to run in excavations if not dewatered or pressurized with air during tunneling.

The lacustrine/estuarine member of the Duwamish formation is composed of hard or very dense silt, clay, and silty fine sand/fine sandy silt with gradational contacts among soil types. In general, the fine soils are of low plasticity. Layers of volcanic ash, pumice, and compressed hard peat and wood are common and some borings encountered interbeds of clean, fine to coarse sand within this member. In general, this member is saturated but has relatively low permeability. These soils are difficult to dewater because of their low permeability. If they are not dewatered, the sands and silts commonly become unstable when disturbed during construction. Some zones of perched water in silty sand were encountered in the upper tunnel heading and caused flowing conditions.

The fluvial and lacustrine/estuarine members were interbedded, especially near the contacts.

Beach or Alluvially Reworked Sediments

Reworked sediments consisting of mixed interglacial and glacial soils are found at some locations overlying the Duwamish formation and underlying glaciomarine drift. The unit is a very dense, gray, clean to slightly silty, fine to coarse sand. The grains are rounded to subrounded and primarily quartz, indicating glacial origin, although scattered red andesite particles are present. The unit is generally water bearing, but its erratic lenticular nature resulted in slow or limited ground-water recharge. Deposits of this unit were at project elevations in the Pioneer Square Station excavation and near the south tunnel portal.

Glaciomarine Drift

Glaciomarine drift is one of the most widespread geologic units along the Pine Street/Third Avenue corridor. It is present along virtually the entire route and was encountered in excavations for the Convention Center Station, Pine Street Line Structure, Westlake Station, University Street Station, and Pioneer Square Station. Deep foundations bear in this unit at the International District Station.

The composition of this unit is varied, owing to its modes of deposition. It consists of clay, sand, silt, and gravel, any one of which can locally be the major constituent. In general it is a mixture of coarser grained particles in a finer grained matrix. This soil varies radically in grain-size distribution over short distances. The clay component generally has low to moderate plasticity, but may be locally very sticky. Rare clam shells are present in these deposits, but generally the unit can be distinguished from lodgment till only by its higher concentration of exchangeable sodium cations as determined by chemical testing. Its engineering characteristics are typically identical to those of lodgment till, and it has historically been described in engineering reports as till in the downtown Seattle area.

Local zones of sand, gravel, cobbles, and boulders are common. These coarse sediments are particularly likely at the top of a glaciomarine stratum, based on observations of drilled shafts and excavations into this soil unit for other projects in the downtown Seattle area. Cobbles 3 to 12 in. in diameter are encountered commonly in this material, and boulders as much as 6 to 10 ft across are not uncommon. The granular particles are hard granitic or metamorphic materials and are abrasive.

Commonly interbedded with the glaciomarine unit are significant thicknesses of hard, gray, silty clay and clayey silt. Much of this clay/silt is highly fractured and slickensided and has a blocky texture. Locally, it is thoroughly sheared and distorted and has a "diced" look. It is similar in engineering properties to the glaciolacustrine soils discussed below.

Except for sand and gravel zones, the glaciomarine drift is relatively impervious, and in many places ground water is perched on top of it. Water-bearing sand and gravel zones within the glaciomarine drift were sporadically encountered in borings and the tunnel heading, and in places resulted in loss of ground; however, flow rates usually diminished in a few hours. Excavation was usually difficult due to the extremely compact nature of this unit; however, the presence of joints filled with fine sand and silt assisted in breaking up the material.

Glaciolacustrine Deposit

The glaciolacustrine deposit underlies much of the project area. It was encountered in the excavations for the Convention Center Station, Pine Street Line Structure, Westlake Station, and University Street Station.

The glaciolacustrine unit is, in general, a hard, gray, silty clay, although it is locally a clayey silt. It ranges from high to low plasticity; however, much of the unit is highly plastic. While no large clasts were encountered during exploratory drilling, cobbles and boulders were observed in the tunnel bore. Where boulders were too large to be handled with the mucking system, they were hydraulically split into smaller pieces at the face.

The top of the clay unit was eroded near Westlake Station, and the swale is filled with subsequent glacial advance outwash. This condition results in a "bathtub", where ground water in the pervious sand is perched on top of the impervious clay.

The clays and silts range from massive to laminated and are commonly blocky or fractured. Locally they are distorted or sheared, thereby having a lower mass strength than the surrounding soil. Slickensides, indicators of past earth movement, are also locally present. Where open excavation or tunneling is accomplished in this soil unit, pre-existing fractures and shear zones control stability.

In the Pine Street area between Ninth and Seventh avenues is a more or less continuous zone about 1 to 7 ft thick between elevations 70 and 95 ft in which the clay is fractured, sheared, and slickensided; it has lower strength than the clay above and below it. The area in which this condition is found coincides with an ancient landslide noted in other geotechnical reports.

Similar disturbed and stratigraphically jumbled clays and glaciomarine drift were encountered in the Convention Center Station excavation.

Advance and Recessional Outwash

These granular outwash soil units were encountered in the Convention Center Station, along Pine Street, and at the Westlake and University Street stations. Outwash deposits are present at depth at some locations at the International District Station but are overlain by Holocene deposits and fill.

Outwash units are composed of dense to very dense granular soils which range from silty fine sand to clean gravel. These soils are generally stratified. Grain size characteristics change over short distances both vertically and horizontally. The soils are generally water bearing, but the amount of ground-water recharge varies considerably. Ground water in these deposits is perched on underlying glaciomarine drift or lacustrine deposits. Outwash soils will run or flow if ground water is not controlled.

Holocene Deposits

Holocene deposits, which encompass a wide range of soil types, are present near the surface at the Convention Center Station, along the Pine Street Line Structure, Westlake Station, and the International District Station. Holocene deposits are similar to the interglacial deposits previously described except that they have not been glacially overridden and are relatively loose or soft.

In three areas along the Pine Street corridor, the Holocene deposits consist of loose to medium dense granular soils and medium stiff to very stiff clays and silts with organic materials. They were deposited in beds of creeks (swales) which drained north toward Lake Union. Also present in the Convention Center Station area are landslide deposits of wide ranging consistency and disturbed, discontinuous stratigraphy.

At the International District Station, normally consolidated sediments have accumulated in the former Duwamish bay to depths of as much as 100 ft. These sediments are primarily very soft to stiff clays and silts and very loose to medium dense (locally dense), fine to medium sands. In general, these soils are moderately compressible and have low shear strengths.

Fill

Artificial fill was encountered along much of the alignment. Soil types have highly varied density and permeability. Fill as thick as 20 ft supports the pavement along much of Pine Street. Locally some effort at compaction may have been made during placement of this material. Much of the fill near the International District Station is believed to have come from the Jackson Street regrade in the early 1900s. It may have been sluiced to the site or hauled and dumped into shallow water. Previous structures in this area were pile supported, and many of these piles have been covered by the fill.

ENGINEERING IMPLICATIONS OF GEOLOGIC CONDITIONS

The transit project alignment can be divided into three primary segments based on the geologic conditions. The design considerations in each were affected to a considerable degree by the prevalent geologic conditions. The tunnel design had to address a wide variety of soils along the alignment, resulting in a versatile design and construction methodology. The following section discusses the primary design constraints imposed by the geologic conditions.

South Section

The south end of the project, from the south portal to about Cherry Street, is characterized mainly by the basal part of the interglacial Duwamish formation. It comprises relatively clean, water-bearing sands interbedded with silty fine sand, silt and clay. The sands are 30 to 40 ft thick and continuous over long distances. It is likely that these soils are part of a regional ground-water aquifer extending from the hills on the east to Puget Sound. Ground-water levels are fairly uniform throughout the south section, ranging from elevation 10 to 20 ft. The alignment of the tunnel is such that it required excavation to about 40 ft below the water table.

Recent soils are near the ground surface at the south portal. At the turn of the century, the Elliott Bay beach line extended across the south portal area (Lawson, 1875). Soft bay deposits overlie the glacially overridden soils in this area. The area was subsequently filled, probably with material from the Jackson Street regrade to the east. Ground-water levels are very near the ground surface.

Industrial development in the area left hundreds of buried wood piles from piers and wharves, as well as waste from a coal gasification plant which was located in the immediate vicinity of the south portal.

Dewatering of the soils prior to construction was a primary consideration in the design of this section. An active dewatering system consisting of large capacity wells was installed along the alignment prior to the construction of the tunnels and the Pioneer Square Station. Actual output from the wells was considerably less than the design capacity and, in some areas, the water table was not lowered below the bottom of the excavation. However, the wells were generally successful in controlling the flow of water into the tunnel and station excavations.

Dewatering at the south portal was complicated by hydrocarbon contamination of the soil and the potential for dewatering-induced settlement of the normally consolidated soils with consequent damage to buildings, retaining walls, and bridges in the area. A closely spaced wellpoint dewatering system with recharge capabilities was used in the south portal area to minimize the transport of contaminants and to minimize drawdowns near particularly sensitive structures. The effects of the wellpoint system proved to be sufficiently localized that only limited use of the recharge wells was required.

Providing a permanently dry final structure was a major design requirement. Wherever possible, gravity drains were installed to prevent the buildup of hydrostatic pressures. However, in the south end, much of the tunnel and part of the station excavation was below the regional ground-water table. Thus gravity drainage was not expected to be effective. Therefore, in the south section, the tunnel and station were designed to be waterproof and to withstand hydrostatic pressures after the completion of construction.

Even with dewatering in advance of construction, ravelling of the soils into the tunnel was a concern. Thus, a tunneling shield with face breasting was required to provide additional control of the face and perimeter of the excavation. The contractor designed a lining system that could be erected in the tail of the shield and expanded quickly against the soil as the shield progressed. This also satisfied design requirements to minimize the potential for ground surface settlement and for damage to existing structures along the entire alignment. This system proved to be effective along the majority of the south section of the alignment. In the few instances where significant ravelling of the soils did occur, ground-surface settlement was minimized by compaction grouting at depth, immediately above the tail of the shield.

Building settlement was also a consideration in the design of the shoring walls for the station excavations. Shannon & Wilson, Inc., recommended lateral earth pressures which were higher than those conventionally used in the Seattle area. The reason for this was that conventional shoring systems support the sidewalk and streets, which can tolerate some settlements. In the station excavations, the shoring walls were supporting existing buildings with considerably less tolerance for settlement. Thus the higher earth pressures led to stiffer shoring walls, which allowed less ground movement to occur behind the walls.

A significant design challenge in the south section was the crossing under the BNRR tunnel about 500 ft north of the south portal. In order to minimize operating grades, the transit tunnel was kept at the highest elevation possible, leaving just 5 ft clear between the transit tunnel and the railroad tunnel.

Special construction techniques were required to prevent damage to the existing railroad structure. Cellular concrete underpinning structures proposed and designed by the contractor were constructed under the railroad tunnel prior to the excavation of the transit tunnels (Figure 4). These were constructed by jet grouting a series of contiguous columns through the invert slab of the railroad tunnel. Jet grouting was performed by pumping grout at high pressures through nozzles in the side of a drill rod. By simultaneously rotating and withdrawing the drill rod, a circular column was "carved" out of the soils by the high pressure jets of grout. The grout mixed with the soil cuttings in the hole, forming a soil-grout mixture which set into a structural column. These columns supported the railroad tunnel structure and provided additional control of soil and ground-water movement as the transit tunnels were excavated through the "slots" formed by the columnar walls.

Central Section

The central section of the project, from Cherry Street to the Westlake Station, is characterized by a variety of glacial soils, predominantly glaciomarine drift and glaciolacustrine clays and silts. Outwash sand overlies the lacustrine deposits, particularly towards the north end of the section.

Dewatering requirements for the central section were considerably different from those for the south section. Recommended dewatering systems for the University Street and Westlake stations included passive sand drain or chimney drain systems outside the shoring walls and sumps in the bottom of the excavations. Small capacity dewatering wells were used for the tunnel section. Because ground water is perched in this section, small amounts of water were expected to be encountered at the contact between the outwash sand and the lacustrine materials. However, incomplete dewatering led to considerable loss of ground; water-bearing sands ran into the tunnel excavation, particularly near Spring and Madison streets and continuing northward on Third

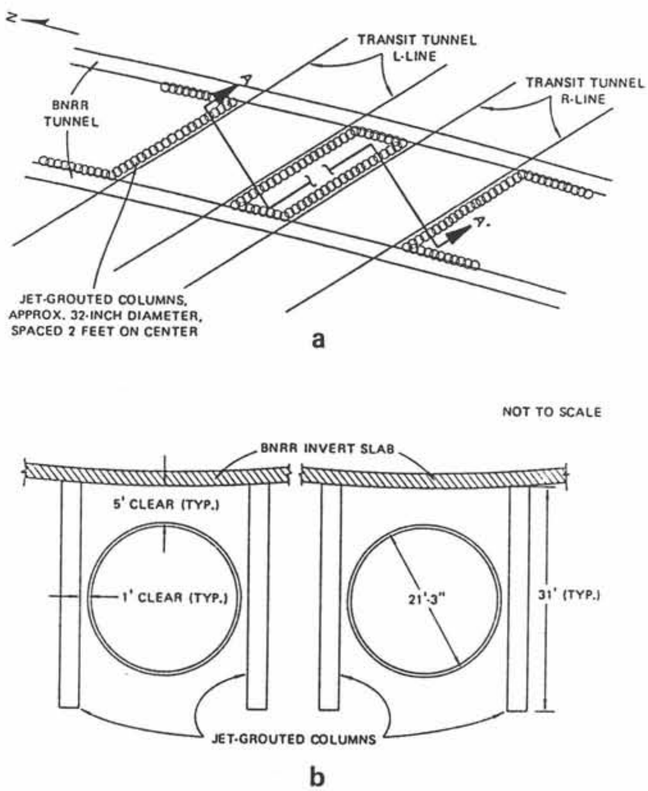

Figure 4. Construction details for Burlington Northern Railroad tunnel undercrossing. a, plan view; b, cross section at A-A',.

Avenue. Extensive chemical grouting was accomplished to reduce ground loss and surface settlement.

Along Third Avenue from Pike to Pine streets, water-bearing silty fine sand and fine sandy silt were encountered, causing flowing conditions in the tunnels. Dewatering was accomplished in these fine sediments with closely spaced eductor wells inside and outside both tunnels.

Because of the boulders in the glaciomarine drift and glaciolacustrine soils, a tunneling machine with an open face was recommended to provide access for the removal of the boulders too large to pass through the normal muck removal system.

This part of the tunnel route is also the most heavily developed of the three, with a number of high-rise office towers lining this segment. Tiebacks were generally used to support the excavations for these buildings; therefore, hundreds of destressed tieback anchors were present within the tunnel horizon along this section. Removal of these anchors required relatively open access to the face of the excavation. However, soil conditions on other parts of the alignment required tight control of the face of the excavation. These contradictory requirements were met through the use of a tunneling machine with retractable breasting doors to provide both full face support and clear access to the face.

North Section

The north section of the project runs along Pine Street from the Westlake Station to the Convention Center Station. It is characterized by glacial soils similar to those of the central section overlain at the surface by substantial thicknesses of fill. Recent channel deposits of sand and organic soils are present in several locations between the glacial soils and the fill. The glacial soils at the Convention Center Station have been disturbed by landsliding to considerable depths.

The tunnel alignment is closer to the ground surface along this section of the project than along Third Avenue. Because of the limited cover and the soil conditions, which were generally less favorable (loose fill) than along other parts of the alignment, tunneling under Pine Street would have resulted in unacceptable ground settlement and potential damage to existing structures. Thus, the design engineers decided to construct this portion of the project by cut-and-cover methods. The temporary shoring along the Pine Street cut was supported in part with internal bracing and in part with tiebacks, depending on the presence of building basements and foundations below the street grade.

Water conditions in the north section were similar to those in the central section. Passive dewatering systems consisting of drains outside of the structure walls and sumps within the excavations were able to satisfy most of the dewatering requirements. However, loss of ground caused by inadequate backpacking of the lagging and seepage through the shoring wall resulted in significant ground surface settlement in small areas.

The Convention Center Station at the north end of the project required retaining walls to heights of 50 ft along Pine Street. The design of these walls was complicated by the presence of fill to a depth of almost 40 ft. The use of conventional tiebacks would have required anchors extending beyond the Pine Street right-of-way. Thus cantilevered walls consisting of 12-ft-diameter cylinder piles at 18-ft centers were embedded to about 70 ft below the bottom of the cut.

Excavation through the fill proved to be difficult in some locations due to the presence of large timbers near the base of the unit. These appeared to have been part of an old structure, possibly a skid road leading towards Lake Union.

CONCLUSION

The Downtown Seattle Transit Project represents one of the largest construction projects ever undertaken in the downtown Seattle area. Because of the lateral extent of the project, it represented a unique opportunity to study the geologic conditions throughout the central business district. Geologic conditions were a dominant factor in shaping the design and construction of the project.

Significant changes in geologic conditions along the alignment required that different sections be designed as separate projects, but the use of common construction methods and technologies resulted in reduced design and construction costs.

ACKNOWLEDGMENTS

We appreciate the help from Michael Kucker, Shannon & Wilson project engineer during construction, for providing us with verification of encountered conditions during the tunneling.

REFERENCES

Lawson, J. S., 1875, *Duwamish Bay (part of) Washington Territory*: U.S. Coast Survey, 1 sheet, scale 1:10,000.

Mackin, J. H.; Mullineaux, D. R.; and Stark, W. J., 1950, Glacial Geology of Seattle: *University of Washington Trend in Engineering*, Vol. 2, No. 4, pp. 19-21.

Yount, J. C.; Dembroff, G. R.; and Barats, G. M., 1985, *Map Showing Depth to Bedrock in the Seattle 30' by 60' Quadrangle, Washington*: U.S. Geological Survey Miscellaneous Field Studies Map MF-1692, 1 sheet, scale 1:100,000.

Members of the Association of Engineering Geologists at the south portal of the Seattle Metro bus tunnel while on a field trip to view the construction, September 19, 1987. This group includes two past presidents of the Association, A. W. Hatheway (standing, third from the right) and R. W. Galster (standing, second from the left).

Exposure of Vashon Till in the Fremont District of Seattle. Photograph by W. T. Laprade.

Engineering Geology in Washington, Volume II
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Landslide Stabilization in an Urban Setting, Fauntleroy District, Seattle, Washington

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GeoEngineers, Inc.

INTRODUCTION

Landslides are relatively common on the hillslopes of the Puget Sound region. Most of the near-surface materials in this region consist of Quaternary glacial and nonglacial sediments of varied composition. Some of these sedimentary units involve a particularly high risk of landsliding where they are exposed on slopes. Urbanization has occurred at many locations where geology, slope, and ground-water conditions combine to create a high risk of landslide activity. Many of these high-risk regions are highly prized for residential development because they are "view properties".

This case history examines a landslide that affected both public and private properties in the Fauntleroy District of metropolitan Seattle. This landslide is hereafter termed the Fauntleroy landslide. Recurrent movement of the Fauntleroy landslide between 1974 and 1983 resulted in damage to streets, sidewalks, subsurface utilities, and residential structures.

Three phases of engineering geologic studies of the landslide were conducted by GeoEngineers, Inc. on behalf of the Seattle Engineering Department. The first phase of study included historical research, surface reconnaissance and mapping, and the drilling of five exploratory borings. The second phase included the installation of ten additional borings, laboratory testing of soil samples, identification of the failure surface, detailed slope stability analyses, recommendations for landslide stabilization, and field observation of corrective actions. The final phase of study included monitoring of landslide movements and ground-water levels following the completion of remedial measures.

No significant movements of the slide mass were detected after implementing the landslide stabilization measures.

GEOLOGIC SETTING

The study area is in a residential area of southwest Seattle, approximately 5.5 mi from Seattle's metropolitan center and 1,500 ft east of the eastern shoreline of Puget Sound (Figure 1). Ground surface elevations near the landslide range from approximately 250 to 325 ft above mean sea level. Local features include streets, residential structures, retaining walls, and undeveloped land occupied by part of Lincoln Park (Figure 2).

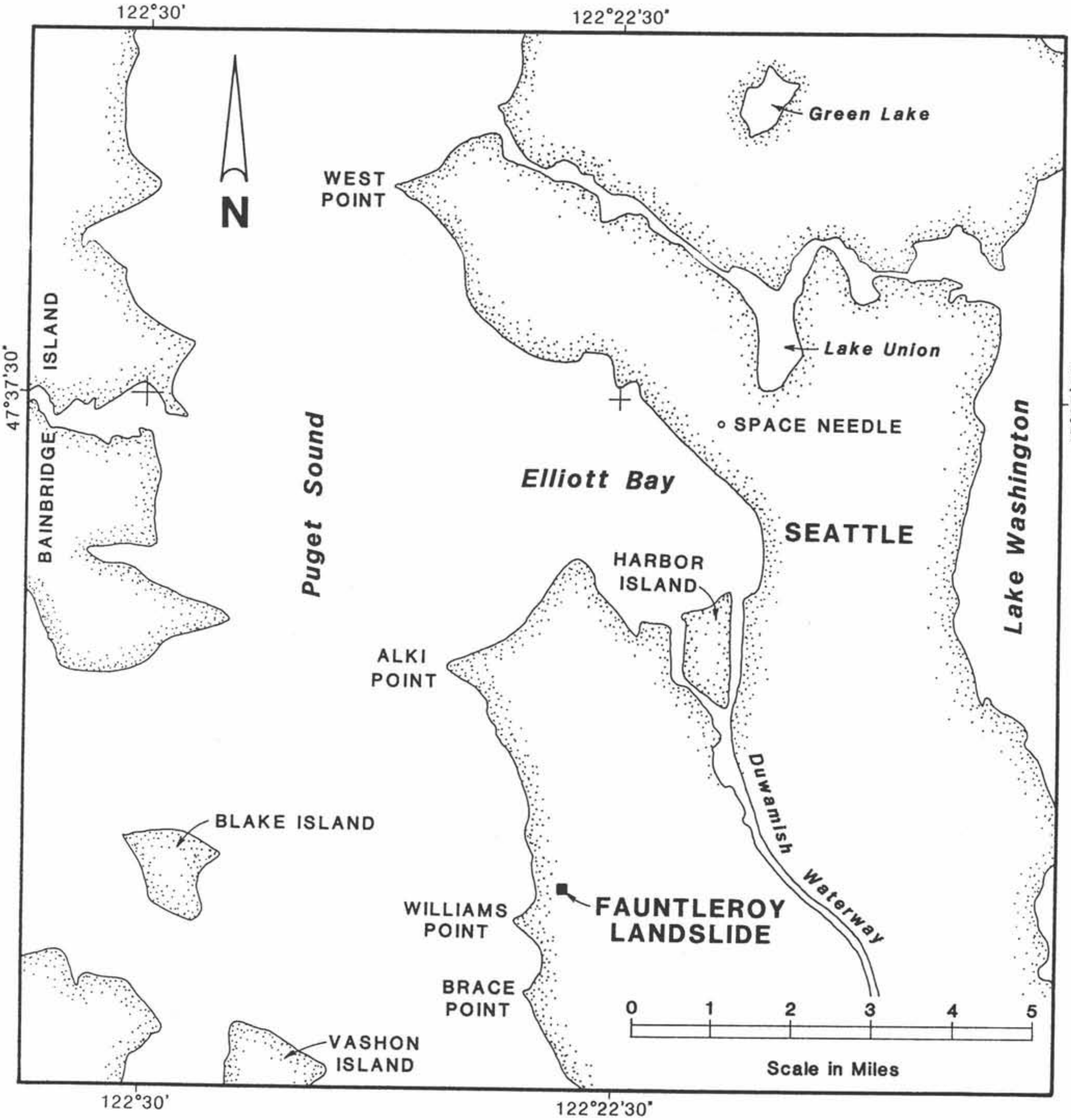
The geologic units of significance to this case history consist of glacial and nonglacial sediments that have been consolidated and densified by glacial ice of the Vashon Stade of the Fraser Glaciation. The sediments in the study area include a unit of sand that overlies bedded silt and clay. The sand unit was deposited as advance outwash during the Vashon glaciation between approximately 15,000 and 13,500 yr ago (Mullineaux et al., 1965; Armstrong et al., 1965). This unit, termed the Esperance Sand Member of the Vashon Drift, is mapped in the eastern portion of the Fauntleroy landslide on the geologic map of the Duwamish Head quadrangle (Waldron, 1967).

The contact zone between the Esperance Sand Member and underlying relatively impermeable silt and clay sediments is typically saturated with ground water. This contact zone has been identified as the locus of frequent landslide activity in Seattle (Tubbs, 1974).

HISTORY OF LOCAL DEVELOPMENT

Landsliding and urbanization are often closely intertwined. In some cases, human modifications such as roads or structures are damaged as a result of completely natural landsliding processes. In other cases, however, physical changes resulting from human activities contribute to slope instability. Human actions that may destabilize landslides include fill placement on slopes, excavations on slopes, interruption of natural drainage, and the introduction of water to the subsurface via "dry wells", septic-tank drain fields, or utility leaks.

Residential development has occurred near the Fauntleroy landslide since the early 1900s. Intermittent construction of private homes in the landslide area has continued into the 1980s. Some of the pertinent milestone dates related to public streets and utilities are listed below.



Base map: U.S. Geological Survey 1:100,000
scale topographic map of the Seattle quadrangle.

Figure 1. Location of Fauntleroy landslide, southwest Seattle.

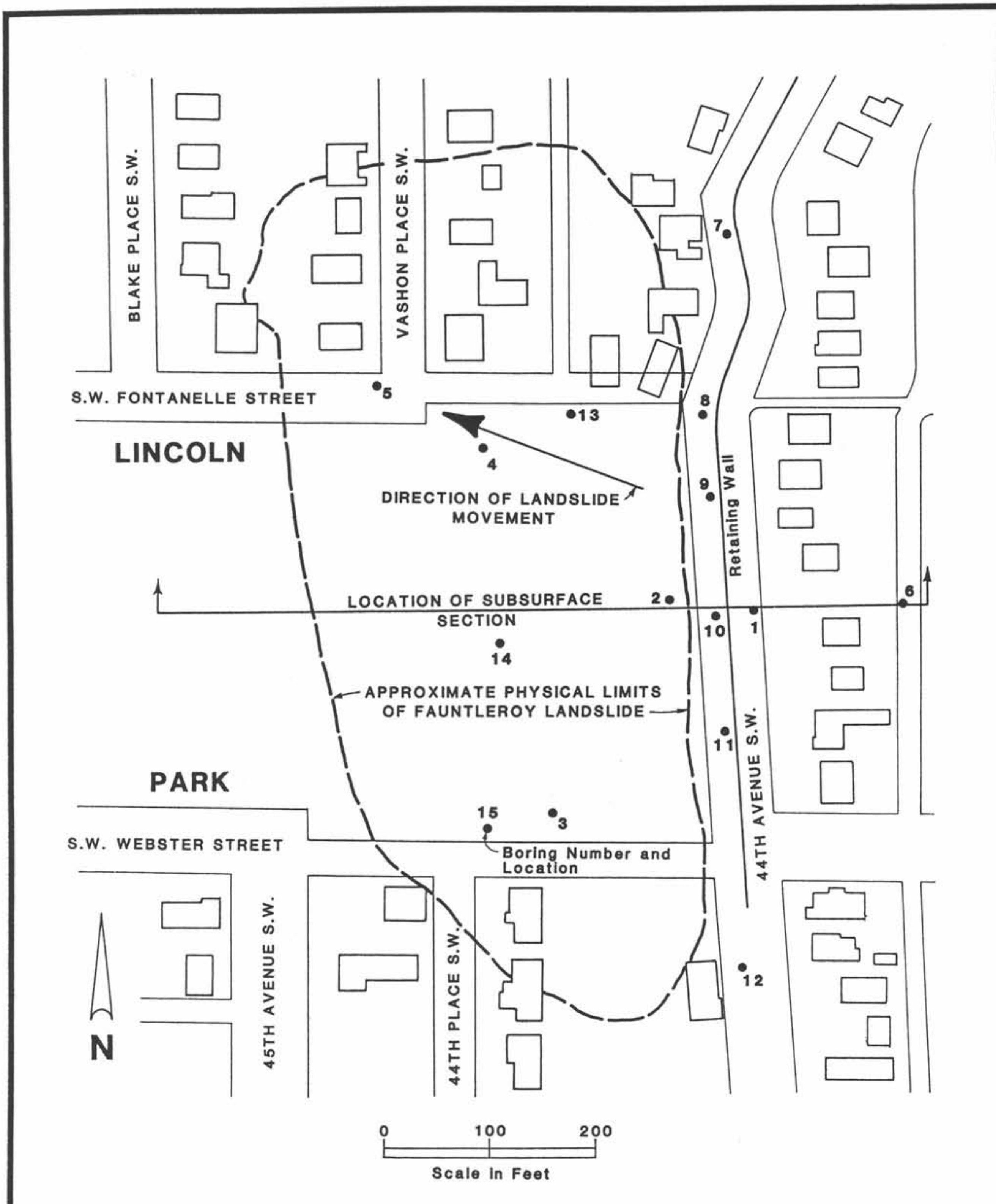


Figure 2. Limits of the Fauntleroy landslide. Section is shown in Figure 3.

1910 - 44th Avenue S.W. graded with construction of a wood retaining wall to separate upper and lower street levels.

1911 - Water main installed along upper level of 44th Avenue S.W.

1922 - Lincoln Park property purchased by City of Seattle.

1926 - Combination storm and sanitary sewers constructed.

1941 - Wood retaining wall on 44th Avenue S.W. replaced with "slab and rail" retaining wall.

1949 - 44th Avenue S.W. paved.

1964 - Vashon Place S.W., Blake Place S.W., and S.W. Fontanelle Street paved and improved.

1973 - Combination sewers converted to separate storm and sanitary sewers.

LANDSLIDE HISTORY

The first reports of landslide activity in the vicinity of the Fauntleroy landslide were received by the Seattle Engineering Department in 1974. Movement at that time damaged residences and underground utilities along Vashon Place S.W., S.W. Fontanelle Street, and 44th Avenue S.W. Several residences along 44th Avenue S.W. were affected by recurrent earth movements between 1974 and 1977. Renewed landslide movement occurred during early 1983, when a 1-ft-high head scarp developed, utilities were broken, and residential structures were damaged.

City records contain information suggesting that the Fauntleroy landslide may have been active prior to 1974. Ground profiles measured in 1909 along S.W. Webster Street (prior to grading of 44th Avenue S.W.) indicated topography suggestive of landslide activity. Also, sewer line breaks were reported in 1971 between 44th Avenue S.W. and Vashon Place S.W.

ENGINEERING GEOLOGIC STUDIES

Geologic studies of the Fauntleroy landslide were begun in March 1983, following reactivation of the slide mass. Surface mapping and research of City records resulted in a map of the limits of the landslide (Figure 2). Mapping of the outer limits of the slide mass was based on visible surface features such as the head scarp and pavement ruptures, together with structural damage to homes and the locations of sewer line breaks. The physical limits of the slide mass were not clearly defined in the relatively undeveloped southwestern portion of the slide mass. As mapped in Figure 2, the Fauntleroy landslide has a surface area of approximately 5.7 acres.

A total of 15 exploratory borings was drilled in two phases within and adjacent to the landslide to develop information regarding subsurface geology and groundwater conditions (Figure 2). Inclinometer casings were installed in borings 10, 13, 14, and 15 to provide data about the depth, magnitude, rate, and direction of landslide movements. Piezometers were installed in all the other borings for the purpose of measuring groundwater levels.

The subsurface explorations indicated that the eastern (uphill) portion of the landslide mass was characterized by a unit of fine sand (Esperance Sand Member) overlying stiff to hard silt and clay (pre-Vashon sediments). This upper sand unit was not present in the western portion of the slide area. The basal portion of the sand unit was saturated with ground water. (Small springs and seeps often marked the western extent of the sand unit within the undeveloped portion of the landslide in Lincoln Park.) Fill was present along the western margin of 44th Avenue S.W., particularly near its intersection with Fontanelle Street S.W.

Generalized geologic conditions are indicated on the subsurface section, Figure 3. The Esperance Sand Member was found to be very dense outside the slide mass and loose to medium dense within the slide area. Many soil samples obtained from the underlying silt and clay unit contained slickensided fractures. These fractures, which indicate past shearing displacements within the soil, were found in samples obtained from zones of suspected landslide movement as well as from areas outside and beneath the slide mass. Although some of the slickensided fractures were undoubtedly related to slide movements, others were probably a result of stress relief during unloading of the land surface by melting of glacial ice.

The landslide failure surface shown in Figure 3 was developed from surface evidence as well as subsurface measurements. For example, the piezometer that was installed in boring 2 in late April 1983 became kinked at a depth of 24 ft in January 1984. The depth of piezometer deflection in boring 2 coincided with a zone of intense fracturing noted in soil core samples obtained from that boring, leading to the conclusion that the zone of landslide movement was 24 ft deep at that location.

A slope indicator casing was installed in boring 14 in early November 1983. Subsequent inclinometer readings documented a shearing offset in the slope indicator casing at a depth of 45 ft . Numerous slickensided fractures were found in soil core samples obtained between the depths of 34 ft and the base of the drillhole (59 ft) for boring 14. The inclinometer casing for boring 13 indicated shearing at a depth of 49 ft. This zone also coincided with observations of shearing in soil core samples. Plots of the inclinometer surveys in boring 13 are shown in Figure 4. These plots indicate progressive creep of the slide mass between November 1983 and March 1985.

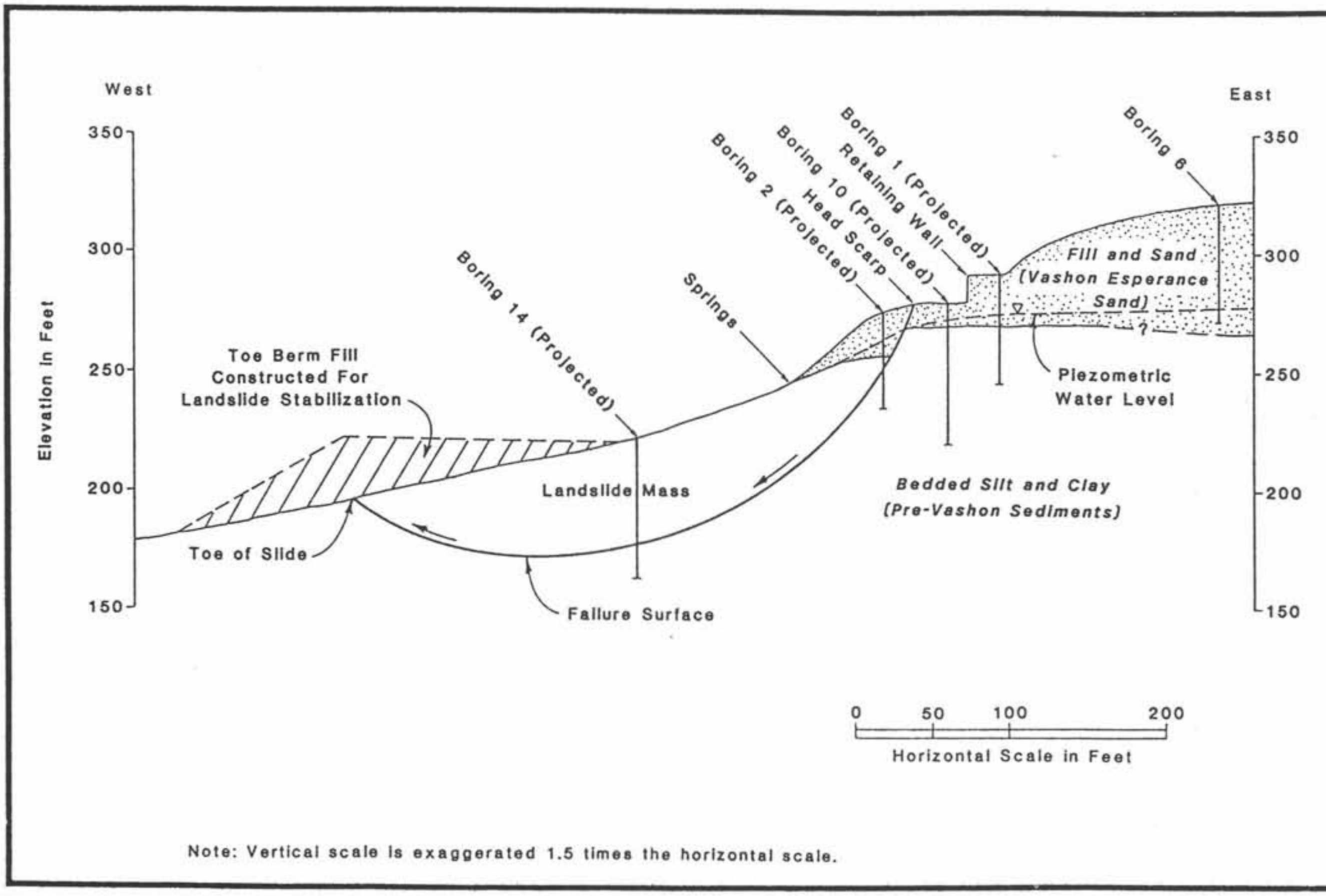


Figure 3. Generalized geologic section of the Fauntleroy landslide.

Analysis of the inclinometer measurements in borings 10, 13, 14, and 15 resulted in the following conclusions regarding the Fauntleroy landslide:

(1) A single, relatively deep failure surface was found at a depth of 49 ft in boring 13 and 45 ft in boring 14.

(2) No definite failure surface was found in boring 15.

(3) No deflections related to slope movements were detected in boring 10, located immediately east (uphill) of the slide mass defined during initial geologic reconnaissance.

(4) The dominant direction of slide movement was west-northwest (approximate compass azimuth 290°).

(5) The northern portion of the slide mass moved more rapidly than the southern portion. The magnitude of shearing deflection measured in boring 13 was nearly double that of boring 14.

Most of the failure surface was found to be within the pre-Vashon silt and clay unit (Figure 3). Numerous soil samples from this unit were tested to determine their pertinent physical characteristics. The laboratory results indicated considerable variation within the unit. Specific samples tested ranged from sandy silt of low plasticity to silty clay of high plasticity. A summary of the laboratory test data for the silt and clay unit is presented in Table 1.

SLOPE STABILITY MODEL

The stability of the slide mass was analyzed with a microcomputer using the STABL computer program (Siegel, 1975). This program has the ability to analyze slope stability for a wide range of failure surface geometries with almost any arrangement of subsurface soil types.

The slope profile, soil conditions, and failure-surface geometry used in stability analyses were basically as shown in Figure 3. The first step in the analyses was to

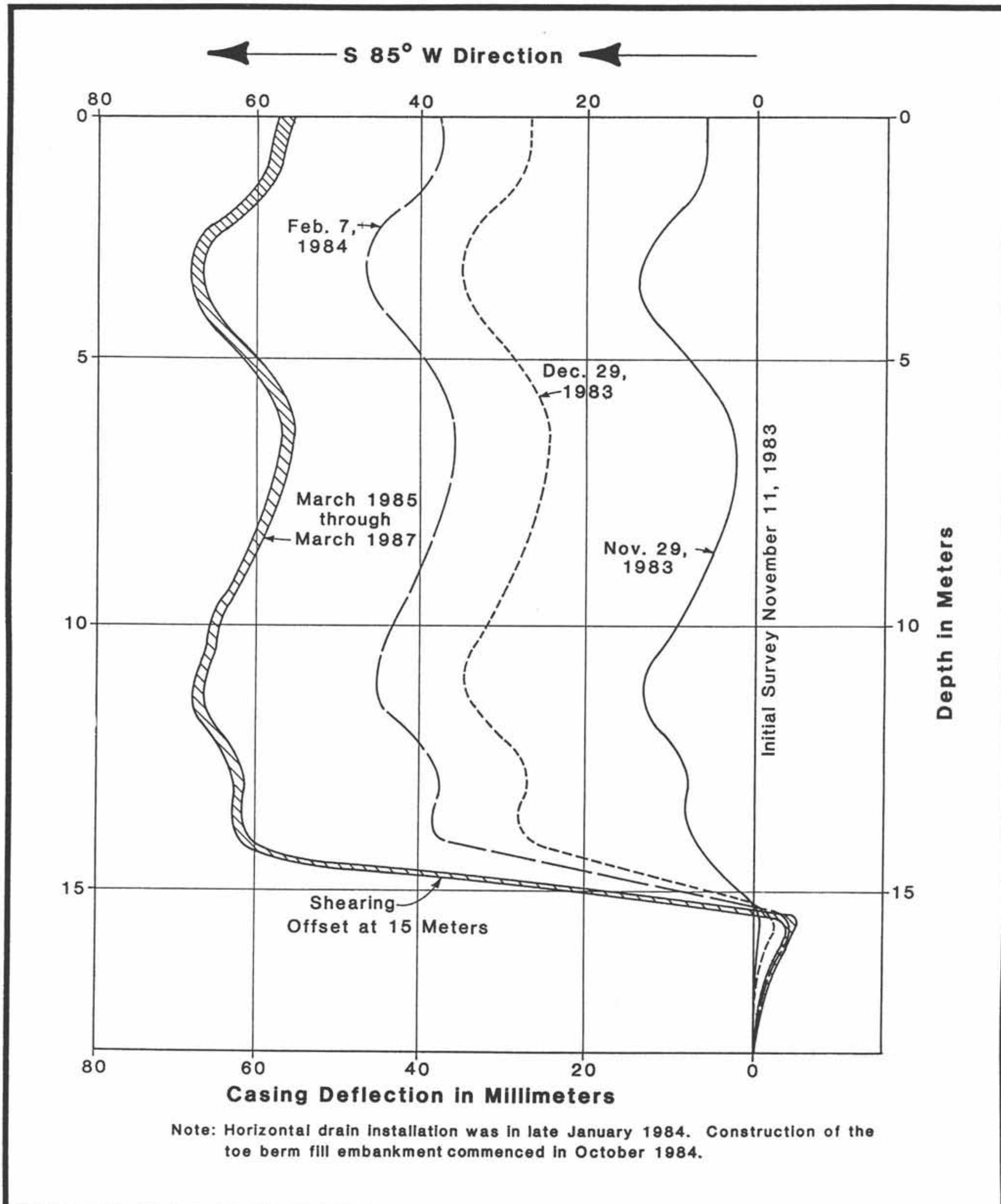


Figure 4. Plots of inclinometer data from boring 13 in the Fauntleroy landslide.

Table 1. Summary of laboratory data for silt and clay unit, Fauntleroy landslide. Parameters with asterisks are based on multiple direct shear tests on the same failure surface using variable axial loads; rate of shearing deflections was 0.5 in./min

Parameter	Range	Number of tests	Mean	Standard deviation
Dry density (pcf)	65-102	65	91	7.7
Field moisture (%)	3.4-58.6	65	31	7.4
Liquid limit (%)	27.8-97.6	13	52	21
Plasticity index (%)	0-60.8	13	24	22
Residual cohesion (psf)*	50-350	9	150	83
Residual friction angle*	11^{o}-29^{o}	9	23^{o}	5.2

back-calculate soil strength values, which resulted in a factor of safety against sliding of 1.0 for the slope model. This presumes that the onset of slope movement corresponds to a factor of safety of 1.0. The soil parameters which resulted from these initial stability analyses are summarized in Table 2.

The general slope geometry (Figure 3) and the soil parameters above comprised the slope stability model for evaluating the effectiveness of potential remedial options for the slide mass.

CORRECTIVE ACTIONS AT THE LANDSLIDE

Movement of a landslide mass occurs when the gravitational forces acting to pull the soil mass downslope (driving forces) exceed the frictional forces available to resist these movements (resisting forces). Landslide stabilization actions involve procedures that either reduce driving forces, increase resisting forces, or both.

Numerous options were considered for improving the stability of the Fauntleroy landslide. Several options were potentially feasible within Lincoln Park, where no residential development or utility construction had occurred. However, remedial options were very limited in the northern portion of the slide mass due to the presence of private residences, streets, and subsurface utilities.

Table 2. Soil parameters

Parameter	Esperance Sand and fill	Silt and clay soils
Total unit weight (pcf)	110	115
Saturated unit weight (pcf)	120	120
Friction angle (degrees)	32	17
Cohesion (psf)	0	50

Three procedures were ultimately employed for improving the stability of the slide mass:

(1) Horizontal drains were installed in the northern portion of the slide, where large-scale construction activities were not feasible.

(2) A toe berm fill with subdrainage was constructed in Lincoln Park.

(3) Several hundred cubic yards of fill were removed from the head of the slide near Fontanelle Street S.W. and 44th Avenue S.W.

The primary corrective actions were installation of the horizontal drains and placement of the toe berm fill. Descriptions of each follow.

Horizontal Drains

The exploration borings indicated the presence of 2 to 12 ft of saturated sand overlying the silt and clay unit beneath 44th Avenue S.W. near S.W. Fontanelle Street. Using subsurface data from borings 7, 8, and 9, a profile of the basal contact for the sand unit was developed.

An array of seven horizontal drains was installed in late January 1984 for the purpose of lowering the ground-water level in the northern portion of the slide mass (Figure 5). The elevation and coordinates of the drilling platform were predetermined so that the azimuth and vertical angle of the drains could be controlled to intercept the "target" locations beneath 44th Avenue S.W. Each drain was installed from the same drilling platform. The drains ranged in length from 175 to 295 ft. Despite the relatively thin target zones of saturation, all but one of the drains resulted in a sustained flow of ground water.

The drainpipes consisted of Schedule 80 PVC pipe with an inside diameter of 1.5 in. The lowermost (westernmost) 30 to 50 ft of pipe was not slotted. The remainder of each drain was perforated with three rows of machined slots having a slot width of 0.010 in. Discharge from the drains was routed to a special catch basin near the western ends of the drains, where the flow from each drain could be measured independently.

The horizontal drains resulted in an almost immediate and sustained lowering of ground-water levels near the drains.

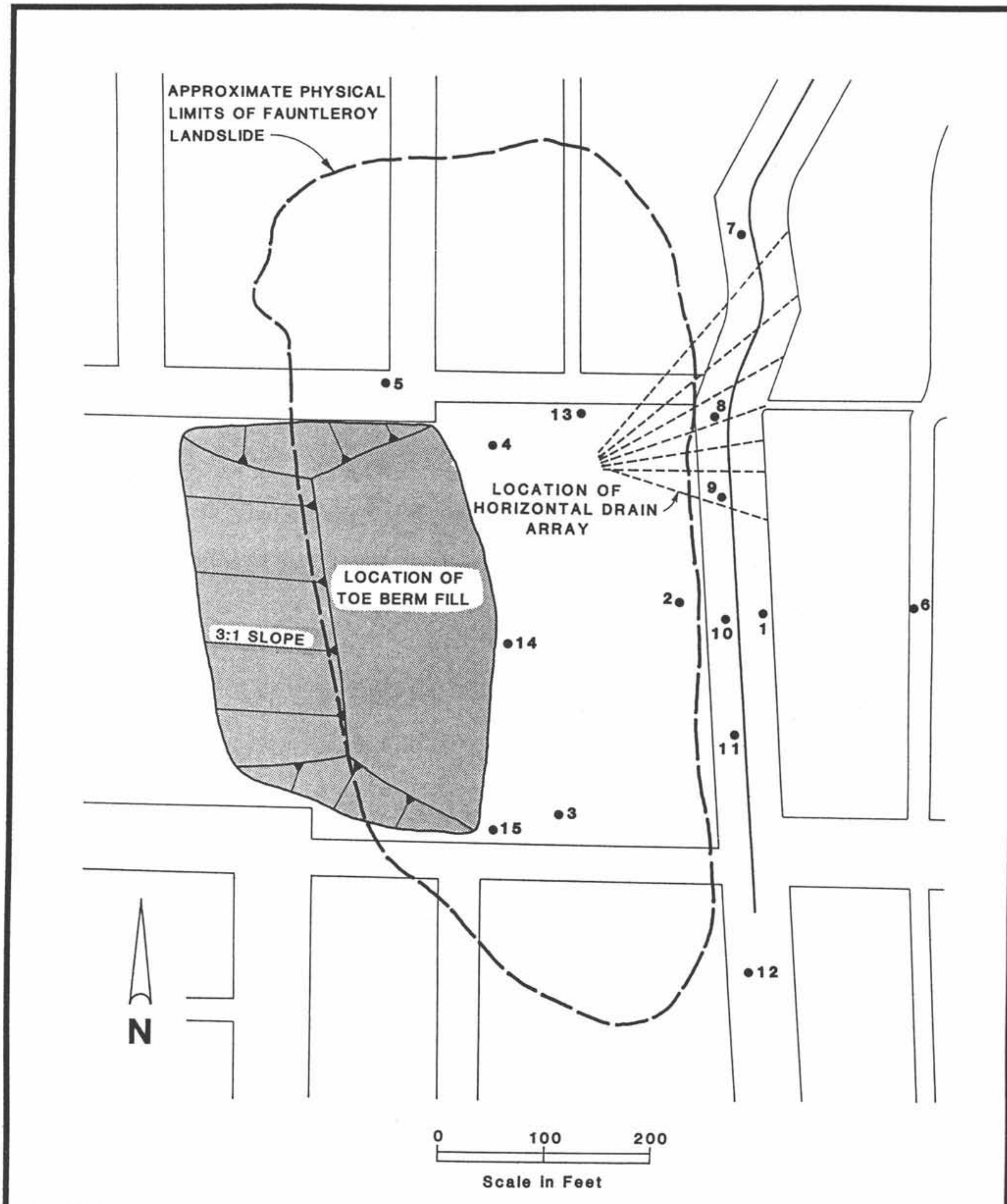


Figure 5. Locations of corrective measures for the Fauntleroy landslide.

Permanent water-level reductions of approximately 5, 12, 9, and 7 ft were documented for the piezometers in borings 7, 8, 9, and 10, respectively. These piezometers are in the immediate area of the drain array. Permanent water-level lowerings of lesser magnitudes were also indicated by the ground-water monitoring data for the piezometers in borings 1, 5, and 6.

The initial volume of water discharge from the six functional horizontal drains was approximately 6.5 gpm on February 3, 1984. The flow volume diminished as water levels declined due to dewatering of the Esperance Sand Member. Total flow measurements between October 1984 and March 1987 ranged between 1.4 and 2.7 gpm.

A lowered ground-water level in a slide mass results in a greater effective weight for the slide mass (due to a reduction in buoyancy forces for submerged soils). This increased weight mobilizes a greater frictional resistance to landslide movement. (Frictional forces are directly proportional to the weight applied normal to the sliding surface.) Slope stability analysis was conducted to estimate the effectiveness of the horizontal drains in improving stability conditions in the northern portion of the slide mass. Based on the stability model used in the analysis, the factor of safety against sliding improved from 1.0 to approximately 1.07 due to dewatering only.

Toe Berm Fill

The Seattle Engineering Department constructed a fill embankment along the toe of the landslide within Lincoln Park, approximately as shown in Figures 3 and 5. Preparation for this toe berm fill involved clearing of vegetation and construction of subdrains to carry drainage under the fill embankment. A total of approximately 32,900 cy of fill was used to construct the embankment. The fill consisted of waste soil materials and demolition debris from City of Seattle construction and maintenance projects. Fill placement occurred intermittently between October 1984 and September 1986.

Construction of a fill embankment at the toe of a landslide results in (1) an increase in soil weight at the base of the slide, and (2) additional shearing resistance gained by forcing the failure surface through the fill. Slope stability analyses indicated an increase in the factor of safety against slide movement from 1.0 to approximately 1.25 due solely to construction of the toe berm fill. This relatively high degree of stability enhancement was possible only in Lincoln Park, where major land surface modifications were feasible without interfering with homes, streets, or utilities.

LANDSLIDE MONITORING

Measurements of piezometric water levels, flow from the horizontal drains, and alignment of the inclinometer casings (borings 10, 13, 14, and 15) were made periodically between November 1983 and March 1987. The monitoring efforts indicated the following:

(1) Ground-water levels in the vicinity of the horizontal drain array remained depressed from a short time after drain installation (January 1984) through March 1987.

(2) Total flow from the six functional horizontal drains stabilized at between 1.4 and 2.7 gpm shortly after drain installation.

(3) Approximately 2.8 in. of shearing displacement was measured across the failure surface in boring 13 between November 11, 1983, and March 1985. Most of this displacement occurred between November 1983 and March 1984.

(4) No significant displacements were detected in the inclinometer casings for borings 13, 14, or 15 between March 1985 and March 1987.

CONCLUSIONS

Remedial actions taken at the Fauntleroy landslide appear to have been successful in arresting landslide movement, at least within the instrumented portion of the slide mass. However, the relative degree of stability improvement is not uniform within the landslide area. A relatively large amount of stability improvement has been provided to the portion of the slide mass in Lincoln Park, where construction of a toe berm fill was feasible. The northern residential portion of the slide mass has been afforded less stability improvement because existing structures and utilities prevented large-scale remedial efforts.

All corrective actions for the Fauntleroy landslide were implemented from within undeveloped portions of public property. Some landslides in urban areas of Washington do not have undeveloped land available for corrective actions. Options for stability improvements are much more limited in totally urbanized landslide areas, and, in general, stability improvements are less effective in these locations.

ACKNOWLEDGMENTS

The geologic studies and remedial measures for the Fauntleroy landslide were made possible by funding from the Seattle Engineering Department. Herb Allwine of that department managed the project, secured city records, and coordinated the project. The Seattle Parks Department cooperated with the implementation of remedial measures within Lincoln Park.

The support staff of GeoEngineers, Inc. provided their time, labor, and talents to the production of this case history. Valuable editorial and technical input was also received from William Laprade of Shannon & Wilson, Inc.

REFERENCES

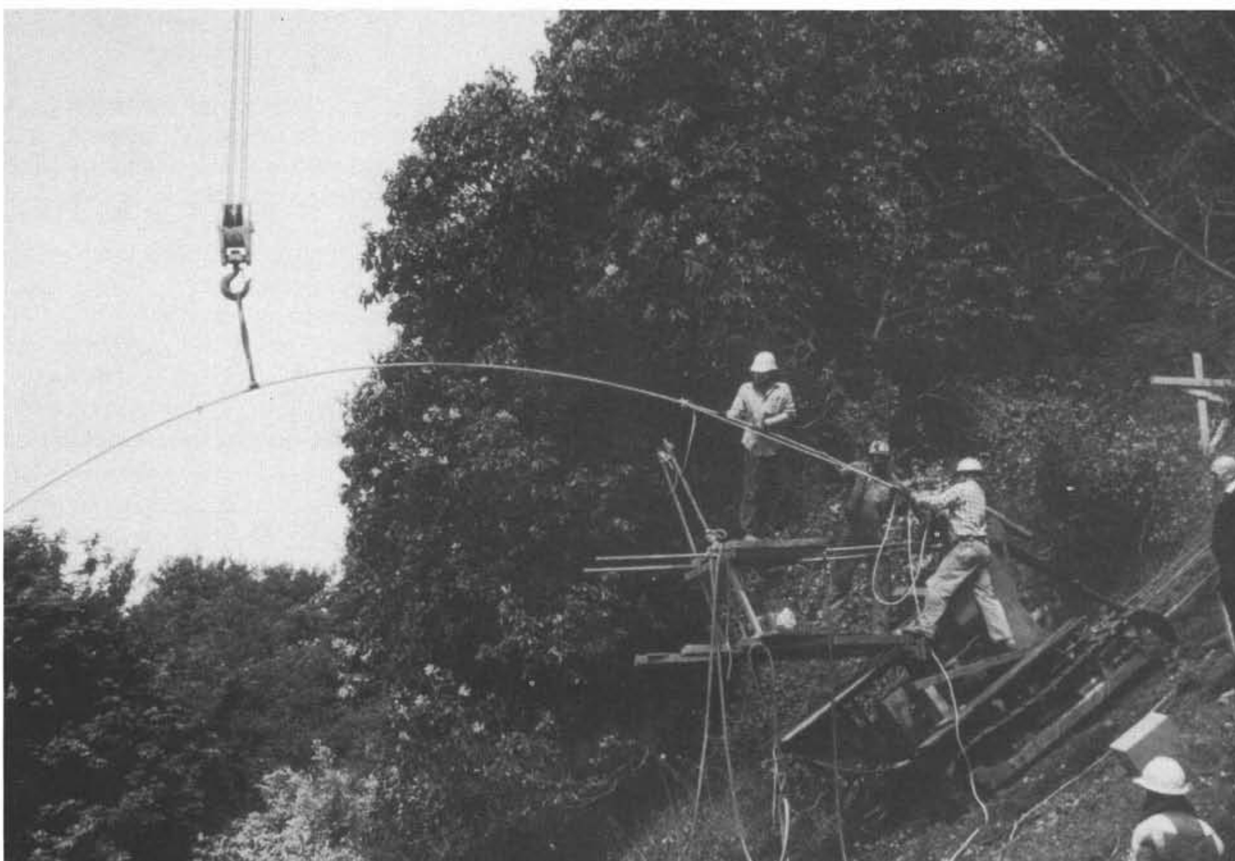
Armstrong, J. E.; Crandell, D. R.; Easterbrook, D. J.; and Noble, J. B., 1965, Late Pleistocene stratigraphy and chronology in southwestern British Columbia and northwestern Washington: *Geological Society of America Bulletin*, Vol. 76, pp. 321-330.

Mullineaux, D. R.; Waldron, H. H.; and Rubin, M., 1965, *Stratigraphy and Chronology of Late Interglacial and Early Vashon Glacial Time in the Seattle Area, Washington*: U.S. Geological Survey Bulletin 1194-O, 10 p.

Siegel, R. A., 1975, *STABL User Manual, a Description of the Use of STABL's Programmed Solution for the General Slope Stability Problem*: Joint Highway Research Project, Engineering Experiment Station, Purdue University, JHRP-75-8, adapted for IBM-PC computers by Design Professionals Management Systems, Kirkland, WA.

Tubbs, D. W., 1974, *Landslides in Seattle*: Washington Division of Geology and Earth Resources Information Circular 52, Olympia, WA, 15 p.

Waldron, H. H., 1967, *Geologic Map of the Duwamish Head Quadrangle, King and Kitsap Counties, Washington*: U.S. Geological Survey Geologic Quadrangle Map GQ-706, 1 sheet, scale 1:24,000.



Tie-back anchors being installed for a residence on a steep hillside, 1982. Photograph courtesy of Shannon & Wilson, Inc.

Engineering Geology in Washington, Volume II
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Regrading Years in Seattle

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Seattle City Engineer, Retired

INTRODUCTION

The fate of Seattle as a great commercial port on the Pacific Ocean might well have been changed had the early pioneers recognized the enormous regrading task to be faced in creating a city where the street grades would help, not hinder, transportation and commerce.

Welford Beaton (1914, p. 64) wrote, in *The City That Made Itself*,

> "Nature apparently grew tired before she finished Seattle. She made a wonderful harbor, produced an empire of timber-hung pictures on the horizon, spread three lakes among the hills and then left the townsite to itself like a tousled, unmade bed."

Seattle's hills are oriented generally in a north-south direction, the result of similarly directed glacial movement and postglacial stream activity in the late Pleistocene Epoch. Lenticular hills are composed of stratified clays, silts, sands, and gravels, usually under a thin veneer of topsoil. Fortunately, in no place in the downtown Seattle area is any bedrock near the surface. This circumstance made the regrading projects less costly.

The Denny, Maynard, Bell, and other plats were laid out with little attention to the steep hills and gullies facing Elliott Bay. Streets suitable for horse-drawn vehicles were to become a necessity. Yesler's Mill was built at the south end of the new town in the late 1850s. Docks were constructed to serve the growing fleet of ships. Surrounding business establishments grew in size and number to serve the expanding population. It became increasingly evident that removal of hills and filling of gullies and tideflats was going to be high on the list of priorities.

The earliest loggers used oxen to haul the heavy old-growth timber logs from the surrounding hillsides to the tide waters of Elliott Bay over a trail on which small logs were placed crosswise at intervals, creating a skid road. First called Mill Street, this first skid road was later changed to Yesler Way. Puncheon (roughly dressed logs) roads soon followed the skid road in the early efforts to permit travel out of the mud in winter and dust in summer. With the opening of Yesler's mill at the west end of Mill Street, planks became available, and Front Street, later First Avenue, was among the first to receive a planked surface.

Regrading by municipal contract was not a factor during Seattle's beginning years. But, as horse-drawn vehicular traffic increased, the steep grades on too many streets had to be corrected. In 1876 a comprehensive regrading of First Avenue from James Street to Pike Street was completed. The hill at First and Cherry was eliminated, and the deep gully in James Street was filled. Contractors graded 28 streets in 1885, including Second and Third Avenues South and Fourth and Ninth Avenues. Downtown Seattle was not to be free of this kind of work until the early 1930s.

Seattle's great fire of June 6, 1889, consumed most of the business district. Plans for new and better municipal facilities could hardly keep ahead of the reconstruction that started immediately. The original wooden Occidental Hotel, on Occidental Avenue south of Yesler Way, was rebuilt following the fire and renamed the Seattle Hotel, then Seattle's finest. But increasing business soon called for an additional fine hotel. This one, the Washington Hotel, was built between Second and Fourth Avenues from Stewart to Virginia Streets on a hilltop that provided a magnificent view of the bay, the hills, and the mountains.

Exercising real foresight, in 1889 the City Council ordered the widening of First Avenue and Yesler Way from 66 to 84 ft and the raising of street grades south of Yesler Way. There was a fast-running stream on Third Avenue south of Yesler Way, and this gully required fill to a maximum of 35 ft (at Jackson Street). The areaways under the sidewalks in the Pioneer Square area were to be left at the early grades, thus creating what has become known as "Underground Seattle".

A far-sighted and dedicated though crusty engineer, Reginald Heber Thomson, better known as R. H. Thomson, was appointed City Engineer on May 25, 1892. His vision of Seattle as a large commercial city quickly centered on the regrading work necessary to create streets with grades no steeper than those that teams of horses could negotiate. Hills would have to be removed, gullies filled, and tideflats raised if Seattle's commerce was to grow as those early pioneers expected. All north-south streets between Yesler Way and Denny Way, from

First Avenue to Sixth Avenue, needed major work, and the connecting east-west streets would have to be adjusted to fit the new grades.

Many contracts were awarded during the two decades following Thomson's appointment as City Engineer. The shaded portions in Figure 1 illustrate the extent of the regrading work that would remake Seattle.

Early in the program it was apparent that Railroad and Western Avenues needed filling. In 1898 these roadways were filled with some of the excavated material from the regrading of First Avenue from Pine Street to Denny Way, where cuts of as much as 17 ft had been made. The regrading program escalated, and numerous contracts were awarded. Table 1 summarizes work done from 1903 to 1914. Typical of the methods used in these early days are the Third Avenue excavation and the Seventh Avenue fill, shown in Figures 2 and 3.

FIRST DENNY HILL REGRADE (1903-1911)

One of the areas north of the main business district that required extensive regrading was the Denny Hill; work there was destined to be done in two major steps.

As the various areas of the city were set up for regrading in accordance with City Engineer R. H. Thomson's grand plan, the City Council established Local Improvement Districts (LID) in which the property owners, after public hearings in the Council Chambers, were assessed part of the cost of the improvement, but with the major part coming from the General Fund. Many property owners were enthusiastic supporters of the plans, for they anticipated the increased commerce

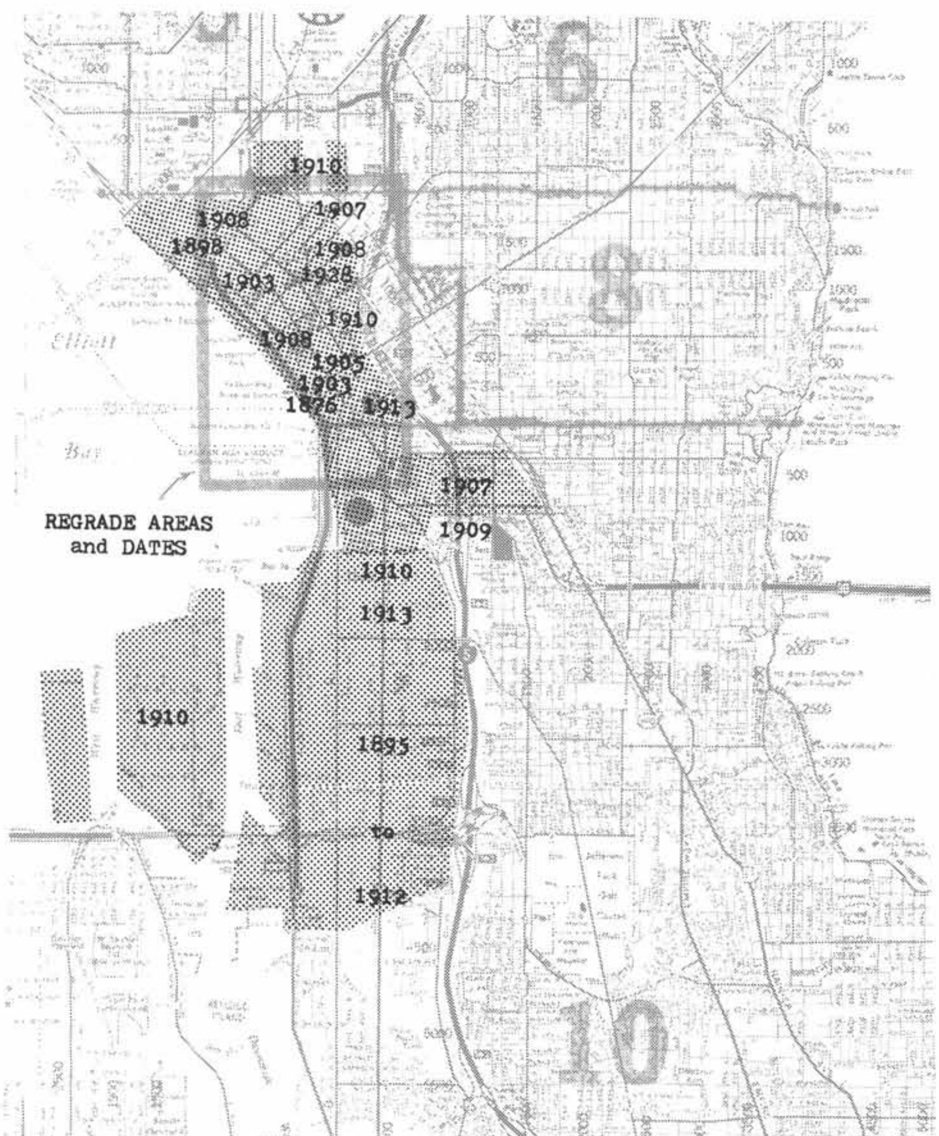


Figure 1. Map of Seattle regrade area, with dates of activity.

Table 1. Data for regrading in Seattle, 1903-1914

Excavation

Contract Title	Material moved (cy)	Start	Finish	Contractor
Pike St. and E. Pike St.	36,479	6-27-03	2-04-04	C. J. Erickson
Pine St.	39,808	8-26-03	4-11-05	C. J. Erickson
Second Ave.	603,862	7-29-03	3-04-06	C. J. Erickson
Third Ave.	92,441	8-04-05	6-28-07	C. J. Erickson
Fairview Ave.	85,294	7-10-07	9-01-08	Ottesen & Jensen
Fourth Ave.	387,168	6-21-07	7-21-09	C. J. Erickson
Jackson St.	1,810,656	4-23-07	2-28-10	Lewis & Wiley
Third Ave.	3,104,604	6-23-08	6-09-11	Rainier Dev. Co.
Ninth Ave. & Ninth Ave. N.	196,748	8-22-10	9-28-11	P. J. McHugh
Harrison St.	91,708	12-27-10	10-31-11	P. J. McHugh
Fifth Ave. & Fifth Ave. S.	173,057	8-12-10	4-09-12	Erickson Constr.
Dexter Ave.	72,441	6-20-10	9-30-12	Olson & Mellen
Dearborn St.	1,259,836	9-24-09	9-30-12	Lewis & Wiley
Leary Ave.	50,417	6-03-13	1-29-14	W. F. Manney & Co.
Sixth Ave.	91,255	5-21-13	6-15-14	Ind. Asph. Pav. Co.
Total excavation	8,095,774			

Embankment

Contract Title	Material moved (cy)	Start	Finish	Contractor
Westlake Ave.	32,397	7-10-07	2-02-08	Hans Pederson
Pine St.	392,121	7-25-07	7-02-09	Hawley & Lane
Fifth Ave. N.	24,374	10-03-08	10-24-09	Grant Smith
Jackson St.	1,356,038	4-23-07	2-28-10	Lewis & Wiley
Western Ave. & Pike Pl.	62,992	12-21-09	4-20-11	Paul Steenstrup
Olive St.	173,670	7-16-08	7-18-11	Hawley & Lane
Twelfth & Twelfth S.	306,635	10-08-10	3-10-12	Erickson Const.
Jackson St. & 22nd Ave. S.	16,646	4-19-13	10-16-13	Andrew Peterson
Total embankment	2,364,873			

and business that would come about, as well as the increases in real estate values. Other citizens were equally unhappy with the prospect of mud, dust, and confusion resulting from Thomson's plans. Their anger and non-support became sharply evident as the work proceeded around their properties, creating small hillocks (Figure 4).

J. A. Moore, of the Denny Hotel Group, adamantly opposed the razing of the Washington Hotel and the regrading of Denny Hill on which it so magnificently stood. With its only access being by horse and carriage from First Avenue, the Washington had not really opened until President Theodore Roosevelt's visit to the city after the Great Fire. So difficult was the approach to the hotel after the regrading that a counterbalanced trolley line was built to connect the entrance with level ground at Pike Street. Built mostly on trestle, the line had a steady grade of 20 percent. Seattle Electric Co. car No. 139 shuttled back and forth on the three-block route for its 12-yr life span (Figure 5). As the years passed, Moore became increasingly unhappy with the problems of his hotel-on-a-mesa location. He finally decided to capitulate, and on April 18, 1906, the Council granted Moore's request to excavate, at his own expense, the streets in the block occupied by the hotel. In addition to the loss of a hotel, this change of mind cost him $133,079.22.

The Seattle Post Intelligencer, on Sunday, August 2, 1903, stated, "It is doubtful whether any public improvement made during many years has created so much

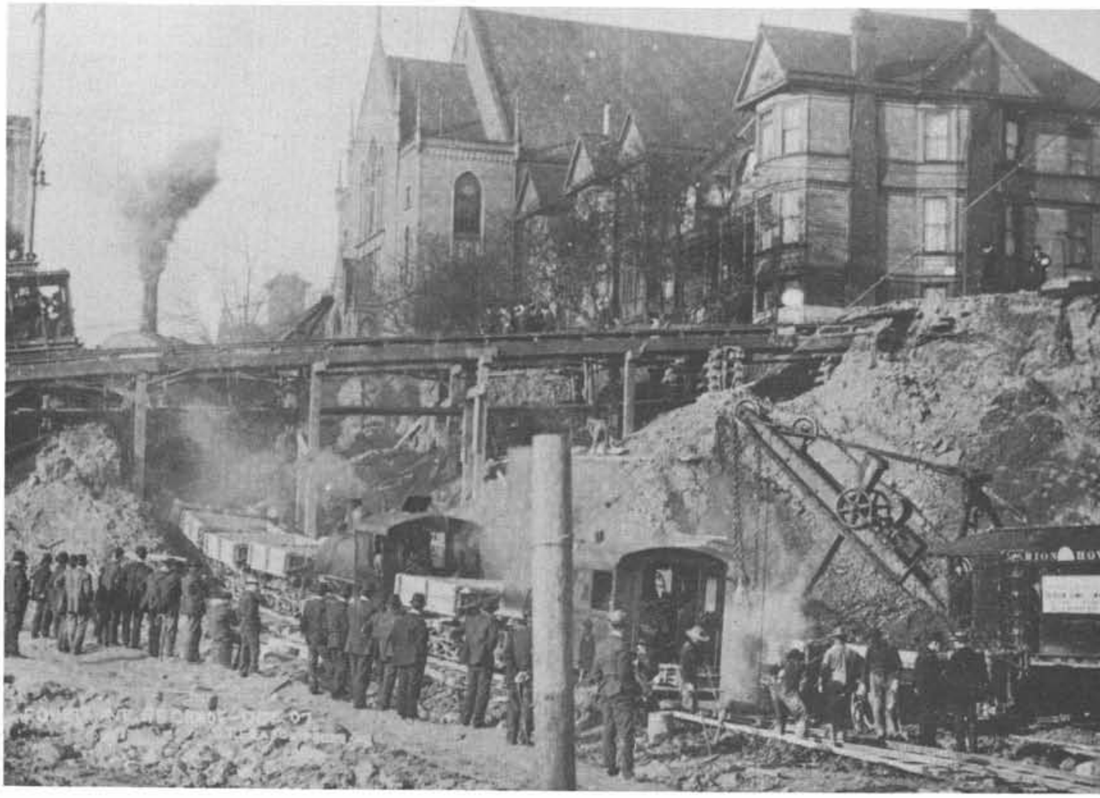


Figure 2. 3rd Avenue excavation, 1905. Photo courtesy of the Museum of History and Industry, Seattle.

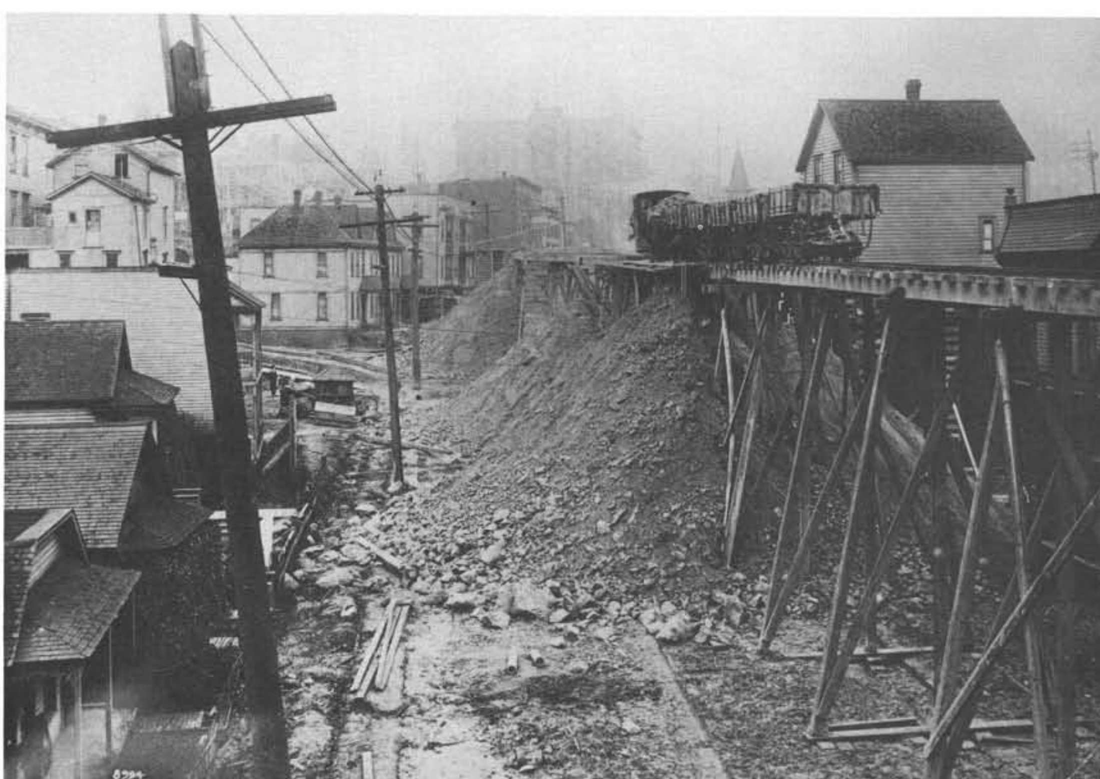


Figure 3. 7th Avenue fill, 1906, view to the south from Stewart Street. Photo courtesy of the Museum of History and Industry, Seattle.

Figure 4. The levelling of the hills to make Seattle. Photo by Asahel Curtis; courtesy of the Special Collections Division, University of Washington Libraries (negative no. 4812).

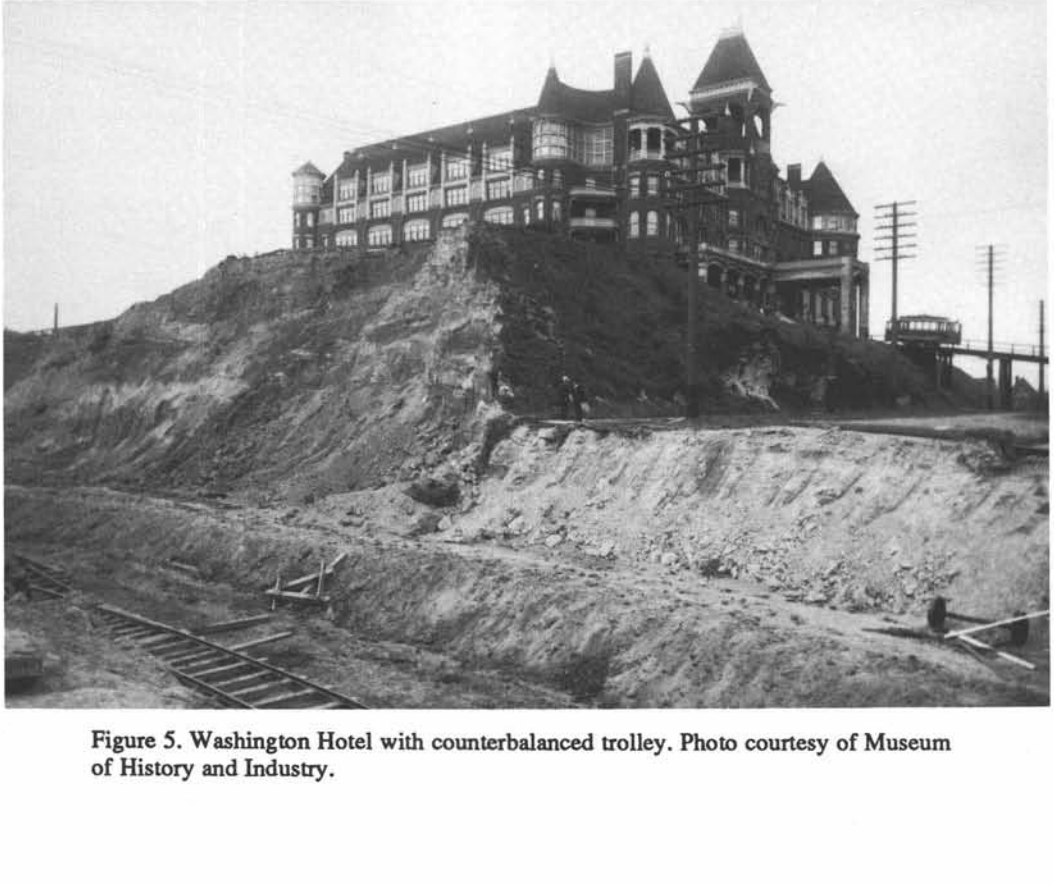


Figure 5. Washington Hotel with counterbalanced trolley. Photo courtesy of Museum of History and Industry.

interest and enthusiasm as has the regrade of Second Avenue from Pike Street to Denny Way." The contract was signed July 28, 1903, by C. J. Erickson and called for cuts of 56.5 ft at Stewart Street and 52.3 ft at Lenora. Adjustments to the side streets and alleys were included. The contract totaled $318,314.54, of which $81,038 was assessed against the property owners in the LID.

Soon other contracts were awarded. In all, this early part of the regrading on the west side of Denny Hill covered 21 blocks and involved 2.5 mi of streets. The cut at Fourth and Blanchard was 107 ft (Figure 6); at Fifth and Blanchard it was 93 ft. With the street grades being changed so drastically, the private property adjoining needed adjusting to fit. Most of the owners made their contracts directly with the contractor, as the city had no responsibility for privately owned property. Some owners, however, did not cooperate, and their lots became isolated pinnacles 50 and 60 ft high as the sluicing went on around them (Figure 7).

H. W. Hawley's private contract with I. A. Moore for Stewart Street from Second Avenue to Fourth Avenue was started in 1905 using steam shovels. This excavation and earthmoving method changed in 1906 with the introduction of hydraulic "giants" to attack the hills, using surplus water from the city water system. This method is illustrated in Figures 6 and 7. That water supply proved inadequate, however, and "the largest pump in the northwest" was installed on Elliott Avenue at Stewart and Virginia, drawing its water from the limitless source of salt water in Elliott Bay. It was powered by an 800-hp motor connected by belt to a four-stage turbine and was guaranteed to deliver 3,500 gal/min against 180 psi at the pump. This was about 5 million gal each 24 hr. Being a relatively new method of excavation, difficulties were experienced, but the sluicing demonstrated the feasibility of the hydraulic method of moving the glacial deposits of clays, silts, sands, and gravel in the hills. Undercutting the embankments caused controlled slides to occur, breaking the deposits down so that the solids represented as much as 16 percent of the water flowing into the flumes or conduits. Cemented tills proved the most difficult to break down, but even these were handled successfully.

Regrading Westlake Avenue required filling of ravines which crossed the Westlake Avenue right-of-way. More than 32,397 cy of embankment was placed in this area by Hans Pederson, contractor. Farther east, the Fairview Avenue contract called for 85,294 cy of

Figure 6. Hydraulic boring into Denny Hill at Blanchard Street. Photo courtesy of Special Collections Division, University of Washington Libraries (negative no. 5021).

Figure 7. Denny Hill Regrade, 1906-1907. Photo courtesy of Museum of History and Industry, Seattle.

excavation by contractor Otteson & Jensen, the material to be placed in the nearby ravines. These two jobs started in 1907 and were completed in 1908.

Denny Hill still presented steep grades on the avenues east of Third Avenue, thus limiting business expansion along Fourth and Fifth Avenues. On August 4, 1908, Grant Smith and Company, subcontractor for Rainier Development Co., was awarded the contracts for regrading Fourth Avenue and North Fifth Avenue. Before this additional phase in the removal of Denny Hill was finished, several contractors were involved, the last being P. J. McHugh, who finished his portion of the work on October 31, 1911.

Much of the work in this Denny Hill regrade was accomplished using hydraulic sluicing methods. A third source of water became available when the Lake Union Shore pumping plant was installed, adding 18,140,000 gal/day to the sluicing operations.

The near-shore area of Elliott Bay seemed an obvious choice to city engineers for wasting the Denny Hill material, thus creating a new beach line. However, the United States Government required disposition of the earth in the deep waters of Elliott Bay, with a provision that a minimum depth of 40 ft below low tide be maintained. The result was that the points of discharge had to be continually shifted into deeper water. Piling driven into the previously sluiced fill supported the flumes, which fanned out to three sluiceways, of which two were in use while the third was being extended by the pile driver. By the end of 1910, the flume was approximately 1,200 ft beyond the outer harbor line; it extended a total length of more than 1,900 ft west of Western Avenue. Pilings of 125 ft length were required for driving into the fill, the water having been approximately 200 ft deep at this point.

Finding a material to line the bottom of the sluiceways and resist erosion of the moving sands and gravel presented a severe problem. Cast-iron blocks, sheet iron, and steel plates were tried from time to time by the several operators, but all were worn out in surprisingly short times. Finally, wooden blocks were placed end-grain up to the wear, and these gave the best performance, lasting about 2 months before being worn to the point that replacement was indicated.

These first Denny regrades removed many of the steep grades that had been serious obstacles to the horse-drawn drays. Roadways and streets around Lake Union, to the north and to the northwest, were opened for much easier travel. Lots near the base of Queen Anne Hill, which prior to the regrades were valued at $2,500, were, upon completion of the work, quoted at $15,000. Figure 8 shows conditions before and after the final regrading of Denny Hill.

JACKSON AND DEARBORN STREET REGRADES

Beacon Hill, originally called Hanford Hill, stretched along the tideflat waterfront and served very effectively as a barrier between the waterfront developments of Seattle and the Rainier Valley settlers. Beach (or River) Road, a puncheon facility, ran along the west base of the hill just above the tide line and for the most part of today's Airport Way, but it served only for communication with settlers to the south who lived along the Duwamish River.

To provide better access to the Lake Washington and Rainier Valley areas, R. H. Thomson and his assistant, J. C. Jeffry, first prepared plans and estimates for either a two-way tunnel or, alternatively, a 90-ft open cut at Ninth Avenue and Jackson to achieve acceptable grades. After several meetings and long discussions, the regrading plan was approved in spite of its greater cost, because of its greater benefit to a larger area.

By 1907 the Jackson Street plans had been developed to include an area of 125 acres, or 56 city blocks, involving 5.75 mi of streets. Jackson Street grade was to be reduced from 15.6 percent to 5.04 percent with the greatest cut, at Ninth Avenue, to be 85 ft, and the street widened from 65 ft to 96 ft. Streets north of Jackson were to be left steep, while the streets south were adjusted to lesser maximum grades, 7.25 percent on Weller, 2.9 percent on Fourth, and 4.5 percent on Sixth Avenue. Of the 56 blocks, 27 were to be excavated and 29 filled.

Contract for the Jackson Street Regrade was awarded April 23, 1907, to a new firm, Lewis and Wiley, Inc. William H. Lewis, a lawyer from the East who had recently acquired some inaccessible property in Rainier Valley, had joined with financier Clifford Wiley to form the company. For their superintendent, they hired young William Chester Morse, who was experienced in the hydraulic sluicing operations. Morse was a civil engineering graduate of the International Correspondence Schools and was destined to become Seattle City Engineer in 1927.

While the Jackson Street Regrade contract was proceeding, plans for the companion changes to the Dearborn Street area were completed, and the contract for this work was also awarded to Lewis and Wiley, Inc., on September 9, 1909. The timing was propitious,

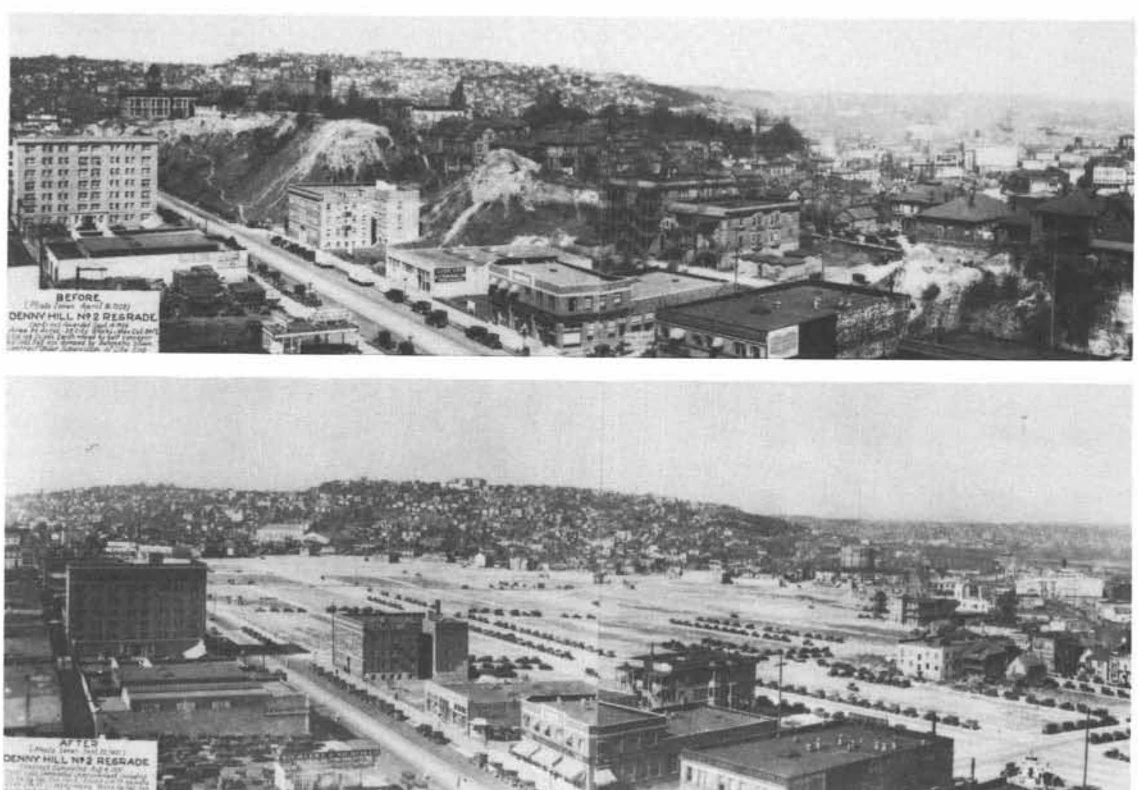


Figure 8. Denny Hill Regrade No. 2: before (April 16, 1928) and after (August 3, 1931). Photo courtesy Special Collections Division, University of Washington Libraries (negative no. 20046).

allowing most efficient use of the machines and equipment on both jobs.

A fresh-water supply by gravity from Beacon Hill Reservoir was used at first. This was supplemented later by 6,000,000 gal/day pumped by the City from the abandoned pumping station on Lake Washington at the foot of Holgate Street. The Lake Washington water was pumped into the Beacon Hill reservoir. This combined flow proved inadequate, so the contractors built a 31-ft x 37-ft frame pump house at the foot of Connecticut Street over salt water, care being taken to protect the intake lines from debris and the pumps from flooding at high tide.

In the hydraulic sluicing a monitor, or "giant", was used to remove sediments and flush them along the flumes. The "giant" is a very large nozzle, usually 7 to 10 ft long, mounted on an articulated base and with a controllable, movable tip 3 to 4 in. in diameter. The operator controlled the direction of the powerful jet stream, using a 3-ft-long handle connected to the movable tip. This required skill and constant attention. Only an experienced operator could keep the face of the excavation pit in proper "working condition" and thus insure that the water carried a capacity load of silt, sand, and gravel. The importance of skillful operation at all times was emphasized one day when Mr. Wiley asked an operator to let him, Wiley, handle the giant for a short time. Chester Morse, in charge of the entire operation, immediately fired the operator for breaking the rule: No amateurs allowed. The crestfallen Wiley quickly disappeared into the ever-present line of sidewalk superintendents (Morse, 1923).

Shear boards, carefully placed, directed the flow of water, carrying about 16 percent of solids, into the penstocks and thence into the flumes and pipes to the disposal sites, often 1,000 to 4,000 ft away. Heavy steel bars over the penstocks prevented large rocks and debris from entering these mud lines. Steel "Hopkirk's Patent Pipe" and wire-wound wood stave pipe of 10- to 30-in. diameters were used here as they had been on the Denny Hill jobs. Special end-grain wooden wearing blocks were placed in the invert (bottom) of the pipe where greatest wear occurred.

Occasionally a mud line would become clogged in spite of great care, and the practice was to send a wooden ball down the pipe. Floating down the pipe to the obstruction point, it reduced the area at the bottom of the pipe for the water to flow through, thus creating a higher velocity scouring action that usually cleared out the obstruction in a few minutes.

The tideflats north of Connecticut Street received the fill materials which accumulated to depths as great as 40 ft. Quoting from City Engineer Dimock's report (1912, p. 15),

> "...to make such a fill it was necessary to put large quantities of earth into the silt and water until by its weight the silt was squeezed out and a bottom formed. On this as a base, along outside slope lines, shear boards were placed. These consisted of two 1 in. x 12 in. boards held in place by 2 in. x 4 in. x 6-ft stakes. As water carrying dirt met these shear boards, it was deflected and the earth precipitated. In this way a fill of about 1-1/2 ft was made when a new line of slopes was determined and the process repeated until a 1.5H to 1V slope was built up step by step."

Other fills were made by the ponding process. The resulting fills were considered ideal, excellent compaction being achieved by the water settlement process.

A total of 3,348,000 cy of excavated material was moved during these contracts, including both the City portion and the private property parts. Certain areas of the tideflats were filled, at the direction of the City, to an elevation 12 ft below the final established grades, thus creating sites for future sanitary landfill operations.

With Dearborn Street graded, Beacon Hill was separated from its former partner hill to the north, Capital Hill. The steel bridge on 12th Avenue, 420 ft long and 110 ft above Dearborn Street, was built to correct this condition. It served as the only connection to downtown Seattle for this north section of Beacon Hill.

In 1895 the Seattle and Lake Washington Waterways Company received a permit to start cutting through Beacon Hill for a water connection from Lake Washington to the southern reaches of Elliott Bay. Material removed was to be deposited in the tideflats, and by 1900 many acres had been filled to desired grade above tide water. But gradually a more northern route became favored, and thus came the end of the effort to establish the southern connection to Lake Washington. The Lake Washington Ship Canal opened in 1916 when the Hiram M. Chittenden Locks were completed.

DENNY HILL REGRADE NO. 2

Removal of the western half of Denny Hill during the decades between 1890 and 1910 (First Denny Hill Regrade) satisfied the growing city until the late 1920s. Business advanced in the downtown areas and the rapidly increasing population pushed to the north. Traffic became increasingly bottle-necked by the obstacle presented by the remaining portion of Denny Hill.

The City Engineer was requested to furnish a report giving estimates on the cost of grading streets and alleys in removing the remaining eastern portion of Denny Hill. The necessary condemnation costs for rights-of-way, installation of water mains and sewers, paving of streets and alleys, and estimates for the grading of private properties affected were to be included. Costs for work on Dexter Avenue, even at that time considered an essential traffic artery along the east slopes of Queen Anne Hill, were to be included in the report.

The proposal created great public interest, and many meetings were held before the contract for Denny Hill Regrade No. 2 was finally awarded in 1928, to George Nelson and Co. Nelson immediately sublet most of the work to subcontractors.

The contractors chose to use mostly electrically powered shovels, loading excavated soils onto moving belts, for the regrade work. Shovels worked on benches, or layers, thus lowering the hill in stages. Each of the three fixed belts was 35 in. wide. Figure 9 shows one built on Battery Street from Sixth Avenue to Third Avenue; this connected with a second continuing from Third Avenue to Railroad Avenue and a third extending from Railroad Avenue to the discharge point on the dock where self-dumping scows were loaded. The elevated timber support structure for the conveyor belts was lined on both sides to protect pedestrians and traffic and to minimize the dust problem. The 500-cy scows were built with identical top and bottom so that, by flooding eccentric compartments, the scow would turn over and dump its full load in 5 min. Dumping valves were operated by a trip rope operated from the tugboat. Scows were 100 ft long, 36 ft wide, and 11-1/2 ft deep.

The three main belts totaled about a half mile in length and were moved at 600 ft/min by large electric motors. Six shorter belts, 30 in. wide and 250 ft long, were used to carry the dirt from the shovels to the main conveyor belts at a speed of 400 ft/min. An ingenious system of movable hoppers straddling the belts allowed the shovels to work along the belt route for some time before it was necessary to swing the belt into a new location. An automatic braking system was arranged so that stoppage of any belt would stop the others. This proved to be a good arrangement, as it prevented damage to the entire system on these occasions.

In searching the dusty archives of the City Engineer's office, the current author was unable to locate the original 1927 records, so the following is taken from page 681 of Clarence B. Bagley's 1929 History of King County:

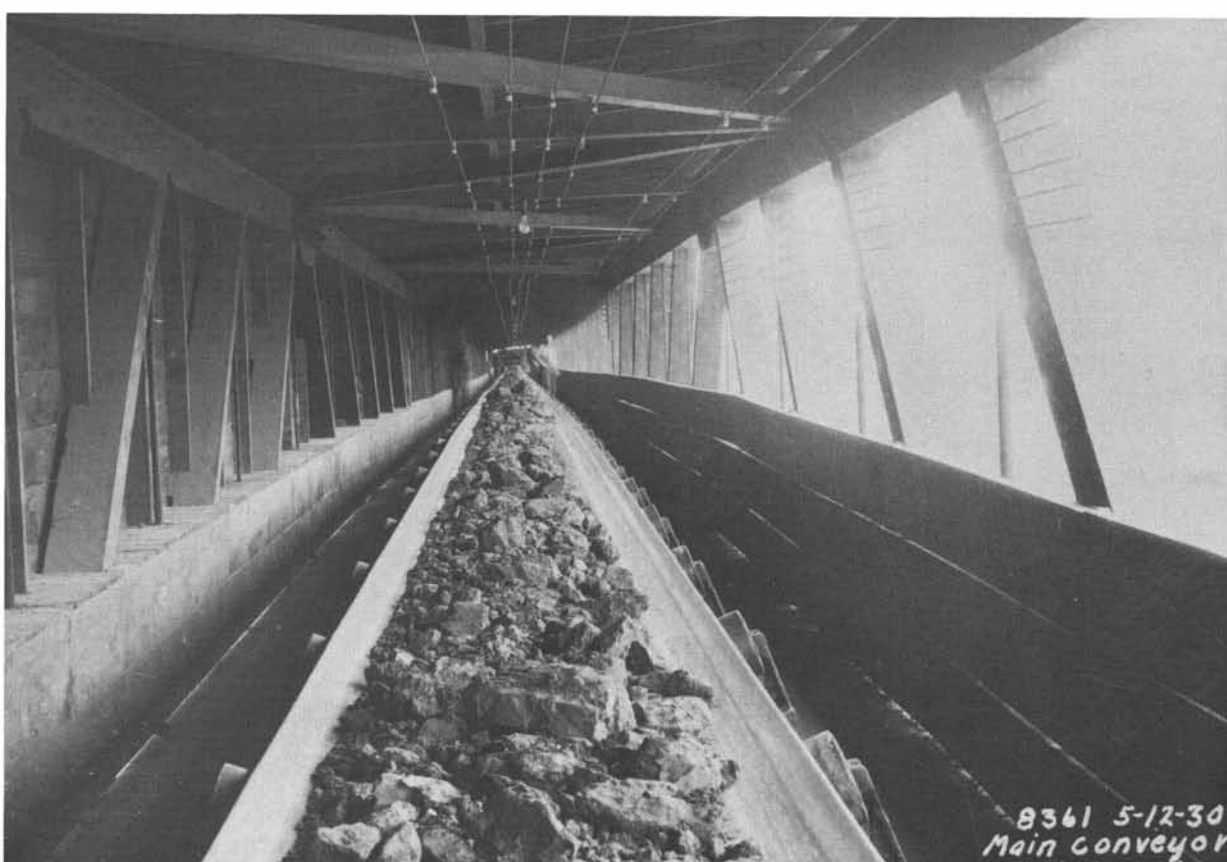


Figure 9. Denny Hill Regrade #2, 1930, main conveyor belt. Photo courtesy of Museum of History and Industry, Seattle.

"Official records in the city engineers' office show the total yardage moved up to 1-1-28 to be 37,085,370 cubic yards. As this figure does not include private property and utility contractor operations, the total yardage to date may be closer to 50,000,000, and this without the inclusion of either the Lake Washington Ship Canal or the Duwamish Commercial Waterway."

SUMMARY

Moving earth is always costly, even with modern equipment. But in reviewing these figures, perspective is improved when we realize that in those early years the work was done by laborers using pick and shovel to load the dirt into horse-drawn wagons or onto platforms below which the wagons were drawn. The one-horse Fresno scraper came into use around the turn of the century, but it was slow and destined to be discarded in favor of steam shovels and steam engines hauling small dump-cars on tracks to the dump sites. The hydraulic "giants" came next, and finally the latest system to be used was the electric shovels and moving belts which loaded their contents onto self-dumping scows for discharging into Elliott Bay deep water.

Seattle's regrades were monumental projects, transforming the cityscape as no other projects have since or are likely to in the future. The desired objectives of opening up the city for commerce and travel were realized as the result of far-sighted engineers and innovative construction practices.

In the decades since Denny Hill Regrade No. 2, the only contracts under which significant quantities of earth have been removed were incidental to highway and freeway traffic improvement projects funded by state and federal agencies. No regrade projects, as such, have occurred.

REFERENCES

Bagley, C. B., 1916, *History of Seattle*: S. J. Clarke Publishing Co., Vol. I, Chicago, IL, 453 p.

Bagley, C. B., 1929, *History of King County*: S. J. Clarke Publishing Co., Vol. I, Chicago, IL-Seattle, WA, 889 p.

Bass, S. F., 1937, *PigTail Days in Seattle*: Binford & Mort, Portland, OR, 191 p.

Beaton, Welford, 1914,, *The City That Made Itself*: Terminal Publishing Co., Seattle, WA, 275 p.

Dimock, A. H., 1912, *Annual Report, City Engineer, Seattle*: Seattle Engineering Department, Seattle, WA. [See also Phelps, 1978].

Morgan, M., 1951, *Skid Road*: The Viking Press, New York, NY, 280 p.

Morse, W. C., 1923, Personal communication, Seattle Engineering Department, Seattle, WA.

Morse, W. C., [March 31,] 1928, *Annual Report, City Engineer, Seattle*: Seattle Engineering Department, Seattle, WA, pp. 1-10.

Overstreet, R. M., [December] 1909, Seattle's regrades - a giant accomplishment: *The Argus*, Seattle, WA, pp. 46-51.

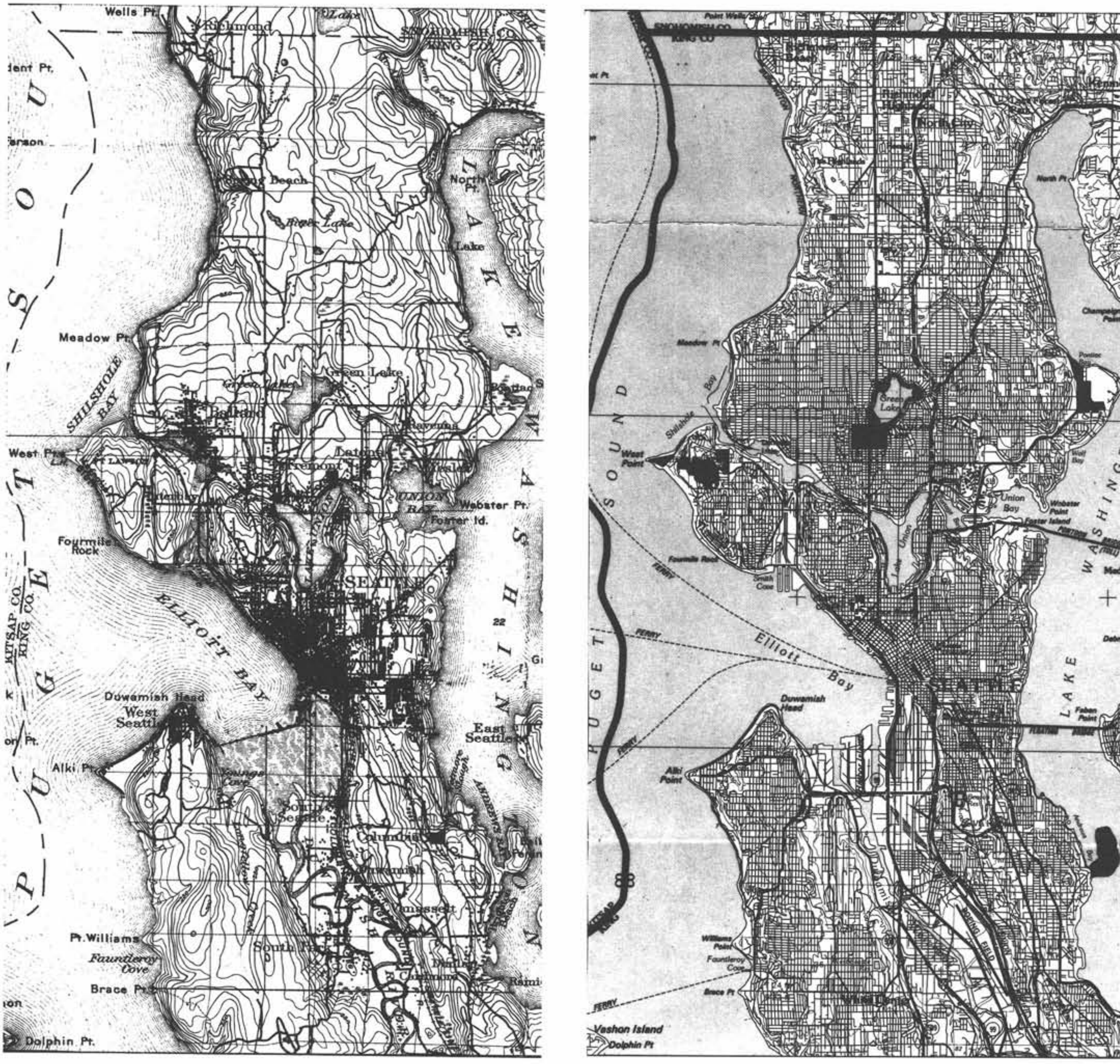
Pacific Building and Engineer, 1907, The Fourth Avenue regrade: *Pacific Building and Engineer*, November 2, 1907, Seattle, WA, Vol. 5, no. 44, p. 44.

Phelps, M. L., 1978, *Public Works in Seattle 1875-1975, A Narrative History*: Kingsport Press, Kingsport, TN, 304 p.

Seattle Daily Times, 1906, Contract signed for razing hotel: *Seattle Daily Times*, April 10, 1906, p. 4.

Seattle Engineering Department, 1910, *Regrading in Seattle, North District (Central Business District and Denny Regrade Area, et al.)*: Seattle Engineering Department Records Center, Seattle, WA, pp. 1-13.

Seattle Post Intelligencer, 1903, To lay low the hills of Second Avenue: *Seattle Post Intelligencer*, August 2, 1903, p. 44.



Maps of Seattle for 1897 (left) and 1975 (right) showing changes in shoreline configuration and urban development. From U.S. Geological Survey topographic maps, Snohomish quadrangle, scale 1:125,000 (1897) and Seattle quadrangle, scale 1:100,000 (1975).

Engineering Geology in Washington, Volume II
Washington Division of Geology and Earth Resources Bulletin 78

Coal Mine Subsidence at Renton, Washington

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INTRODUCTION

Subsidence due to underground coal mining in Renton has been known for many years, but it is only since the passage of the Surface Mine Control and Reclamation Act of 1978 (SMCRA) that funding has been available in the state of Washington for dealing with subsidence hazards. The city of Renton has a long mining history, and subsequent urbanization of lands overlying the mine workings has made that area one of prime concern for subsidence hazards. This paper discusses a subsidence that occurred beneath a house in Renton in 1986.

AREAL GEOLOGY

The coal-bearing rocks of the Renton area belong to the Renton Formation of middle and late Eocene age. The unit is composed of fine- to medium-grained feldspathic sandstone, siltstone, intraformational conglomerate, and coal [that was] deposited in an upper delta plain environment. The Renton Formation is disconformably overlain by the Blakeley Formation, a continental and nearshore to marine turbiditic sequence of Oligocene age. The Blakeley Formation, in turn, is unconformably overlain by Pleistocene glacial deposits of middle and late Wisconsin age. The Renton is underlain, with gradational contact, by a sequence of tuffs, laharic breccias and conglomerates, volcanic sandstones and siltstones, and bony coals that is referred to the middle and upper(?) Eocene Tukwila Formation. No older rocks are exposed in the Renton area.

Glacial deposits mantle the Tertiary section; therefore most of the geologic structure in the Renton area is known only from mapping in coal mines. The Tertiary rocks are folded into open anticlines and synclines whose axes trend about N60°W and plunge gently to the southeast. Dips are gentle except near faults. All significant faults exposed in the mines are steeply dipping normal faults that strike approximately east-west. The age of faulting and folding is not well constrained but is likely to be Oligocene (Walsh, 1986). Figure 1 shows the geology of the Renton area.

MINING HISTORY

Coal was first discovered in the Renton area in 1853 when Dr. M. Bigelow uncovered a coal seam while clearing his land (Bagley, 1929; Phillips and Walsh, 1981). Bigelow shipped small quantities of coal, but it was not until 1874 that a commercial mine was put into production (Watson, 1887). Initially, the Renton mine was opened by a drift from the Cedar River valley into Renton Hill along the Number 2 seam and by a slope on the Number 3 seam, which underlies Number 2 by about 75 ft (Figure 2), approximately a mile to the south. The two mines were later joined by a rock tunnel but were abandoned in 1886 (Evans, 1920). In 1895, the mine was reopened by the Renton Cooperative Coal Company, which accessed the mine by a slope entry on the west side of Renton Hill. In 1901, the mine was taken over by the Seattle Electric Company, which drove a rock tunnel from west of the present location of Interstate Highway 405 to the Renton Cooperative Coal Company's slope at the second level. The Seattle Electric Company worked the Number 3 seam on 12 levels before abandoning the mine in 1922. In 1923, William Strain, previously a mining engineer with the Seattle Electric Company, took over the property and robbed pillars (mined coal that had been left for permanent support) until 1933.

In a separate development, the Denny-Renton Clay and Coal Company opened a drift mine on the Number 1 seam, stratigraphically about 110 ft above the Number 2 seam. The Denny-Renton mine ceased operations in 1914, and it is shown on available mine maps as having had all pillars pulled.

No maps are available detailing Strain's activities, but the configuration of the mine workings on all three seams as of 1922 is recorded in Figures 3 and 4.

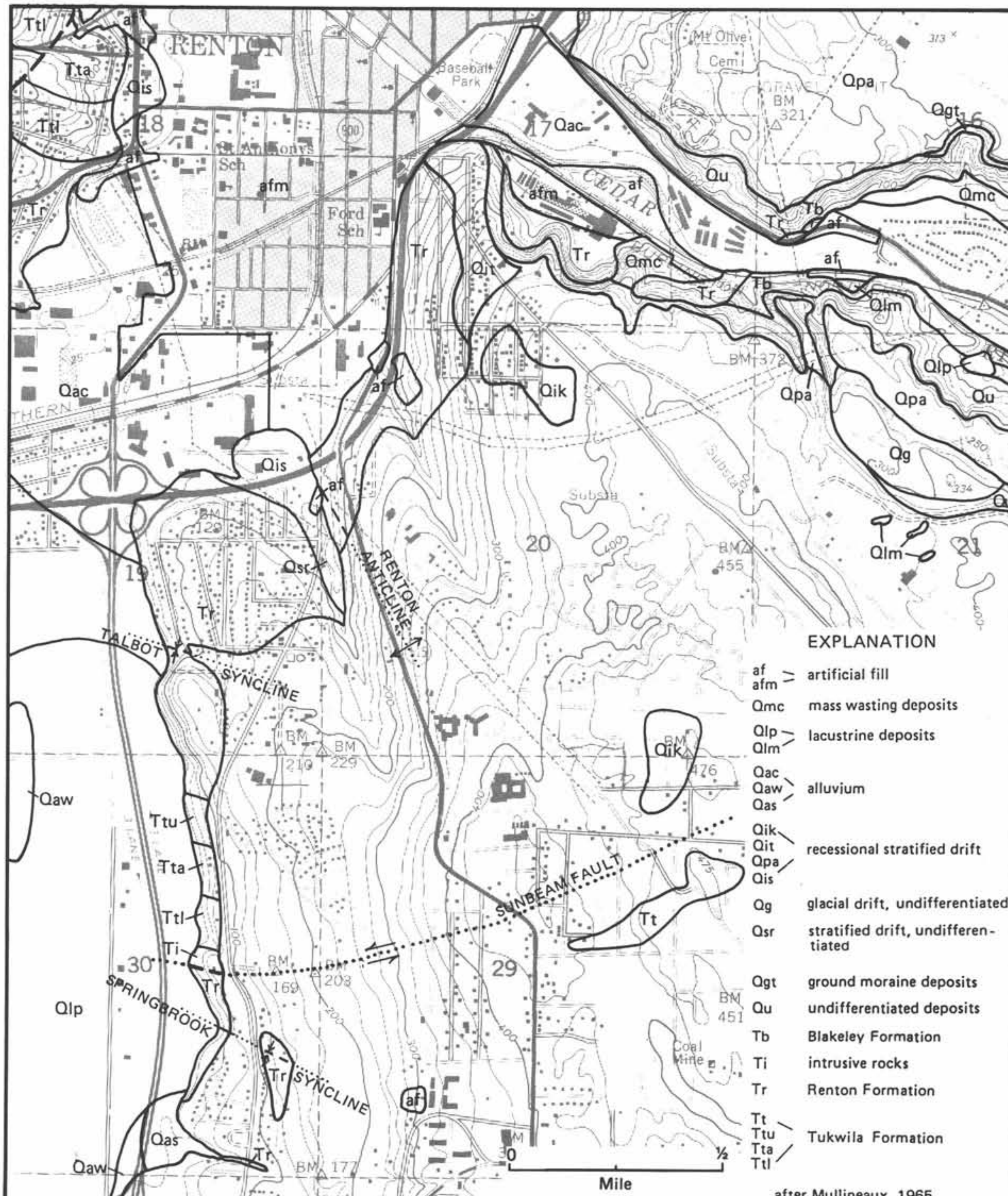

Figure 1. Geologic map of Renton area. Scale 1:24,000.

SUBSIDENCE HISTORY

Subsidence before 1982 in Renton is not well documented, in part because there was no governmental agency that systematically documented it, and in part because development above the mines is recent. In 1982, an air shaft into the Denny-Renton opened up on Renton Hill, probably as a result of the failure of a previous attempt to fill the shaft. The homeowner on whose property the subsidence occurred filled the hole, but this attempt to fill the shaft failed within the year. The Office of Surface Mining Reclamation and Enforcement

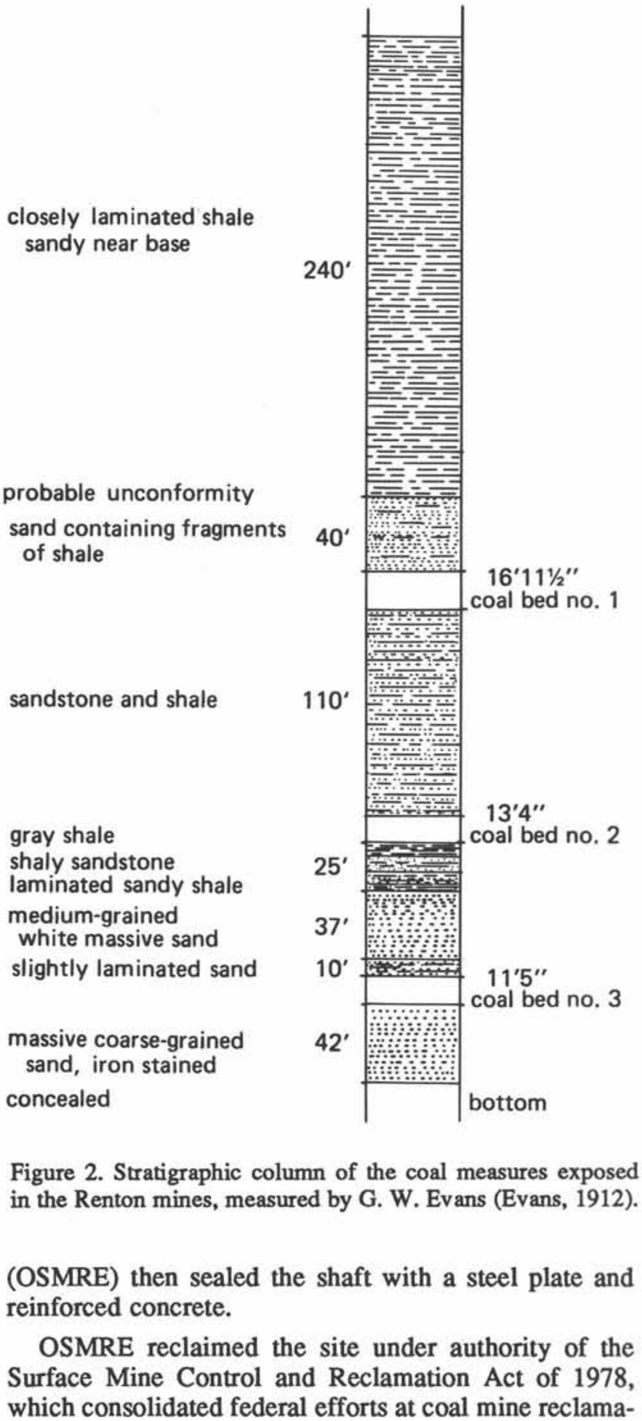

Figure 2. Stratigraphic column of the coal measures exposed in the Renton mines, measured by G. W. Evans (Evans, 1912).

(OSMRE) then sealed the shaft with a steel plate and reinforced concrete.

OSMRE reclaimed the site under authority of the Surface Mine Control and Reclamation Act of 1978, which consolidated federal efforts at coal mine reclamation and created a system of cooperation with states. Funding was set up through a tax on active coal mining, part of which is dedicated to the inventory and amelioration of coal mine hazards. Emergency reclamation responses were provided for, either by states or by OSMRE cooperatively with states.

Within the next 2 yr, three suspected coal mine subsidences in Renton were reported to the Washington Division of Geology and Earth Resources. In 1984, OSMRE commissioned an investigation of these and other coal-mine-related problems in Renton (Morrison-Knudsen, 1985) as part of a national effort to inventory hazards resulting from abandoned coal mines. Only two additional areas of subsidence were found. The inventory, however, only identified existing subsidence. If any mine voids remain, there is a risk of future subsidence, the timing of which is not predictable (Kratzsch, 1986).

In March 1986, another subsidence was noticed by a homeowner who lived about 600 ft south of the subsidence on the Denny-Renton mine that has been reclaimed by OSMRE. The Washington Division of Geology and Earth Resources was notified and one of us (Walsh) investigated the site. A roughly circular area approximately 6 ft in diameter had subsided as much as 10 in., apparently during the preceding night (Figure 5). The most obvious damage had occurred on the concrete patio; subsidence had tilted the footing for the second-story deck 10° to the east (Figure 6). The ground immediately to the east of the patio had also subsided, endangering the stairway, and a tension crack 2 in. wide had opened (Figure 7). Cracking was also visible along the chimney.

The location of the subsidence was projected relative to the known coal mine workings and found to lie an estimated 290 ft above the third level north on the Number 3 seam and about 205 ft above the Number 2 seam. The top of the Number 1 seam projects to a depth of about 65 ft below the subsidence (Figures 3 and 4).

OSMRE was contacted, and the site was declared eligible for emergency reclamation. OSMRE retained the engineering firm of Hart Crowser, Inc. to further investigate the subsidence and develop recommendations for mitigation.

SUBSURFACE EXPLORATIONS AND OTHER INVESTIGATIONS

Subsurface explorations at the site consisted of five borings within and around the observed subsidence, a downhole geophysical survey, and hand excavations and probing to assess whether loose soils extended beyond the visible subsided area. Other investigations at the same time included further review of available records and collection of oral history, as well as detailed observations and monitoring of the structure.

The borings were accomplished in March 1986 by hollow-stem auger and rotary drilling techniques. At a location on the edge of the subsided area, the hollow-stem auger was advanced to a depth of about 38 feet through very loose to medium-dense sand and about 6 feet of coal. The auger met refusal at a depth of about 25 ft, in very dense weathered sandstone, at another boring only a few feet from the subsidence. Thereafter, rotary drilling techniques were used. The rotary drilling used a casing and tricone bit to advance through the soil overburden; this was followed by coring with an NQ

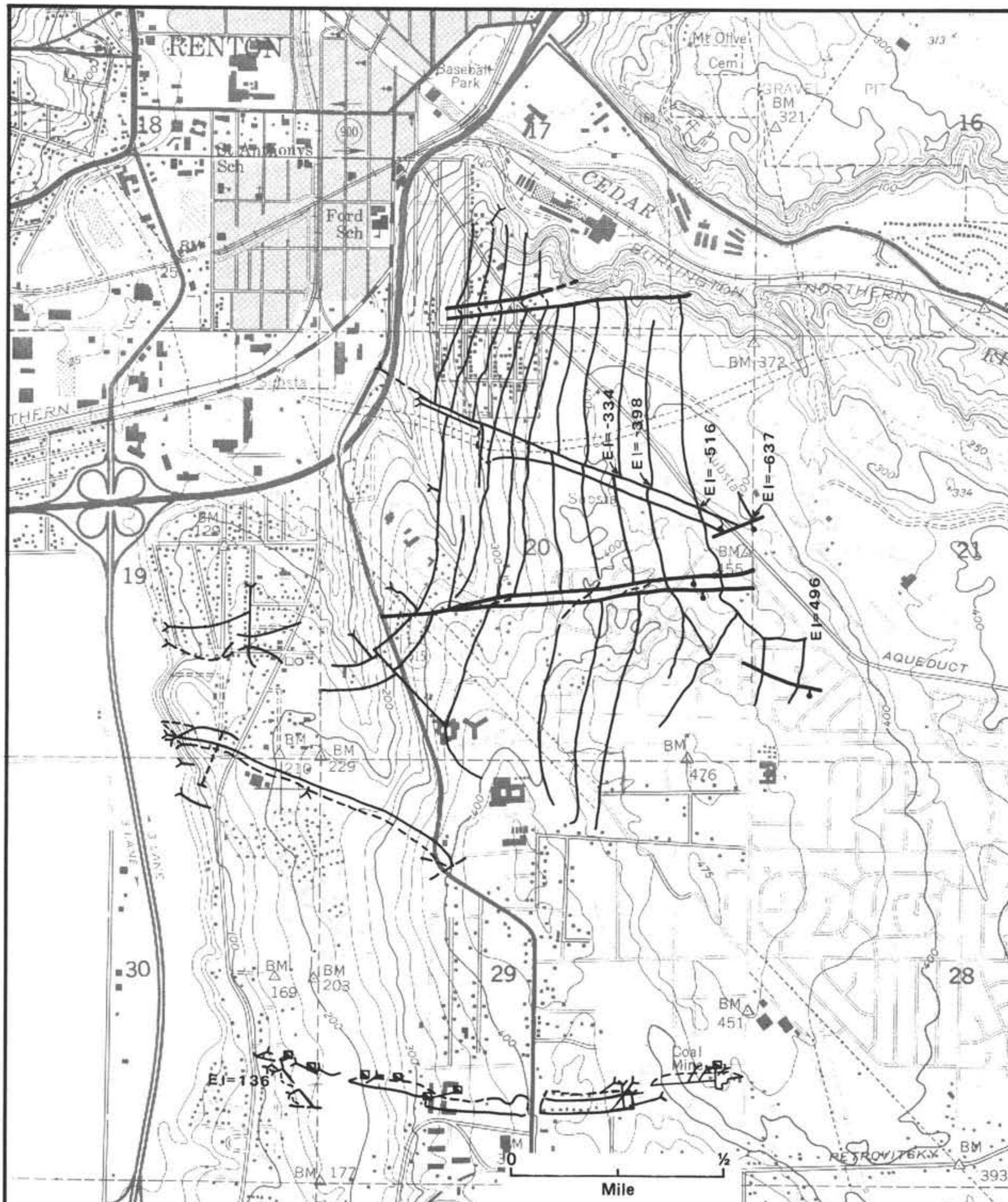

Figure 3. Mine workings on the Renton Number 3 seam. Solid lines are coal slopes and gangways; dashed lines are rock tunnels; heavy lines are faults, with bar and ball on the downthrown side. Elevations are relative to sea level. Compiled from mine maps on file at the Washington Division of Geology and Earth Resources. Scale 1:24,000; base from USGS Renton 7.5′ quadrangle.

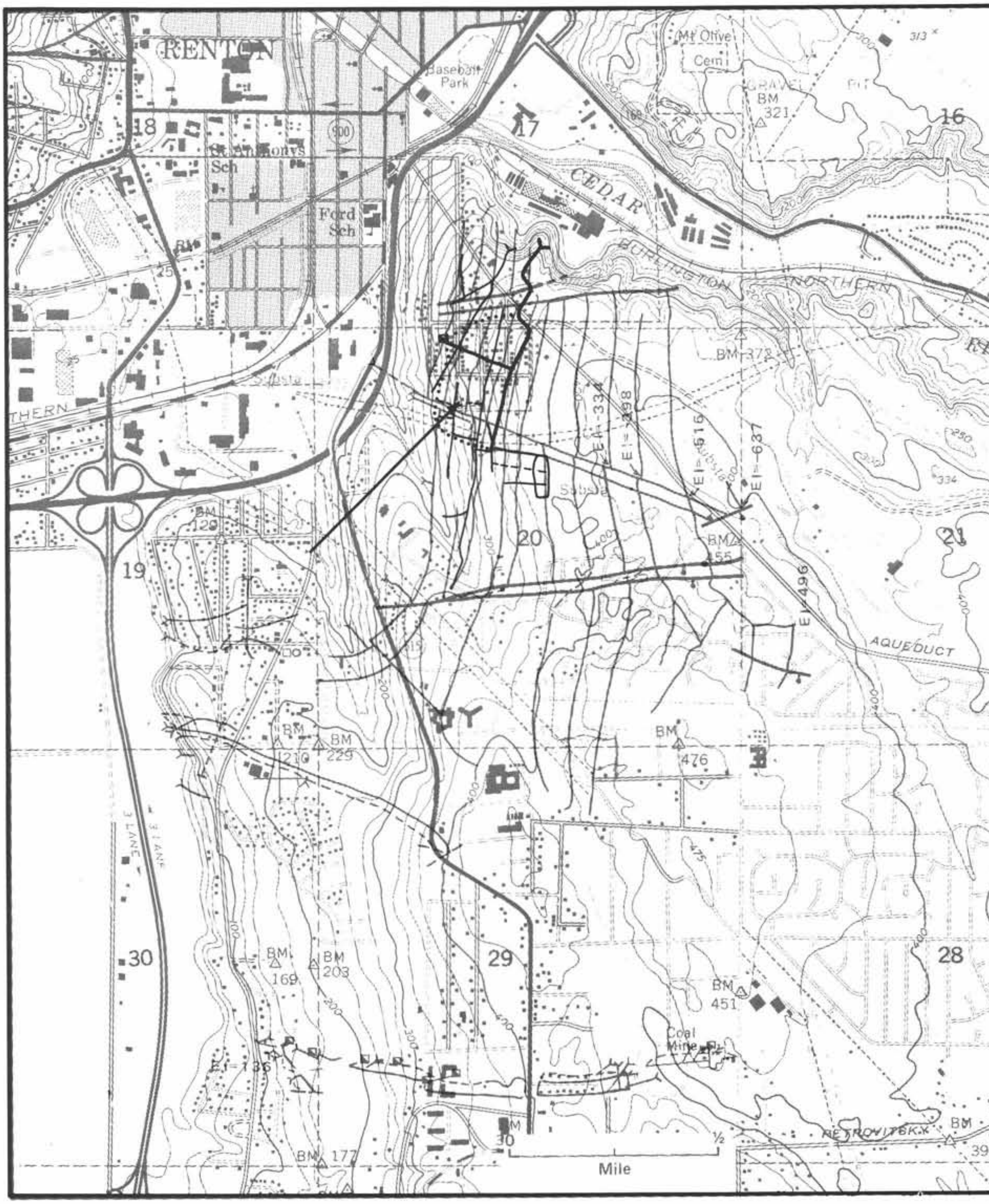

Figure 4. Mine workings on the Renton Number 2 and Renton Number 3 seams, located relative to Figure 3. Dotted line shows maximum extent of workings on the Number 1 seam. The subsidence discussed herein marked with arrow. Scale 1:24,000; base from USGS Renton 7.5′ quadrangle.

Figure 5. Subsidence on Renton Hill. Tension cracks are visible under staircase (arrow) and patio concrete is cracked. Diameter of the depression is about 6 ft.

Figure 6. Close-up of damage to patio. Surface slopes about 10° to the right. Note tilt of footing for deck support.

Figure 7. Close-up of tension crack (arrow) under staircase.

size double or triple tube core barrel. Standard Penetration Tests (ASTM D 1587) were accomplished in all the borings to assess density of the soil overburden. Details of the coring operation were varied in the field in an effort to improve both the total amount and relative length (as indicated by Rock Quality Designation) of recovered core.

The first four borings were located within the subsidence area, on the southwest edge of the subsided area, about 6 ft from the east side of the subsidence, and about 45 ft from the east side of the subsidence. The eastern boring locations were chosen because review of mine maps suggested that the mine workings extended irregularly along the strike of the seam to the north and south and downward to the east. Records of local mining typically show a pattern of mined areas (referred to variously as rooms, chutes, or breasts) extending up-dip from haulage tunnels (gangways) that roughly followed the strike of the seam. The subsidence was thought to have most likely occurred first in the upper part of the workings, where depth of cover is least. Subsequently, both solution of a three-point analysis and review of published data (Smith, 1911) indicated a strike more nearly N35°E (Figure 4). The fifth boring was consequently located by an area of foundation cracking on the west side of the house, which also was roughly up-dip of the subsided area considering the revised strike.

The three borings outside the subsided area passed through from 2 to 10 ft of dense to very dense fill soils overlying 20 to 35 ft of very dense sand (weathered sandstone) in turn overlying a sequence of relatively less weathered sandstone, coal, and shale. The depths to rock varied among borings. The two borings on the edge of and within the subsided area encountered about 20 ft of very loose to loose fill over about 20 ft of very loose to medium-dense sand, below which was sandstone, coal, and shale. In addition to the contrasting density within and adjacent to the subsided area, the subsidence correlated spatially with variations in the elevation of the top and appearance of the coal. It could not be determined whether the relatively loose fill and sand soils were initially dense soils loosened by subsidence, loose backfill of a shaft (or collapsed area), or the results of some combination of natural and manmade conditions. Drilling action was closely observed, but did not provide any positive indication of the presence of voids in any of the borings. A second coal seam was encountered about 54 ft below the first coal seam in the single boring which extended to such depth. The other borings were terminated in apparently undisturbed shale below the first seam.

Within the subsided area, coal was encountered several feet below where it would have been anticipated for a seam of uniform thickness, compared to the borings on either side. The elevation of the bottom of the coal was consistent among all the borings that penetrated the first seam. Core recovery was poorer and lost circulation (and blocking of the bit) was more of a problem within the coal zone in the subsided area. In addition, luster of the coal was duller and core fragments shorter and more irregular within the subsided area compared to fragments from other borings. These data were interpreted to indicate that the surficial subsidence actually corresponded to an area of mining that had subsequently collapsed.

Recognizing that the borings were an expensive and incomplete means of assessing the problem, other sources of information were also investigated. Mine maps and published reports were obtained from the Division of Geology and Earth Resources. Other sources of documents included the local building department, the University of Washington, and the Renton Historical Society. Aerial photography from 1936 was compared with current topography to assess whether there was subsidence beyond the limited area readily visible at the back of the house. Telephone interviews were conducted with the builder of the house and long-time area residents to obtain information about the structure and preconstruction conditions at the site.

Following completion of the borings, additional explorations included hand excavations to observe condition of the footing wall at several locations. The back yard was probed by hand with a steel rod, on a regular grid pattern, in an attempt to delineate any additional loose soils which could indicate potential settlement areas. Also, downhole gamma gamma and natural gamma logging were used to determine whether low density zones (i.e., voids) existed in areas of bedrock where core recovery was poor. The logging was considered to be generally unproductive, in that results were inconsistent and provided little additional information.

Additional data collection included a survey of cracks and damage both within and outside the house. Eighteen damage areas were identified and documented. As frequently occurs in such cases, the residents reported new observations as well as old recollections of damages as the investigation proceeded. Separation of cracks in the lower portion of the chimney, as well as inclination of the chimney, were monitored on a regular basis to determine whether movements were ongoing.

A structural engineer experienced in assessing damage to existing structures assisted in developing plans for remedial measures.

DIAGNOSIS

Review of the information obtained strongly suggested that the visible subsidence in the patio and chimney area was only the most recent manifestation of a problem that had been developing for some time. Most of the damage was concentrated in a limited area at the back of the house, suggesting a localized rather than an areal problem. Most of the observed distress to masonry and plaster appeared to be concentrated in the chimney and in the patio area. Additional damage to the house seemed to be related to distortion of the frame structure due to inadequate foundation support in the chimney area. The apparent cause was settlement of loosened soils within a very limited area that likely corresponded to an inadequately backfilled shaft, or possibly progressive caving from the mine workings towards the surface.

Several questions were not answered by the investigation; no voids were encountered, and no record was found of a shaft at this location. Drainage of the area was thought to play a role, but the effects on the subsidence were unclear. However, the preponderance of data obtained, as well as proximity to prior areas in which mine subsidence problems had been noted, qualified the site for treatment as an abandoned coal mine hazard, according to OSMRE.

SUBSIDENCE MITIGATION

Mitigation Methods Considered

Five methods were considered to mitigate adverse effects of the subsidence problem in the chimney area at the back of the house. The different methods were evaluated with respect to the effectiveness of avoiding additional progressive damage to the structure and eliminating risk of sudden, potentially catastrophic, toppling of the masonry chimney while recognizing that uncertainties would remain. The selected method had to be adaptable in order to benefit from information about the structure and subsurface conditions gained during mitigation work. Further ground movements and structural distress were anticipated to follow periods of increasing precipitation in the fall. Costs of protective and remedial measures were weighed against the cost of additional study. Replacement value of the structure was not explicitly evaluated. The five mitigation methods focused on providing support to the base of the chimney and/or slab and footing wall in the area of the apparent subsidence. In addition to gravity loading and differential vertical settlement, overturning and tension forces were considered. Options were categorized as either (1) improving the support capability of the ground or (2) improving the structure's ability to support itself. Temporary support of the structure during construction also had to be provided. The alternatives for improving the ground are briefly summarized below:

- Compaction grouting: This consists of injection of stiff portland cement grout to increase relative density of the loosened zone of soil and severely weathered bedrock by displacement. This technique has reportedly worked well in cases of subsidence related to poorly constructed soil fills and in soils overlying karst sinkholes and tunnels with caving ground conditions. However, relatively few contractors have experience with this technique, the size of the project was not likely to attract experienced contractors from outside the area, and there was risk of further structural damage.

- Slurry or chemical grouting: These approaches involve injection of a fluid cementitous grout into the loosened soil and severely weathered bedrock to increase load-carrying capacity of the soil by increasing its cohesion. More local contractors are familiar with this approach than with compaction grouting, and use of relatively lower grout pressures compared to compaction grouting would reduce risk to the structure. However, it is difficult to control actual location of grout flow and to obtain uniform penetration. Verifying the degree of soil improvement with this technique is problematic.

The methods considered for improving the structure's ability to support itself were:

- Load removal: This would have required structural modifications including removing the masonry chimney and fireplaces and installation of new framing, insulation, siding, and trim. The result would be to reduce load on the damaged structure and thereby reduce risk of further progressive damage. This would also eliminate the risk of the chimney toppling, but it would reduce the owner's beneficial use of the property. This work was mostly outside funding guidelines for the federal abandoned mine program.
- Underpinning: This would have involved construction of a special foundation element underneath the chimney and adjacent areas of the slab and footing wall, in order to provide increased support to the structure. The underpinning could have provided support by using augercast piles installed by drilling to sound bearing below as much as 40 to 60 ft of loose material or, alternatively, by using a large shallow footing to bridge the area of loose ground. These are relatively conventional construction techniques, but they would have required excavation under the damaged structure with shoring and bracing in close quarters. The cost of this approach was also significant.
- Addition of a grade beam: This approach focused on reinforcing the footing wall to increase support to the slab and chimney. The grade beam would also redistribute load on the footing wall by extending its support area to undisturbed soils and reducing the bearing pressure. The grade beam would resist overturning and further differential settlement of the chimney. The grade beam utilizes available support adjacent to the subsidence area, and it could be sized so that applied loads would cause little additional settlement even if further loosening of the ground around the subsidence area occurred. In addition, the grade beam was anticipated to increase resistance to tension on the bottom of the footing wall. Construction was anticipated to be relatively straightforward, with less need for shoring and risk of construction damages compared to complete underpinning.

Selected Approach

Based on construction cost, risk of further damage during construction, and potential for successfully preventing further settlement damages, the grade beam method was selected. Design was completed in a matter of a few weeks, and the necessary building permit was obtained on behalf of the owner. OSMRE determined that selection of a construction contractor would be accomplished on a competitive bid basis that was administered by OSMRE. The project was not formally advertised; instead, bids were solicited from contractors experienced in foundation construction and repair. A number of local contractors were contacted and informed of the project. Five indicated interest, but only one attended the initial pre-bid meeting, and no bids were subsequently received. Telephone follow-up indicated the contractors were hesitant to bid because of (1) uncertainty about conditions that would be encountered in the excavation, (2) the amount of paperwork required to meet OSMRE contract requirements, and (3) high risk of owner dissatisfaction compared to anticipated construction cost if the problem were not solved. A second round of contacting perspective contractors was initiated, and two bids were received. The contract was awarded in mid-November 1986, and work was completed within about 3 weeks (Figure 8).

CONCLUSION

Construction of the grade beam was successfully accomplished. Removal of the original patio slab exposed additional loose soil conditions extending west of the originally visible subsidence area, as well as additional tension cracks in the footing wall. Accordingly, design of the grade beam was modified slightly during construction. Original footing drains in the area were also replaced with a new tightline system leading to a dry well located well away from the house.

At this writing (1987), less than a year has passed since construction was completed. It is not clear whether further ground settlement in the area may occur. The following general conclusions may be drawn from this case history; however, final conclusions are pending:

(1) Mining subsidence damages are virtually impossible to predict, and once they occur it is difficult to identify the specific cause or mechanism.

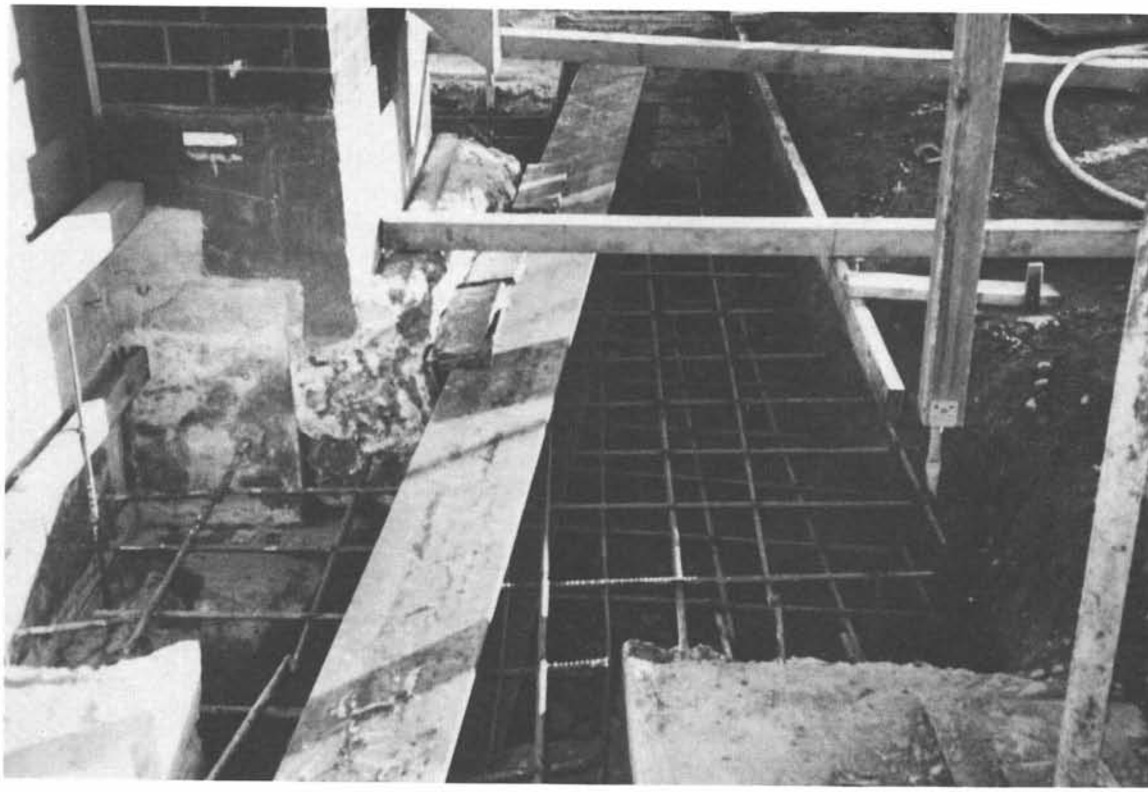

Figure 8. Grade beam during construction.

(2) Cost of investigation and mitigation is great, typically beyond the means of a homeowner.

(3) An array of investigative techniques is available and should be considered. No one technique will necessarily provide information on the source or mechanism of subsidence, and various methods may give conflicting results.

REFERENCES

American Society for Testing and Materials (ASTM), 1987, *D 1586 Penetration Test and Split-Barrel Sampling of Soils*: Annual Book of Standards Volume 04.08, Philadelphia, PA, pp. 296-301.

Bagley, C. B., 1929, *History of King County, Washington*: S. J. Clarke Publishing Co., Vol. 1, Seattle, WA, 889 p.

Evans, G. W., 1912, *The Coal Fields of King County*: Washington Geological Survey Bulletin 3, Olympia, WA, 247 p.

Evans, G. W., 1920, *A Report on the Property of the Renton Coal Company, Renton, Washington*: Unpublished report to the Puget Sound Traction, Light, and Power Co., 75 p. [Available for inspection at Washington Division of Geology and Earth Resources Library]

Kratzsch, H., 1986, Mining subsidence engineering: *Environmental Geology and Water Sciences*, Vol. 8, No. 3, pp. 133-136.

Morrison-Knudsen Company, Inc., 1985, *Engineering Investigation for the Renton Area, Washington*: Prepared for the U.S. Office of Surface Mining, Denver, CO, 64 p.

Mullineaux, D. R., 1965, *Geologic Map of the Renton Quadrangle, King County, Washington*: U.S. Geological Survey Geologic Quadrangle Map GQ-405, 1 sheet, scale 1:24,000.

Phillips, W. M. and Walsh, T. J., 1981, Coal geology of King County, Washington: *Washington Geologic Newsletter*, Vol. 9, No. 2, pp. 1-11.

Smith, E. E., 1911, *Coals of the state of Washington*: U.S. Geological Survey Bulletin 474, 206 p.

Walsh, T. J., 1986, *Age of deformation along the Olympic-Wallowa Lineament near Seattle, Washington* [abstract]: Geological Society of America, Cordilleran Section, 82nd Annual Meeting, Abstracts with Programs, Vol. 18, No. 2, p. 195.

Watson, J. H., 1887, *Report of the Inspector of Coal Mines and Ventilation of Washington Territory*: By Authority [of the Territorial Printer], Olympia, WA, 27 p.

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Washington Division of Geology and Earth Resources Bulletin 78

Stabilization of a Roadway Slide by Drainage Installation and Light-Weight Fill

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Shannon & Wilson, Inc.

INTRODUCTION

A landslide destroyed the access roadway to six residences in Seattle, Washington, on January 12, 1980, after a period of heavy precipitation. The slide prevented vehicular access to the residences for approximately 3 yr. Remedial schemes for reconstruction of the city street were developed by others but not implemented because of a lack of funding and disputes concerning financial responsibility for the project. Initial cost estimates for construction of a proposed stabilization scheme involving a cantilever soldier pile wall approached $300,000. Because no municipal funds were available for construction, the affected homeowner group retained Shannon & Wilson, Inc. in May 1983 to evaluate the cause of the slide and to develop plans and specifications for a more cost-effective solution.

The landslide is located in one of the many slide-prone areas of Seattle and is adjacent to Puget Sound. Because of legal issues, the specific project site is not identified.

SITE CONDITIONS AND GEOLOGY

The roadway is on a relatively continuous bench at about midslope on a hillside overlooking Puget Sound. As shown on Figure 1, the landslide encroached onto the roadway, destroying most of the pavement except for about a 5- to 7-ft-wide intact area at the top of the slide scarp. The roadway in the slide area is bounded by a steep bluff to the east and a more gentle slope to the west. The east bluff is approximately 90 ft high and extends upward to a residential neighborhood. A 4-ft-high concrete panel retaining wall supported with steel rails provides local restraint to the bluff and also aids in keeping materials from sloughing onto the roadway. The area west of the road includes an undeveloped lot that slopes downward at grades of about 50 percent for approximately 100 ft, and then continues on a slope of about 80 percent to the Puget Sound shoreline.

The east bluff is composed of competent Vashon till standing nearly vertical in most locations. The till, consisting of gravelly, silty, clayey sand, is generally underlain by very dense sand (Esperance Sand Member of the Vashon Drift). The bench area and the more gentle slope west of the roadway are composed of fill and slide debris consisting of loose, silty sand and gravel. The bluff farther downslope along the shoreline consists of hard clay and silt of the Lawton Clay Member of the Vashon Drift.

Ground water seeped from the Esperance Sand at numerous locations onto the slope, including near the head of the scarp at the roadway. In addition, a 6-in.-diameter steel pipe carrying a flow of about 1 gpm drained onto the slope immediately west of the road. The upstream origin of the pipe was unknown. A typical subsurface profile of the area after the slide occurred is presented on Figure 2.

As described by Tubbs and Dunne (1977), the bench, which functions as the foundation for the road, was created by slope regression due to movement of ground water along the contact between the Esperance Sand and the Lawton clay and silt. Ground-water seepage at this contact caused the Esperance Sand to erode and slump, resulting in undercutting and subsequent instability of the overlying soils. The slumped material accumulated on the bench, creating a convenient location for development.

The history of the project area indicated that the disturbed soils on the bench overlying the Lawton clay and silt experienced slope movements prior to the 1980 slide. City of Seattle Engineering Department records showed that slides occurred near the project in 1930, 1950, and 1973. Although documentation is incomplete, it appears that these earth movements occurred during periods of wet weather. Remedial measures for these past slides included drainage installation and additional fill placement.

On the basis of results of the subsurface explorations and a history of the area, the slide appeared to have been caused by heavy ground-water seepage. The seepage saturated the existing slide debris that formed the foundation for the roadway embankment fill. Upon saturation, the foundation materials lost effective strength, causing a failure of the roadway embankment. From the existing soil exposures, the depth of slope movement

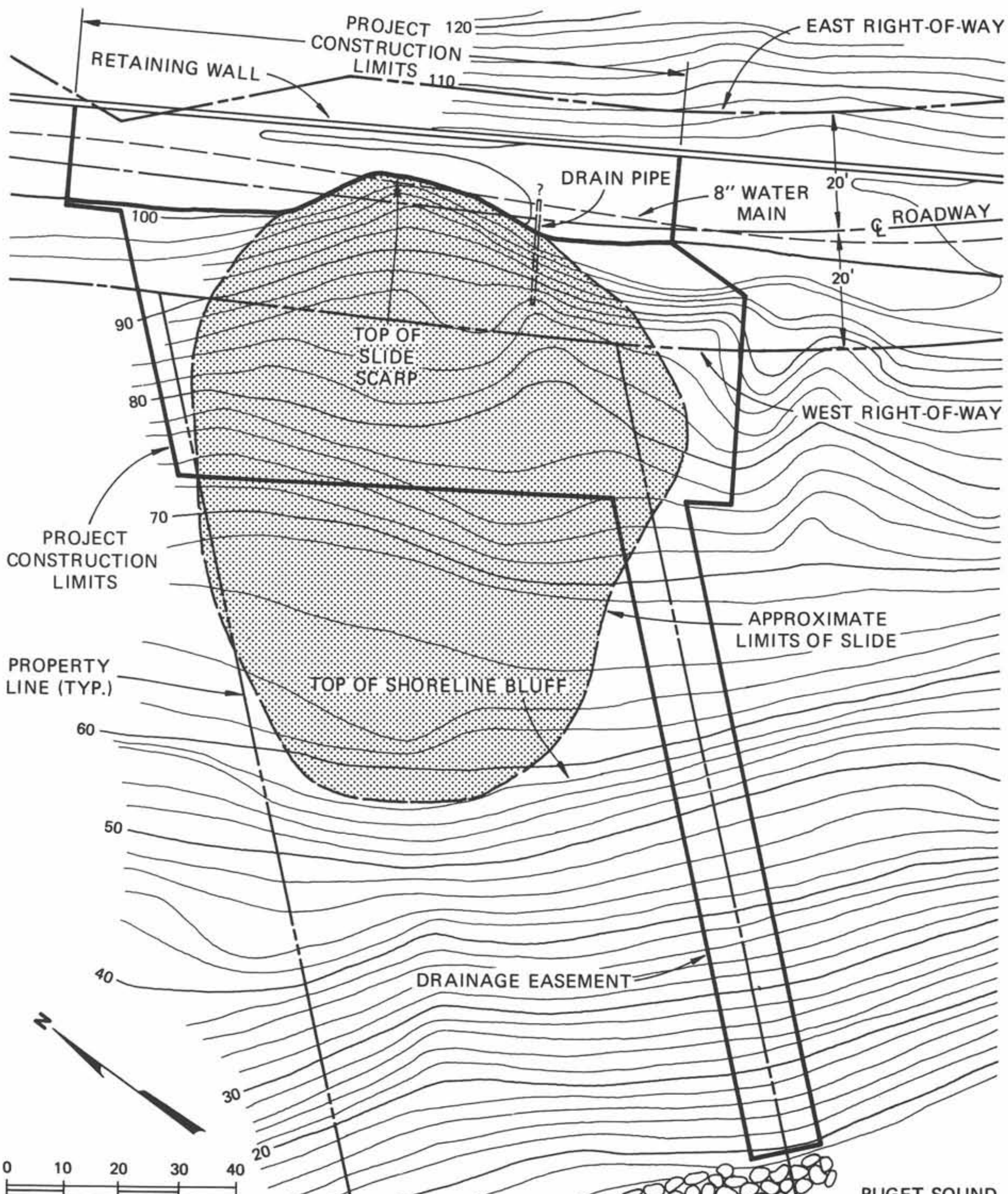

Figure 1. Site plan, including project construction limits and the top of the slide scarp.

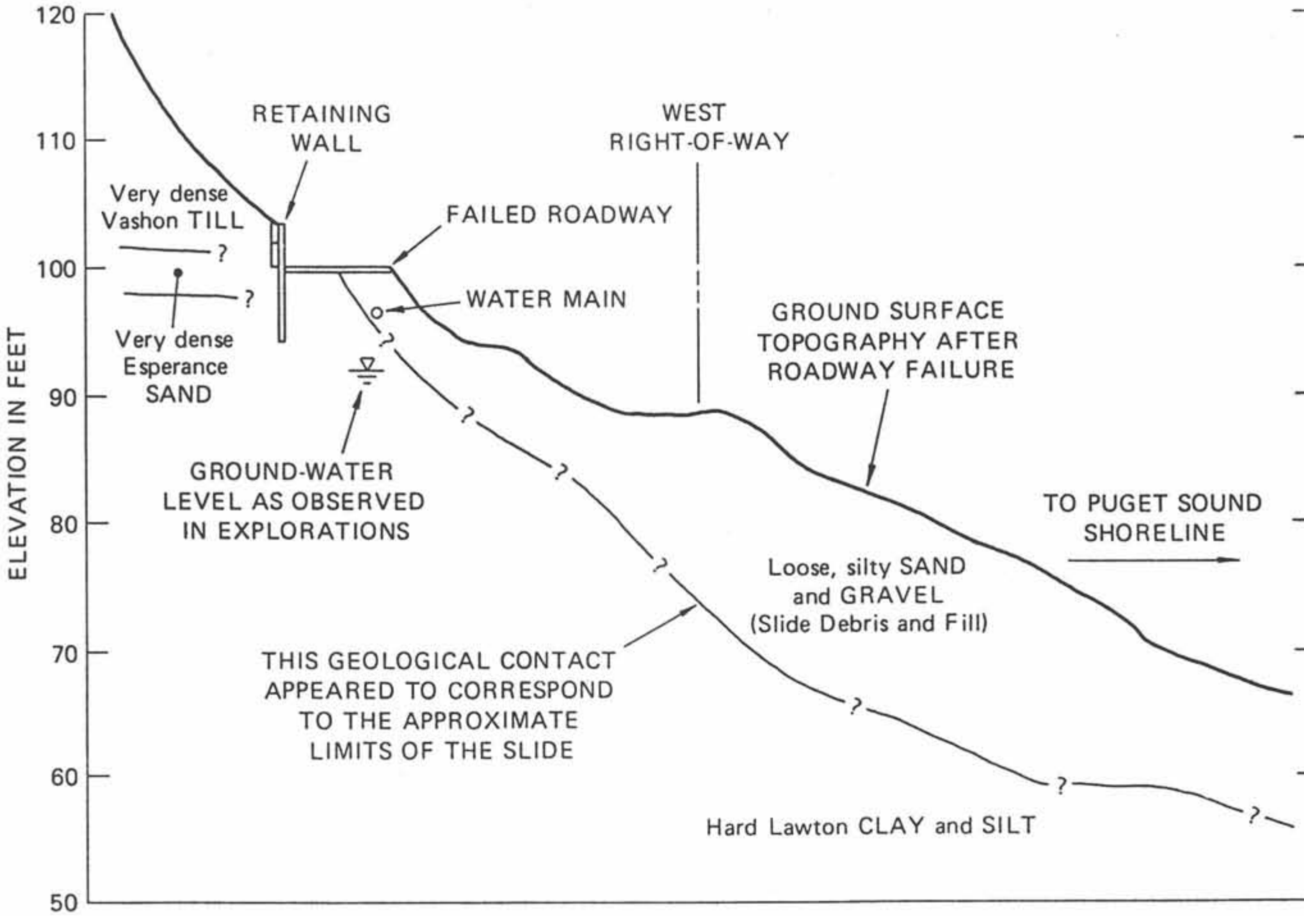

Figure 2. Typical subsurface profile after roadway failure.

appeared to be limited to the previously disturbed surficial soils overlying the undisturbed Lawton clay and silt. The approximate limits of the 1980 slide are shown on Figure 1.

DESIGN OF REMEDIAL MEASURES

At the request of the homeowners, Shannon & Wilson, Inc., performed preliminary analyses and cost estimates for several design schemes to restore the roadway. These studies were based on existing information of the slide area. It was understood that, upon determination of a preliminary workable design concept, additional explorations and engineering studies would be accomplished to further evaluate the initial assumptions. In addition, it was also understood that a suitable, less permanent scheme could be implemented at the site, provided the associated costs for the proposed fix were significantly less than the previously proposed permanent cantilevered soldier pile wall. The main criteria established for design of the remedial measures included:

(1) Safe reconstruction of the roadway without creating additional instabilities near the project;

(2) Maintaining the integrity of the existing water main located below the roadway;

(3) Performing required excavations and constructing permanent structures or drainage within city right-of-way boundaries and easements;

(4) Improving the stability of the undeveloped property located between the roadway and Puget Sound; and

(5) Providing an economical fix that could be funded by the Local Improvement District process and additional funds provided by the homeowners, based on property assessments.

Other remedial schemes that were considered included drainage installation, buttress fills, various retaining structures, and a timber trestle bridge. After an evaluation and comparison of other preliminary designs, it was concluded that a remedial scheme less costly than the cantilevered soldier pile wall was possible. Assuming that ground-water seepage contributed significantly to the slide, it was determined that the slope could be stabilized and the roadway could be reconstructed by removing portions of the slide mass, installing appropriate drainage systems, and reconstructing the roadway embankment with light-weight fill. From this preliminary design scheme, additional explorations and engineering studies were accomplished to verify the assumptions and develop a set of contract documents.

Subsurface Explorations and Conditions

In order to determine the limits of the disturbed materials, and to locate the contact between the slide debris and the competent native soils, additional subsurface explorations were accomplished. The explorations included two soil borings drilled by track-mounted equipment, two borings drilled by portable hand-operated equipment, and five backhoe test pits. In addition, two seismic refraction lines were run to establish the elevation of the intact native materials between the boring sites. These explorations supplemented four soil borings made by others for the design of the initial stabilization scheme (cantilever soldier pile wall).

The results of the explorations (Figure 2) indicate that the thickness of the loose slide debris materials varied from about 2 to 17 ft. The explorations also confirmed that the slide zone was limited to the previously disturbed surficial soils and that no major deep-seated movements had occurred.

In addition, the large volume of ground water observed at the top of the relatively impermeable Lawton clay and silt in several of the explorations indicated that the lack of appropriate drainage facilities probably contributed greatly to the slide. Water discharge in several of the test pits caused localized sloughing of pit sidewalls, which were composed of both slide debris and hard Lawton clay and silt.

Stabilization Scheme

On the basis of the subsurface conditions encountered in the explorations, it was determined that the stability of the slide area could be increased by installing a trench subdrain to intercept seepage along the top of the hard Lawton clay and silt. The subdrain would be installed parallel to the roadway and would drain into a tightline extending along a City of Seattle easement to Puget Sound. The roadway embankment could then be restored by filling the area with a light-weight fill composed of bottom ash material keyed into the competent native soil, as shown on Figure 3. Bottom ash is a sand-like, inorganic by-product of burning bituminous coal in steam generating power plants (Seals et al., 1972; Vas Srinivasam et al., 1977).

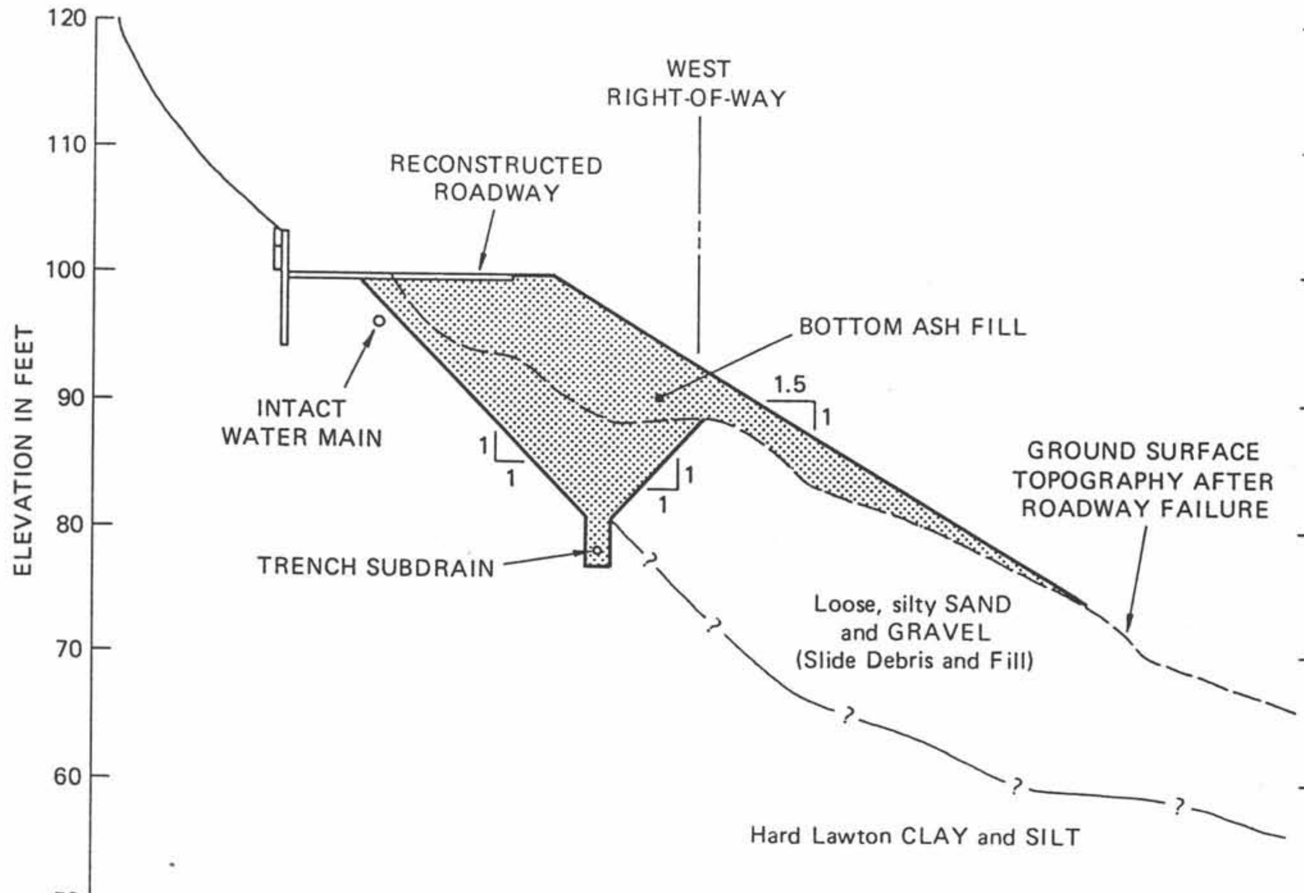

Figure 3. Typical subsurface profile after roadway reconstruction.

Stability analyses were performed in order to evaluate the effects of the drainage installation and light-weight fill placement. These studies were accomplished using the Modified Bishop's method of analysis. Typical soil strengths of the native materials were back-calculated, assuming that the factor of safety for the topography after the roadway failure was approximately equal to 1.0. The ground-water conditions used in the stability analyses were assumed to correspond to those determined by the exploration program. The stability of the slope reconstructed with drains and light-weight fill was then determined using these estimated strength parameters for the existing materials, the unit weight and strength parameters of the light-weight fill as determined by laboratory testing, and the assumed ground-water conditions upon drain installation.

The results of the stability studies indicated that the proposed restoration would result in an increase in site stability of approximately 10 to 20 percent, depending upon the effectiveness of the subdrain. The key to the proposed remedial scheme was the excellent drainage, frictional, and light-weight characteristics of the bottom ash. It was determined that the same roadway configuration would not be stable using sand and gravel. Although the drainage and frictional properties of well-graded sand and gravel are similar to those of the bottom ash, the greater unit weight of sand and gravel placed at the head of the slide would have destabilized the area.

Bottom Ash Fill Material

The bottom ash was selected for its light weight, excellent frictional properties, and good drainage characteristics. A supply of the bottom ash material was readily available at a coal-fired steam generating plant in Centralia, Washington, approximately 100 mi south of the project site. The grain-size distribution curve of typical bottom ash material used in the project (Figure 4) shows that the material is a poorly graded, slightly silty, gravelly sand. Its maximum dry unit weight (Figure 5) is approximately 50 to 60 percent of that determined for a similar naturally occurring sand and gravel deposit. Its light weight is derived from the porous texture that results from intense burning (Seals, et al., 1972).

The permeability of the ash when compacted to a dry unit weight of about 70 pcf (90 percent of its maximum dry density based on the Modified Proctor test) was determined by laboratory testing to be about 10^{-1} cm/sec. This value is approximately equivalent to that

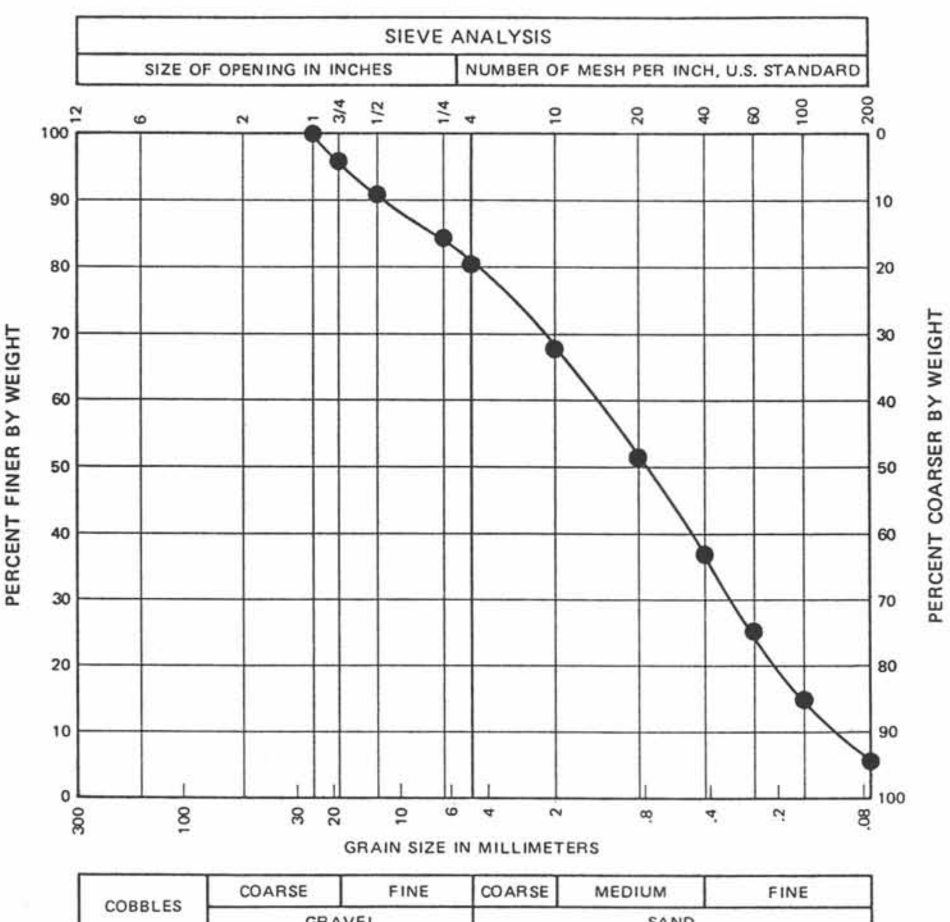

Figure 4. Grain-size distribution for bottom ash.

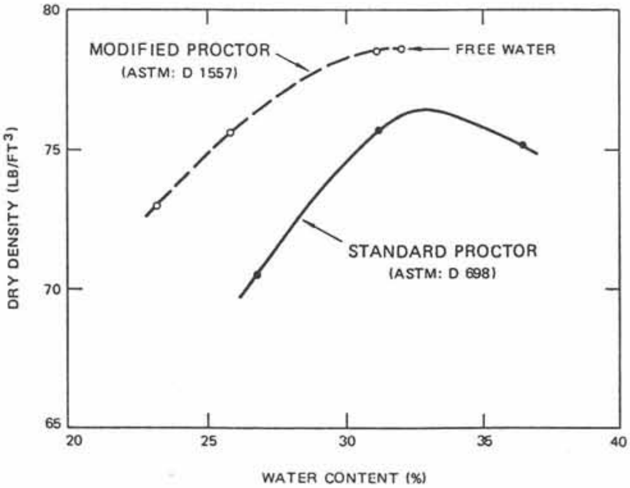

Figure 5. Compaction curves for bottom ash.

for the permeability of drainage sand and gravel or fine pea gravel as determined by Shannon & Wilson, Inc. laboratory tests. The frictional properties of the bottom ash were determined by consolidated-undrained triaxial compression tests with pore pressure measurements. The test results for a specimen compacted to a dry unit weight of 70 pcf indicated that its angle of internal friction varied between about 36° and 38° with a zero cohesion intercept. These frictional values appeared to be fairly high, but were actually less than the 38° to 42.5° values in the literature for various sources of bottom ash (Seals, et al., 1972). An internal angle of friction equivalent to 36° was used in the stability analyses made for the project.

PROJECT CONSTRUCTION

Shannon & Wilson, Inc. incorporated design recommendations into project plans and specifications. In addition, advertisement for bids, evaluation of submitted bids, interviews of prospective contractors, and selection of a contractor to perform the work were all accomplished by Shannon & Wilson, Inc.

Construction began on August 8, 1983. Shannon & Wilson, Inc. monitored construction progress and functioned as project manager. The bottom ash was obtained from the Centralia source. Forty-cy capacity trucks, under contract to the Seattle Metro sewage plant, delivered the bottom ash to a stockpile at the project site. These trucks hauled Metro sewage sludge to Centralia. Utilizing the returning trucks was arranged by the City of Seattle, thereby controlling project costs.

The contractor began installation of the subdrainage system from the Puget Sound shoreline, working upslope. The drain trench along the steep bluff was hand excavated. An unperforated pipe was nailed to the base of the excavation and the trench was subsequently backfilled with crushed rock and quarry spalls. Perforated subdrains were then installed parallel to the roadway and along the drainage easement by a large track-mounted backhoe. The pipes were bedded in pea gravel; the remaining backfill consisted of bottom ash. In general, the contractor was limited to short trench excavations and was required to backfill all excavations prior to completion of the work day. In this manner, the subdrainage system was successfully installed without extensive shoring or bracing systems. Upon completion of the drainage installation, the roadway embankment was reconstructed with bottom ash. The material was placed with a small dozer and was compacted with a towed vibratory roller.

Construction was essentially complete by September 1, 1983, in general accordance with the project plans and specifications. Approximately 1,400 cy of bottom ash were used in the embankment construction. The contract cost for construction totaled $63,160. The final result was a natural appearing, safe, traversable roadway.

PROJECT PERFORMANCE

Upon completion of the project, survey points were established along the west edge of the roadway to monitor settlement of the fill. In August 1987, the 23 surface settlement markers were monitored. None of the points exhibited settlement greater than about 1/4 to 1/2 in. In addition, the relatively steep bottom ash side slope appeared to be intact; no displacement or sloughing were observed.

This project was presented the 1984 Outstanding Local Civil Engineering Project Award by the Seattle Section of the American Society of Civil Engineers. It was selected because of its unique design, innovation, and cost-effective use of materials and resources.

ACKNOWLEDGMENTS

The cooperative efforts of the City of Seattle Department of Engineering, Country Boy Dozing (bottom ash supplier), O'Neill & Sons Trucking Company, and Kohl Excavating, Inc. (prime contractor) contributed to the successful completion of this project.

REFERENCES

City of Seattle Department of Engineering, Right-of-Way Division, Landslide Files.

Seals, R. K.; Moulton, L. K.; and Ruth, B. E., 1972, Bottom ash–An engineering material: *Journal of the Soil Mechanics and Foundations Division*, American Society of Civil Engineers, New York, NY, Vol. 98, No. SM4, pp. 311-325.

Tubbs, D. W. and Dunne, T., 1977, *Geologic hazards in Seattle–A Field Guide for the Geological Society of America Annual Meeting, 1977*: Geological Society of America, 37 p.

Vas Srinivasam, A. M.; Beckwith, G. W.; and Burke, H. H., 1977, Geotechnical investigations of power plant wastes. In *Geotechnical Practice for Disposal of Solid Waste Materials*: American Society of Civil Engineers, New York, NY, pp. 169-187.

Engineering Geology in Washington, Volume II
Washington Division of Geology and Earth Resources Bulletin 78

The East Whitney Hill Landslides

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INTRODUCTION

This study addresses two landslides that occurred within a 450-ft-long segment of 212th Way S.E. in King County, Washington (Figure 1). One of the landslides was a debris flow, the other a rotational landslide. A site plan showing the configuration of the two landslides is presented on Figure 2. Figure 3 is an aerial view of part of the two landslides prior to construction of remedial measures.

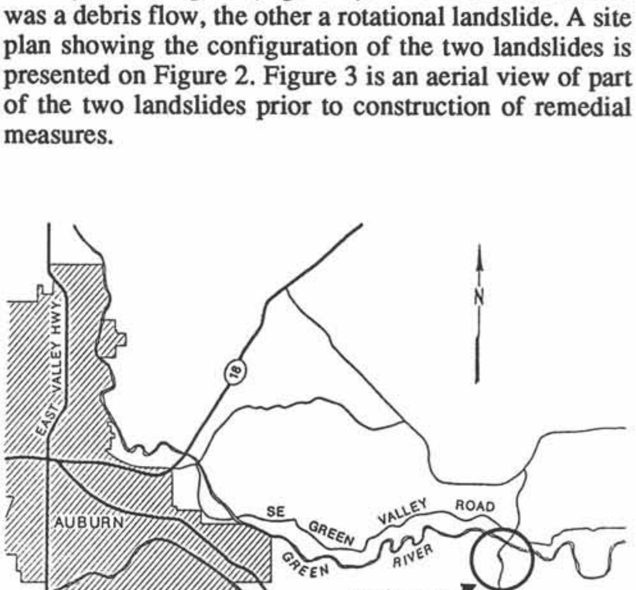

Figure 1. East Whitney Hill landslide area, King County.

The debris flow (Figure 4, cross section A-A′) occurred early in 1982 and resulted in closure of one lane of 212th Way S.E. The slide involved approximately 3,500 cy of material and reportedly occurred within a short time during a moderate rainfall. The debris flow extended into an undeveloped area and caused little damage other than the lane closure. No injuries or fatalities were reported as a result of the debris flow.

The history of the rotational slide is not as well documented; however, evidence suggests that pavement cracking and earth movement occurred prior to 1982. Aerial photographs taken at the time of road construction (late 1950s) show the road crossing the main body of the landslide. The rotational slide is approximately 280 ft wide; vertical offset near the center of the slide and upslope of the road is approximately 50 ft. Other than requiring an occasional pavement overlay, the rotational slide had not caused injury or damage.

After the 1982 debris flow, the King County Department of Public Works installed a new guardrail and traffic control signal and posted a 5-ton load limit. Reid, Middleton & Associates, Inc., a consulting engineering firm in Lynnwood, Washington, was retained by the County in early 1985 to provide design drawings and specifications for reconstruction of the roadway. Landau Associates, Inc. provided geotechnical services under a subconsultant agreement to Reid, Middleton & Associates, Inc.

PREVIOUS STUDIES

Preliminary geotechnical investigations were performed by the King County Department of Public Works (Bishop, 1982, 1983). These preliminary investigations determined that the debris flow was probably caused by hydrostatic pressure associated with groundwater and storm-water infiltration and that roadway damage in the vicinity of the rotational slide was associated with road shoulder failure (rather than attributable to large-scale movement). During a September 1985 field reconnaissance by Landau Associates, Inc., the full magnitude of the rotational slide was recognized.

REGIONAL GEOLOGIC SETTING

The project site is along the south wall of the Green River valley, approximately 5.5 mi east of the city of Auburn. Similar to most of the southern part of the Puget Lowland, the study area is within the glacial drift plain that extends from the Olympic Mountains to the foothills of the Cascade Range. The drift plain surface typically has rolling topography with 100 to 200 ft of relief.

The drift plain is not continuous, but is dissected into a number of individual plateaus, separated from each other by bedrock hills and ridges, river valleys, and the

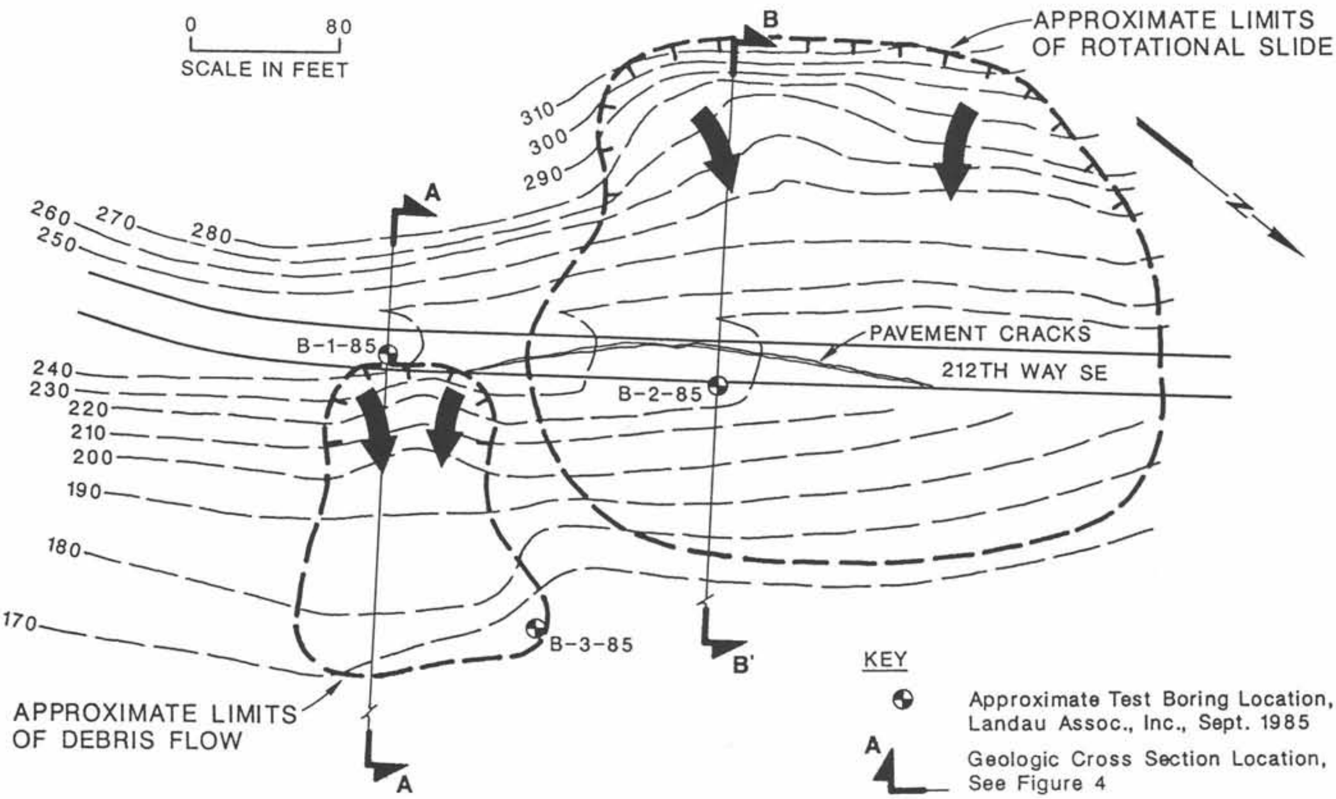

Figure 2. Site plan showing debris flow, rotational slide, and location of test borings and cross-sections.

waters of Puget Sound. Bedrock exposures are relatively few, although sedimentary rocks of early Tertiary age are exposed 1 to 2 mi east of 212th Way S.E. (Mullineaux, 1970).

The materials comprising the drift plain near the project site consist of an alternating series of glacial and interglacial deposits. The deposits vary widely in both texture and depositional history and include silty to clayey till, coarse stony till, outwash sand and gravel, and lacustrine sands, silts, and clays.

Four major glacial episodes spanning the past 500,000 to 2 million yr have been identified in the soil materials of the drift plain (Mullineaux, 1970; Blunt et al., 1987). These glaciations are (from youngest to oldest) the Fraser, Salmon Springs, Stuck, and Orting. Orting glacial sediments dominate the upland terrain in the study area.

The Green River runs through a valley that is approximately 1 mi wide at the project site. Coarse alluvial materials, capped in places by a veneer of flood deposits, comprise the valley bottom sediments. Erosion of the valley sidewalls by the Green River has produced oversteepening and landsliding at many locations. Based on a review of aerial photographs, the Green River once flowed along the toe of the slope at the site of the two landslides.

SITE GEOLOGY AND HYDROLOGY

Geologic and hydrologic conditions at the site were investigated by drilling three test borings, performing a geologic reconnaissance, reviewing available published geologic reports and maps, and studying aerial photographs. The borings were drilled using a truck-mounted, hollow-stem auger; soil samples were collected on about 5-ft intervals using a 3-1/4-in.-outside-diameter drive sampler. Figure 2 shows the locations of the borings.

Boring B-1-85 was drilled along the outside road shoulder above the headscarp of the debris flow. This boring extended to a depth of 80 ft, and three piezometers were installed in the borehole to monitor ground-water levels within known or potential water-bearing zones. Data from this boring indicated that near-surface soil at the debris flow consists of about 10 ft of roadway fill (a mixture of clay, silt, sand, and gravel), a thin zone of weathered till, a fine to medium sand horizon, and a basal sequence of interbedded clay, silt, and fine sand (Figure 4, cross-section A-A′). Of primary interest at this location was the water-bearing sand horizon encountered below the till.

Boring B-1-85 indicated that all the encountered soil units, with the exception of the roadway fill, were not disturbed by mass movement. Thus the debris flow was

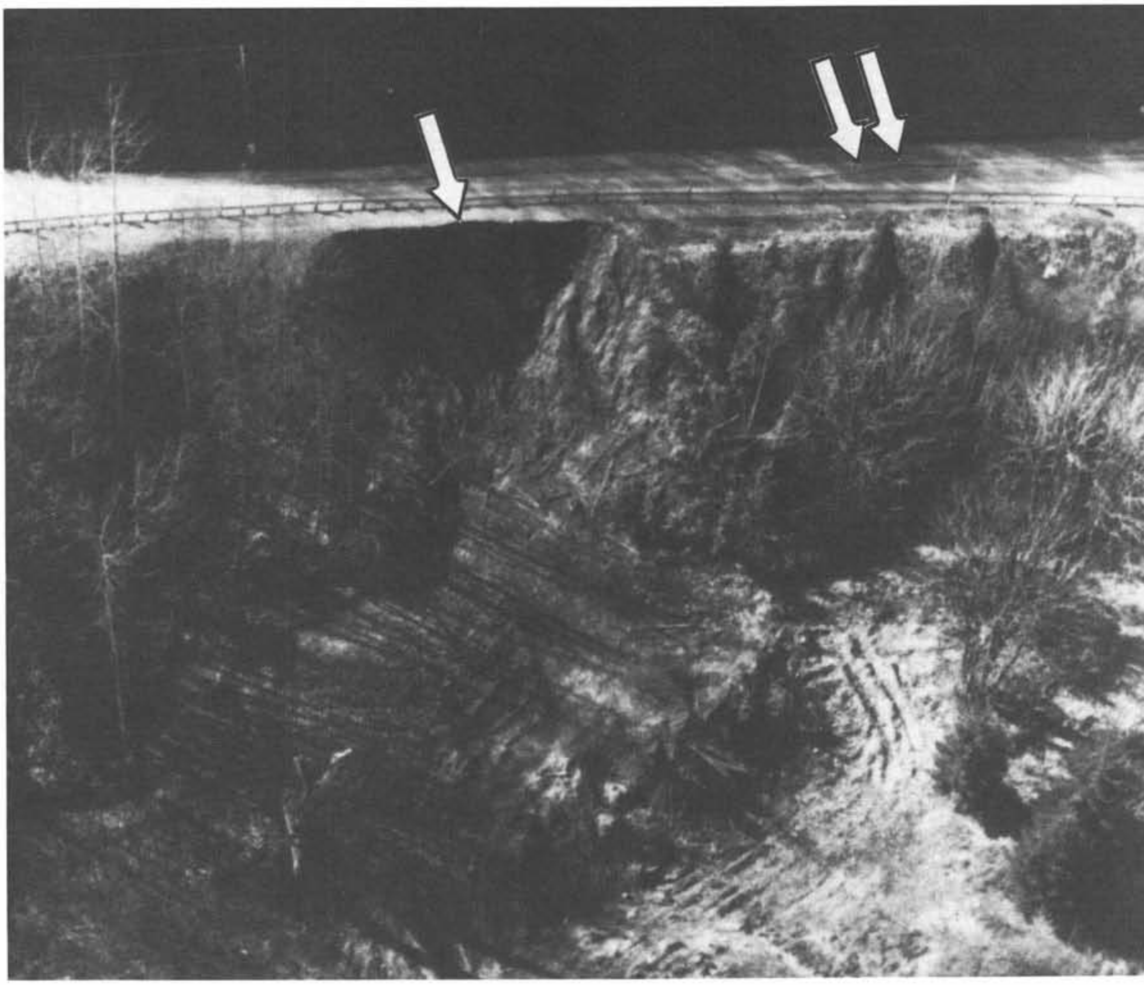

Figure 3. Aerial view of the East Whitney Hill site showing the debris flow (single arrow) and pavement cracking at the rotational slide (double arrow). Debris pile has been partially regraded to facilitate drainage. (Photograph by King County, March 1985.)

a relatively shallow failure, probably triggered by hydrostatic pressure within the above noted sand unit and by poorly controlled surface runoff.

Subsurface conditions at the rotational slide (Figure 4, cross section B-B′) were explored by a boring (B-2-85) along the outside road shoulder at the point of maximum pavement offset and a shallow boring (B-3-85) at the toe of the hillside. The boring drilled along the road shoulder extended to a depth of 61 ft and encountered soft and disturbed soil to a depth of 31 ft. Slickensided, but more competent, soil was encountered to a depth of 46 ft. Soil below 46 ft did not show evidence of movement. Three piezometers were installed in this borehole.

The boring drilled at the base of the slope penetrated 5 ft of the 1982 slide debris flow and approximately 4 ft of sand and gravel valley alluvium; it was terminated within hard undisturbed silt and clay at a depth of 19 ft. A single piezometer was installed in this borehole.

Based on the soil and ground-water information obtained from borings B-2-85 and B-3-85, it appeared that the basal slippage plane for the rotational slide was at or just above the elevation of the valley floor. Internal failure within the landslide mass was probably responsible for the greater disturbance noted within the upper 31 ft of boring B-2-85 and also probably for the arcuate pavement cracking on 212th Way S.E.

LABORATORY TESTING

Laboratory tests to determine physical soil properties were performed on selected samples obtained from the three borings. The testing included moisture content, density, grain-size determination, Atterberg Limit, direct shear, and unconfined compressive strength.

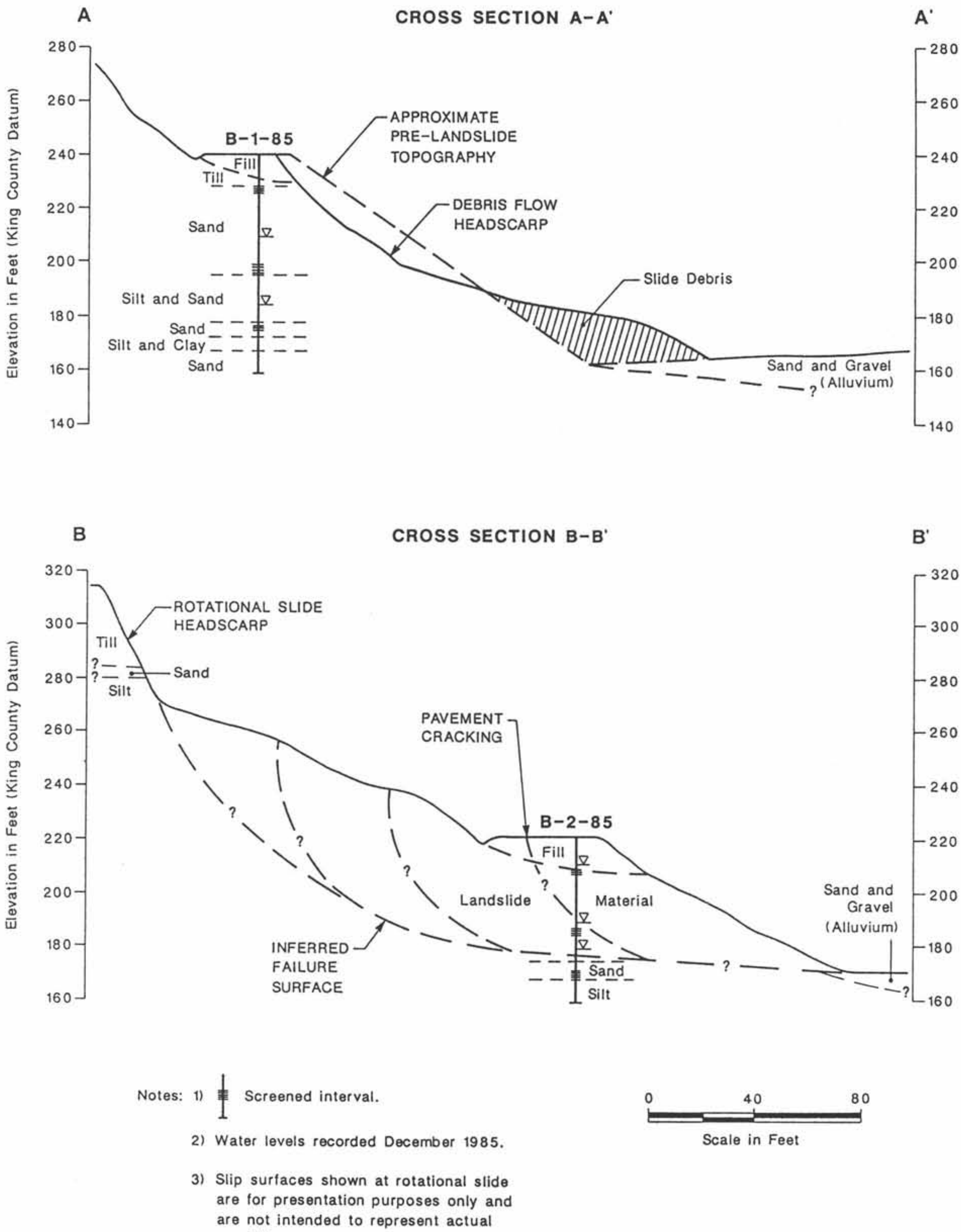

Figure 4. Geologic cross-sections A-A′ and B-B′ at the East Whitney Hill debris flow and rotational slide, respectively. Water levels were recorded in December 1985. Slip surfaces shown at the rotational slide are diagrammatic and are not intended to represent actual conditions.

EVALUATION OF STABILIZATION ALTERNATIVES

During the study, several possible remedial actions were evaluated with respect to technical feasibility and cost effectiveness. They included: (1) road relocation; (2) retaining walls; (3) horizontal and vertical drains; (4) light-weight roadway fill; and (5) a buttress fill. The fifth action was selected because it (a) addressed stabilization of both landslides; (b) was more cost effective than road relocation, retaining walls, and drains; (c) allowed continued use of the existing roadway during construction (required by King County); and (d) provided long-term protection with minimal maintenance. Availability of undeveloped land at the toe of the landslides and nearby gravel borrow sources contributed to the cost effectiveness of this action.

BUTTRESS DESIGN

Design of the buttress had to accommodate irregular and steep topography, heterogeneous soil types (including road fill and landslide debris), operational constraints (keeping the roadway open at all times), and equipment access limitations. Also, since surface and ground water contributed to the instability of the area, the design included drainage above the roadway and provided for an impervious lining of the inside ditch line. Figure 5 is a plan view of the buttress and drainage network. Figure 6 presents a representative cross section through the buttress.

Several analyses were performed to evaluate the stability effect of the buttress and determine its optimum configuration. The Modified Bishop method was used for both static and dynamic stability analyses using soil strength parameters derived from soil boring information, laboratory test results, and assumed earthquake-induced ground accelerations.

The buttress built from the final design was approximately 580 ft long, 200 ft in maximum width, and varied from 35 to 80 ft high. A 5-ft-thick (minimum) layer of select granular borrow and two longitudinal underdrains provided drainage beneath the buttress. Approximately 61,000 cy of select and general borrow materials were used for buttress construction.

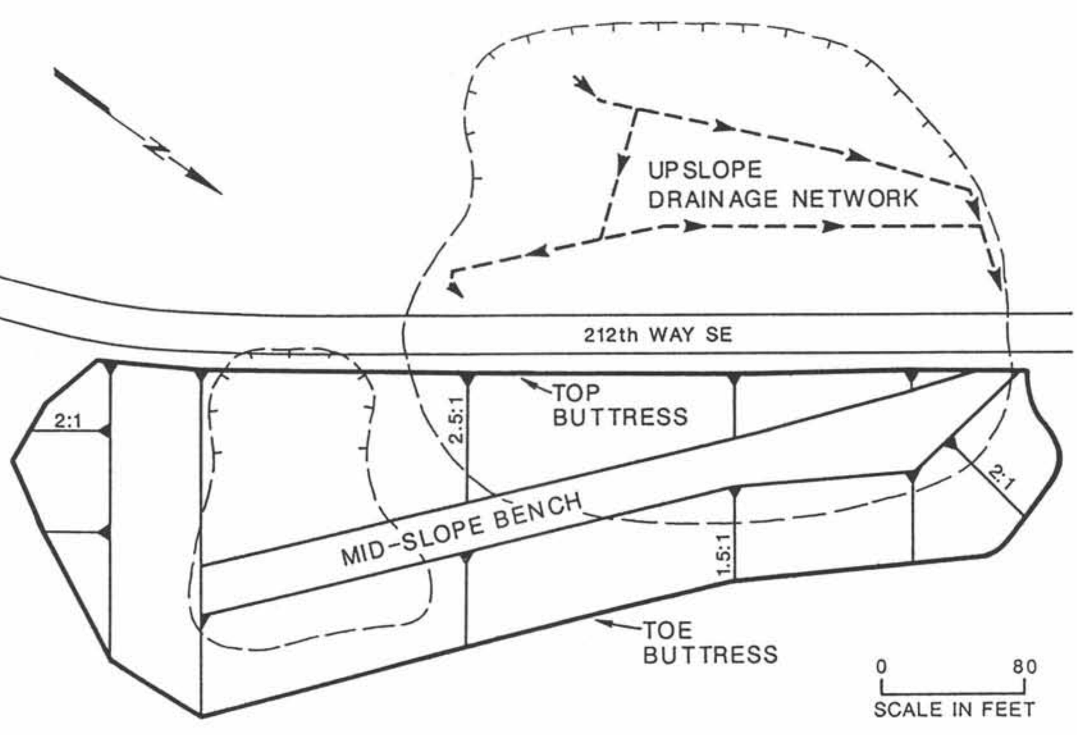

Figure 5. Buttress and drainage network configuration for remediation of the East Whitney Hill landslides.

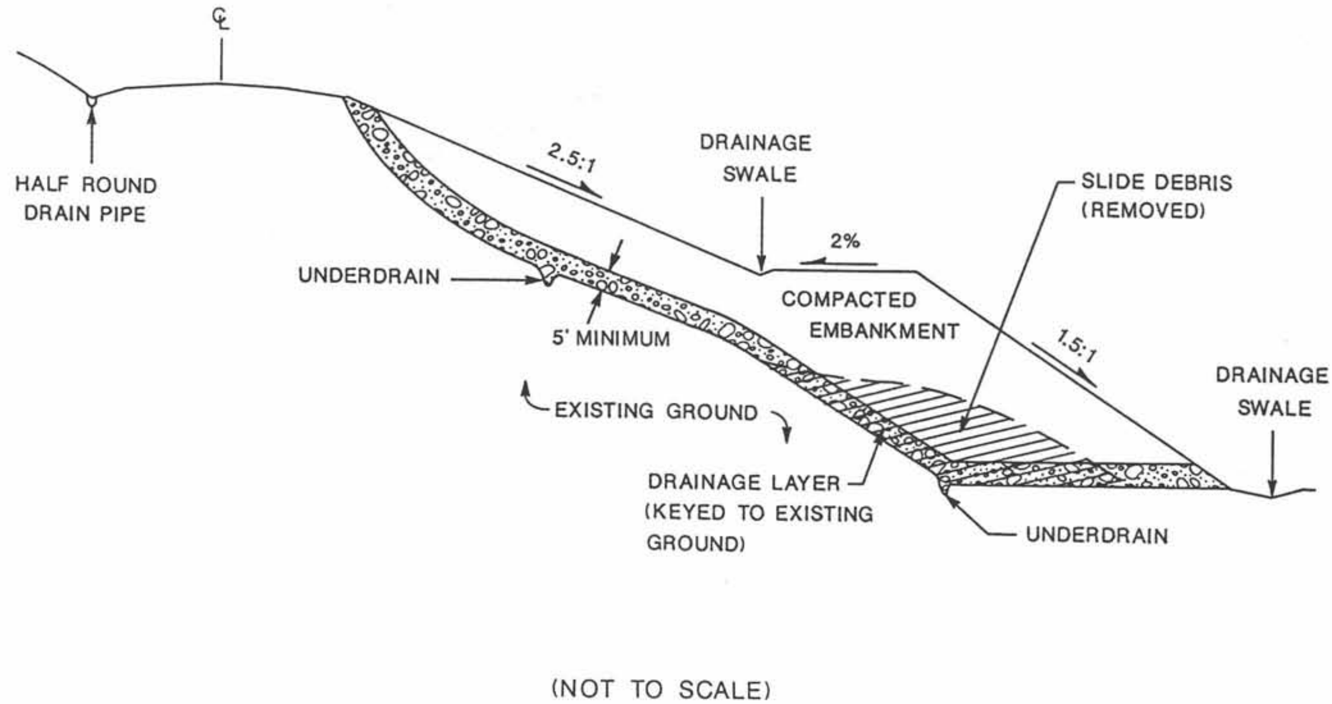

Figure 6. Typical cross-section through buttress at the East Whitney Hill site.

PROJECT CONSTRUCTION

Bids on the project were received from six contractors in May 1986. Estimates ranged from \$577,000 to \$805,000; the County's estimated cost was \$510,000. Actual construction cost, including several change orders, was approximately \$590,000. Construction began in September 1986 and was completed in January 1987. Post-construction field inspections by King County maintenance crews in 1987 and early 1988 indicate that the buttress and drains are performing as designed.

ACKNOWLEDGMENTS

The cooperative efforts of several departments within the government of King County, and the design team selected and coordinated by Reid, Middleton & Associates, Inc., are responsible for successful completion of the project.

REFERENCES

Bishop, L., 1982, *Preliminary Investigation of the East Whitney Hill Road Slide*: King County Department of Public Works, Seattle, WA, 6 p.

Bishop, L., 1983, *East Whitney Hill Road Slide Investigation*: King County Department of Public Works, Seattle, WA, 6 p.

Blunt, D. J.; Easterbrook, D. J.; and Rutter, N. W., 1987, Chronology of Pleistocene sediments in the Puget Lowland, Washington. In Schuster, J. E. (editor), *Selected Papers on the Geology of Washington*: Washington Division of Geology and Earth Resources Bulletin 77, pp. 321-354.

Landau Associates, Inc., 1986, *Geotechnical Investigation and Slope Stabilization Recommendations, 212th Way Southeast Landslides (East Whitney Hill Road), King County, Washington*: Edmonds, WA, 28 p.

Mullineaux, D. R., 1970, *Geology of the Renton, Auburn, and Black Diamond Quadrangles, King County, Washington*: U.S. Geological Survey Professional Paper 672, 92 p.

Engineering Geology on Transportation Routes

Robert A. Robinson, Chapter Editor

Engineering Geology in Washington, Volume II
Washington Division of Geology and Earth Resources Bulletin 78

Engineering Geology on Transportation Routes: Introduction

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The development of transportation routes and corridors in the state of Washington is heavily impacted by a range of difficult geotechnical conditions. To cross the state, most routes must traverse mountainous terrain with inherently marginally stable rock and soil slopes alternating with valleys filled with soft, compressible sediments. The design and construction of a single transportation route may have to accommodate virtually all elements of engineering geology including: unstable soil and rock slopes, soft organic soils, glacially overridden soils, soft to hard rock, faults and shear zones, and volcanic and earthquake hazards. To cope with these conditions, transportation route design may require a large variety of slope stabilization procedures such as reinforced earth or cantilevered cylinder pile retaining walls, deep and shallow bridge foundations, ground-water drawdown and recharge, and major embankments on soft, compressible subgrades.

The history of design and construction of transportation corridors in Washington is short in comparison to that for the eastern United States. Numerous local railroads were constructed, beginning in the 1870s, to transport timber, ore, and farm goods. The first common carrier was completed in 1875 to carry produce from Walla Walla to Wallula on the Columbia River, a distance of 30 mi. Railroad construction was spurred on by the rich natural resources of the state, as well as the potential for major commerce with the Orient, resulting in the development of so-called "tea and silk trains" from Puget Sound. Major railroads were not extended into the state until the late 19th century when the Northern Pacific Railroad was built from Duluth, Minnesota, to Kalama, Washington, on the Columbia River between 1879 and 1883. The transcontinental Great Northern Railroad soon followed in 1893, terminating at Everett. The last transcontinental railroad to enter the state was the Milwaukee Road, extending from Missoula, Montana, to Tacoma across Snoqualmie Pass in 1909.

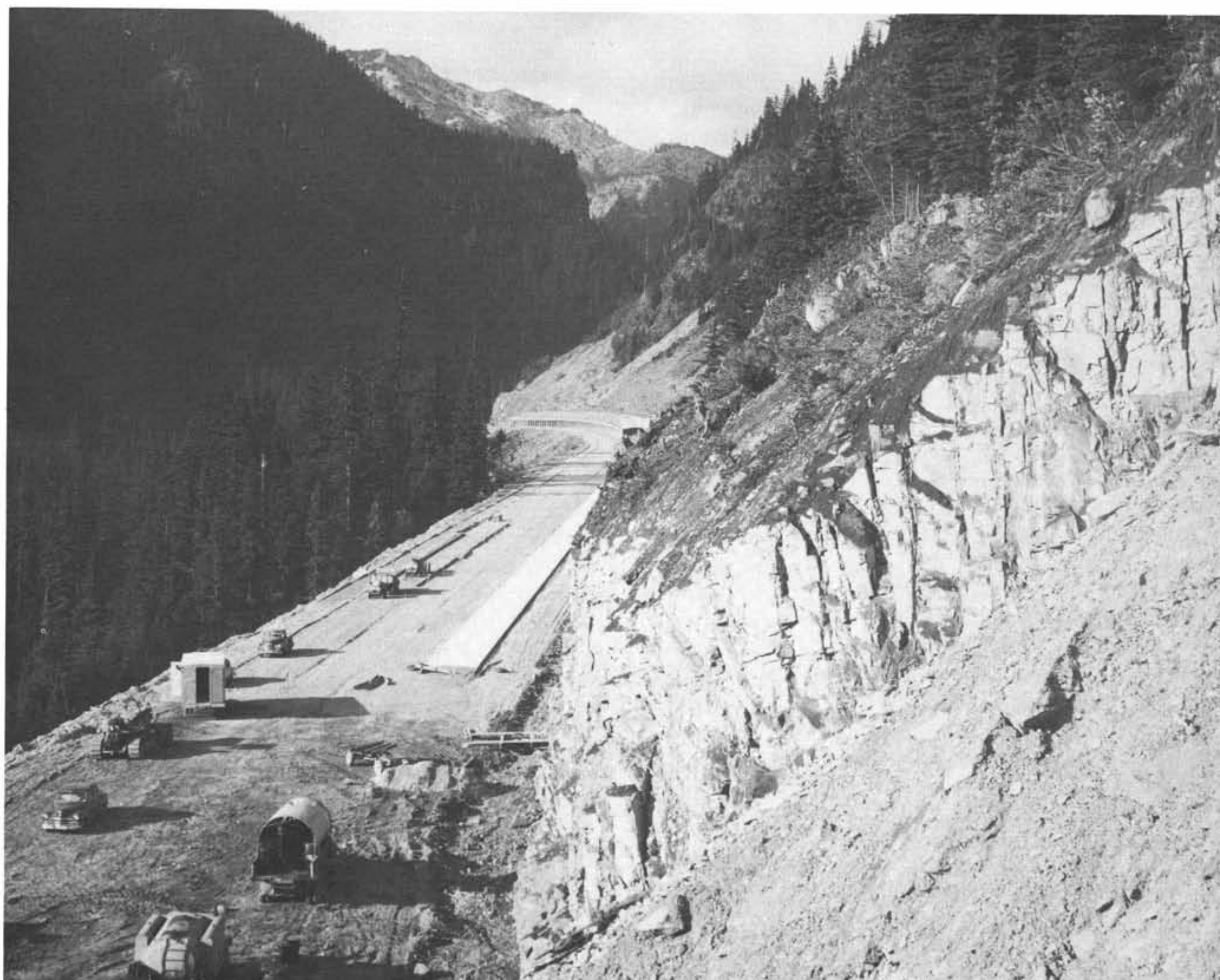
It took the advent of the automobile along with the formation of the State Highway Commission to initiate the development of the extensive, well designed highway system now in use. The first roads were wagon roads, many built as toll roads by towns to route raw materials and farm products to ports and markets. As an example, citizens on either side of Snoqualmie Pass jointly constructed a toll road across the pass in the 1860s to facilitate commerce between both sides of the Cascades. When the State Highway Commission was formed in 1905, it was responsible for the construction and maintenance of 100 mi of roadway comprising 12 state roads. By 1910, the highway system had expanded to more than 35,000 mi of roads including 11,267 mi of improved dirt, gravel, plank, corduroy, and macadam. Major highway construction in the state underwent a dramatic increase in the late 1950s and early 1960s with construction of the federal interstate system including Interstate Highways 5 and 90. Currently there are more than 41,000 mi of paved and 39,000 mi of unpaved state and federal highways and roads in Washington.

The papers selected for this chapter reflect the great diversity of geotechnical conditions and engineering solutions for a wide range of transportation corridors. The topics span the history of construction of major transportation routes in the state, beginning with the paper by Bauhof chronicling construction of the Great Northern (now the Burlington Northern) Railroad tunnel beneath Stevens Pass and ending with the paper by Sondergaard et al. on a portion of the ongoing Interstate 90 project into downtown Seattle.

The papers describe very diverse geotechnical environments and conditions. Higgins and his co-authors discuss road construction on cutslopes in loess. Bronson presents information about soft, compressible peat and organic silt and clay along the West Valley Freeway. Witek's contribution covers foundation conditions beneath an interchange along State Route 16 on the Kitsap Peninsula. The range of exploration environments is also represented in the papers by Yamane and Wu on design and construction of the new West Seattle Bridge in an urban environment, as compared to exploration and design for the new Mount St. Helens Memorial Highway in a remote environment that initially required helicopter access, described in the paper by Burk and others.

The papers in this chapter also provide a cross section of the various construction elements of transportation corridors such as bridge and interchange structure foundations. Harwood and others discuss exploration for the Glenn Jackson Bridge across the Columbia River. Mikkelsen presents monitoring techniques for fills and embankments. Soil slope stabilization for Interstate Highway 5 is reviewed by Squier and Klassel. Rock slope stabilization along a highway in eastern Washington is discussed by Washburn.

These papers also represent a number of triumphant firsts for the engineering and construction industries. The 7.79-mi-long Great Northern tunnel (Bauhof), constructed in 1929, was the western hemisphere's longest railroad tunnel, and currently is the second longest in North America. The construction of Interstate Highway 5 through downtown Seattle encountered major slope instabilities, which were corrected by the first known use of large-diameter cast-in-place cylinder piles (documented by Squier and Klasell) and included the first use in Washington of cone penetrometers for soil testing and of light-weight cinder fills to correct potential slope instabilities (discussed by Johnson). Construction of the last remaining 6 mi of Interstate Highway 90 into Seattle has included the construction of the world's largest diameter soil tunnel, excavated through Mt. Baker Ridge (discussed by Robinson). Also a notable engineering achievement constructed as part of Interstate 90 is one of the world's five floating bridges; four of the five are in Washington, and the first was the initial floating bridge across Lake Washington.



Interstate Highway 90, eastbound lanes construction west of the summit showing the north end of the Snoqualmie Pass rock slide after the road was cleared. Snowsheds (center) have subsequently been removed. Photograph by H. A. Coombs.

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Construction History of the Cascade Railroad Tunnel

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INTRODUCTION

When the 7.79-mi-long Cascade Tunnel was opened on January 12, 1929, by the Great Northern Railway, it became the longest railroad tunnel in North and South America and the fifth longest in the world. The Cascade Tunnel was dedicated to James J. Hill, the founder of the Great Northern and in whose mind the transcontinental line was first conceived. The dedication and opening ceremonies for the tunnel were broadcast on a nationwide radio program and featured numerous political and entertainment dignitaries. In his speech, Herbert Hoover, President-elect of the United States, praised the miners and engineers involved in the construction of the record-breaking tunnel.

The Cascade Tunnel is now operated by the Burlington Northern Railroad and is the longest railroad tunnel in the United States. Until December 1988, when the Canadian Pacific Railroad began operation of the 9.11-mi-long Mount MacDonald Tunnel through the Rogers Pass area in British Columbia, the Cascade Tunnel was the longest in North America. Currently, the Cascade Tunnel is the 19th longest railroad tunnel in the world (Marshall, 1985).

CASCADE MOUNTAIN CROSSING

Initial Switchback

In 1890, when the Great Northern began the extension of its line from Havre, Montana, to the Pacific Coast, the major engineering problem consisted of finding the best route through the Cascade Range. James J. Hill, president of the Great Northern and who was to become known as the Empire Builder, directed John. F. Stevens to locate a line far enough north to serve a territory not already reached by any railway and to get as direct a line to Puget Sound as possible with favorable distance, grades, and curvature (Great Northern Railway, 1929). Just prior, in December 1889, Stevens had discovered the Marias Pass in Montana which gave the Great Northern the lowest crossing over the Rocky Mountains north of New Mexico. At the end of 1890, Stevens recommended a route over a mountain pass, about 100 mi from Seattle, which was named in his honor, Stevens Pass. Stevens eventually became the railroad's chief engineer and later was the chief engineer in charge of construction of the Panama Canal.

The route selected has 2.2 percent grades and a 2.63-mi-long summit tunnel. However, in order to complete the line and provide transcontinental service to Seattle on schedule, a series of temporary switchbacks with grades as steep as 4 percent were constructed over the 4,061-ft pass summit in 1892 (Figure 1).

Old Cascade Tunnel

Construction of the original 2.63-mi tunnel began in January 1897, and the switchbacks were used until the tunnel was opened in 1900. The summit elevation of this crossing was 3,383 ft, located at the east end of the tunnel (Figure 1). The concrete-lined tunnel was 24 ft high by 20 ft wide and cost about $3 million (Engineering News and American Railway Journal, 1900). In 1909, the old tunnel was electrified to improve operations and safety during the summit crossing.

The summit area of Stevens Pass was originally heavily timbered and presented very little avalanche danger. However, forest fires and timber cutting soon destroyed the dense timber stands, and snowslides became an increasingly serious problem. Heavy snowfall at the east portal of the old tunnel reached a maximum in one season of nearly 56 ft. In order to continue operation of the railroad during the winter, snowsheds were built over the tracks at avalanche chute locations. Snowsheds eventually covered about 76 percent of an 8-mi section located west of the pass (Engineering News-Record, 1926a).

On March 1, 1910, one of the worst railroad disasters in the United States occurred in the Wellington siding at the west end of the old tunnel. Two westbound trains from Spokane, which had been stalled at the siding for 10 days by major snowslides to the west, were swept off the tracks and down the Tye River valley by a massive snowslide, killing 101 people (Hubbard, 1981).

Operation of the tunnel line was difficult and hazardous during the winter, and maintenance costs for the snowsheds and tunnel were excessive. This gave impetus to studies of a new tunnel alignment (Wood, 1967).

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1 Present affiliation: CH2M Hill, Inc.

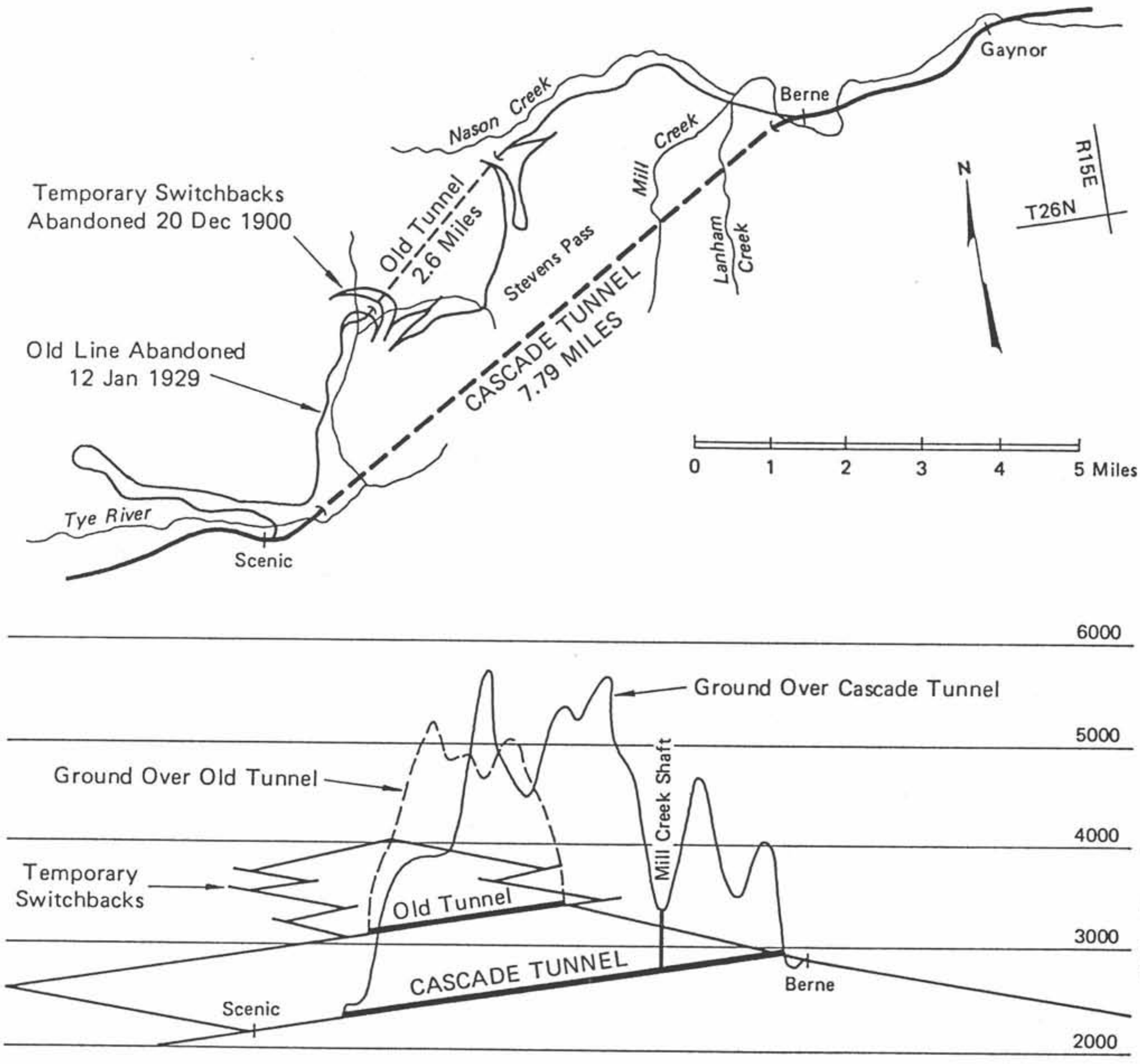

Figure 1. Plan and profile of Cascade Range crossing.

New Cascade Tunnel

In order to improve the summit crossing, four possible new tunnel routes, ranging from 6 to 17 mi in length, were seriously studied throughout 1916 and 1917. However, the start of World War I postponed additional studies until 1921. Cost estimates for a 14-mi tunnel were developed from 1921 to 1925, but the economic expenditure could not be justified. In 1925, the present 7.79-mi tunnel alignment was chosen as the minimum tunnel length that could be economically justified by eliminating all the snowsheds on the west slope (Kerr, 1932).

On Thanksgiving Day, 1925, the railroad directors authorized the most extensive improvement project ever undertaken on the Great Northern at that time. It involved the construction of a new long tunnel through the backbone of the Cascade Range, the relocation of all but 7 mi of the old 50-mi line between Peshastin and Scenic, the elimination of nearly 12 mi of tunnels and snowsheds, and the electrification of 75 mi of road between Wenatchee and Skykomish. The total construction cost of this program was $25 million, which included $14 million for the new tunnel (Great Northern Railway, 1929).

The Cascade Tunnel is a 41,152-ft-long, single-track, tangent tunnel and is on a 1.56 percent grade descending from east to west (Figure 1). The tunnel substantially lowered the summit elevation, reduced distances and curvature, and eliminated snowslide hazards, expensive maintenance on about 6 mi of snowsheds, and costs for snow removal along the Cascade crossing. Table 1 compares the old and new lines between Berne and Scenic (Kerr, 1932).

In order to avoid heavy renewal cost of the existing west-side snowsheds and to save interest during construction, it was imperative that the new tunnel be constructed within 3 yr (Kerr, 1932). To avoid delay, a fixed-fee construction contract for the tunnel work was given to A. Guthrie & Company of St. Paul, Minnesota, on November 27, 1925.

PROJECT PLANNING

Surveying

Initial tunnel alignment and grade options were selected from a study of U.S. Geological Survey topographic maps which located an 8-mi-long tunnel with the west and east portals near Scenic and Berne, respectively. Detailed location surveys were done at the portals to determine the exact portal location and the new approach grade. The portals were selected to provide surface- and ground-water drainage and adequate cover to "portal under" and to avoid snowslide hazards (Mears, 1932). Investigations along the tunnel route indicated continuously solid rock except for about 1,000 ft at the west portal.

The first alignment surveys were 10-mi traverses along the operating line that connected both portals and were used to throw the tangent line over the mountains (Mears, 1932). The tunnel and shaft excavation work was begun based on this preliminary tangent line. For the final adjustments to the preliminary tunnel alignment survey, large theodolites were mounted on concrete monuments at mountain-top observation stations.

To determine the length of the tunnel axis, a precise traverse was accomplished with four courses. A triangulation survey was also accomplished, as an additional check, by using one of the courses as a base line.

Construction Approach

To complete the tunnel construction within the 3 yr desired by the railroad, it was necessary to develop as many excavation faces as possible, in addition to the east and west portals. The deep valley of Mill Creek, which crossed the tunnel alignment 2.41 mi from the east portal, provided a location for a 622-ft-deep shaft and the development of two additional working faces (Figure 1). Therefore, the 5.38-mi tunnel to the west of the shaft became the controlling factor in the construction schedule of the tunnel.

The method chosen to provide the additional work faces in the tunnel west of the shaft was a small pioneer tunnel, parallel to the main tunnel and with crosscuts to the main tunnel alignment. The pioneer tunnel method had been used successfully to achieve high excavation rates in the Simplon Tunnel in the Swiss Alps and the Connaught Tunnel in the Selkirk Mountains of British Columbia (Baxter, 1932). This method also provided flexibility in dealing with different ground and water conditions. The pioneer tunnel was driven eastward from the west portal and westward from the Mill Creek shaft.

The pioneer tunnel was 8 ft high by 9 ft wide and located 66 ft south of the main tunnel (Figure 2). This distance was considered sufficient to avoid disturbance to the rock surrounding the main tunnel without complicating the construction of the cross cuts or the movement of muck, equipment, materials, or men between the tunnels (Mears, 1932). Cross cuts to the main tunnel were at intervals of about 1,500 ft and, at one time, 11 faces in the main tunnel were being worked simultaneously (Great Northern Railway, 1929).

Table 1. Comparison of old and new rail lines between Berne and Scenic

	Old line	New line	Favorable to new line
Length of line	17.67 mi	9.99 mi	7.67 mi
Summit elevation	3,382 ft	2,881 ft	501 ft
Rise westbound	546 ft	45 ft	501 ft
Fall westbound	1,325 ft	824 ft	501 ft
Snowsheds	6.04 mi	0.00 mi	6.04 mi
Bridges	0.23 mi	0.04 mi	0.19 mi
Tunnels	3.66 mi	7.79 mi	(4.13 mi)
Maximum curve	10 deg	6 deg	4 deg
Degrees of curvature	2,128	187	1,941
Maximum grade	2.2%	2.2%	0%
Grade in tunnel	1.69%	1.56%	0.13%

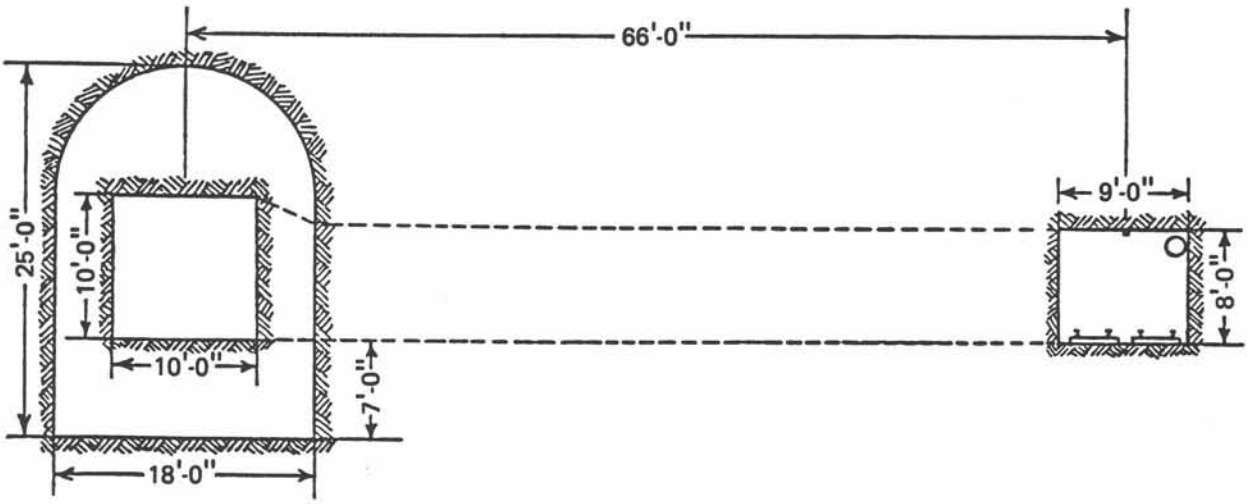

Figure 2. Cross section of center heading, main tunnel, and pioneer tunnel. From Baxter (1932) by permission, American Society of Civil Engineers.

The pioneer tunnel also served as the primary access for supplies and for muck removal from faces opened up from the cross cuts, all of which improved the efficiency of the enlargement drilling, blasting, and mucking operations in the main tunnel. Also, compressed air, water, electrical power lines, and forced air for ventilation were supplied through the pioneer tunnel. The pioneer tunnel also provided drainage for water encountered during construction and operation of the main tunnel. Additionally, if future business warranted, a pioneer tunnel could be enlarged to a single-track tunnel at a significant cost saving.

In general, a 10-ft x 10-ft drift was constructed in the center of the main tunnel from the portals, shaft, and cross cuts. The center heading was enlarged by ring drilling and blasting to the full tunnel section (Figure 2).

In order to avoid possible delays in constructing the pioneer tunnel due to the anticipated bad ground conditions under the Tye River, a 30° inclined drift, 251 ft long, on the east bank of the river was driven down to the line of the pioneer tunnel.

All the rock excavated from the tunnel was used as embankment fill to straighten a total of 7.36 mi of line outside the tunnel (Great Northern Railway, 1929).

As a positive means of rockfall protection, the entire tunnel was lined with concrete. The three-centered roof arch, in combination with the trolley line support recesses (Figure 3), reduced the quantities of excavated rock and concrete liner. The size of the tunnel excavation was based on the nature of the rock and a minimum thickness of concrete liner.

In order to complete the tunnel on time, the lining and excavation operations in the main tunnel were carried on simultaneously. Eight concrete mixing and placing units were operated at different locations in the tunnel, three being served from the west portal and five from the east portal (Baxter, 1932).

CONSTRUCTION METHODS

Drilling and Blasting

In the pioneer and center heading on the west side, a Sullivan drill carriage was used, from which four compressed air drills could be operated simultaneously (Figure 4). This drill carriage required the heading to be mucked out, rail laid, and a horizontal stabilizing bar jacked against the sidewalls before drilling began (Engineering News-Record, 1926c). On the east side, four drills were mounted on a horizontal bar which was set up to drill the upper face after the heading was partially mucked out and again to drill the lower face after it was completely mucked out.

Typical drilling patterns are shown in Figure 5, but these varied, depending on the character of the rock. The enlargement of the main tunnel required ring drilling on 4-ft centers, and these holes were located with a clinometer, as shown in Figure 6.

About 8 to 10 lb of 60 percent dynamite per cubic yard of rock was used for each round in the pioneer tunnel and center heading. The rounds used as many as six different delays and were fired with electric blasting caps.

The shaft was sunk with a stepped bench method that permitted simultaneous drilling and mucking operations in different halves of the shaft, as shown in Figure 5. Drilling of the blast holes was done on the bench while the other side was being mucked out. The shaft round was also charged with 60 percent dynamite and fired with electric blasting caps.

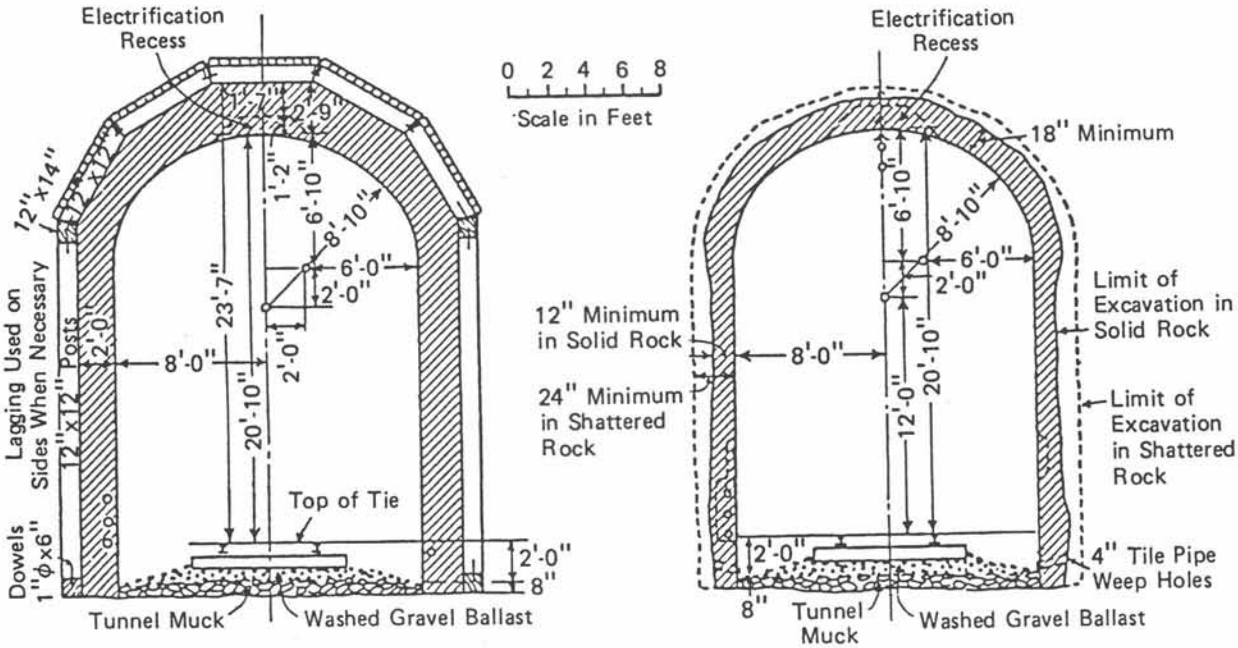

Figure 3. Typical timbered and self-supporting tunnel sections. From Mears (1932) by permission, American Society of Civil Engineers.

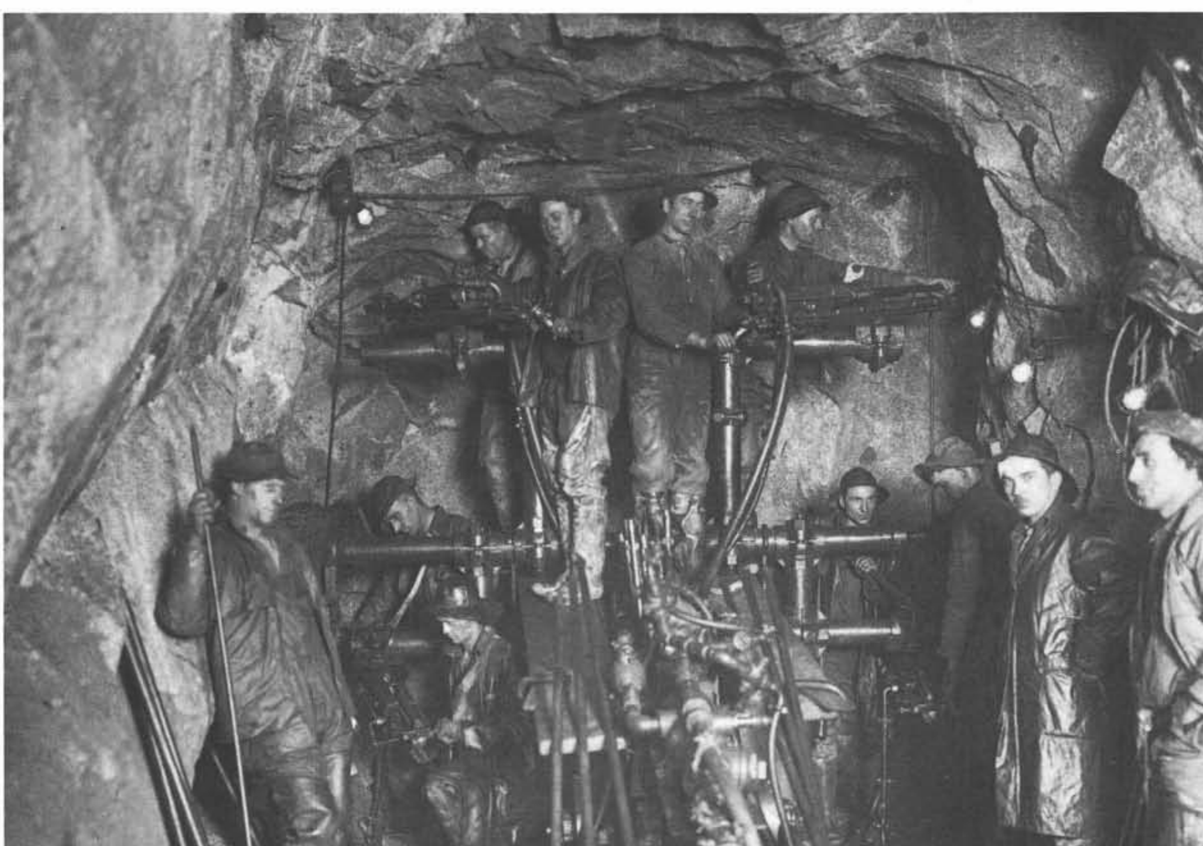

Figure 4. Sullivan drill carriage in center heading. Courtesy of Special Collections Division, University of Washington Libraries; photo by L. Pickett, negative 3762.

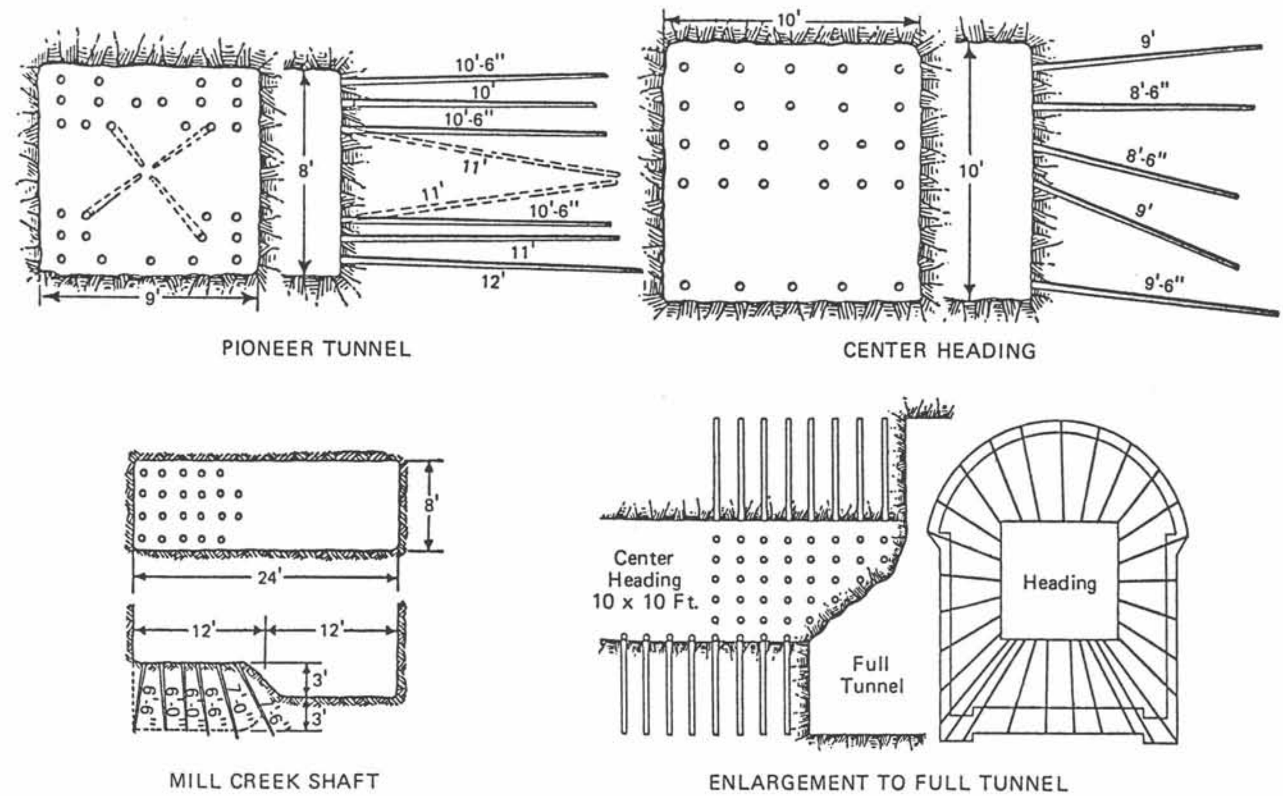

Figure 5. Typical blast hole patterns. From Baxter (1932) by permission, American Society of Civil Engineers.

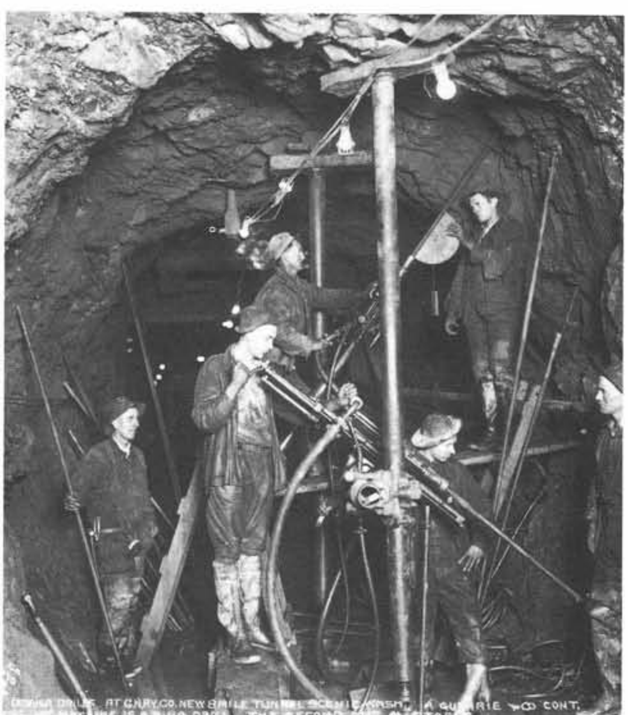

Figure 6. Ring drilling in center heading for enlargement to full tunnel. Courtesy of Special Collections Division, University of Washington Libraries. Photo by L. Pickett, negative 3534

Mucking

Mucking in all the smaller headings was generally accomplished with track-mounted Myers-Whaley mucking machines that loaded side dump cars (Figure 7). Two 24-in. gauge tracks were used in the smaller drifts from the heading to the last cross cut and spaced so that the two inner rails were at 24-in. gauge also. The mucker and drill carriage used this inner track. The 50-cu-ft dump cars were transported in and out of the tunnel, as shown in Figure 8, with 6-ton trolley electrical locomotives.

The two full tunnel faces at the portals and the westward face from the shaft were mucked out with Marion air-driven power shovels, modified from steam power and sized to work within the tunnel (Figure 9). The 6-cy dump cars were transported by 20-ton, 3-ft-gage electric locomotives operated on a trolley (Baxter, 1932).

Main Tunnel Excavation and Support

Five different excavation and support methods were generally used to construct the main tunnel, depending on the ground conditions encountered (Baxter, 1932). The first method was the center heading, which was later enlarged by ring drilling as shown in Figure 10. This was the basic method used in the main tunnel, where the ground in the full section was able to stand without timber supports.

Figure 7. Myers-Whaley mucking machine. Courtesy of Special Collections Division, University of Washington Libraries. Photo by L. Pickett, negative 3257.

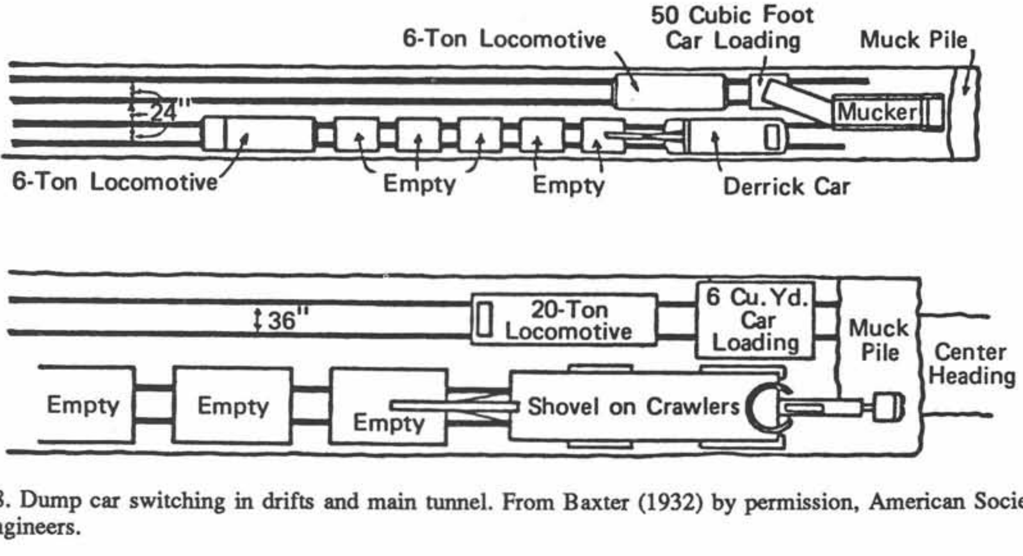

Figure 8. Dump car switching in drifts and main tunnel. From Baxter (1932) by permission, American Society of Civil Engineers.

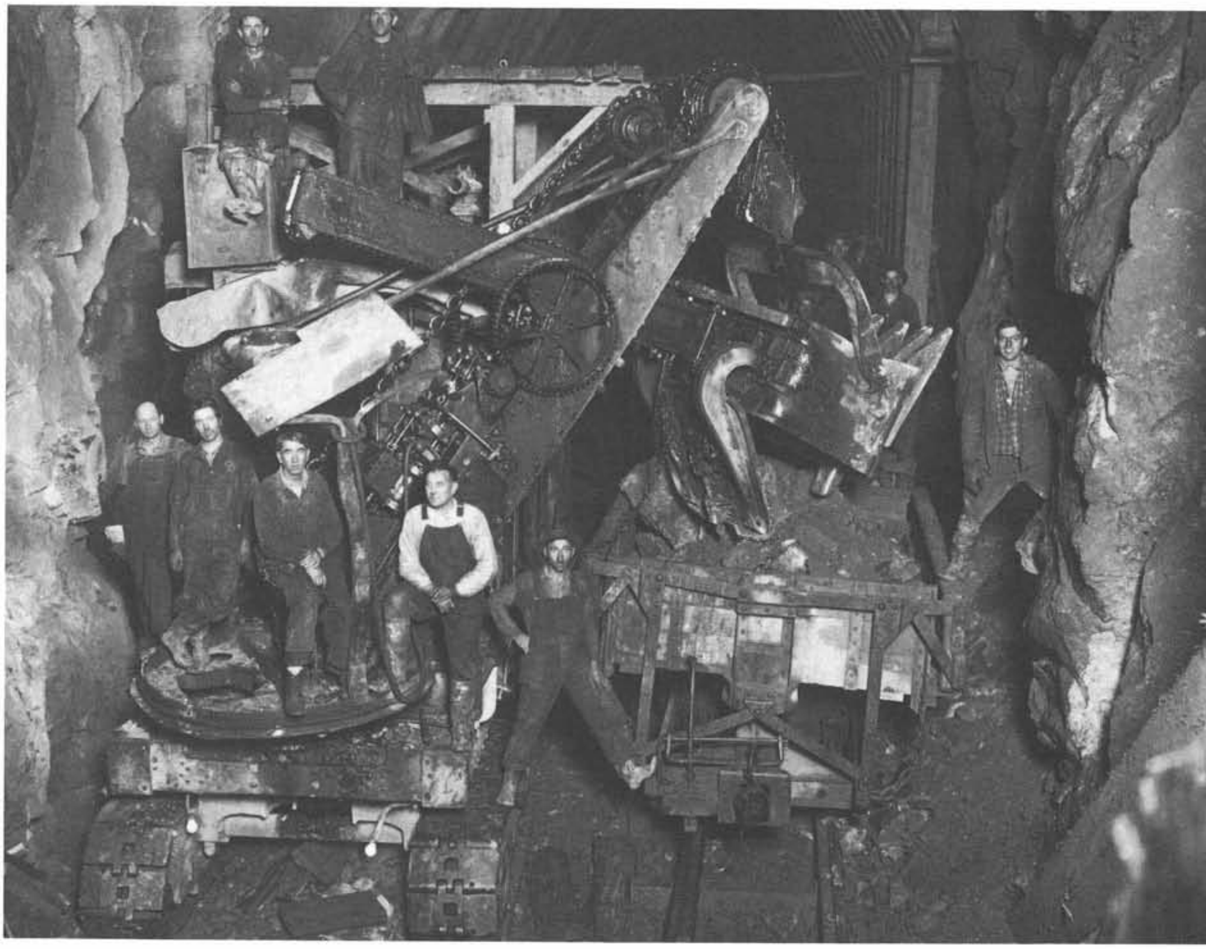

Figure 9. Marion power shovel and dump car in main tunnel. Courtesy of Special Collections Division, University of Washington Librairies. Photo by L. Pickett, negative 3535.

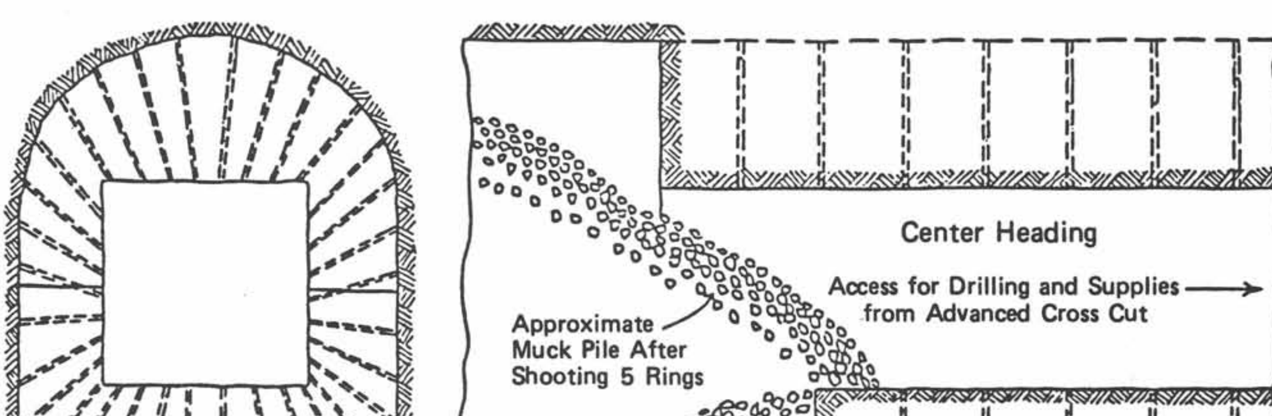

Figure 10. Excavation by ring drilling from center heading. From Baxter (1932) by permission, American Society of Civil Engineers.

The second method was the center heading followed by a top heading and bench, as shown in Figure 11. This approach was used in ground that could not stand without timbering the full section. Following the top heading excavation, five wooden arch segments, 12 in. by 12 in., were placed on a wooden wall plate or beam located at the springline. Later, excavation of the bench allowed the placement of wooden posts to support the wall beam and arch. The timber supports were spaced according to the ground conditions, but they were generally on 2- to 4-ft centers.

The third method involved a top heading and bench, as shown in Figure 12. This was used in advance of the center heading where the ground conditions in the pioneer shaft indicated the full tunnel section could not stand without timber supports. Timbering of the top heading is shown in Figure 13.

The fourth method, wall plate drifts followed by top heading and bench, was also used in advance of the center heading, as shown in Figure 14. This was used in ground that could not stand unsupported in a full top heading.

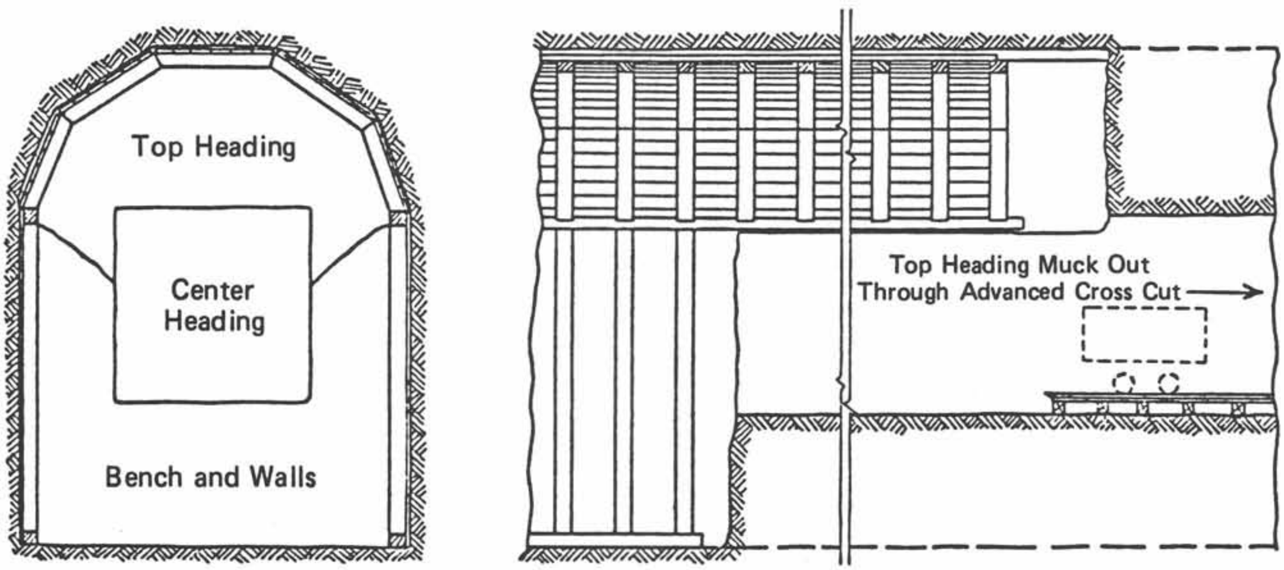

Figure 11. Excavation by center heading followed by top heading and bench. From Baxter (1932) by permission, American Society of Civil Engineers..

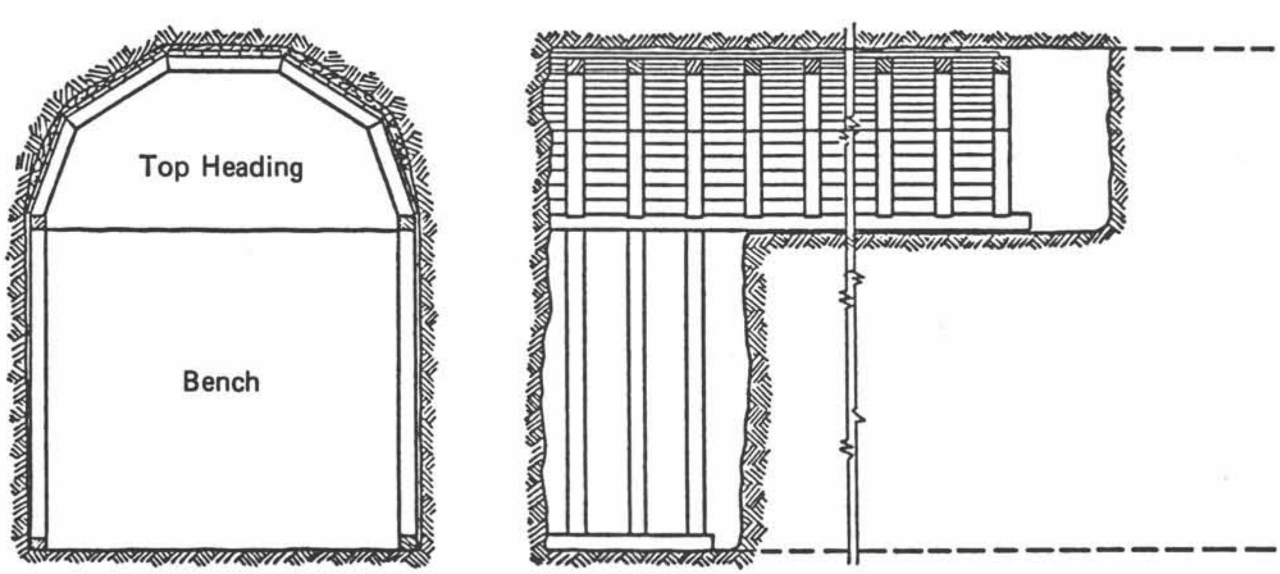

Figure 12. Excavation by top heading and bench. From Baxter (1932) by permission, American Society of Civil Engineers.

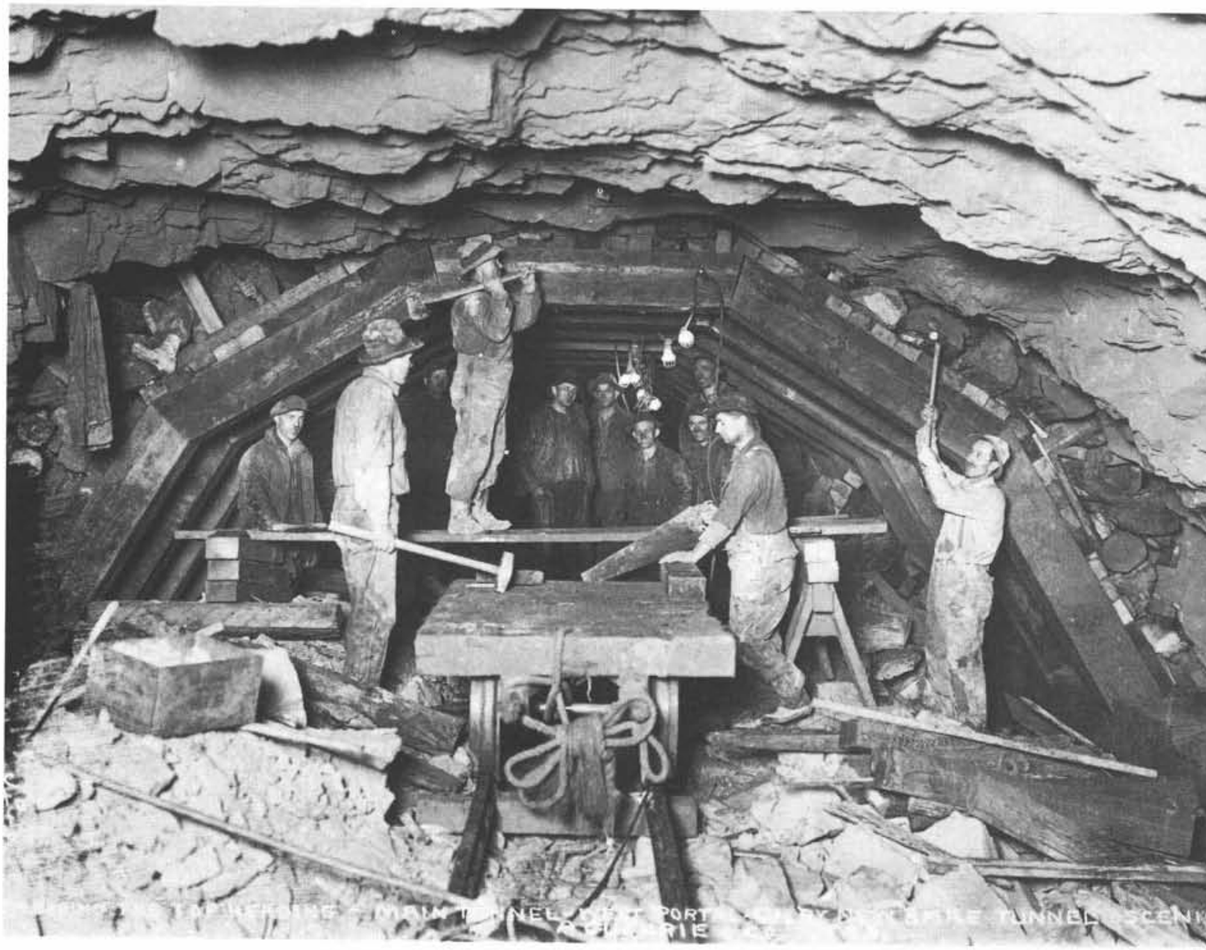

Figure 13. Placement of timber arch supports in top heading. Courtesy of Special Collections Division, University of Washington Libraries. Photo by L. Pickett, negative 3306.

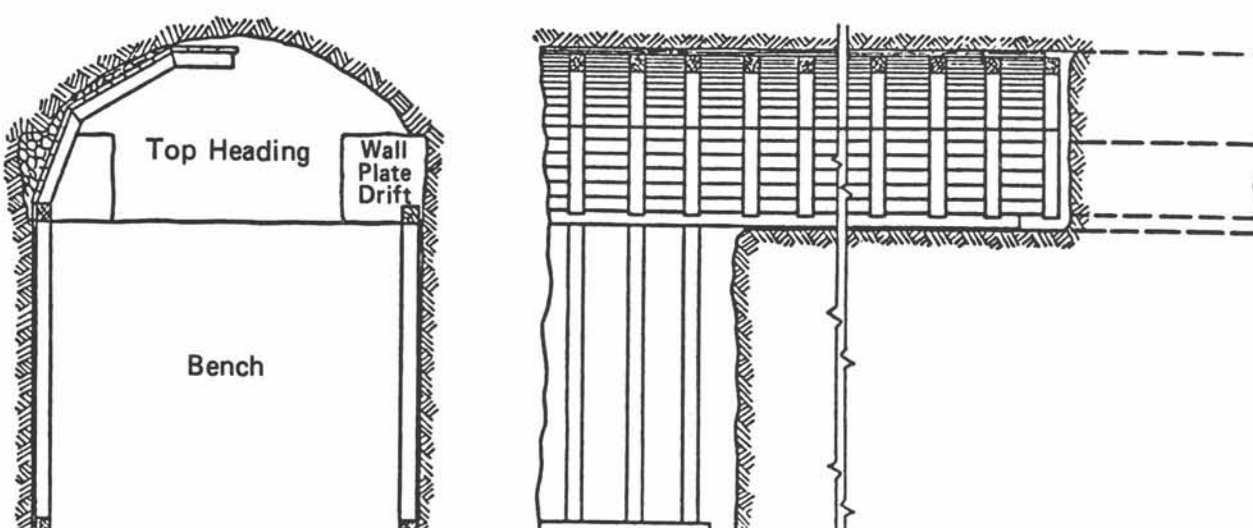

Figure 14. Excavation of wall plate drifts followed by top heading and bench. From Baxter (1932) by permission, American Society of Civil Engineers.

The fifth method used both a top center heading and wall plate drifts to support very poor ground, as shown in Figure 15. These drifts were advanced with crown bars driven ahead of the excavation to provide complete ground support. Once the top was supported, small drifts were used to place the timber posts. Breast boards, which supported the excavation face, were used where the unsupported ground would run or flow into the drift. This method was used to excavate the poor ground at the west end of the tunnel.

Concrete Lining

Due to a haul length of almost 4 mi near the completion of the lining operations, it was decided to transport the dry concrete materials from the proportioning plant to a mixing plant jumbo coupled to the steel concrete forms. The jumbo car carried the concrete mixer, hopper, distributing chutes, conveyor, and pneumatic concrete gun. The steel forms were 37.5 ft long, allowing two set-ups between electrification recesses spaced at 75 ft. The jumbo car was built so concrete and muck cars could pass below and not interfere with the work.

Concrete operations began with the delivery of the gravel, sand, and cement in batch boxes. The concrete mix was 1 part water, 2 parts cement, and 4 parts aggregate. To increase the flowability of this fairly dry mix, 2 percent of diatomaceous earth was used.

At the jumbo, the box was lifted and the contents dumped into a mixing drum (Figure 16). After mixing, the concrete was placed in a skip that discharged into the main hopper. From the hopper, the concrete was directed into chutes for the lower wall, to a belt conveyor for the higher side walls, or into a 1.5-cy compressed-air concrete gun for delivery to the arch of the tunnel.

CONSTRUCTION PERFORMANCE

The construction contract for the tunnel work was given to A. Guthrie & Company on November 27, 1925, and within 2 weeks actual tunneling operations began (Great Northern Railway, 1929). Several world records for tunnel construction were established and helped to complete the tunnel in about half the time previously required for a job of the same magnitude, as shown in Table 2 (Great Northern Railway, 1929). This could be attributed to project planning, modern equipment, and the interest of the miners. A performance bonus program, the chance to break a world's record, and a friendly but intense rivalry between crews at the different headings encouraged innovative and rapid work.

The miners involved with drilling, blasting, and mucking in the pioneer and center drifts were paid an hourly rate and also received a footage bonus (Engineering News-Record, 1926c). The camp with the best excavation record for the month flew a "high camp" pennant from the flagpole for the next month. The pennant had on it a Rocky Mountain goat design, the emblem of the Great Northern, and it created the desire to "get the other camp's goat".

The work was carried on around the clock, three 8-hr shifts per day, including Sundays and all holidays. Crews were changed at the face so that work stoppages

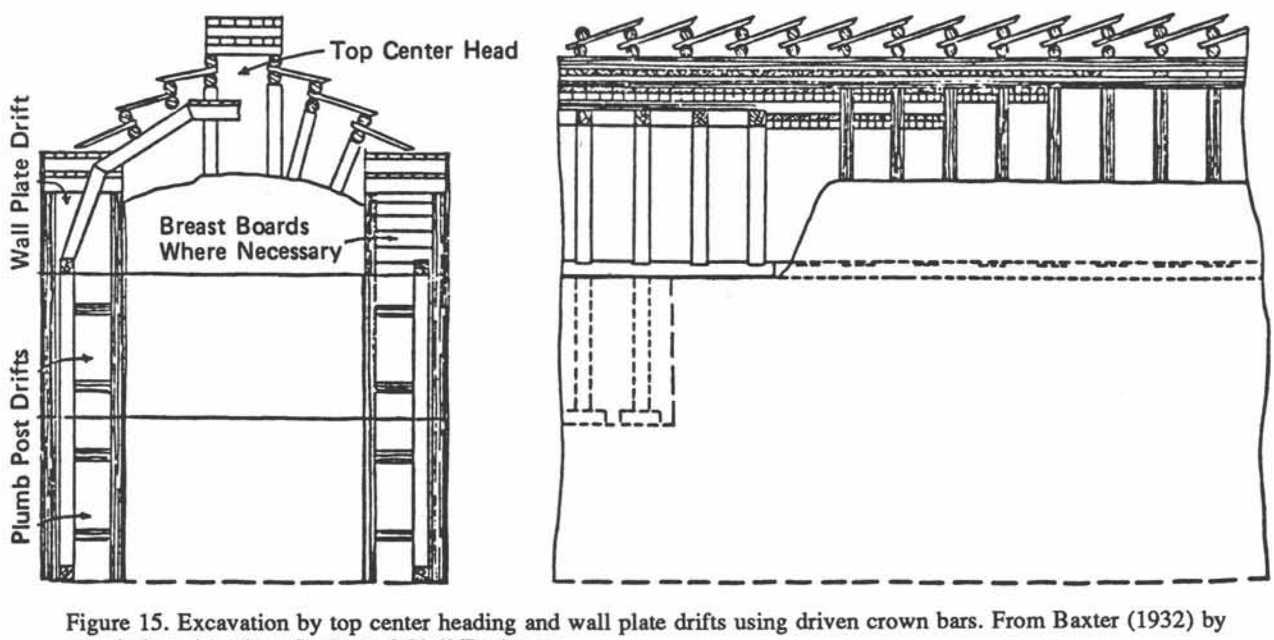

Figure 15. Excavation by top center heading and wall plate drifts using driven crown bars. From Baxter (1932) by permission, American Society of Civil Engineers.

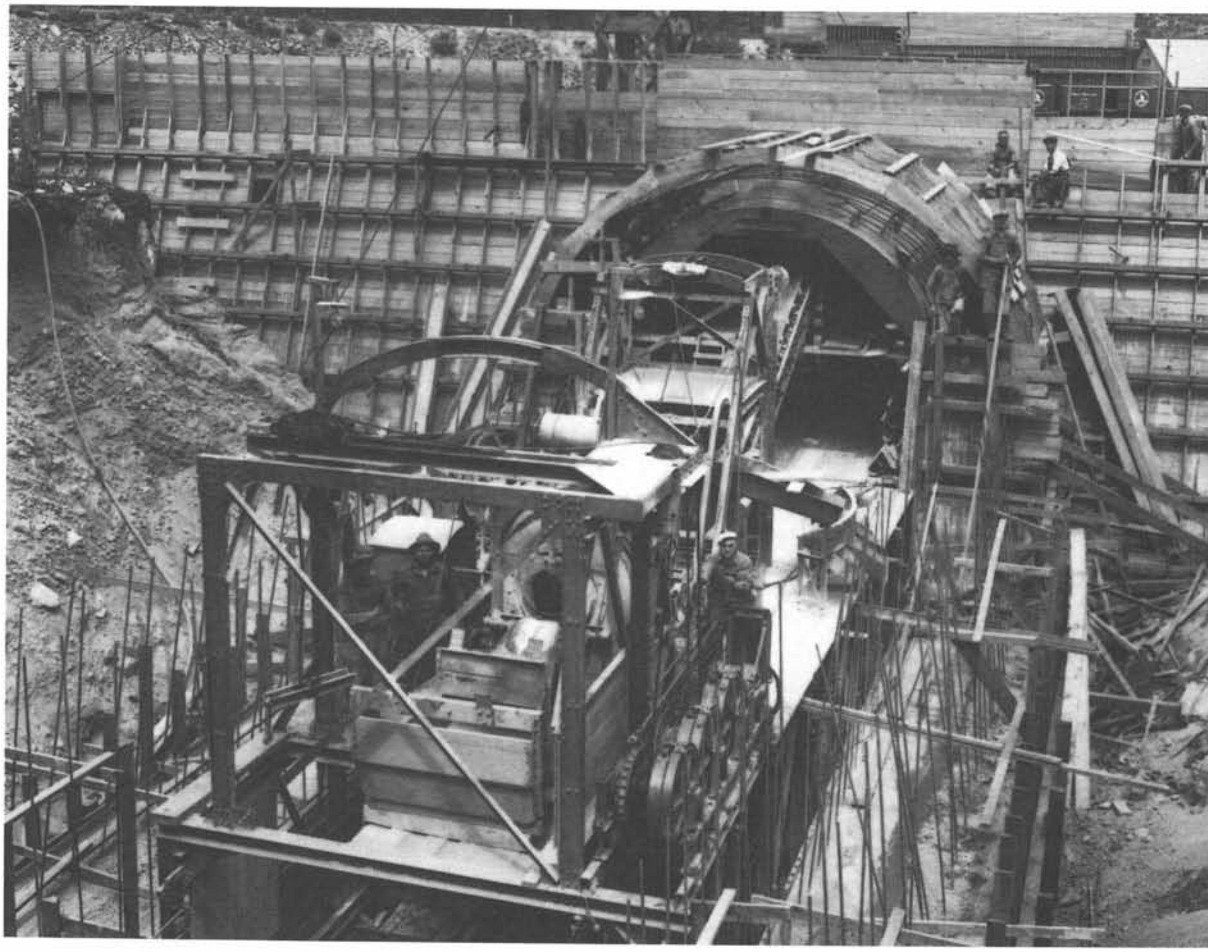

Figure 16. Concrete jumbo at east portal. Courtesy of Special Collections Division, University of Washington Libraries. Photo by L. Pickett, negative 3550.

Table 2. Best tunnel excavation records

Great Northern Railway Cascade tunnel		Canadian Pacific Railway Connaught tunnel	
Pioneer tunnel 8 x 9 ft		Pioneer tunnel 6.5 x 8 ft	
1 day	52 ft	1 day	37 ft
2 days	90 ft	2 days	68 ft
3 days	140 ft	3 days	98 ft
1 month	1,157 ft	1 month	932 ft
Main tunnel enlargement		Main tunnel enlargement	
1 month	1,220 ft	1 month	1,030 ft

did not occur at shift change. At one time there were nearly 1,800 men on site. Three main construction camps with modern facilities were provided—at Scenic, Mill Creek, and Berne.

Geologic Conditions

The tunnel encountered two Late Cretaceous bedrock units consisting of metamorphic rocks of the Chiwaukum Schist and intrusive igneous rocks of the Mount Stuart batholith. The Chiwaukum Schist is composed of biotite schist and amphibolite, whereas the Mount Stuart plutonic rocks are composed of quartz diorite and granodiorite (Tabor et al., 1982).

Much of the rock in the tunnel was good, drilling and blasting easily (Engineering News-Record, 1926b, 1927b). The schistose rocks and the quartz granodiorite, which contained graphitic streaks, formed heavy blocky ground. This rock broke back to the seams during blasting and air slaked, which required continual scaling for protection of the miners (Engineering News-Record, 1927b). In some areas, a light coat of gunite was applied to reduce the slaking and rockfall.

Faulted or badly shattered zones of granitic rock containing water under pressure were encountered and significantly slowed progress in the middle third of the tunnel. The maximum total discharge was 10,300 gpm, which gradually decreased to about 5,000 gpm (Baxter, 1932).

Recent alluvium consisting of poorly sorted gravelly sand with boulders was present for about 800 ft at the west end in the valley of the Tye River. This soft ground had the consistency of sloppy concrete, which required solid timbering and caulking (Engineering News-Record, 1926b).

Preliminary studies of the possible rock temperatures, using the highest known geothermal record of 2°F/100 ft of depth and a 3,300-ft maximum overburden, indicated the rock temperature could reach a maximum of 115° at the deepest point. Actual rock temperatures were 55° at the portal and reached a maximum of 70° within the tunnel (Baxter, 1932).

Pioneer Tunnel

In October 1926, the 8-ft-high x 9-ft-wide pioneer tunnel progressed 1,157 ft and set a world record for excavation in one month (Great Northern Railway, 1929). A typical cycle for one 8-ft round took a shift of 16 men about 4 hr and 40 min and consisted of the following:

	Hours	Minutes
Drill 28 holes, 8-1/2 ft long	1	17
Remove drill jumbo, load holes, fire round	0	18
Vent heading	0	27
Move mucker to face, install utilities	0	38
Muck heading	1	22
Remove mucker, move drill jumbo to face	0	38

By throwing an electric switch in Washington, D.C., on May 1, 1928, President Coolidge set off the blast that holed through the pioneer tunnel. The survey work between the west portal and the Mill Creek shaft, a distance of 5.38 mi, was accomplished with a closure error of only 0.64 ft for the alignment, 0.78 ft for the elevation, and 1.00 ft for the distance (Great Northern Railway, 1929).

The pioneer heading was completed in 867 days, during which 28,268 ft was excavated from two faces. In addition, 21 cross cuts, each 100 ft long, were excavated for a total of 30,368 ft at an average rate of 17.5 ft/day (Baxter, 1932).

Center Heading

Construction of 12,733 ft of center heading driven east from the Mill Creek shaft and west from the east portal was completed in 430 days and averaged 14.8 ft/day. The maximum month's progress at one face was 954 ft (Baxter, 1932).

The survey work between the east portal and the Mill Creek shaft, a distance of 2.41 mi, was also accomplished with great accuracy. The closure error was 0.23 ft for the alignment, 0.20 ft for the elevation, and 0.90 ft for the distance (Great Northern Railway, 1929).

Tye River Incline

The 8-ft-high x 9-ft-wide Tye River incline was 251 ft long and intersected the pioneer tunnel 2,280 ft from the west portal. The development of the Tye River incline was justified due to the poor ground conditions of the west portal. The tunnel crews had excavated 3,420 ft beyond the intersection of the incline and the pioneer by the time the pioneer crews broke through into the existing heading.

Mill Creek Shaft

The Mill Creek shaft had a cross section of 8 by 24 ft and contained four 6-ft x 8-ft compartments. One compartment contained a ladder, two ventilation pipes, compressed air, and three pump lines (Engineering News-Record, 1926c). The second compartment contained the man cage, and the other two were for the two 10-cu-ft balanced skips. The skips transported about 140 tons/hr (Engineering News-Record, 1929).

Initial progress of the Mill Creek shaft was slowed by ground-water inflows that were pumped out at a maximum of 215 gpm, all of which entered the shaft within about 250 ft of the surface (Engineering News-Record, 1926c). The shaft was sunk and timbered in 174 days (Engineering News-Record, 1927a). As the center and pioneer headings were driven from the shaft, ground-water inflows that had to be pumped up the shaft increased to a maximum of 2,800 gpm (Baxter, 1932).

Main Tunnel Enlargement

Enlargement of the main tunnel between the east portal and Mill Creek shaft was completed in 452 days, during which 12,733 ft of tunnel was enlarged, for an average rate of 28.2 ft/day. The maximum month's progress at one face was 1,220 ft (Baxter, 1932). The 28,419 ft of main tunnel enlargement between the west portal and the shaft was completed only 8 days behind schedule. The total volume of rock excavated from the main tunnel was 839,700 cy (Great Northern Railway, 1929).

Concrete Lining

Concrete lining of the tunnel was completed 16 days after tunnel excavation was finished. The full tunnel length of 41,152 ft was concrete lined at an average rate of 75.5 ft/day and attained a maximum month's progress of 3,002 ft (Baxter, 1932). The total amount of concrete used in the lining was 275,218 cy.

Tracklaying and Electrification

After the tunnel was built and leveled off with tunnel muck, 21 in. of washed gravel ballast was placed. The track was laid with a 110-lb rail, 7-1/2- x 10-1/2-in. steel tie plates and 7-in. x 9-in. x 8-1/2-ft-long creosoted wooden ties (Mears, 1932). Tracklaying and ballasting of the entire tunnel was completed in 18 days.

The purpose of electrification was to provide clean and pleasant train operation through the long tunnel and to avoid duplication of electric and steam power through the mountain crossing (Great Northern Railway, 1929). Approximately 75 mi from Appleyard (near Wenatchee) to Skykomish were changed to electrical operation. The 11,000-volt contact system consisted of a stranded copper messenger from which was suspended a single copper contact wire.

In 1956, the installation of a tunnel ventilation system permitted diesel engines to operate within the tunnel, and the electric locomotives were abandoned (Wood, 1967).

CONCLUSIONS

The Cascade Tunnel was completed in 3 yr and 47 days from the notice to proceed, at an average rate of 36 ft/day. The primary reasons for the successful completion of the tunnel were the use of the most modern machinery and skilled workmen in the construction effort.

There were numerous benefits due to the opening of the tunnel and improvement of the mountain crossing. Shorter distance and lower grades improved the transport schedule for freight and passenger traffic. The dependability and safety of the crossing were also greatly improved. The upgrading of this important northwest transport link was a significant advancement in the development of the country's transcontinental transportation network.

In the near future, additional construction to improve the clearance dimensions of the Cascade Tunnel will be undertaken. This work will allow the transport of larger double-stack containers through the tunnel. When accomplished, this upgrading will constitute another important step in the continual development of the Pacific Northwest's and our nation's rail transportation system.

ACKNOWLEDGMENTS

Gerald Millar and Harvey W. Parker provided constructive comments and suggestions, as well as the opportunity to develop an interest in railroad and tunnel history through our work for the western railroads. Kenneth Bruestle and Bryan Tompari of the Burlington Northern Railroad provided valuable historical information and assistance. Considerable help was provided by Dorothy Pickett and Louise Sumner related to the review of Lee Pickett's photograph catalogs of the Cascade Tunnel construction.

REFERENCES

Baxter, J. C., 1932, Construction Plans and Methods. In *A Symposium on the Eight-Mile Cascade Tunnel, Great Northern Railway*: American Society of Civil Engineers, Transactions, Vol. 96, Paper No. 1809, New York, NY, pp. 950-988.

Engineering News and American Railway Journal, 1900, Cascade Tunnel: June 14, 1900, Vol. 43, No. 24, p. 385.

Engineering News-Record, 1926a, 7-3/4-Mile Cascade Tunnel for the Great Northern Railway: Jan. 7, 1926, Vol. 96, No. 1, p. 28.

Engineering News-Record, 1926b, Progress and Methods on the New Cascade Tunnel, Great Northern Railway: July 15, 1926, Vol.. 97, No. 3, pp. 100-101.

Engineering News-Record, 1926c, Methods and Progress in Driving the 8-Mile Great Northern Tunnel in the Cascades: Nov. 25, 1926, Vol. 97, No. 22, pp. 858-863,

Engineering News-Record, 1927a, Short Section of Cascade Tunnel Holed Through: April 19, 1927, Vol. 98, No. 15, p. 626.

Engineering News-Record, 1927b, Progress of the Cascade Tunnel, Great Northern Railway: Aug. 11, 1927, Vol. 99, No. 6, pp. 224-225.

Engineering News-Record, 1929, Driving the Second Cascade Tunnel: Feb. 28, 1929, Vol. 102, No. 8, pp. 334-338.

Great Northern Railway, 1929, Dedication and Opening of the New Cascade Tunnel: Great Northern Railway, 21 p. [Available at the Seattle Public Library]

Hubbard, F. H., 1981, *Encyclopedia of North American Railroading*: McGraw-Hill, Inc., New York, NY, 377 p.

Kerr, D. J., 1932, Preliminary Studies and Results of Improving Cascade Crossing. In *A Symposium on the Eight-Mile Cascade Tunnel, Great Northern Railway*: American Society of Civil Engineers, Transactions, Vol. 96, Paper No. 1809, New York, NY, pp. 917-925.

Marshall, J., 1985, *Rail: The Records*: Guinness Superlatives Ltd., Enfield, Middlesex, UK, 192 p.

Mears, F., 1932, Surveys, Construction Methods and a Comparison of Routes. In *A Symposium on the Eight-Mile Cascade Tunnel, Great Northern Railway*: American Society of Civil Engineers, Transactions, Vol. 96, Paper No. 1809, New York, NY, pp. 926-949.

Tabor, R. W.; Frizzell, V. A., Jr.; Booth, D. B.; Whetten, J. T.; Waitt, R. B.; and Zartman, R. E., 1982, *Preliminary Geologic Map of the Skykomish River 1:100,000 Quadrangle, Washington*: U.S. Geological Survey Open-File Report 82-747, 25 p., 1 plate, scale 1:100,000.

Wood, C. R., 1967, *Lines West*: Bonanza Books, New York, NY, 192 p.

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Washington Division of Geology and Earth Resources Bulletin 78

Engineering Geology and Construction of Interstate Highway 82 From Thrall Road to Selah Creek, Washington

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PROJECT BACKGROUND

Interstate Highway 82 (I-82) begins about 2 mi southeast of Ellensburg at its junction with Interstate Highway 90 (I-90) (Figure 1). It continues for about 133 mi in a southeasterly direction, terminating at the Washington-Oregon border on the Columbia River near Plymouth, Washington.

This paper covers the 20-mi segment from Milepost (MP) 3.79 to MP 23.79 (Station 96+30 to Station 1152+00), which is located in south-central Kittitas County and north-central Yakima County. It starts near the south edge of the Kittitas Valley near Thrall Road and terminates at the north rim of Selah Creek canyon. In so doing it traverses some prominent topographic features. From north to south these features are known locally as Manastash Ridge (el. 2,830 ft); Squaw Creek Canyon (el. 1,690 ft); North Umtanum Ridge (el. 2,395 ft); Burbank Creek Canyon (el. 1,785 ft); and South Umtanum Ridge (el. 2,250 ft). Beginning and ending elevations are 1,450 ft and 1,650 ft, respectively. Most grade lines are in the 4 to 5 percent range.

The roadway consists of two southbound lanes (L^R Line) and two northbound lanes (L^L Line), constructed on independent alignments. Travel lanes are 12 ft wide, with 10-ft outside shoulders and 4-ft inside shoulders. The traffic lanes are surfaced with 0.75 ft of portland cement concrete (PCC) placed on 0.25 ft of crushed stone surfacing top course and 0.50 ft of ballast. Shoulders consist of 0.15 ft of Class B asphalt concrete on 0.85 ft of top course and 0.50 ft of ballast.

Nine separate contracts were involved in the overall construction of this segment, which took place from the spring of 1969 to the fall of 1972. Two prestressed concrete girder bridges 140 to 145 ft long and four reinforced concrete box girder bridges ranging from 341 to 567 ft in length were included with the work. There was a total of 18,900,000 cy of planned roadway excavation on the grading contracts. Additionally, several hundred thousand cubic yards of removal was required for slide corrections and cleanup.

This interstate route replaced a two-lane highway, then called Primary State Highway (PSH) 97, located 1/2 to 4 mi west of I-82. It is situated within the deep and crooked Yakima River canyon and closely parallels the east side of the river; this road is now designated as a scenic highway, State Route (SR) 821.

The project is in a desert setting. Data from the U.S. Weather Bureau stations for Ellensburg and Yakima closely reflect job-site conditions. Annual precipitation ranges from nearly 9 in. at Ellensburg to 7-3/4 in. at Yakima. Average yearly snowfall depths are 30 to 35 in. and 20 to 25 in., respectively. Normal winter depth of snow on the ground ranges from 3 to 10 in. Temperature extremes are moderately severe, with recorded maximums of 107°F at Ellensburg, 111°F at Yakima, and minimums of -29°F and -25°F, respectively. The maximum recorded frost depth of 24 in. was measured during February 1950 near both Ellensburg and Yakima. This figure is deemed to be typical for the job site as well.

There are no perennial water courses within the project limits. Several low-yield springs occur adjacent to the central and south portions of the project.

ROUTE GEOLOGY

Stratigraphy and Lithology

Construction involved two principal geologic formations, the Yakima Basalt (a subgroup of the Columbia River Basalt Group), and the Ellensburg Formation (Bentley and Campbell, 1983a, 1983b; Campbell, 1975). Both are of Miocene age. The former comprises a thick sequence of basaltic lava flows. The basalt flows are typically 25 to 200 ft thick, with the uppermost 5 to 10 ft commonly being vesicular. Where fresh, the rock is gray to black; it weathers brown or reddish brown. Most of the flows are fine to medium grained. Jointing ranges from massive and blocky to spacings of an inch or less. Many fractures are filled with silt and clay. Substantial changes in joint patterns occur not only between flows but also within the same flow, locally over short

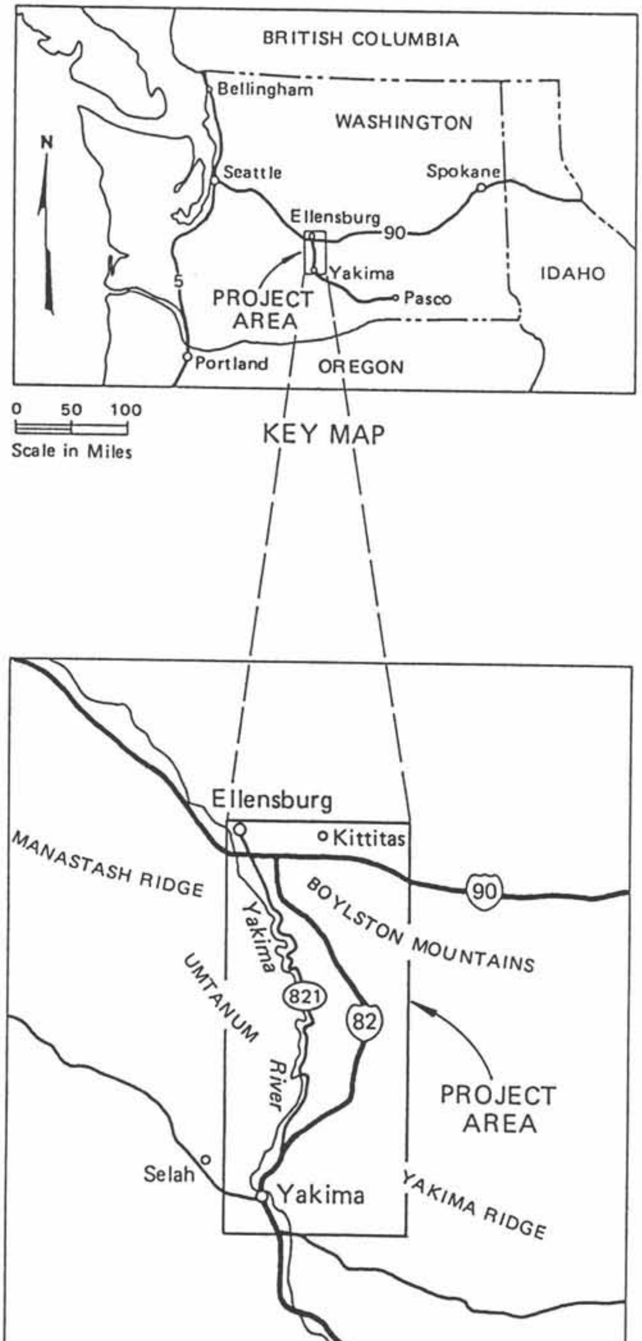


Figure 1. Location of Interstate Highway 82 project.

distances. Much of the extensive fracturing and open jointing is the result of intense folding. The fracturing is most pronounced in areas of greatest stress—that is, where folding is most severe.

Scattered scoriaceous zones as much as 10 or 20 ft thick also are present, and pillow basalt flows containing palagonite are common. Hydrothermal action has been responsible for subsequent alteration and deterioration of the basalt in several areas.

The coeval Ellensburg Formation is sedimentary in origin, consisting of andesitic and pumiceous conglomerate, sandstone, siltstone, and claystone. All are weakly to moderately indurated. Members of the Ellensburg occur as interbeds between Yakima flows.

Overburden soils are generally 10 ft or less thick, and in many places they do not exceed 2 to 3 ft. These soils are mainly silt or clay derived from weathering of the underlying basalt or Ellensburg units. Locally, eolian silt and sand (loess) are present.

Structure

Several geologic factors influenced the design and construction of the highway. Of particular importance was the folding that created a series of west-trending anticlines, three of which are traversed by this alignment: Manastash Ridge, North Umtanum Ridge, and South Umtanum Ridge (Figure 2).

Manastash Ridge is an asymmetrical fold with northerly dips of as much as 50°, while those on the south limb are generally no more than 10°. On North and South Umtanum ridges there is less disparity in the dips, though the dips on the north limb are somewhat steeper than those on the south slope. Accompanying the major deformation are localized secondary flexures along both flanks.

Along with fracturing of basalt, this deformation caused faulting in both the basalt and the sediments. Within the project confines, most faulting is minor; maximum displacements are perhaps 30 ft. Larger faults with offsets of as much as 250 ft have been mapped in the area (Waters, 1955). Not all fault zones in the basalt are readily apparent, the movement having taken place along flow contacts or joint planes in many places. In the Ellensburg sediments the results of folding and faulting are much more visible and striking. A few good examples have been exposed in some of the more extensive roadcuts. As will be discussed later, faulting and folding played key roles in the development of most of the landslides that occurred during construction of the highway.

Landslides

Several ancient landslides were noted in and near the project environs. Most were along the steeper flanks of the anticlines, with the north limb of Manastash Ridge exhibiting the greatest concentration of slides. In some instances two or three slide blocks were situated one above the other. The steep dips coupled with a silt or clay stratum underlying individual basalt layers seem to be the primary cause of a majority of the landslides. Prior to construction all slides within or close to the project appeared to be stable.

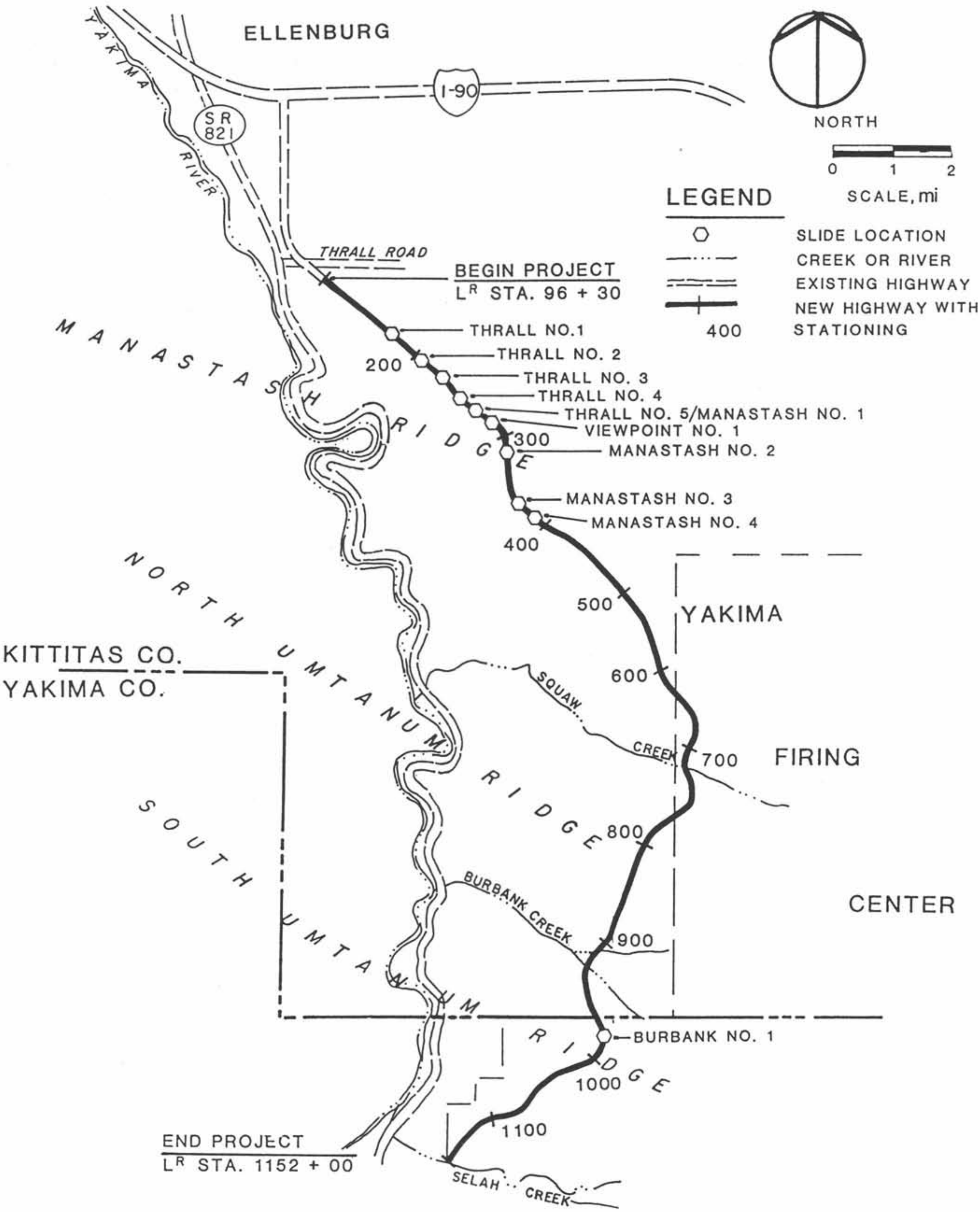


Figure 2. Slide locations along the project area.

Ground Water

Little or no ground water was encountered during the soils investigation or during construction, even where some cuts exceeded 200 ft in depth. Several small springs and seeps are present along the slopes of the major draws and canyons in adjacent areas, however. They apparently emanate from the contacts between basalt flows or between a sedimentary bed and a basalt flow. The exposed sediments appear to be too dense to be aquifers.

An irrigation ditch situated above the initial 2-1/4 mi of the alignment was a matter of concern. However, no ground or surface water was encountered in the roadway section downslope of the ditch.

During the installation of numerous borehole inclinometers in slide areas on the north flank of Manastash Ridge, ground water was encountered in only one or two isolated test borings. Ground water did not appear to play a significant role in any of the project's slides. How important a factor ground water was in producing the ancient landslides is conjectural.

LOCATION AND DESIGN FACTORS

General

In selecting the I-82 alignment a number of route corridors were considered by District 5 of the Washington State Department of Highways (WSDOH). The principal ones were as follows:

(1) An alignment several miles east of the selected route was the initial choice, as it would traverse somewhat more subdued terrain. It took advantage of lower gaps in the aforementioned ridges, thus moderating the grade lines. Also, WSDOH personnel felt that certain potential geologic problems might be less severe along this route. The route was several miles longer, however, leaving the Kittitas Valley via Badger Gap, 7 to 8 mi east of the present highway. The main problem with this alignment was that it was mostly within the confines of the Yakima Firing Center, an extensive U.S. Army training facility. The U.S. Government rejected this corridor because of the extensive encroachment into the manuever areas.

(2) A route within the Yakima River canyon, basically a re-alignment and upgrading of the existing road, was given serious consideration. The principal advantage to this alignment was its near water-level grade along the Yakima River, thereby avoiding most of the steeper terrain of the other proposed corridors. It also presented the shortest route of all. There were some serious drawbacks, however. The confined area within the deep, steep-sided canyon posed some monumental construction problems and attendant costs, especially in view of the four-lane interstate design standards involved. A considerable number of bridges, tunnels, and retaining walls would have been needed, and a large quantity of earthwork, most of it in basalt, would have been required. Another serious handicap would have been traffic control and routing while the road was under construction—while not impossible, it would have been highly impractical and costly. Furthermore, the canyon is aesthetically and environmentally sensitive. A four-lane interstate highway would have been quite detrimental to the environment and, because of its limited-access design, access to and from the river for recreational purposes would have been greatly curtailed. Because of the avoidance of most of the steep grades and possibly better winter driving conditions, most of the trucking industry favored this route. Excessive costs, environmental concerns, and handling of traffic were the primary reasons for rejecting this route.

(3) Another potential alignment choice was one situated about 3 to 5 mi west of the Yakima River canyon. It followed an existing gravel county road. However, it was rejected, mainly because of the steep terrain and higher maximum elevations, a somewhat out-of-direction alignment, and possible access and right-of-way problems to the south around the town of Selah and adjacent, more populated areas.

(4) The final alignment considered was the route ultimately picked. Normal practice is for WSDOH personnel to be responsible for the location and design of highway projects. In the case of this segment of I-82, the available WSDOH personnel to meet the tight schedule on a project of such length was insufficient, and the work was delegated to a private consulting firm, The Ken R. White Co., in the summer/fall of 1966. By this time, the WSDOH had conducted reconnaissance studies on the several corridors under consideration and had selected the one encompassing the route ultimately built. The consultant was directed to locate the specific route within this corridor.

The consultant's responsibilities included the field soils investigations and report (Noble, 1967). Initially, the consultant planned to handle this with an extensive geophysical investigation coupled with detailed on-site observations and geologic mapping, tied in with a minimum of test drilling for correlation. As the geologic complexities became apparent, considerably more test drilling was needed to present a sufficiently detailed picture of the soil and rock conditions.

Several geologic factors were involved in locating and designing the final alignment. These included the anticlinal structures and their related adverse dips, existing landslide blocks, bedrock competency, low-strength sediments, and swelling clays.

While the anticlines could not be avoided completely, attempts were made to cross them in a direction that moderated the impact of the adverse dips. Both North and South Umtanum ridges were traversed on an alignment essentially parallel to the direction of the dip, minimizing potential slope instability. Manastash Ridge, however, was a special case. The elevation at the base of the ridge is about 1,440 ft. The ridge crest at Vanderbilt Gap, the crossing point for the alignment, is nearly 2,800 ft. This required a 5 percent grade for the 5-mi stretch across the moderate dips of the north limb at angles almost normal to the dips. A further complication was the ancient slide blocks dotting the north flank of Manastash Ridge. Their distribution was such that complete avoidance was not practical. The ascending 5-mi grades of the two independent alignments passed through a variety of soil conditions because of the landslide topography and the aforementioned geologic complexities. The consultant called for back slopes ranging from 0.5H to 1V to 2H to 1V to accommodate these variations. Rock buttresses were also specified in two places where old slide blocks were crossed with deep cuts. Most cut heights ranged from 25 to 150 ft in the 5-mi stretch. At Vanderbilt Gap proper, the common cut for the two alignments reached a maximum depth of about 230 ft and involved some complexly folded and faulted rock and abrupt changes in soils conditions within short distances.

Once beyond the summit of Manastash Ridge, the L^L and L^R lines were laid out to avoid other existing slide areas. However, this did not end the back slope design problems. Most of the numerous cuts were in the 20 to 100 ft depth range, but a few reached 150 to 190 ft. The materials to be excavated included competent basalt with widely spaced joints, pillow basalt containing clay-like palagonite, severely weathered basalt with abundant joints filled with clay and silt, moderately competent sandstone and conglomerate, and low-strength siltstone and claystone. In many places several of these conditions occurred in a single cut, creating difficult cut-slope design problems. Steep terrain imposed further design constraints.

Because of the extreme variation in the competency of the basalt, the consultant devised a rock classification system based principally on joint spacing and considering the amount of clay and silt in the joints. The rock was graded on a scale of I, representing "rock so highly fractured it resembles sand" (Noble, 1967, p. 9), to V, sound basalt with widely spaced joints. This classification system, while used mainly to aid in slope design, also helped to determine the ease of ripping the rock. Some correlation was made between geophysical test results and the basalt classifications.

A requirement for the use of pre-splitting or pre-shearing blasting techniques was discussed in the consultant's soils report (Noble, 1967). Blasting was deemed unnecessary in the Class I and II basalt as these classes were judged to be rippable. The report noted that Class III and IV likely would benefit from pre-splitting, but that predetermination of cuts to be pre-split could unnecessarily add to construction costs in excess of benefits. Hence the requirements for pre-splitting rock in these classes were left open for determination at the time of construction. The report further recommended that pre-splitting of Class V rock was superfluous, owing to the near-vertical columnar jointing. The report did not consider excavation methods for rock with widely spaced joints.

The Ellensburg Formation was also classified as follows: I for very soft sandstone, I-S for swelling claystone and siltstone, II for moderately cemented sandstone, and III representing hard sandstone and conglomerate.

Basalt and sedimentary rocks in classes I and II were considered as rippable, while Class III rocks of both groups were barely rippable, and Class IV and V would require drilling and shooting. No pre-splitting was necessary in any of the sedimentary rocks.

Given all the foregoing variables, it is easy to understand the problems involved in arriving at safe and suitable, yet cost-effective cut slopes.

While the designer's main concern was to avoid major slope failures, the problem of slope ravelling had to be dealt with as well. Though of a less serious nature, it could become a long-term maintenance and safety problem. Accommodating the abrupt changes in rock types as well as the variety of the igneous and sedimentary rocks themselves was difficult. No ground water was encountered during the soils investigation; this simplified selection of the design criteria.

The varied geologic conditions resulted in a compromise in the consultant's design of the cut slopes. This prompted differences of opinion regarding design parameters among WSDOH staff from District 5, Headquarters Materials Laboratory, Headquarters Construction Division, and the consulting firm. Several joint meetings were held, some involving Federal Highway Administration personnel, but no consensus was reached. The outcome of these discussions was a call for a program of test cuts to be constructed and evaluated in advance of the ultimate grading work.

Test Cuts

A test program, titled SR-82, Thrall to Burbank, Test Cuts, was implemented; it included two cuts representing the conditions and design criteria in question. Each was to be part of an alignment cut to be excavated to finished grade and thus could be observed to evaluate full-scale performance. A 4,000-ft-long cut segment along the upper part of Manastash Ridge was excavated, but it did not include the deep summit cut itself. A 3,500-ft-long cut segment ascending the moderately sloping north flank of North Umtanum Ridge was also excavated. Most of the planned 1,500,000 cy of roadway excavation was on the L^R, or southbound lanes, where the cuts were more extensive. Work on the L^L Line mainly involved embankment construction using the material from the L^R Line excavation. No pre-existing slide areas were included within the test cut sections.

To aid in evaluating slope performance, a variety of instruments and tests was used: (1) surface reference points, (2) heave points, (3) borehole deflectometers, (4) shear modulus tests, and (5) strain islands. The strain islands were areas used to measure the *in-situ* strains during excavation and release of the residual stresses locked into the expansive formations (Noble, 1970). Two 10-ft-square strain islands were constructed near the bottom of a deep cut near the upper portion of Manastash Ridge. One partially failed during construction because of the dipping strata. Data acquired on the second one were used by the consultant for analyses (Noble, 1970).

Also included with the contract plans were three different rock buttress proposals to be implemented should back slope failures occur or appear imminent. The three designs were provided to accommodate differing slide conditions that might occur.

Only one relatively minor slide occurred during the test cut contract work. It was along the upper slope of Manastash Ridge in the vicinity of L^R Line Station 275. However, correction of this slide was left for the ultimate grading contract because there appeared to be no immediate need for treatment and it was not of major proportions.

No major slope re-designs grew out of the test cut project. Buttress dimensions were increased, and moderate back slope flattening was recommended in some cuts to increase stability and reduce slope ravelling. It was observed that 0.75H to 1V to 1.5H to 1V faces allowed many rocks to roll down the slope with sufficient velocity to reach the shoulders or travel lanes. For 0.5H to 1V to near-vertical cuts, rockfall ditches proved capable of containing most of the rockfall even from the highest faces. As a result, some rock cut slopes were steepened where the basalt quality was deemed sufficiently sound, whereas elsewhere in some of the sedimentary rocks and highly fractured basalt, back slopes were moderated. Where this could not be done effectively, rock protection fences were added to prevent rolling rocks from reaching the highway.

Surfacing and Paving Requirements

A challenging design problem involving the subgrade was identified during the initial soils investigation. Discontinuous deposits of highly expansive siltstone and claystone (Soils Class A-7-5 and A-7-6) (American Association of State Highways and Transportation Officials, 1986) were found at subgrade elevation. Tests on undisturbed samples revealed swell pressures as great as 2,000 psi. The problem was resolved by using a waterproof membrane of catalytically blown asphalt shot directly on the swelling soils. Plans required the actual limits of coverage to be established as grading operations neared subgrade level. The final 1 ft of roadway excavation in such areas was to be delayed until all other grading was completed. Removal of the last 1 ft, application of the asphalt membrane, and placement of a 0.50-ft protective cover of crushed filler and ballast was the final order of work. All except lightweight, noncleated construction traffic was prohibited from traversing the treated areas. The swelling soils taken from the roadway cuts were not to be used within the uppermost several feet of any completed embankment. The above treatment was patterned after one used successfully by the Colorado Department of Highways (Merten and Brakey, 1968) on extensive deposits of swelling clays.

The 1.5 ft of combined surfacing and paving depth used throughout the project was dictated by frost design criteria rather than the Hveem Stabilometer "R" value requirements (WSDOH, 1969). On the main-line travel lanes this consisted of 0.50 ft of ballast, 0.25 ft of crushed stone surfacing top course, and 0.75 ft of PCC. The PCC pavement was selected primarily because it was expected to hold up well under the semi-mountainous conditions that would require tire chains and snow plowing.

Because of the lack of nearby gravel sources and the proximity of state-owned quarries, three basalt quarries were provided by the state for the production of all surfacing and paving aggregates. Haul distances to the roadway ranged from 1/4 to 1-1/4 mi. While WSDOH specifications permit the use of crushed aggregate in PCC, it had not been used before in central Washington WSDOH work or perhaps even in the state. The PCC grading specifications are strict and difficult to meet, especially if crushed aggregate is used. Furthermore, a crushed aggregate results in a harsh concrete mix that is hard to finish and requires a modified mix design. However, the paving contractors elected not to use the quarries for the PCC aggregate. Instead, one acquired aggregate from a privately owned gravel pit about 1 mi north of the north end of the project, while the other obtained concrete from a commercial source several miles south of the job.

The sources for the rock buttress material were roadcuts containing sound, medium jointed to blocky basalt.

LANDSLIDE PROBLEMS

A total of 10 landslides occurred within the construction limits (Figure 2). Three of the slides were categorized as major, and all except one occurred along Manastash Ridge. The exception was located south of Burbank Creek on South Umtanum Ridge and was a major slide.

In addition, a potential slide was detected in a cut in highly fractured and deeply weathered basalt that was generating considerable rockfall. Although there was no actual slide movement prior to remedial action, it was considered to pose future stability problems and warranted remedial treatment.

Each landslide was given a combination name and number, used in the following descriptions.

Thrall #1 Landslide

Thrall #1 Landslide is located on the middle part of the north slope of Manastash Ridge between Stations L^R 173 and 183. This 2H to 1V cut is 40 to 45 ft deep on centerline and 75 to 80 ft deep on the uphill, or right side. It traverses an ancient landslide block composed of slightly indurated clay, silt, and sand together with highly fractured basalt. Failure occurred on or about March 21, 1970, when the cut was still 5 to 10 ft above finished grade (Figure 3). Grading had been previously halted here when the contractor shifted his operations to another area. At that time, the cut was still 20 to 25 ft above final grade level. It was left in this state for a few weeks before excavation resumed. During this period no instabilty was observed. Upon resumption of grading, 10 to 15 ft of additional excavation was accomplished in a few days, and it was then that the failure occurred. This major slide was rotational in nature; the back scarp was 150 to 160 ft above and 480 ft upslope of highway centerline. Numerous small cracks were noted another 70 to 75 ft farther upslope. The toe, located near the centerline of the two L^R lanes, heaved 5 to 6 ft above the cut floor. Subsequent drilling and excavation revealed the slip surface to be a 1/8- to 1/4-in. seam of highly plastic, gray-brown clay. A ravine not far behind the slide separated it from a second old slide lying above it.

Several inclinometers were installed to ascertain the location of the slip plane and monitor ongoing movement. Data thus obtained, together with continued field observations, were used to determine the corrective action (Noble, 1971). The need to maintain adequate support for the second old slide above was also critical. First, the new slide was unloaded by removing an estimated 150,000 to 200,000 cy of material from the head of the slide. When ongoing monitoring indicated the area to be relatively stable, a second step was initiated. A 30-ft-thick rock buttress was constructed. This was accomplished by excavating into the 2H to 1V cut in 50-ft-long segments, each segment being immediately backfilled with buttress rock. This prevented excessive removal of the toe support. The buttress extended 6 or 7 ft below the slip plane to provide a shear key. The slip plane was revealed in the shear key trench. Slow movement into the open trench was observed just ahead of the buttress backfilling. Movement continued, though at a considerably reduced rate, and a few cracks appeared a short distance behind the top of the removal area. Additional unloading of the upper part of the slide arrested slide movement.

Figure 3. Thrall #1 Slide, L^R Line Station 180+ to 181. Shows slip plane down cut slope and across grade, along west edge of slide. Toe of slide in center foreground. Highway cut is 5 ft above final grade.

Thrall #2 Landslide

Thrall #2 Landslide occurred between L^R Stations 211 and 213 in part of an ancient slide block. The 1H to 1V cut, about 60 ft deep at centerline and more than 100 ft high to the southwest, was primarily in basalt containing some palagonite. Failure occurred during construction when the cut was about 20 ft above grade. The combination of highly weathered, severely fractured basalt with palagonite and the 1H to 1V slope triggered the slide. Remedial work consisted of benching back some 75 ft from the original cut face, then resloping the failed area on a 1.5H to 1V to 2H to 1V slope. The remaining lower 20 ft of cut was excavated on the planned 1H to 1V slope because the basalt was found to be more competent. There were no further stability problems.

Thrall #3 Landslide

Thrall #3 was a major slide located on the L^R Line generally between Stations 222 and 228. It was situated in a larger old landslide mass that contained sedimentary rocks ranging from claystone to conglomerate and weathered and broken basalt with clay and silt-filled joints. The landslide, which began moving soon after roadway excavation was started in this 120- to 140-ft-deep cut, appeared to be a block slide. Inclinometers, 50 to 220 ft in depth, were installed. The slip plane was found to be in a highly plastic clay layer that passed at a shallow depth below finished grade. Direction of movement was somewhat diagonal to the alignment.

A massive rock buttress, about 70 ft high and 100 ft thick, was constructed to bring the slide under control. Though a rock buttress was included in the plans for this cut, its size was appreciably smaller than the one built. Slow movement continued after completion of the buttress. An analysis of the data showed that total failure likely would result. Several unloading plans were studied. The plan selected included excavation to begin a short distance behind the buttress on a 4H to 1V slope and continued to a catch point about 725 ft upslope. Planned excavation volume was 77,000 cy. An analysis of the inclinometer data by the consultant showed diminishing deformation rates, which suggested that total collapse would not occur for several years. On this basis, corrective measures were deferred as the slide posed no serious threat. The remedial work, completed in the fall of 1972 on a separate contract, successfully terminated the slide movement.

Thrall #4 Landslide

Thrall #4 is located near Station 250 on the L^R Line. This 1.5H to 1V cut traversed an old landslide block consisting mainly of siltstone and sandstone and minor amounts of claystone and conglomerate. The maximum cut depth was nearly 140 ft on the southwest. Grading had been essentially completed when several thousand cubic yards of the siltstone and claystone sloughed off the face. The slide was 50 to 60 ft high and 75 to 100 ft long. It was confined wholly to the cut face; the head was 30 to 40 ft below the top of cut, and the toe was well above the ditch line. To correct the problem, the 1.5H to 1V slope was moved back 30 ft along a several hundred foot stretch, and a rock buttress of equal dimensions constructed to an elevation just above the back scarp (Figure 4). The cut slope excavation and buttress construction were carried out simultaneously to prevent excessive removal of toe support. This treatment stopped further slide activity.

Thrall #5 Landslide

Thrall #5 was a small slide occurring between L^R Line Stations 270 and 271, which is the initial 100 ft of an 800-ft-long cut. The 30- to 40-ft-high back slope had been excavated on a 1H to 1V slope; the soils here were a mix of weathered, severely fractured basalt and sandy silt containing rock fragments. The area did not appear to be within an older slide mass as had all the others previously described. Corrective measures for this slide are described with those for the Manastash #1 slide.

Manastash #1 Landslide

This shallow block glide failure was located on the L^R Line between Stations 276 and 278, which is the upper end of the cut described in the Thrall #5 slide. Moderately dipping sandstone, siltstone, and claystone beds were undercut by the 1H to 1V cut slope, thereby

Figure 4. Thrall #4 Slide, near L^R Line Station 250. Note slide debris removal and rock buttress construction proceeding simultaneously.

removing the toe support (Figure 5). The result was a landslide of several thousand cubic yards. The slip plane was the upper surface of a claystone layer, the strike of which diverged from the alignment to the west, thus limiting the slide potential in that direction. Because both Thrall #5 and Manastash #1 slides basically resulted from overly steep back slopes and were in opposite ends of the same cut, the recommended treatment called for flattening the entire cut face to a 1.5H to 1V slope. This required some 35,000 cy of additional excavation. Since neither area presented an immediate or particularly hazardous stability problem, correction was deferred and completed in July 1972, under a separate contract.

Figure 5. Manastash Slide #1, L^R Line Station 276 to 278. Note dipping sedimentary rocks at right center. Slip surface in clay strata is exposed as the smooth lobate area at the left edge of the slide.

Viewpoint #1 Rockslide

A small rockslide occurred on the access road to the southbound area viewpoint at "Rest Area, Southbound" Line Stations 19 to 22, which is on the upper northerly slope of Manastash Ridge. The 1H to 1V cut was entirely in basalt bedrock which was dipping into the cut slope. The basalt was weathered and highly fractured in the upper part, but less broken and more competent below. It was between these two parts that the failure occurred. In addition to the rockslide, slope ravelling had taken place, mainly from the upper portion of the face. The slope stability problems were successfully alleviated by resloping the top part of the cut on a 1.75H to 1V back slope. This involved 13,000 cy of additional excavation.

Manastash #2 Landslide

This landslide encompassed the entire 3,500-ft-long cut traversing the summit of Manastash Ridge between L^L and L^R Line Stations 300 and 335. Maximum cut heights exceeded 200 ft, and the geology was quite complex, involving extensive folds and faults in both igneous and sedimentary rocks. The basalt varied from hard and moderately competent to weakly indurated pillow basalt with palagonite. Intercalated sedimentary rocks consisted of sandstone, siltstone, claystone, and a bed of diatomaceous earth. Through the central and deepest part of the cut, where basalt predominated, 0.75H to 1V slopes were excavated. In adjoining portions, the sedimentary rocks dominated, and slopes were laid back to 1.5H to 1V to 2H to 1V with appropriate transitions to the basalt slopes. Two or three minor slides took place within the sedimentary rocks soon after grading was completed, and differential erosion in some of the less durable materials created vertical faces and overhangs, which were considered areas of possible future small failures. Short narrow cracks were detected just behind the top of each side of the cut. These appeared to be an indication of stress release in the tightly folded and faulted formations, but did not seem to pose serious stability problems.

Nonetheless, several inclinometers were installed behind each cut face. Also, several horizontal extensometers as much as 100 ft in length were placed in boreholes on the high side of the cut just above the ditch line. Minor sporadic movements were measured in one or two of the inclinometers, while the extensometers showed no movement. In order to ameliorate some of the poorer slope conditions, benches were constructed on each side of the cut, 50 ft wide on the left (east) and 30 to 40 ft wide on the right (west). The former bench is about 750 ft long and traverses the deepest part of the L^L Line cut. The latter, because of the presence of less competent materials, is situated south of the highest part of the cut and is somewhat shorter. Both benches are some 65 to 75 ft below the top of cut. Slopes have remained stable, except for some minor sloughing in the sedimentary rock and ongoing differential erosion creating rough, uneven faces.

Manastash #3 Landslide

This rock slide occurred on the gently dipping south slope of Manastash Ridge on the L^L Line from Station 380 to 382, where the cut was 75 to 80 ft deep (Figure 6). The material was predominantly weathered, highly fractured basalt, none of which required drilling and shooting. Some thin sedimentary interbeds were also encountered. The 1H to 1V slope failed while under construction along what appeared to be an old fault plane. Approximately 20,000 cy of finely jointed basalt was involved, and it partially buried a piece of earth-moving equipment at the toe of the cut. The problem was cor-

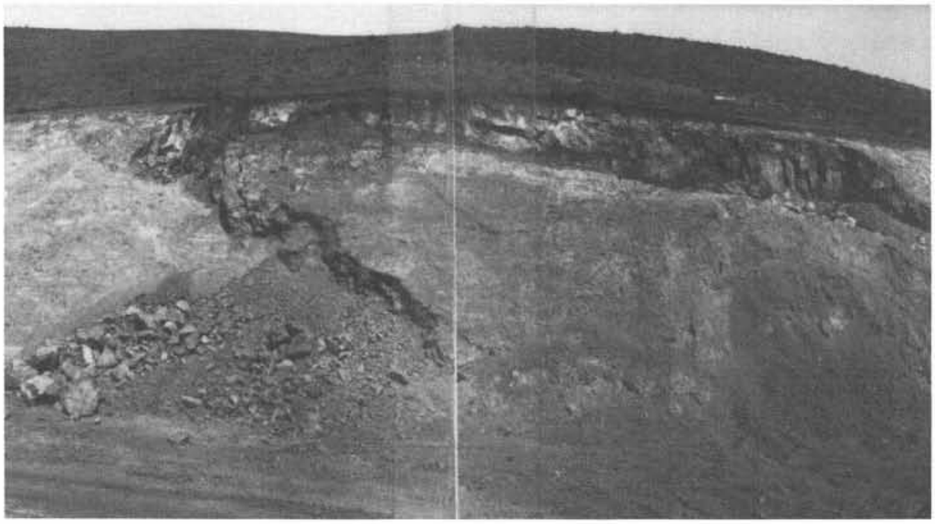


Figure 6. Manastash #3 Slide, L^L Line Station 380 to 382. Most of the cut was on a 1H to 1V slope in weathered and highly fractured basalt.

rected successfully by flattening the cut to a 1.75H to 1V slope, which required about 75,000 cy of additional excavation.

Manastash #4 Landslide

Manastash #4 was designated as a slide area, even though no failure ever occurred. It is located on the L^L Line between Stations 394 and 407 along the southerly slope of Manastash Ridge. This cut, with a maximum depth of more than 100 ft, was built with a 0.75H to 1V slope. Except for the fact that some of the basalt in this cut required drilling and shooting, geologic conditions were similar to those at Manastash #3 slide. There were several open fractures, possibly faults, and a substantial amount of ongoing ravelling on the face. These conditions prompted flattening of the entire northeast side of the cut to a 1.5H to 1V slope.

Burbank #1 Landslide

This massive block glide failure between L^L Line Stations 968+50 and 972+50 was the most interesting and spectacular slide on the project. The cut itself was impressive, reaching 160 ft in depth and 2,400 ft in length, and involved 2.3 million cy of planned roadway excavation. It is located about 0.7 mi north of the summit of South Umtanum Ridge in a small anticlinal fold that has 10° to 15° dips to the northeast and southwest. The alignment trends almost due north-south through the cut. The bulk of the cut material is weakly to well indurated siltstone, sandstone, and conglomerate and minor inclusions of clay. Capping all the sediments is a 50- to 60-ft-thick flow of sound basalt. Several reverse faults showing displacements of 4 to 10 ft are exposed in the cut. Waters (1955) mapped a fault with 200 to 250 ft of displacement in this area; this may be exposed near the extreme north end of the east side of the cut. At about the time grading had reached a depth of 100 ft in the early spring of 1970, minor lateral movement was noted on the east slope of the L^L Line. The basal slip plane was found to be a several inch-thick layer of highly plastic clay near a 100-ft depth. The south edge of the failure was along one of the reverse faults, while a narrow crack running diagonally upslope across bedding planes formed the north boundary.

By early April 1970, displacement had reached approximately 1 in., but the rate was still quite slow. To determine the magnitude and extent of the slide, three inclinometers, a series of surface survey hubs behind the cut face, and numerous survey pins placed on either side of the basal slip plane were installed. Several weeks of monitoring showed very slow movement along the base, with no hint of acceleration. No movement was detected on the surface reference hubs, and a close scrutiny of the ground behind the cut face revealed no surface cracks. Grading operations, consisting of ripping the massive sandstone and conglomerate, continued while daily measurements were taken.

One morning in late May, with the cut approaching finished grade, the contractor's superintendent observed several puffs of dust emanating from the lateral surfaces of the massive slide block and within a very short time noted actual movement. After ordering immediate removal of all construction equipment from the grade, he started to take photos of the phenomenon from atop the opposite cut slope, as by then movement was clearly visible. The slide continued for 30 to 35 minutes, and about 100,000 cy of material fell onto the grade and lower cut slope. The contractor's superintendent documented the whole event on film.

Inspecting the slide aftermath, the anticline was found to have an apparent 6° to 8° plunge to the west (almost normal to highway centerline). It was along this gentle slope that the block appears to have moved. Removing the ~100,000 cy of slide debris left a 400-ft wide by 100-ft deep "amphitheater" perched some 40 to

50 ft above the roadway. Additional excavation was undertaken to bring the bottom below the basal clay layer, and the back and sides were reshaped on 2H to 1V slopes. A rock buttress approximately 50 ft high, 30 ft wide, and 400 ft long was built across the back of the cavity (Figure 7). As evidenced by continued monitoring all slide movement was arrested.

On the L^R Line a rock buttress was also built from Station 950+65 to 958+50 west of centerline. This item had been specified in the plans because the bedding planes dipped somewhat into the cut face at this locale. No sign of instability was noted during or after construction.

MISCELLANEOUS CONSTRUCTION PROBLEMS

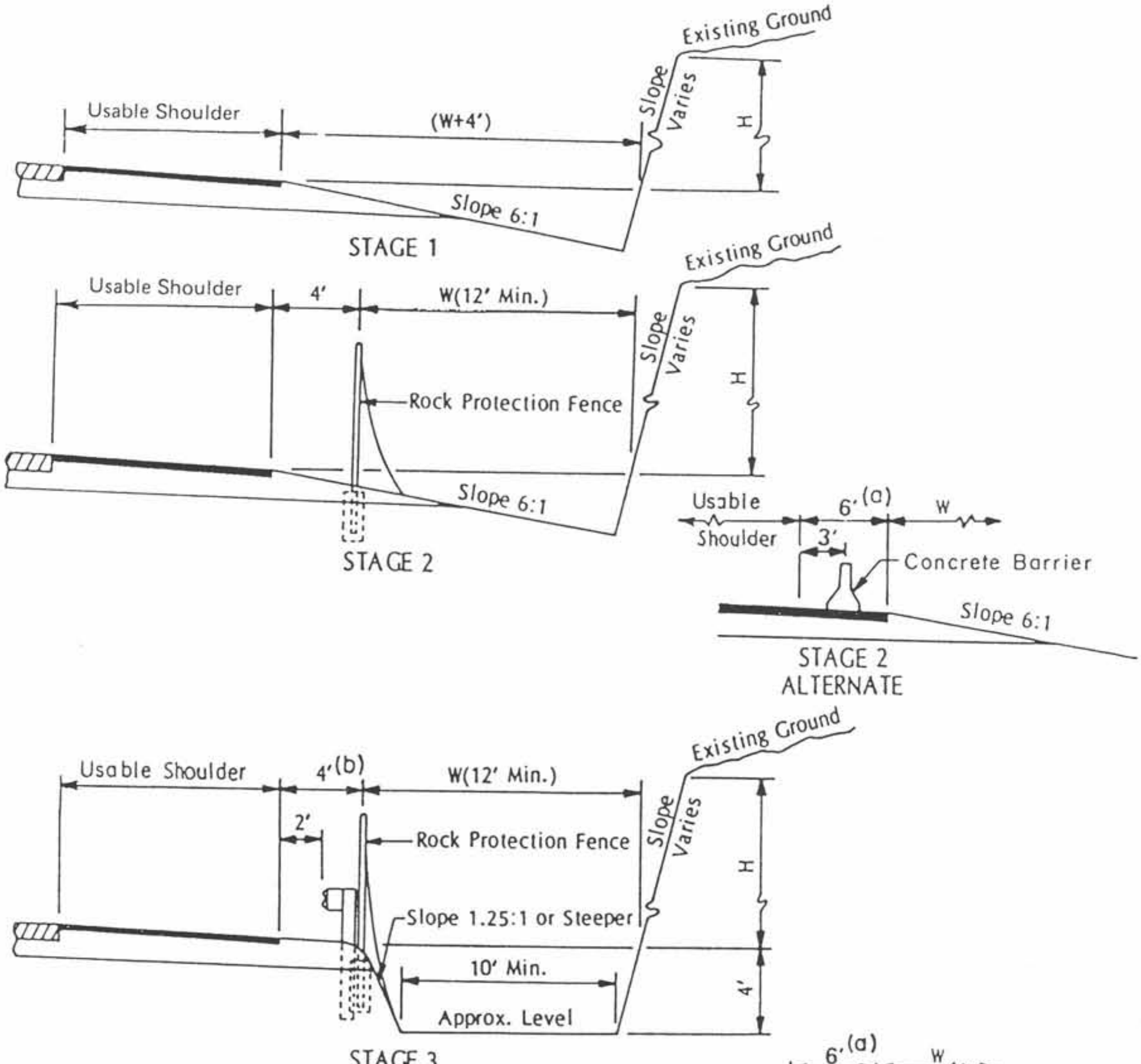
Another potential slope stability problem arose during construction. In a few of the competent rock cuts, cracks were discovered behind the top of the cut and paralleling the cut faces. All cracks were noted in through cuts 50 ft or more in depth and having 0.25H to 1V or near-vertical faces. All were near an anticlinal crest or along a dipping flank. The cracks likely resulted from stress release in the folded rock, triggered by the extensive roadway excavation. When the cracks were discovered in the first two cuts, concern over possible slope failure resulted in one cut being benched and the other resloped from 0.25H to 1V to 0.5H to 1V. Observations of other similar cuts indicated no further movement after initial cracking. Hence further corrective action was deemed unnecessary. Continued observations and monitoring with surface reference points confirmed the stability of these cut slopes.

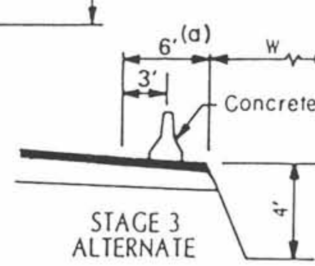
A less serious slope stability problem was severe ravelling in some of the cuts in highly fractured and weathered basalt. At most locations such conditions were anticipated, and standard rockfall ditches were constructed in accordance with the WSDOH Design Manual (1968). In a few places this problem became apparent only after it was too late to build an appropriate rockfall catchment area. At such locations rock protection fences were placed near the toe of slope (Figure 8). These proved to be quite effective; few of the fallen rocks are large, and rockslides of any appreciable volume are uncommon. The amount of accumulated rock in many ditches and behind the fences attests to their effectiveness.

Pre-splitting was used in many of the more competent rock cuts to minimize overbreak and long-term rockfall problems. However, on one grading project the contractor chose to rip all of the cuts, eliminating the pre-splitting requirement. Although this proved difficult in some areas, he persevered, and only minor drilling and shooting were necessary.

Discontinuous deposits of swelling clays had to be dealt with throughout the project. The use of catalytically blown asphalt waterproof membrane was most effectual. No heaving of the PCC pavement occurred except in two isolated areas. The first was near Thrall #1 slide area, and the second was a short distance north of the Burbank slide; both were in the northbound lanes (L^L Line). In both instances narrow, steeply dipping beds of clay traversed the roadway in a cut section and, because of their limited width, were overlooked in application of the membrane. In the Thrall #1 area, when a small bump developed across the northbound lanes, it

Figure 7. March 1987 photo of Burbank #1 slide area, L^L Line Station 968+50 to Station 972+50. Note dipping sedimentary rock, also rock buttress across the center of the photo.





ROCK SL.	H(FT.)	W(FT.)
NEAR VERTICAL	20 - 30	12
	30 - 60	15
	> 60	20
0.25:1 or 0.30:1	20 - 30	12
	30 - 60	15
	60 - 100	20
	> 100	25
0.50:1	20 - 30	12
	30 - 60	15
	60 - 100	20
	> 100	25

Notes:

Cut heights less than 20 feet shall be treated as a normal roadway unless otherwise determined by the District Materials Engineer.

Stage 2 and 3 Alternates may be used when site conditions dictate.

Fence may be used in conjunction with the Stage 3 Alternate.

(a) Four feet when usable shoulder is 8 feet or more and centerline of barrier 1 foot from edge of usable shoulder.

(b) Two-foot minimum when usable shoulder is 8 feet or more and face of guardrail placed at edge of usable shoulder.

Figure 8. Roadway sections in rock cuts, Design A from Washington State Department of Highways (1968).

was feared the slide area was becoming active. However, no movement had been reflected by the inclinometers. A closer examination revealed a 1- to 2-ft-wide, untreated band of swelling clay to be the culprit. In the Burbank area, a somewhat thicker deposit of untreated clay caused two or three of the concrete paving panels to buckle and break.

No significant embankment foundation problems were encountered on the projects, though fill heights of 50 to 125 ft are common. Terracing along steep hillsides was necessary to key the embankments to the original ground.

ROADWAY PERFORMANCE

The highway has been in operation some 15 years at this writing (1988). There have been no slides of any consequence. A small one occurred at the summit of South Umtanum Ridge on the L^L Line near the top of the cut in the folded sediments. The quantity of slide debris was a few thousand cubic yards, and little material reached the travel lanes. The area was cleaned out, reshaped, and the slope flattened slightly. There has been no subsequent instability.

A few minor slumps have taken place in the steeply dipping sediments of the Manastash Ridge summit cut (Manastash #2 slide). Some of the debris fell on the shoulders, but none appears to have reached the traffic lanes. Cleanup and minimal reshaping were the only remedial measures required.

For the most part, rockfall has been limited to slope ravelling; one local rock slide involved a few hundred cubic yards of material. The rockfall ditches and fences have been effective in containing the debris—after 15 yr of rockfall into the ditches, the accumulation is light to moderate, heavier only at the base of some of the near-vertical fault zones which commonly ravel severely.

The membrane treatment of the swelling clays has been very successful, with no noticeable heaving of the pavement in treated areas. Conversely, the two untreated clay layers have heaved significantly, causing pavement distortion and breakage. Embankments show no signs of settlement, and no cut-to-fill transition bumps are in evidence.

Overall, the PCC has held up well, despite the heavy use of studded tires and chains.

The highway has proven to be well designed and well built, successfully overcoming a number of difficult engineering and geologic problems. An interesting sidelight is that despite the size and complexities of the different contracts, no major claims arose on any of the nine contracts involved in the work.

The highway is a tribute to the engineers and geotechnicians from the WSDOH, Federal Highway Administration, and The Ken R. White Co., who, despite differing design and construction concepts, nevertheless coordinated their ideas and efforts to produce this vital piece of highway engineering.

REFERENCES

American Association of State Highway and Transportation Officials, 1986, *Materials, Part I Specifications, 14th Edition, The classification of soils and soil-aggregate mixtures for highway construction purposes*: American Association of State Highway and Transportation Officials, Washington, DC, Designation M-145-82, pp. 213-219.

Bentley, R. D. and Campbell, N. P., 1983a, *Geologic map of the Ellensburg Quadrangle, Washington*: Washington Division of Geology and Earth Resources Geologic Map GM-28, Olympia, WA., 1 sheet, scale 1:62,500.

Bentley, R. D. and Campbell, N. P., 1983b, *Geologic map of the Yakima Quadrangle, Washington*: Washington Division of Geology and Earth Resources Geologic Map GM-29, Olympia, WA., 1 sheet, scale 1:62,500.

Campbell, N. P., 1975, *A geologic road log over Chinook, White Pass and Ellensburg to Yakima highways*: Washington Division of Geology and Earth Resources Information Circular 54, Olympia, WA., 82 p.

Merten, F. K. and Brakey, B. A., 1968, *Asphalt Membranes and Expansive Soils*: The Asphalt Institute, Asphalt Institute Information series 145 (IS-145), College Park, MD, pp. 1-7.

Noble, H. L., 1967, *Final soils and geologic report, Thrall interchange to East Pomona Road, Kittitas and Yakima Co. SR 82*: Prepared for The Ken R. White Co., Denver, CO, 85 p.

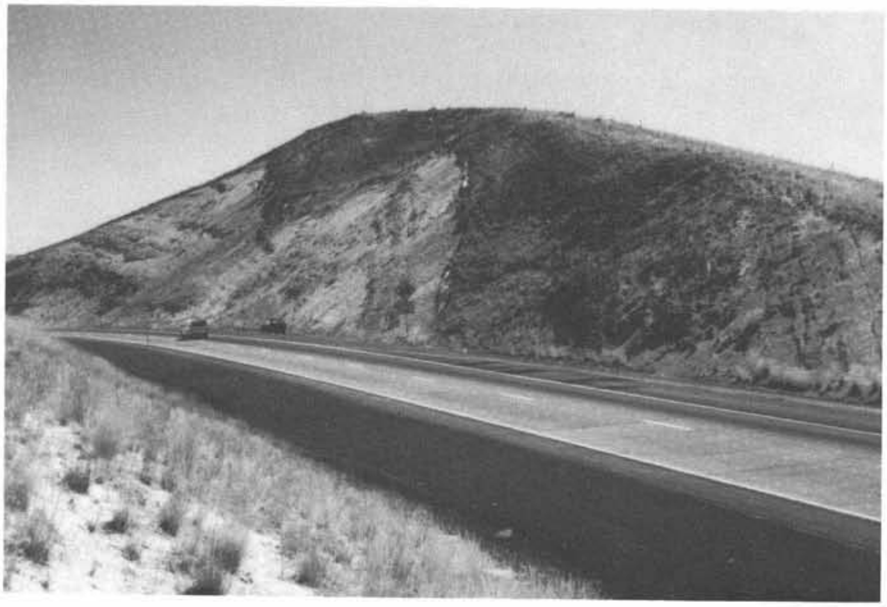
Noble, H. L., 1970, *Analysis of test cuts on Thrall-Pomona project*: Prepared for The Ken R. White Co., Denver, CO, 52 p.

Noble, H. L., 1971, *Investigation of landslides, Thrall one and three, Thrall-Pomona Interstate project SR 90 to Squaw Creek Interchange, Yakima, Washington*: Prepared for The Ken R. White Co., Denver, CO, 15 p.

Washington State Department of Highways, 1968, *Design Manual*: Washington State Department of Highways, Olympia, WA [variously paginated].

Washington State Department of Highways, 1969, *Laboratory Manual, Volume II*, WSHD Test Method No. 611: Washington State Department of Highways, Olympia, WA [variously paginated].

Waters, A. C., 1955, *Geomorphology of south-central Washington, illustrated by the Yakima East Quadrangle*: Geological Society of America Bulletin, Vol. 66, No. 6, pp. 663-684.



View northwest to Kittitas Basin from Vanderbilt Gap where Interstate Highway 82 crosses Manastash Ridge in a deep rock cut at the crest of the Manastash anticline. Photograph by R. W. Galster, August 1987

View of a highway cut along Interstate Highway 82; west-bound lanes at Selah Ridge crossing through (left to right) the Selah Member of the Ellensburg Formation, the Pomona Member of the Saddle Mountians Basalt, and tilted post-Pomona sediments. Photograph by R. W. Galster, August 1987

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Washington Division of Geology and Earth Resources Bulletin 78

Engineering Geology of a Portion of the Spirit Lake Memorial Highway

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INTRODUCTION

On May 18, 1980, mass wasting events related to the eruption of Mount St. Helens destroyed more than 20 mi of Washington State Route (SR) 504 located in the valley bottom of the North Fork Toutle River (Figure 1). Reconstruction of the highway is proposed along the north slope of the valley, well above the level of the 1980 debris avalanche and mudflow. On behalf of Washington State Department of Transportation (WSDOT), Golder Associates Inc. conducted a geotechnical engineering investigation of a 17-mi portion of the new Spirit Lake Memorial Highway from Hoffstadt Mountain to Coldwater Lake (Figure 2). This investigation was intended to provide sufficient information on which to base plans, specifications, and estimates for final roadway design. Because of the lack of previous geologic descriptions of the deposits and rocks along the alignment corridor, the geologic mapping and borehole logging effort focused on describing material properties for engineering purposes and providing the geologic framework within which to interpret the geotechnical field and laboratory data.

METHODOLOGY

The engineering geology investigation for this project consisted of review of previous reports, maps, and technical publications, interpretation of aerial photographs and images, reconnaissance geologic mapping, a primary drilling phase, seismic refraction work, a secondary drilling and test pit excavation phase, and additional geologic investigations at selected sites. The alignment was divided into six segments based on WSDOT construction requirements, and separate reports were issued for each segment. In addition, separate investigation reports were prepared for seven bridges along the route.

The review of available information consisted of examination of previous geological work conducted at the project site on behalf of WSDOT, a review of published literature, and discussions and field trips with individuals knowledgeable about the area.

We examined color vertical aerial photographs at a scale of approximately 1:24,000, taken May 17, 1985, by WSDOT. Image analysis was conducted using false-color infrared images, at a scale of approximately 1:63,000, taken August 6, 1981, as part of the U.S. Geological Survey's (USGS) National High Altitude Program. The results of these analyses were plotted on mylar overlays to the photographs and used as a starting point for field mapping. Black and white vertical aerial photographs, at a scale of approximately 1:45,000, taken July 28, 1952, by the USGS were used for comparison purposes to evaluate any changes in the distribution of surficial deposits over the last 35 yr.

In our geologic mapping we used the WSDOT color aerial photographs, which were enlarged to a scale of approximately 1:12,000. Selected terrain-unit mapping techniques (Kreig and Reger, 1976; Vita, 1984) were used for all geologic mapping. These techniques were chosen because they emphasize the stratigraphy of the surficial geologic units and, unlike traditional surficial and bedrock geologic mapping, permitted inclusion of the borehole data on the alignment plans (Figure 3).

The subsurface investigations included drilling 382 boreholes totaling 17,194 ft and excavating 205 test pits along the alignment and at seven bridge locations and two borrow areas during a 108-day period in the winter and spring of 1987. Single standpipe piezometers were installed in approximately 250 boreholes, and nested piezometers were installed at six locations to more definitively monitor ground-water conditions. Drilling was conducted in two phases so that additional holes could be drilled if the results of the initial subsurface investigations suggested that further exploration was warranted. To investigate subsurface conditions in cut areas and to better define the landslide deposits at the base of Hoffstadt Mountain, 18,704 ft of seismic refraction data were collected.

Subsurface investigations were undertaken using four track-mounted drill rigs and one excavator. Drilling and sampling techniques were consistent with American Society for Testing and Materials (ASTM) standards D1452, D1586, D1587, D2113, and D3550 (ASTM,

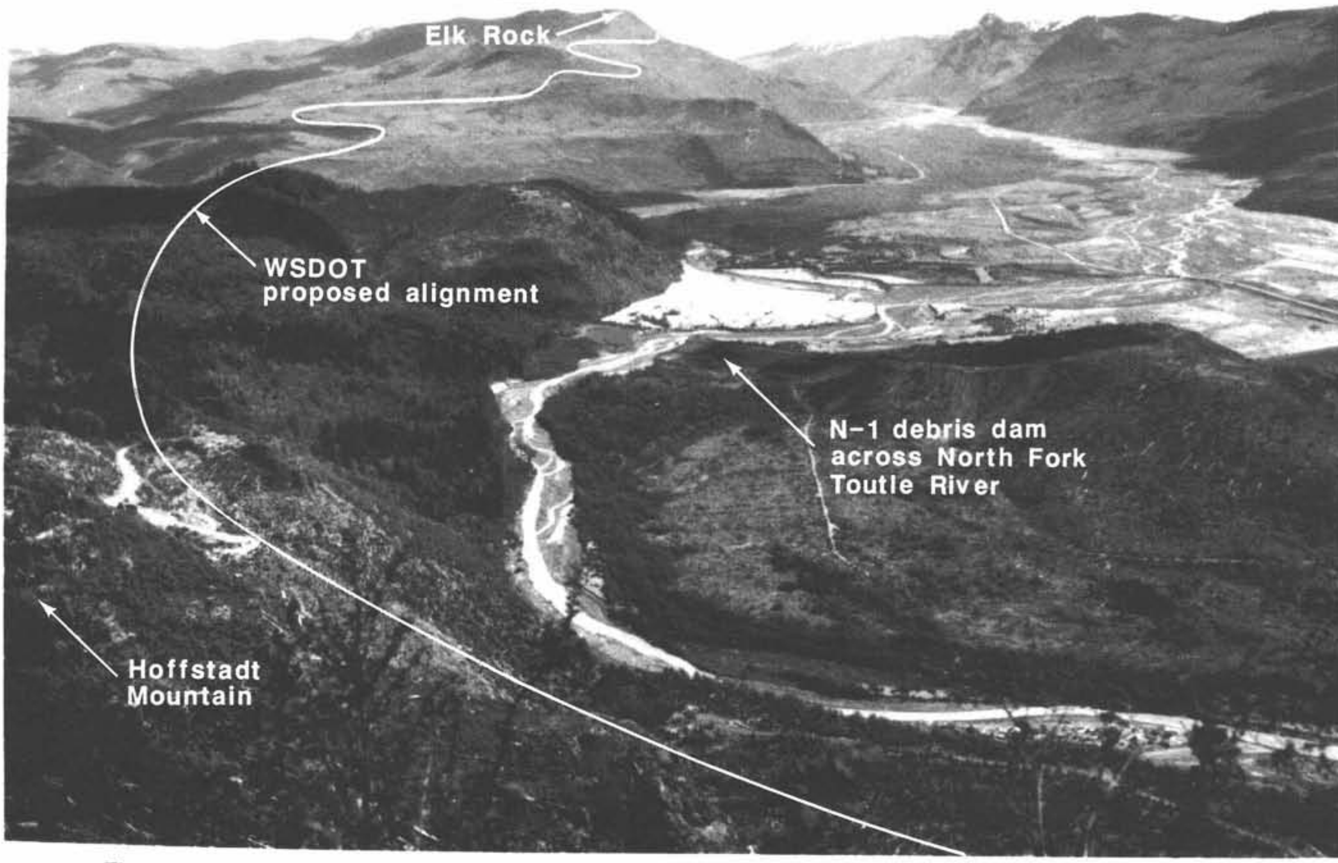


Figure 1. Portion of the Spirit Lake Memorial Highway alignment along the north side of the North Fork Toutle River valley. The original State Route 504 was located in the valley bottom and is now buried beneath the 1980 mudflow and debris avalanche deposits from Mount St. Helens.

1985). Sampling in the unconsolidated deposits was accomplished by drilling casing, washing, and then driving either standard split-spoon samplers or larger diameter California samplers using both standard penetration test (SPT) methods and modified heavy hammer methods alternately. In rock and indurated deposits, diamond core drilling using a double-tube system with a split inner barrel was used to obtain core samples. Test pits were dug with a track-mounted excavator. Rig geologists or engineers were present to log all boreholes and test pits. Logging was accomplished using Golder Associates Inc. standard procedures for soil descriptions (Golder Associates Inc., 1987) and a standard method for rock descriptions, which is based on the International Society for Rock Mechanics (ISRM) Suggested Methods (Brown, 1981). All soil samples and core were independently reviewed in a field laboratory for consistency of description prior to production of final borehole logs. Because of the large number of boreholes and the necessity of on-site engineering analysis concurrent with the investigation program, final borehole logs were produced at the project site using Golder Associates microcomputer borehole logging program (Figure 4). Data entered into the logging program was linked to a data base management program to facilitate data retrieval and analysis.

Golder personnel and subcontractors had offices and living facilities in a trailer camp established for this project at the former site of Weyerhaeuser's Camp Baker (Figure 2).

REGIONAL GEOLOGY

During the Quaternary this area was subjected to extensive alpine glaciation, as well as volcanic activity, and glacial, periglacial, and tephra deposits left by these geologic processes have mantled the older volcanic rocks. After deglaciation, hillslope development occurred by fluvial, colluvial, and mass movement processes. On the steeper slopes, mass movement processes have been particularly important.

The Pacific Northwest is a zone of convergence between the Juan de Fuca and the North American tectonic plates. Subduction of seafloor materials beneath the North American continental plate and subsequent partial melting of these rocks at depth forms the magma

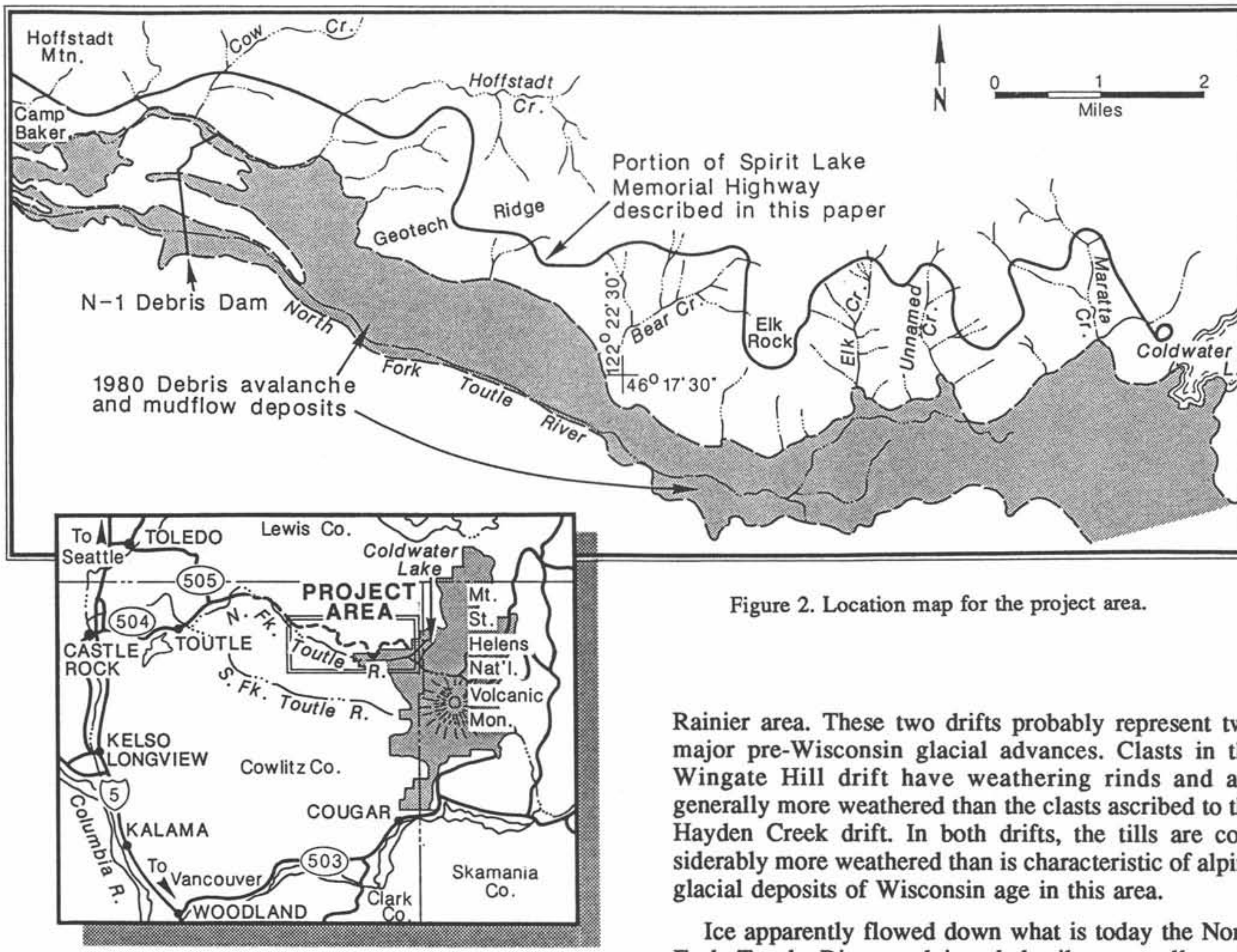


Figure 2. Location map for the project area.

necessary for the eruptions that have produced the volcanic peaks in the Cascade Range. Igneous activity in the southern Washington Cascades spans at least the last 35 m.y. (Fiske et al., 1963), and recent activity is exemplified by the 1980 eruptions of Mount St. Helens.

Surficial Geology

Limited investigations of the glacial and volcanic history of this area have been undertaken to describe the distribution and stratigraphy of the surficial deposits. On the basis of an analysis of the geomorphology and the physical properties of the surficial deposits in the Mount St. Helens area, three major glacial advances are recognized. During the last major glacial advance (Wisconsin, Evans Creek), ice apparently did not emerge from the valley now occupied by Coldwater Lake (Figure 2), and hence, all glacial drift within the project area is considered to be pre-Wisconsin in age.

Glacial deposits in the project area have not been dated; however, based on weathering criteria and stratigraphic position, they are considered to be correlative with the Hayden Creek and Wingate Hill drifts described by Crandell and Miller (1974) in the Mount Rainier area. These two drifts probably represent two major pre-Wisconsin glacial advances. Clasts in the Wingate Hill drift have weathering rinds and are generally more weathered than the clasts ascribed to the Hayden Creek drift. In both drifts, the tills are considerably more weathered than is characteristic of alpine glacial deposits of Wisconsin age in this area.

Ice apparently flowed down what is today the North Fork Toutle River and invaded tributary valleys, in some places forming proglacial lakes. Lacustrine and fluvial deposits in Maratta Creek, Cow Creek, and the Hoffstadt Creek area (Figure 2) form the evidence for these lakes. Remnants of a delta that was formed by the deposition of sediments from Hoffstadt Creek into one of the proglacial lakes were found at an elevation of approximately 2,000 ft, adjacent to Hoffstadt Creek. Other lacustrine deposits and peat have been found between Geotech Ridge and Cow Creek. The proglacial fluvial and lacustrine sediments were subsequently overridden by glacial ice, and the upper parts of these deposits were deformed. The evidence for this deformation is well displayed near the mouth of Maratta Creek.

Two prominent end moraines are present in the upper portion of the Cow Creek drainage, and a major end moraine is present in the Hoffstadt Creek drainage. In the Cow Creek drainage, proglacial fluvial sediments were deposited against one of the moraines on the Toutle valley side and lacustrine sediments were deposited on the north side.

The downvalley limit of ice was not mapped. However, Hayden Creek till is present on the hill adjacent to the U.S. Army Corps of Engineers N-1 debris

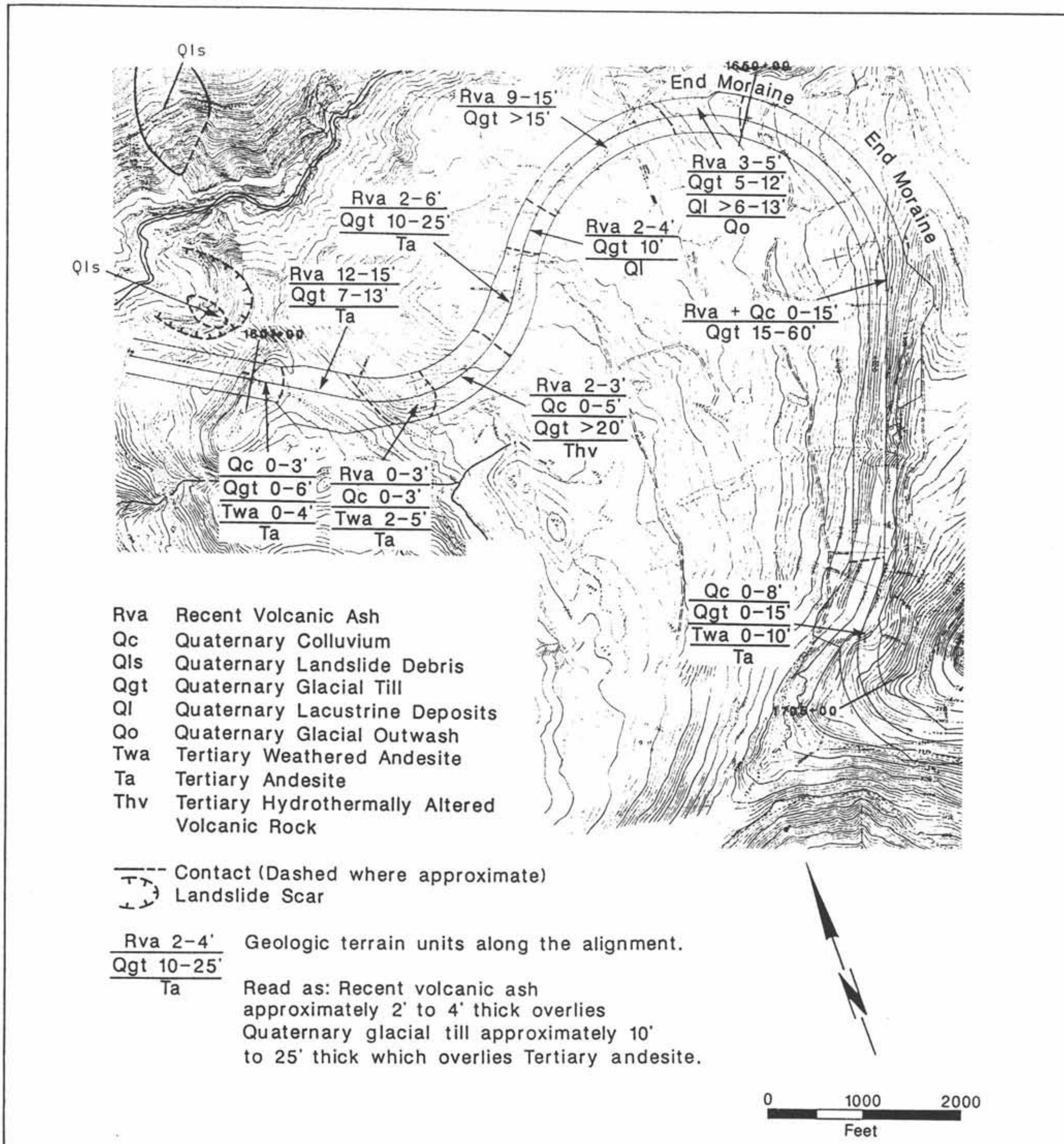


Figure 3. Typical geologic terrain-unit map prepared for each of the six segments of the highway alignment.

dam (Figure 2). Brief reconnaissance efforts did not delineate till farther downvalley. The terminal moraine for that glacier may be buried under the valley fill. Seismic refraction data indicates approximately 200 ft of valley fill on the north side of the valley near the N-1 debris dam.

Prior to glaciation, the volcanic and plutonic rocks in the project area were more directly exposed to the weathering environment. Much of the weathered rock has been stripped away by glacial erosion; however, remnants of the weathered horizons are present locally.

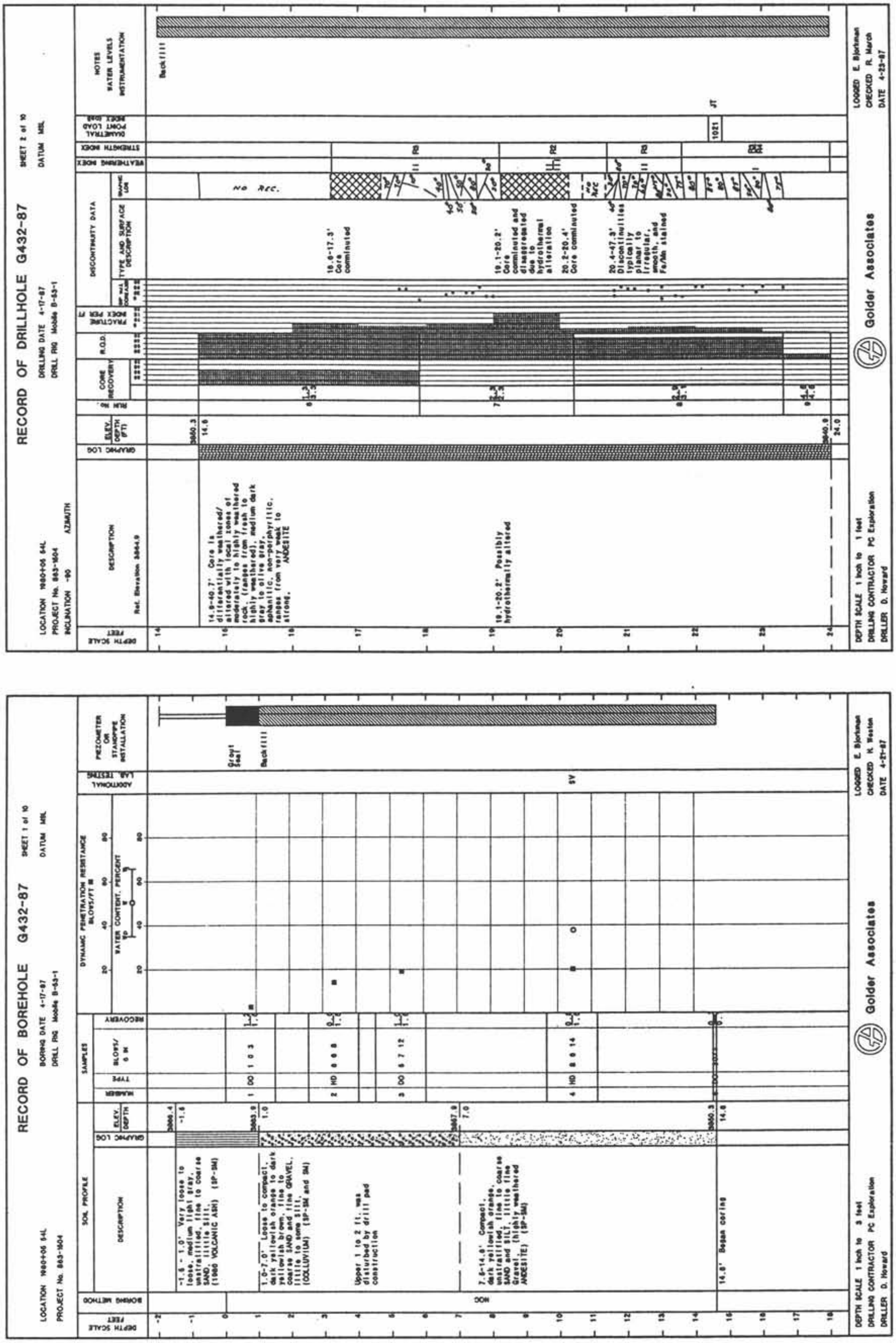


Figure 4. Example of the final borehole logs that were produced at the project site using Golder Associates Inc.'s SEATLOG computer program. The discontinuity graphic log is the only portion that is hand-drafted. An associated database management system allows the retrieval of the log data.

Tephra was found to be nearly ubiquitous in the project area. Tephra layers have been recognized above, below, and within the glacial deposits and form a significant portion of the fines in the colluvium. Quaternary lahars were not mapped within the alignment corridor; however, the Pine Creek lahar (Mullineaux and Crandell, 1981) was noted in the valley bottom near Camp Baker. Ten major eruptive periods for Mount St. Helens are known over the last 40,000 years, including the 1980 eruption (Mullineaux and Crandell, 1981). The presence of ash beneath the till suggests that there may be even more eruptive periods, although the source of the pre-glacial ash is unknown.

Because of the topographic relief and the weathering environment in this area, the surficial deposits have been subjected to mass movement activities. Colluvial deposits are present throughout much of the area and are particularly extensive on steep slopes such as those near Elk Rock and Hoffstadt Mountain (Figure 2). Various landslide scars and landslide deposits were noted in the general project area. A large (approximately 50 acres) Quaternary landslide deposit is present below the steep southeast face of Hoffstadt Mountain.

Bedrock Geology

Extrusive and intrusive igneous rocks of various ages are exposed within the Mount St. Helens area. Extrusive rocks consist of andesites and pyroclastic rocks in various states of weathering and alteration. Most of these rocks exhibit a mineralogy indicative of very low grade metamorphism. The extrusive rocks dip gently to the southeast and probably form part of the northern limb of the southeast-trending Napavine syncline (Phillips, 1987a,b). Interflow zones, which appear to contain thermally altered soils, have been observed, and permineralized wood has been found adjacent to flow contacts. Diamicts, which probably represent Tertiary lahars, are exposed near the western boundary of the project area and may exist in other areas.

The volcanic pile was intruded by dikes and sills of andesitic and basaltic composition, as well as by gabbro. Gabbro was found exposed along Bear Creek (Figure 2) below an elevation of approximately 2,500 ft and was penetrated in one borehole along the alignment beneath till.

Hydrothermal alteration of the volcanic rocks is probably related to both shallow and more deep-seated intrusive activity proximal to the altered rocks. In the Bear Creek area and probably in other parts of the study area, plutonic rocks intrude older volcanic rocks, and the adjacent volcanics exhibit evidence of hydrothermal alteration. Silicification, disseminated pyrite, calcite veinlets, and zeolite mineralization are considered as evidence of hydrothermal alteration. This alteration may be pervasive throughout the entire rock unit or may be localized along specific discontinuities. Rocks adjacent to hypabyssal intrusions are similarly altered.

All bedrock units in the project area contain discontinuities. In some areas, the orientation, spacing, and persistence of the discontinuities appear to be related to rock type. In some intrusive rocks, cooling joints were formed perpendicular to the cooling surface; these joints terminate at the boundary with the host rock. In the Hoffstadt Mountain area, the number of discontinuities was observed to be generally higher in the andesite dikes and sills than in the tuff and agglomerate units. However, discontinuities tended to be more persistent (up to 50 ft in length) in the tuff and agglomerate than those in the andesite. Although local areas (Cow Creek and Elk Rock) have fairly well defined discontinuity sets, the orientation of discontinuities is not consistent over the project area. In general, the majority of the discontinuities are steeply dipping. Mount St. Helens is in a tectonic environment which can be characterized by horizontal compression associated with plate motion. Seismicity following the May 18, 1980, eruption had a different spatial distribution than that occurring before the eruption. Prior to the eruption, earthquakes were confined to a small area in the shallow crust beneath the north side of the volcano. After the eruption seismic events occurred at levels as deep as 10 mi and were distributed both north and south from the mountain. These more recent seismic events are consistent with a strike-slip seismic zone trending northwest from the mountain (Weaver et al., 1981). Cataclastic rocks were found along the highway alignment, in places coinciding with lineaments. Evaluation of fault activity and seismic hazards in the project area was beyond the scope of this investigation.

GEOLOGIC UNITS

For mapping purposes, the surficial geologic units were divided into the following informal units: 1980 tephra, topsoil, colluvium/Recent volcanic ash/talus, alluvium, landslide deposits, Quaternary tephra, till, lacustrine deposits, peat, and glacial outwash. The bedrock was divided into the following informal lithodemic units: basalt, gabbro, andesite, and pyroclastic rocks. The definition of pyroclastic rocks and the nomenclature is based on the International Union of Geological Sciences (IUGS) system (Schmid, 1981). The basalt and some of the andesites are considered shallow intrusive rocks.

Surficial Geologic Units

1980 Tephra

Ash and lapilli from the 1980 eruptions of Mount St. Helens discontinuously mantle the ground surface in most of the project area. The tephra consists predominantly of very loose, unconsolidated, gray, sand-size ash. Between Hoffstadt Mountain and Cow Creek, ash is present at only a few locations along the alignment and is generally less than 1 in. thick. East of Cow Creek, the maximum thickness of ash gradually increases to about 3 ft near Elk Rock. Between Elk Rock and the eastern end of the project at Coldwater Lake, the ash is as much as 6.5 ft thick locally. The thicker ash deposits have accumulated in small gullies and depressions mainly by colluvial and alluvial processes.

Topsoil

Topsoil refers to the organic litter layer and the A and B horizons. In this investigation, the C horizon materials have been included with the parent material. Using this definition, topsoil thickness varies from less than 1 in. to 1 ft in most of the project area. The topsoil was thickest (as much as 2 ft) in the gently sloping area between Cow Creek and Hoffstadt Creek. The root zone for the forest vegetation extends to a depth of approximately 6 ft; however, the thickness of this zone is varied and was not specifically defined. The topsoil is composed of very soft, dusky yellowish brown, unstratified, organic rich silt, 5 to 30 percent of fine to coarse sand, with charcoal and roots.

Colluvium/Recent Volcanic Ash/Talus

Colluvial materials are those materials carried downslope primarily under the influence of gravity, although other slope processes may also be involved. This unit underlies topsoil throughout most of the project area. When the term colluvium is used, it refers to a material composed of gravel- to boulder-size rock fragments in a matrix of silt and sand. This matrix is dominated by volcanic ash, which is considered to be Recent or late Pleistocene in age based on its stratigraphic position and degree of oxidation and weathering. Where the quantity of ash greatly exceeds the amount of gravel and coarser fragments, the deposit is termed Recent volcanic ash. Materials labeled Recent volcanic ash are restricted to post-glacial ash which is older than the 1980 tephra unit. Ash units below till or between till units are designated Quaternary tephra. Where the gravel- to boulder-size rock fragments are predominantly angular to subangular and lack significant fine matrix, the deposit is termed talus.

Colluvium is typically from 1 to 15 ft thick and is found on moderate to steep slopes and in gullies and ravines throughout the project area. On the basis of the borehole results, the colluvium is thickest adjacent to the steep slopes of Hoffstadt Mountain (19 to 48 ft) and Elk Rock (>31 ft).

The Holocene volcanic ash unit consists primarily of reworked material that has been moved downslope by alluvial and colluvial processes. The reworked volcanic ash is commonly overlain by topsoil and was found above, below, and within deposits of colluvium. The reworked volcanic ash is thickest between Hoffstadt Mountain and Geotech Ridge, where it is commonly 2 to 15 ft thick and locally as much as 21 ft thick. East of Geotech Ridge, the reworked volcanic ash is present locally and is less than 4 ft thick.

Alluvium

The numerous creeks and tributary drainages of the area contain alluvium composed of gravel and sand, with 12 percent of silt. This material has been largely derived from the colluvium and till units. Alluvium in the North Fork Toutle River is mostly derived from the 1980 mudflow and debris avalanche deposits from Mount St. Helens.

Landslide Deposits

Landslide deposits have been mapped at the base of Hoffstadt Mountain, at Hoffstadt Creek, and at Maratta Creek. The geometry and stability of the landslide deposits at the base of Hoffstadt Mountain were investigated because the alignment crosses these deposits (discussed in the section on Potential Geologic Hazards); the other landslide deposits are below the alignment and do not pose potential hazards to the alignment.

The landslide deposits at the base of Hoffstadt Mountain are typically composed of unstratified, fine to coarse sand, some clayey silt, some fine to coarse gravel, with 5 to 40 percent subangular to subrounded, matrix-supported cobbles and boulders as much as 2 ft in diameter, with rare boulders as large as 10 ft in diameter. Near the steep cliffs of Hoffstadt Mountain, the deposits contain a higher percentage of cobbles, boulders, and gravel, and near the toe of the deposits adjacent to Hoffstadt Creek, the material is mostly sand and clayey silt. Borehole and seismic refraction data indicate that the landslide deposits are from 40 to 170 ft thick and overlie weathered tuff and tuff.

Quaternary Tephra

Quaternary tephra includes any tephra deposit beneath the Recent volcanic ash and above the bedrock. All of these materials are in the ash-size range. Quaternary tephra layers of at least two different ages were encountered in the drilling investigation. Between Hoffstadt Mountain and Cow Creek and in the Maratta Creek area, a Quaternary tephra layer was penetrated beneath colluvium and Holocene volcanic ash but above the till. Less than 6 ft of that tephra is present near Maratta Creek, but from 13.5 to 17.5 ft of the tephra is present between Hoffstadt Mountain and Cow Creek. In the Bear Creek area, a test pit exposed a Quaternary tephra layer that was as much as 5 ft thick between two till layers. This was the only area where a volcanic ash layer older than the upper till unit was found. Based on its stratigraphic position, this ash layer is considered to be between Wingate Hill and Hayden Creek in age.

Till

Till, which was deposited by alpine glaciation, underlies topsoil, colluvium, and Recent volcanic ash in many places within the project area. The till is widespread between the hill adjacent to the N-1 debris dam and the west side of Elk Rock. Till was not found west of the area of the N-1 debris dam. Between Elk Rock and the east side of the unnamed creek between Elk Creek and Maratta Creek, till was not found along the alignment, although evidence of glaciation was present. Glacial striations are present on a bedrock outcrop at elevation 3,728 ft along the east side of Elk Creek, and granitic boulders, which are believed to have been glacially transported from the Spirit Lake pluton about 5 mi east of the project area, were found at elevation 3,885 ft near the headwaters of the unnamed creek. Between the unnamed creek and the eastern end of the project area, thin deposits of till are present locally, although granitic boulders are widespread in the colluvium. Because the bedrock exposures on Elk Rock (elevation 4,200 ft) do not show evidence of glaciation, the glacial striations along Elk Creek and the granitic boulders in the headwaters of the unnamed creek are believed to have been near the upper surface of the glacial ice mass, a position where glacial scouring would have been more prevalent than deposition. The local occurrence of till east of Elk Rock and the widespread occurrence of glacially transported boulders suggest that the glacial ice mass tended to scour rather than deposit till in this area. The thin drift (<1 to 21 ft thick) that was deposited would have been susceptible to weathering and colluvial processes, which could account for the widespread colluvium containing granitic boulders.

Although till of two different ages was recognized in the project area, the till unit was not subdivided for the purposes of mapping and borehole logging on this project. West of Elk Rock, the till is typically composed of gravel and sand, some silt or clayey silt, with as much as 20 percent of the unit consisting of cobbles and boulders to 1.5 ft in diameter. The fines fraction is typically between 20 and 40 percent in this unit. In the Bear Creek area, the till contains significant quantities of the local hydrothermally altered bedrock. Prominent end moraines are present between Cow Creek and Geotech Ridge.

Lacustrine Deposits

Rhythmically layered sand and silt deposits of lacustrine origin are present in the Cow Creek, Hoffstadt Creek, and Maratta Creek areas. These deposits range from 1 to 13 ft thick where penetrated by the boreholes and were generally found within till or associated with glacial outwash deposits. These materials were deposited in proglacial lakes near the margins of a glacial ice mass and show evidence of subsequent deformation by glacial ice.

Peat

Peat from 5.5 to 8.5 ft thick was found interlayered with lacustrine deposits between Cow Creek and Hoffstadt Creek, and sandy peat within glacial till was found in one borehole near Geotech Ridge. This relationship indicates that the proglacial lakes accumulated both organic materials and sediment on a periodic basis.

Glacial Outwash

Glacial outwash deposits comprised of moderately well sorted sand and gravel were found between Hoffstadt Mountain and Geotech Ridge and in the Maratta Creek area. Except near Hoffstadt Mountain, where the outwash overlies bedrock, the deposits are usually associated with the lacustrine deposits or found within till. Outwash thicknesses range from less than 1 to 7.5 ft in the boreholes. An interlayered sequence of outwash deposits and lacustrine sediments is exposed along the south side of Hoffstadt Creek near the alignment.

Bedrock Geologic Units

Basalt

Basalt was found in the boreholes near Hoffstadt Mountain and Geotech Ridge and during mapping in the Bear Creek area. Field relationships suggest that the basalt occurs as dikes or sills within the older volcanic rocks. The lack of significant alteration associated with the basalt suggests that it probably represents the latest phase of shallow intrusive activity in the project area.

Gabbro

Gabbro was found exposed along Bear Creek below an elevation of approximately 2,500 ft and was encountered in a nearby borehole along the alignment beneath till. A weathering horizon in which the gabbro has been converted to grus is present in this area. The gabbro is considered to be younger than the adjacent volcanic rocks, and hydrothermal alteration of the volcanic rocks in the area is probably partly related to the intrusion of the gabbro. Although more mafic than the rocks of the Spirit Lake pluton, which is located about 7 mi east of the Bear Creek area, the gabbro may be related to or coeval with this pluton, which has a K-Ar radiometric age of 20.6 to 22 Ma (Phillips, 1987b).

Andesite

Those volcanic rocks with predominantly plagioclase phenocrysts and a color index of less than approximately 50 were considered andesites for the purposes of this project. Mafic phenocrysts are predominantly hornblende. Andesite was encountered throughout the project area. Contact relationships with other bedrock units in the boreholes and in exposures at Hoffstadt Mountain, together with the aphyric nature of some of the andesite, suggest that while most of the andesite occurs as flow units, some of it probably represents dikes and sills. A weathering horizon, which varies from 1 to 29 ft thick, is present at many locations; the weathered andesite generally has a lower rock strength than the fresh rock.

Hydrothermally altered andesite is encountered locally between Hoffstadt Creek and the Maratta Creek area. Hydrothermal alteration has produced clay minerals from the feldspars and mafic minerals and deposited iron-manganese minerals along discontinuity surfaces.

This alteration has generally resulted in a loss of rock strength, although the strength of local areas of silicification is not appreciably lower than that of the fresh rock.

Andesite flows between Hoffstadt Mountain and Elk Rock, and locally in the Maratta Creek area, contain subangular to subrounded andesite clasts in an andesite matrix. These rocks would be classified as lapilli tuffs or agglomerates in the IUGS classification system (Schmid, 1981). However, these rocks are identified as andesite autobreccias because their texture suggests that the fluid portion of the flow incorporated solidified flow crust. It is in places difficult to distinguish the andesite autobreccia from the andesite, especially where the matrix and the clasts are the same color.

In the Geotech Ridge area, two boreholes penetrated 1.4 ft and 12.7 ft, respectively, of protomylonite within the andesite. This rock is the result of tectonic movements along faults within the andesite unit. Throughout the project area, boreholes encountered cataclastic rock produced by shears or faults within the andesite. The evidence of tectonic deformation included clay fillings or slickensides along discontinuities and brecciated rock fabrics (Higgins, 1971). The orientation of most of these faults is unknown. The association of cataclastic rock in some boreholes adjacent to linear drainages suggests that at least some of the northeast- and northwest-trending drainages in the project area are structurally controlled.

Between Geotech Ridge and the Bear Creek area, several quarry exposures were found where porphyritic andesite boulders, which appeared to have been competent, strong rock when excavated, were decomposing into sand- and gravel-size clasts. The surfaces of the boulders that remain wet for long periods appeared to be more susceptible to degradation than the surfaces that were better drained. Because there is no visible difference between the andesite that is susceptible to this degradation and competent andesite, samples were tested for Los Angeles (L.A.) abrasion, Washington degradation, and sulfate soundness, and mineralogy of the samples was determined by thin-section petrography and X-ray diffraction.

A sample of the andesite that did not appear to be visibly degrading and had a field strength classification of strong to very strong had a 26.3 percent loss for L.A. abrasion, a Washington degradation value of 3, and 16 percent loss for sulfate soundness. While the L.A. abrasion value indicates a material which is acceptable for many uses, the degradation and soundness values indicate a lower quality material.

X-ray diffraction results for a sample of the decomposed material indicated the presence of both kaolinite and montmorillonite clays, which are considered to be weathering products. The feldspars are apparently highly susceptible to weathering, and when they weather to clay, the volume expansion that results breaks the rock apart. While it is difficult to visually recognize the andesite with this susceptibility to decomposition, test results appear to be reliable indicators of future performance. The distribution of this andesite in the project area is unknown.

Pyroclastic Rocks

Consolidated pyroclastic rocks that are classified as tuff, lapilli-tuff, lapilli-ash tuff, ash-lapilli tuff, or agglomerate based on the IUGS classification system (Schmid, 1981) are present throughout the project area. In most of the area, these rocks are fresh to slightly weathered and medium strong to strong, although a weathering horizon that varies from less than 1 ft to 27 ft thick is present locally. The weathering process has resulted in an overall decrease in rock strength.

Hydrothermally altered tuff was encountered east of the Bear Creek area. The hydrothermal alteration in the tuffs and its effect on rock strength are similar to those characteristics in the andesite. X-ray diffraction analyses of the tuffs indicated that the rocks were converted to an assemblage of quartz and kaolinite or quartz, feldspar, and montmorillonite. Many Rock Quality Designation (RQD) values of the non-silicified, hydrothermally altered tuffs did not correlate with the strength of the rock mass. In some cores these rocks were unfractured (i.e., 100 percent RQD), even though the rock was extremely weak, broke easily when handled, and was very susceptible to slaking. Because the tuffs weakened by the hydrothermal alteration process exhibited more soil-like than rock-like engineering characteristics, the RQD of these rocks was not a reliable indicator of material competency and durability.

Throughout the project area, boreholes penetrated cataclastic rock produced by shears or faults within the pyroclastic rocks. The evidence for and structural significance of the cataclasis in these rocks is the same as that previously discussed for the andesite.

ENGINEERING GEOLOGY AND DESIGN CONSIDERATIONS

Engineering Geology of the Surficial Deposits

Glacial deposits, colluvium, and volcanic ash form the primary surficial deposits along the alignment. Recognition of the frequent interchange between glacial and ice-marginal conditions along the North Fork Toutle River valley allowed the extent of the various glacial deposits to be predicted between boreholes and provided a framework for interpreting the subsurface information. Peat and lacustrine deposits were confined to areas where lobes of ice had formed proglacial lakes. Glacial outwash was found where ice-marginal lakes were breached and in areas where ice-marginal streams were present.

Till has a lower density where it has been affected by weathering and colluvial processes. Weathering of the till appeared to be dependent, in part, on the presence of shallow ground-water conditions and, in part, on the composition of the predominant clasts. Locally, upper portions of the till have a texture which suggests some modification by colluvial processes. In many places it was difficult to distinguish between weathered till and colluvium.

Engineering Classification

Where test results indicated similar engineering behavior for different geologic units, the units were combined into a single engineering unit to simplify design considerations. Since the completely altered rocks are considered to have soil-like properties, they were evaluated as soil units.

Based on the investigations and test results, the soil units were grouped into three general units for engineering purposes: upper loose soils, till, and completely altered rock. The upper loose soils primarily consist of 1980 tephra, manmade fill, topsoil, peat, Recent/late Quaternary tephra, colluvium, talus, and lacustrine sediments. The till unit represents drift from both Wingate Hill and Hayden Creek glaciations. Completely altered rock generally comprises tuff and andesite, although the parent rock was commonly difficult to determine.

Upper Loose Soils

For engineering purposes these materials were considered to be loose with relatively poor in-place properties. Although no direct strength or compressibility tests were performed on the upper loose soils, they are anticipated to behave as cohesionless soils with moderate strength and low compressibility, based on Standard Penetration Tests (SPT) and classification test results. It was estimated that the materials have an *in situ* friction angle of 30° to 36° with zero cohesion, with the angles in 1980 ash, talus, and other types of colluvium being somewhat higher because of particle angularity. For design purposes, a friction angle of 32° was assumed.

Till

Till is the principal soil unit along the western half of the alignment. It comprises compact to dense sands with varying amounts of fines, gravels, cobbles, and boulders. The fine contents are generally in the 20 to 40 percent range, and some of the deposits are graded from silt to clayey silt. The thickness of the till is generally less than 25 ft. Where thicker tills were encountered, density increased below a depth of approximately 25 ft. In some areas this change in the density of the till appears to be related to weathering and ground-water conditions, whereas in other areas the change was probably related to differences between Hayden Creek and Wingate Hill drift. The upper, looser tills are primarily found in the Bear Creek area. East of Geotech Ridge the till is locally sufficiently dense to produce 2-3 ft sticks of unbroken core as recovered in drilling operations.

It was not feasible to obtain adequate undisturbed samples of the till for laboratory strength and compressibility testing. To assess *in situ* strengths, a slope inventory of both natural slopes and logging road cut slopes was completed. Back analyses of existing slopes was conducted using CSLOPE, a Golder Associates Inc. microcomputer slope stability analysis program. Based on these analyses and engineering judgment, 37° was selected as the effective stress friction angle and 200 psf was selected for the effective stress cohesion intercept for the upper tills. For the lower, denser tills, 500 psf was selected as the effective stress cohesion intercept.

Completely Altered Rock

Because of the nature of the weathering and alteration processes, completely altered rock can be expected to exhibit a wide range of strengths varying from a dense soil-like behavior to a weak rock behavior. Triaxial strength testing on relatively undisturbed samples obtained by coring indicated a friction angle of approximately 38° and a cohesion of approximately 400-800 psf. Because of the variability of the material, 36° was selected as the effective stress friction angle, and 400 psf was selected as the effective stress cohesion intercept. These values reflect the opinion that the altered rocks probably have a higher cohesion than the upper tills because of relict rock structure but a somewhat lower friction angle because of their relatively lower density and finer gradation.

Rock and Soil Cut Slopes

Using the shear strength parameters outlined above, stability calculations were undertaken to determine acceptable cut slope configurations. Typical design recommendations for soil and rock are shown in Table 1. Rock cut slope design is discussed in a subsequent section.

Embankments

The stability of embankment fills depends on the stability of the foundation materials, seepage conditions, the shear strength parameters of the fill materials, and the geometry of the fill. In general, analyses indicated that the foundation soils would provide a stable base for the fills provided that appropriate stripping of loose materials and foundation terracing were undertaken, as required in normal WSDOT construction practice. In terms of the materials available for use as fill, however, it was determined that the high fines content of the soil materials and of the completely altered rock made these materials unsuitable for placement in fill sections.

Virtually all of the upper loose soils, the till, and the completely altered rock were found to have high fines contents because of the effects of weathering, glacial abrasion, volcanic ash content, or the effect of

Table 1. Design cut slope recommendations in soil and rock. Cut slope height defined from the toe to the crest of the slope; recommendations are subject to modification to suit specific slope conditions

Material	Cut slope height (ft)	Maximum allowable cut slope angle
Upper tills	< 30	1H:1V
	30 - 50	1.5H:IV
	50 - 120	2H:1V
Lower tills	< 50	1H:1V
	50 - 120	1.5H:1V
Completely altered rock	< 50	1H:1V
	50 - 100	1.5H:1V
Poor quality rock	< 50	1H:1V
	50 - 100	1.5H:1V
Fair quality rock	> 50	0.25H:1V
	< 50	0.50H:1V
Good quality rock	< 50	Vertical
	50 - 100	0.25H:1V

hydrothermal alteration, and they are considered to be moisture-sensitive materials in terms of their compaction characteristics. Because many of these materials also had high in-place moisture contents (well above optimum in some) and recognizing that many of the fills will be placed under rainy conditions, it was concluded that all soils and completely altered rock excavated from cuts along the route must be considered as waste materials.

Materials which can be placed as fill under all weather conditions will be derived both from cut slope sections located in rock along the route and from two major quarry areas located near the U.S. Army Corps of Engineers N-1 debris dam and near Elk Rock, respectively.

Engineering Geology of the Bedrock Units

Bedrock along the alignment is comprised predominantly of layers of agglomerate, tuff, and andesite that were locally intruded by andesite and basalt dikes and sills and in one place by gabbro. The volcanic rocks have been weakly metamorphosed and some have been hydrothermally altered, whereas the intrusive rocks are some of the freshest rocks in the project area. The alteration of the volcanic rocks was probably related to the intrusion of plutonic or hypabyssal rocks and has increased the susceptibility of these rocks to weathering processes. Prior to glacial erosion these rocks were exposed to the weathering environment. The weathering horizon developed on the volcanic rocks is preserved where it was not stripped by erosion. Some interflow permineralized wood and paleosols were also noted.

Table 2. Rock mass classification. No single parameter is definitive, and classification is based on an overall assessment of sections of core

Rock Mass Classification	Good Quality	Fair Quality	Poor Quality
Description:	Fresh to slightly weathered rock, with widely to moderately spaced fractures	Slightly to moderately weathered rock, with closely spaced fractures	Moderately to highly weathered rock with closely to very closely spaced fractures
Classification Parameters			
RQD	> 60%	< 60%	< 60%
Fracture Index (Joints/foot)	< 3	3 to 9	> 9
Joint Friction	> 45	35 to 45	< 35
Weathering Category	I to II	II to III	III to V
Strength Category	R3 to R5	R3 to R5	< R3
Core Recovery	> 90%	> 80%	< 80%
Point Load Strength	400 to 1500	< 800 psi	< 300 psi
Uniaxial Strength	9000 to 33,000	< 18,000 psi	< 7000 psi
Jar Slake Index	> 4	< 4	< 4
Slake Durability Index	> 55%	< 55%	< 55%

Structurally, the rock layers along the alignment dip gently (<15°) to the southeast. Analysis of aerial photographs and images revealed numerous lineaments along the highway corridor. Some boreholes proximal to the lineaments penetrated cataclastic rocks that were produced by fault activity of unknown age. Evidence of tectonic deformation included clay fillings or slickensides along discontinuities and rock fabrics that ranged from fault breccia to fault gouge (Higgins, 1971). Most of the rock cores showed little or no evidence of cataclasis.

Engineering Classification

In general, the bedrock lithodemic units did not have a sufficiently narrow range of engineering properties to be used for engineering design purposes. In order to identify units with better engineering significance, selected areas of the rock mass were classified into one of three engineering rock mass classes: good quality rock, fair quality rock, and poor quality rock, based on broad consideration of ten classification parameters which collectively indicated the overall quality of the rock mass as an engineering material. Table 2 summarizes the parameters and values used to distinguish these engineering units. The choice of parameters and their values was based on review of a number of published classification systems (Barton et al., 1974; Bieniawski, 1974; Hoek and Brown, 1980) that were modified by judgment based on the rock engineering experience of the project team members.

Derivation of Engineering Properties

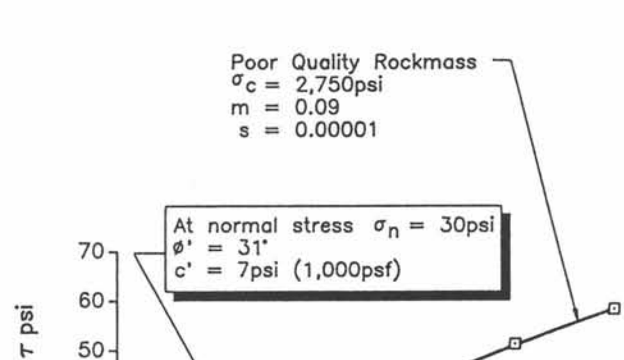
Where exposures were present, the orientation of discontinuities was measured. No evidence was found of widespread joints sets which would consistently control the stability of rock cut slope faces, although such structural control was evident in a few localized areas. For general cut slope design purposes, where clear structural control was lacking, the engineering properties of the *in situ* rock mass were based on the overall properties of the rock mass rather than the particular properties of a specific discontinuity or discontinuities.

Anticipated shear strength properties for each of the engineering rock mass classes were evaluated based on the empirical relationships between rock mass quality and shear strength developed by Hoek and Brown (1980) and reviewed by Hoek (1983). For each rock mass engineering class, non-linear shear strength envelopes were developed based on the Hoek-Brown method, and the values of the shear strength parameters, effective angle of friction, and effective cohesion were selected based on the tangent to the curvilinear shear strength envelope at a normal stress level judged to be appropriate for the depths of the anticipated cut slopes. A typical shear strength envelope for poor quality rock resulting from this procedure is shown in Figure 5.

Values of mass modulus of deformation were selected based on the work of Serafim and Pereira (1983) and unpublished work by Hoek correlating rock mass quality with the modulus of deformation. Evaluation of degradability was based on the results of slake durability testing and on the work of Franklin and Chandra (1972).

Poor Quality Rock Mass

Throughout the alignment the areal distribution of poor quality rock was largely controlled by the presence of zones of hydrothermal alteration. A secondary factor was the presence of weathered horizons. The distinction between hydrothermal alteration and weathering was made, where possible, because hydrothermally altered zones generally have varied engineering properties both laterally and vertically, and commonly have unpredictable alteration boundaries. Weathering horizons tend to be near-surface and show some lateral variation, and rock quality in them generally improves with depth. Hydrothermal activity was especially predominant proximal to the gabbro body exposed in the Bear Creek area. Other zones of hydrothermal alteration, for ex-



Envelope based on Hoek–Brown failure criteria:

$$\sigma_1 = \sigma_3 + \sqrt{m\sigma_c\sigma_3 + s\sigma_c^2}$$

Figure 5. Interpreted shear strength envelope for the poor-quality rock mass classification. Similar envelopes were prepared for good- and fair-quality rock mass classifications for each of the alignment segments.

ample, in the Elk Creek area, were not demonstrably associated with intrusive activity; however, the subsurface geometry of the plutons is unknown. Completely altered rock with few vestiges of the structure of the former rock was considered a soil and was described in a previous section.

Poor-quality rock is moderately to completely weathered (the ISRM designation does not distinguish between weathering and alteration [Brown, 1981]) and broken by closely to very closely spaced fractures which cause the rock mass to behave in a manner that is intermediate between a weak rock and a soil. The relict structure of the rock will cause the material to dilate during shear at low normal stresses, although this behavior will cease at higher normal stresses as shear failure will pass through the material regardless of the relict structure. As shown in Figure 5, this behavior gives rise to a curvilinear shear strength envelope. Shear strength parameters selected for engineering purposes, in this example, are an effective angle of friction of 31° and an effective cohesion of 1,000 psf. Curves similar to those shown in Figure 5 were constructed for fair and good quality rock masses for each of the alignment segments.

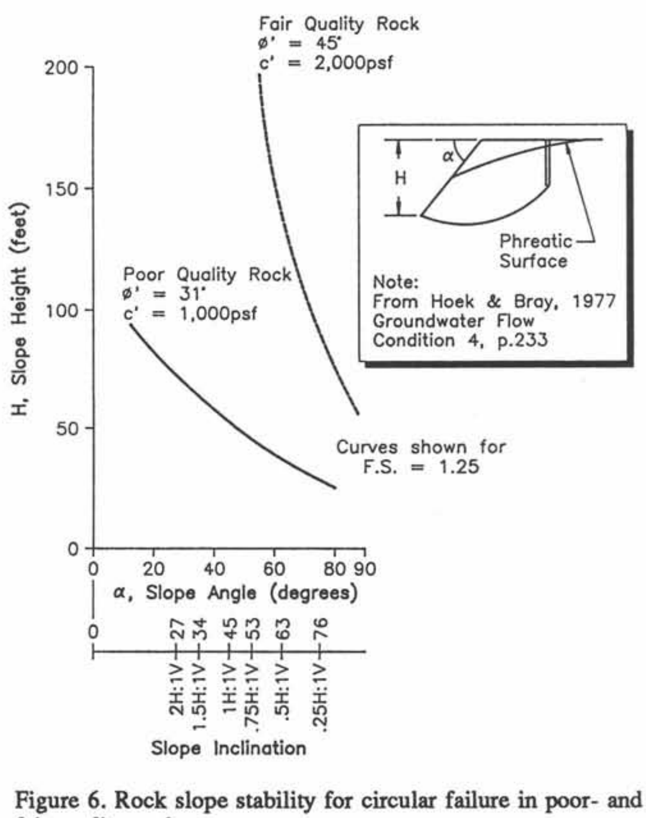


Figure 6. Rock slope stability for circular failure in poor- and fair-quality rock masses.

Fair Quality Rock Mass

The designation as fair quality rock resulted from both fracture intensity and loss of intact strength because of weathering and/or slight to moderate hydrothermal alteration. In general, fair quality rock refers to a rock mass comprising slightly to moderately weathered rock material with closely spaced fractures. This rock is generally more broken than the good quality rock mass, allowing individual fragments to more readily rotate and translate. An effective angle of friction of 45° and an effective cohesion of 2,000 psf were selected based on estimated shear strength envelopes similar to the one shown in Figure 5.

Good Quality Rock Mass

Rock in this category is relatively fresh, and any metamorphism or hydrothermal alteration does not materially affect rock strength. This rock is relatively fracture free and has RQD values greater than 60 percent. An effective angle of friction of 55° and an effective cohesion of 6,000 psf were selected based on estimated shear strength envelopes similar to the one shown in Figure 5.

Rock Cut Slope Design

The first stage in the design of cut slopes in rock was to determine if joints or other discontinuities in the rock mass would control the slope angle. These kinematic analyses were based on structural data collected from surface mapping and logging of oriented core and were presented in the form of stereonets. If a structural control was identified for the cut face, then either the design was modified to accommodate the structure or remedial support was recommended to stabilize the adverse structure. This support most commonly consisted of tensioned and grouted rock bolts or untensioned, grouted rock dowels.

If a structural control was not identified, which was the most common situation, then the rock cut was designed on the basis of rock quality. Based on the shear strength parameters given above, the stability of cut slopes in rock was analyzed based on limit equilibrium techniques applied to a partially saturated slope containing a water-filled tension crack behind the crest of the slope, as presented in Hoek and Bray (1977) and shown in Figure 6. Allowable slope angles with a factor of safety of 1.25 vary as a function of slope height and rock mass quality. Curves are not shown for a good quality rock mass, as the analyses indicated that vertical slopes would maintain overall stability in these materials for all slope heights of practical interest to the project.

Recommendations for rock slope designs included slope angle and requirements for benching, rockfall control, ditch width, drainage, and remedial support.

Rock slopes as much as 100 ft in height were designed at inclinations ranging from 1H to 1V (45°) to 0.25H to 1V (76°) as uniform slopes without benches. Benching was recommended only for those cuts with contrasting material strengths, such as till over fresh rock or weathered rock over fresh rock, which required a compound slope design. Where the overlying material exceeded 20 ft in thickness, a minimum 15-ft-wide bench was recommended at the slope design change.

In general, ditch widths were designed as the primary control of rockfall. Where the roadway width was limited or the potential of rockfall was higher than normal, additional remedial controls were recommended. These included galvanized, double twisted, hexagonal wire mesh draped over the slope or Jersey barriers erected at the shoulder of the roadway.

As rock cuts were being excavated, it was recommended that newly exposed faces be inspected and uncased drainage holes be drilled into areas of significant seepage. Upon completion of all rock cuts in excess of 30 ft in height, it was recommended that uncased drainage holes be drilled on 25-ft centers from the ditch to a depth equal to one-half the slope height or to a maximum depth of 30 ft.

Based on the varied quality of the bedrock throughout the project, it was recommended that all cuts be inspected as excavation proceeded to verify slope design and to determine local support requirements. Depending on actual conditions encountered, remedial support could include rock bolts or shotcrete, and for each design segment, nominal quantities of both were recommended for inclusion in the contract bid items.

Retaining Walls and Bridges

The preliminary highway design called for approximately 7,300 ft of wall to retain both cut slopes and fills. Throughout the geotechnical investigation, steeper cut slopes and alignment shifts were evaluated as alternatives to cut retaining walls. As a result, only a few cut wall locations remain. The preferred retaining system at these sites consists of soil nailing, a relatively new concept which involves pattern bolting of the soil slope and placement of a shotcrete facing. The advantages of this system include reduced cost and the ability to build the wall in lifts as the slope is excavated without the need for temporary support.

In many areas, embankments could not be designed to "catch" on the steep natural slopes, and walls were required to retain the fill. Geotechnical design recommendations were provided for cantilever, gravity, and tieback walls. A number of proprietary walls were also considered appropriate at specific sites.

Foundation designs were required for seven major bridges. Geotechnical conditions ranged from soft, compressible Recent volcanic ash on flat to gently sloping terrain to talus and bedrock on very steeply sloping side slopes. For each bridge pier, a preferred design and, if appropriate, an alternate design were provided to WSDOT. The designs included spread footings, piles, and drilled or socketed shafts. The last were considered where steep slopes would necessitate major excavation to locate a pier footing.

Potential Geologic Hazards

Geologic hazards along the project corridor consist of mass movement activity, snow avalanches, volcanic activity, and seismic hazards. Assessment of mass movement and snow avalanche activity included investigating the evidence of past mass movement activity and suggesting areas with the potential for future activity. The project area is seismically active (Weaver et al., 1981) and proximal to Mount St. Helens; however, assessment of seismic and volcanic hazards was outside the scope of the project.

Rockfall was initially a concern where the alignment was proximal to the face of Hoffstadt Mountain (Figure 2). Blocks of rock which range in size to as much as 10 ft in diameter are present in colluvial cones at the base of gullies along the mountain front. The current alignment is sufficiently distant from the free face to avoid any rockfall hazard.

The colluvial deposits at the base of Hoffstadt Mountain overlie deposits which, on the basis of geomorphic expression and evidence from borings and test pits, are interpreted as resulting from landslide activity (Figure 7). These landslide deposits are of particular significance because, without major re-routing, SR-504 must cross them. Evaluation of whether these deposits would be subject to future mobilization during the expected lifetime of the highway was undertaken. Reconnaissance geologic mapping, excavation of test pits, and seismic refraction techniques were used to assess the depth, areal extent, composition, and inferred processes of slope failure of the landslide. Some subsurface information was also available from boreholes located and drilled as part of the alignment drilling program, and limited additional drilling was undertaken within the slide area. Based on these data, a series of slope stability analyses was conducted to assess the potential effect of highway fill construction on the stability of the slide.

On the basis of the results of these analyses and the lack of any geomorphic evidence of recent slide activity, the landslide deposits are considered to be stable in terms of their effect on highway design and construction. Nevertheless, an inclinometer was placed in one of the exploration boreholes in order to confirm that no movements are occurring.

Hoffstadt Creek has large amounts of logging/blowdown debris and volcanic ash entrained in its channel. Much of this material was mobilized during the December storm of 1980 and could be mobilized again during low-frequency, high-intensity runoff events. In the

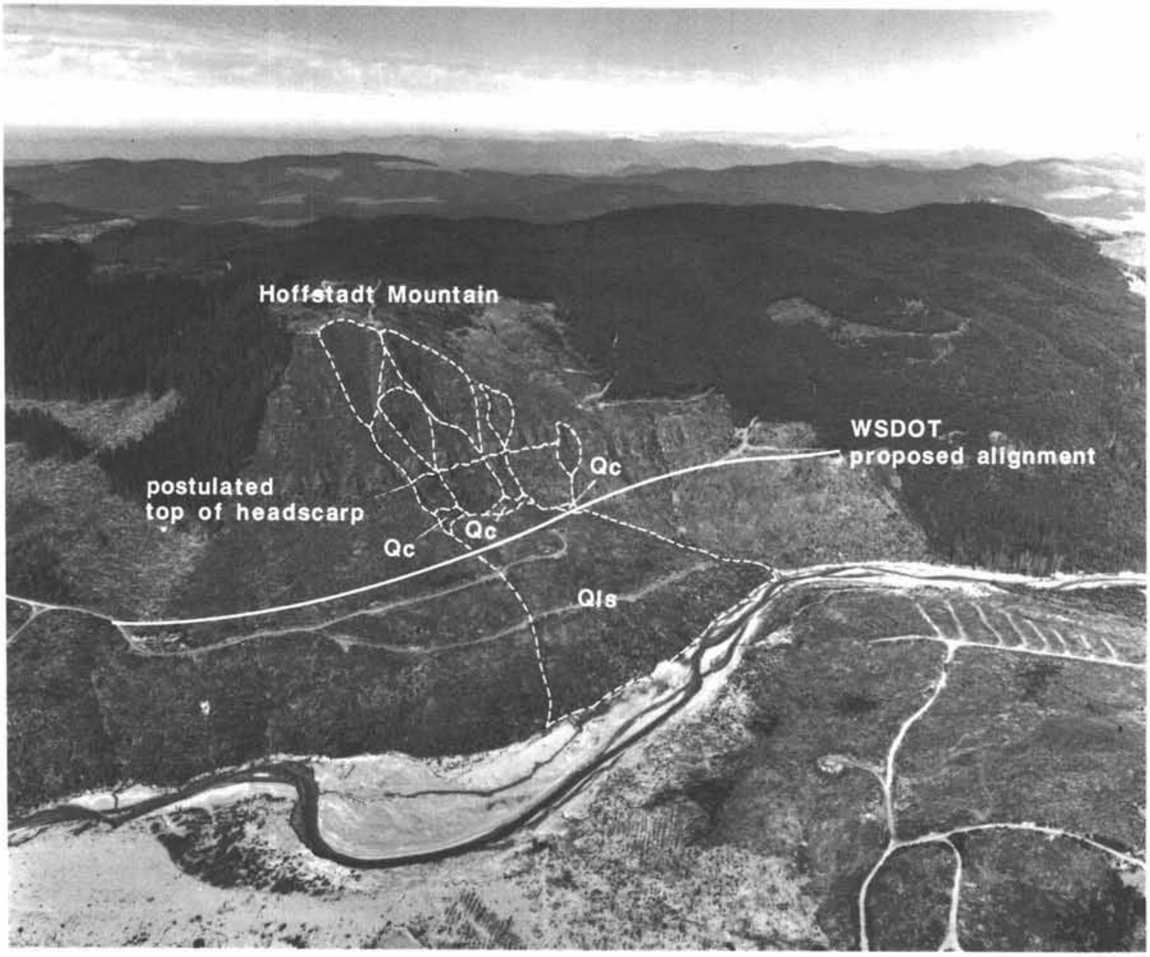


Figure 7. The highway alignment crosses a large Quaternary landslide deposit (Qls) that is overlain by colluvium and talus (Qc) derived from Hoffstadt Mountain.

upper portion of Hoffstadt Creek, the debris formed a temporary dam which impounded water. When this dam broke, the discharge of the creek almost certainly rose very rapidly, as did the capacity of the stream to transport boulders and large logs. Such debris torrents could potentially compromise the integrity of bridge piers, should they be located in the channel area used by the torrent.

Hoffstadt Creek could also be dammed if the active landslide which was mapped approximately a mile northeast of the west abutment of the Hoffstadt Creek bridge were to fail rapidly and enter the channel area. This landslide has a mapped area of approximately 6 acres and an unknown thickness. The head scarp is in andesite bedrock, and much of the slide debris consists of relatively intact andesite blocks, suggesting that bedrock as well as surficial materials are involved in this slide. Ground cracking and vertical offset of a post-1952 logging road by as much as 3 ft are evidence that the slide continues to be active. Multiple scarps are present, both above and below the logging road. Given the proper pore water pressure and/or seismic acceleration conditions, it is credible that this slide could fail into Hoffstadt Creek as a single mass. The blockage could form a dam of water and debris which could fail rapidly and potentially affect bridge piers, if they were located at a sufficiently low elevation in the creek valley.

The head of the Bear Creek valley has appropriate slope angles and catchment area to form snow avalanches, if snow loading is sufficient. The part of the basin above an elevation of approximately 3,600 ft has an average slope of about 27°. The average slope above the road, which is at an elevation of approximately 3,300 ft in this area, is about 19°. A terrace, which lies approximately 100 ft above the proposed route of SR-504, represents the upper limit of Wingate Hill drift and is sufficient to catch minor avalanche events. However, major events could cover the highway.

ACKNOWLEDGMENTS

Discussions during a 1-day field trip with William E. Scott, Richard B. Waitt, and Richard Janda of the U.S. Geological Survey, discussions with Steve Lowell and Jim Luker of WSDOT and with Anthony Irving and

Thomas Dunne at the University of Washington, and discussions and a field trip with William M. Phillips of the Washington Division of Geology and Earth Resources aided the interpretation of the stratigraphy and the geologic framework of the project area. Project participants included many colleagues from Golder Associates Inc., whose input and assistance are acknowledged.

REFERENCES

American Society for Testing and Materials [ASTM], 1985, *Annual Book of ASTM Standards*: American Society for Testing and Materials, Vol. 4, 912 p.

Barton, N.; Lien, R.; and Lunde, J., 1974, Engineering classification of rock masses for the design of tunnel support: *Rock Mechanics*, Vol. 6, No. 4, pp. 189-236.

Bieniawski, Z. T., 1974, *Geomechanics Classification of Rock Masses and its Application to Tunneling*: Proceedings of the 3rd International Congress on Rock Mechanics, International Society of Rock Mechanics, Vol. IIA, pp. 27-32.

Brown, E. T. (editor), 1981, *Rock Characterization Testing and Monitoring - ISRM Suggested Methods*: Pergamon Press, Oxford, UK, 211 p.

Crandell, D. R. and Miller, R. D., 1974, *Quaternary Stratigraphy and Extent of Glaciation in the Mount Rainier Region, Washington*: U.S. Geological Survey Professional Paper 847, 59 p.

Fiske, R. S.; Hopson, C. A.; and Waters, A. C., 1963, *Geology of Mount Rainier National Park, Washington*: U.S. Geological Survey Professional Paper 444, 93 p.

Franklin, J. A. and Chandra, R., 1972, *The slake durability test:* International Journal of Rock Mechanics and Mining Sciences, Vol. 9, pp. 325-341.

Golder Associates Inc., 1987, *Technical Procedure for Field Identification of Soil*, TP-1.2-6, Approved 1-27-87: Golder Associates Inc., Redmond, WA, 13 p.

Higgins, M. W., 1971, *Cataclastic Rocks*: U.S. Geological Survey Professional Paper 687, 97 p.

Hoek, E., 1983, Strength of jointed rock masses: *Geotechnique*, Vol. 33, No. 3, pp. 187-223.

Hoek, E. and Bray, J. W., 1977, *Rock Slope Engineering*: Institute of Mining and Metallurgy, London, UK, 402 p.

Hoek, E. and Brown, E. T., 1980, *Underground Excavations in Rock*: Institute of Mining and Metallurgy, London, UK, 527 p.

Kreig, R. A. and Reger, R. D., 1976, Preconstruction terrain evaluation for the Trans-Alaska Pipeline Project. In Coates, D. R. (editor), *Geomorphology and Engineering*: Dowden, Hutchinson and Ross, Inc., New York, NY, pp. 55-76.

Mullineaux, D. R. and Crandell, D. R., 1981, The eruptive history of Mount St. Helens. In Lipman, P. W. and D. R. Mullineaux (editors), *The 1980 Eruptions of Mount St. Helens, Washington*: U.S. Geological Survey Professional Paper 1250, pp. 3-15.

Phillips, W. M., 1987a, personal communication (with K. Moser), Washington Department of Natural Resources, Division of Geology and Earth Resources, April 6, 1987.

Phillips, W. M., 1987b, *Geologic Map of the Mount St. Helens Quadrangle, Washington*: Washington Division of Geology and Earth Resources Open File Report 87-4, Olympia, WA, 59 p., 1 plate, scale 1:100,000.

Schmid, R. 1981, Descriptive nomenclature and classification of pyroclastic deposits and fragments; Recommendations of the IUGS Subcommission on the Systematics of Igneous Rocks: *Geology*, Vol. 9, pp. 41-43.

Serafim, J. L. and Pereira, J. P., 1983, Consideration of the geomechanics classification of Bieniawski: *Proceedings of the International Symposium on Engineering Geology and Underground Construction*, Lisbon, Portugal, pp. II33-44.

Vita, C. L., 1984, Route geotechnical characterization and analysis: *Journal of Geotechnical Engineering*, Vol. 110, No. 10, pp. 1715-1734.

Weaver, C. S.; Grant, W. C.; Malone, S. D.; and Endo, E.T., 1981, Post-May 18 seismicity; Volcanic and tectonic implications. In Lipman, P. W. and D. R. Mullineaux (editors), *The 1980 Eruptions of Mount St. Helens, Washington*: U.S. Geological Survey Professional Paper 1250, pp. 109-121.