SEATTLE LANDSLIDE STUDY

Volume 1
Geotechnical Report
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EXECUTIVE SUMMARY

In light of the devastating landslides of the winter of 1996/97, Seattle Public Utilities (SPU) created a new landslide policy and undertook a program of landslide awareness and mitigation. One part of that program is an evaluation of landsliding in Seattle.

Landsliding in Seattle is caused by a combination of geologic conditions, steep topography, concentration of rainfall in the winter months, and the influences of an urban environment. The geologic conditions are primarily a legacy of repeated glacial incursions during the past 2 million years. The topography is the result of mass wasting in the past 13,000 years, since the disappearance of the last glacial ice. Although Seattle does not receive a large volume of precipitation, concentrations of rainfall in the winter months can be significant enough to saturate the glacial and colluvial soils. Overlying this natural setting is the human pattern of residential, commercial, and industrial development, and the infrastructure that binds it together.

Seattle is unique in that it has a rich record of landslides that dates back as far as 1890. A database was created with 1,326 landslides. Information in the database includes the location, date, type of landslide, geologic conditions, and possible contributing factors. The landslides were also plotted on maps using Arcview Geographic Information System (GIS) technology, and then the locations were field checked to reduce the error. In spite of this verification process, some of the locations are still approximate. The database and GIS maps are useful tools for City of Seattle (City) departments.

Four landslide types were recognized from the data amassed in the study:

(1) High Bluff Peeloff – blockfalls of soil from the high bluffs that are found primarily along the cliffs of Puget Sound.

(2) Groundwater Blowout –catastrophic groundwater/soil bursts caused by the buildup of groundwater pressures along the contact of pervious/impervious soil units.

(3) Deep-Seated Landslides – deep, rotational or translational sliding and slumping caused by groundwater pressures within a hillside.

(4) Shallow Colluvial (Skin Slide) – shallow rapid sliding of the outer rind of a hillside slope, sometimes also resulting in a debris flow.

The most common type of slide is the shallow colluvial slide, particularly in response to an intense, short-duration storm. The largest and commonly most destructive are deep-seated landslides; however, they are not activated as frequently as the other types of slides. The preponderance of landslides occur in January after the water table has risen in the previous months, although destructive landsliding can sometimes last until March. The landsliding occurs in only about 1 percent of the City, around the edges of the steep, mostly linear hills. Although all of the steep slopes on the hill margins are susceptible to sliding, the GIS maps clearly show that certain areas are highly susceptible to slope instability. Contributing causes of landsliding may be myriad, but water is involved in nearly all of the cases. Consistent with other studies in the City and the region, 84 percent of the reported landslides may have had some factor of human influence associated with them.

Of the total number of landslides in the database, 58 percent were within existing potential slide areas and 76 percent were within the steep slope areas, as defined by the Department of Design Construction and Land Use (DCLU). The percentage of landslides within either a steep slope or existing potential slide area was 88. Several dense clusters of slides were clearly outside of existing mapped potential slide areas, so studies were performed to remap the potential slide areas throughout the City using the historical record as the primary factor.

Typical improvements to slope instability in Seattle are presented for each of the types of landslides. They include surface water and groundwater improvements, retaining structures, soil reinforcement, grading, and catchment or diversion structures. Unit cost estimates were prepared for each of the landslide improvement features. The role of vegetation to maintain stable slope conditions and reduce erosion is discussed. The role of utilities and roads in landslides and how to reduce landsliding through the design of utilities are also presented.

Forty-three stability improvement areas were defined throughout the City. They are areas that share somewhat similar geologic and groundwater conditions, and are geographically contiguous. For each of these stability improvement areas, engineering solutions were tabulated, so rough cost estimates could be made by the City; however, no site-specific subsurface explorations were performed.

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SEATTLE PUBLIC UTILITIES
SEATTLE, WASHINGTON

PREFACE

1.0 INTRODUCTION

1.1 Purpose

This report presents the results of a comprehensive study of landslides in Seattle, Washington, using data that dates back to 1890. The first study of landsliding in Seattle was performed by Dr. Donald Tubbs in his doctoral studies at the University of Washington between 1972 and 1973. The result, *Landslides in Seattle,* was published in a circular by the Washington Department of Natural Resources in 1974 (Tubbs, 1974). Tubbs based his study on the wet 1971/72 winter that resulted in the largest number of reported landslides since the winter of 1933/34. Since then, three major winter storm regimes have heavily impacted Seattle in terms of landslide damage: 1985/86, 1995/96, and 1996/97.

Seattle is unique among cities in the United States in that it instituted a full-time position to gather information on landslides, categorize it in files, and coordinate landslide information among City of Seattle (City) departments. Since the pioneer efforts in the 1960s, this landslide file has been updated periodically and the files have been open to the public. Despite some spatial and temporal gaps in the information contained in these files, it is probably one of the most comprehensive records of landslides in this country. It is this remarkable record that allows this study to be completed.

When the city experienced the impact of two succeeding rainy seasons (1995/96 and 1996/97) of abnormally high rainfall, the City, with input from the public, decided to consider new approaches to deal with landsliding in the City. In order to adopt new City of Seattle policies for landsliding proaction and response, two approaches were solicited: (1) public outreach and opinion, and (2) a scientific approach to understanding the landsliding and formulating remedial measures to combat it. The first approach was met by holding a series of five public meetings during which citizen comments were taken. The second approach is addressed with this report on the landslides in Seattle.

The purpose of this study is four-fold:

1. To inventory and catalog landslides in the City and Shannon & Wilson, Inc. (Shannon & Wilson) files.

2. To better define landslide hazard zones within the City.

3. To aid in the landslide policy decisions by City officials.

4. To increase public knowledge of landslides and landsliding in the City.

1.2 Scope of Services

The scope of services for this study was developed in an iterative manner with officials from Seattle Public Utilities (SPU). They also received comments from representatives from Seattle Transportation Department (SEATRAN), Seattle Department of Design Construction and Land Use (DCLU), and Seattle Law Department. The following are the topics of the scope of work:

1. Files dating from 1890 were searched in the landslide file at SEATRAN. Electronic files were received from DCLU, with landslide data starting in 1986. Shannon & Wilson files were researched internally. The landslides covered by the report are those reported through June 1999.

2. The data was categorized and plotted using a Geographic Information Systems (GIS) base for manipulation and presentation.

3. Landslides in three selected areas (West Seattle, Magnolia/Queen Anne, and Madrona) were field checked. Subsequently, the locations of all landslides in the city were field checked.

4. Shannon & Wilson personnel attended public and policy meetings to discuss technical issues regarding landsliding and to relate the results of this study.

5. The interrelationship between city streets, underground utilities, and landslides were evaluated citywide.

6. Shannon & Wilson formulated typical geotechnical engineering solutions and unit costs related to landslide problems typical to Seattle, and developed Stability Improvement Areas to provide information for prioritizing remedial efforts and developing approximate cost estimates.

7. The citywide locations of potential slide areas were updated based on our evaluations and field checking.

8. The results of this study were summarized in this report.

1.3 Report Organization

This report is presented in two volumes. Volume 1 contains the report and figures. Volume 2 contains the Map Folio and the Landslide Database. Volume 1 is organized into five Parts:

Part 1 - Landslide Inventory and Analysis

This Part describes the data sources and methods used to develop, field check, and analyze landslides in Seattle. It includes descriptions of the conditions that lead to landsliding in Seattle, including the topography, geology, groundwater, surface water, climate and cultural features. This Part also describes the types of landslides that occur in Seattle, when they occurred, and how their locations relate to currently mapped hazard areas.

Part 2 - Geotechnical Evaluation of Landslides Citywide

Part 2 presents a geotechnical engineering evaluation of the landslides described in Part 1. It describes typical stability improvements that can be made for each of the landslide types and typical details for the stability improvements. We present unit cost estimates for the typical improvements that could be used for preliminary budgeting purposes. Finally, this Part evaluates how City utilities and streets can affect stability.

Part 3 - Landslides in Three Study Areas: West Seattle, Magnolia/Queen Anne, Madrona

Part 3 presents the results of geological and geotechnical studies we made in the original three study areas of the City. These areas are: West Seattle, Magnolia/Queen Anne, and Madrona. The emphasis in this part of the report is on evaluating factors that contribute to slope instability and the remedial measures that could be implemented to improve stability in these areas.

Part 4 - Landslides in North, Central, and South Seattle

Part 4 presents the results of geologic and geotechnical studies we made in four additional study areas in Seattle: Northwest Seattle, Northeast Seattle, Capitol Hill, and South Seattle. Similar to Part 3, this section presents measures for improving stability in these areas.

The original scope of work resulted in the preparation of Parts 1, 2, and 3. In Part 3, three study areas were originally selected for detailed evaluations including field verification of landslide locations. This resulted in the identification of 26 Stability Improvement Areas where landsliding was prevalent in West Seattle, Magnolia/Queen Anne, and Madrona. Subsequently, Part 4 was authorized to extend the field verification process to include landslides throughout the City, and to identify additional Stability Improvement Areas. The result was the delineation of 17 additional Stability Improvement Areas in Northwest Seattle, Northeast Seattle, Capitol Hill, and South Seattle. Thus, Parts 3 and 4 provide similar results regarding stability improvements in the respective areas; however, Part 3 includes more description and GIS analyses of the landslide inventory data than are presented in Part 4.

Part 5 - Potential Slide Areas

This part describes the process and results of updating the locations of potential slide areas.

The Figures and Tables for each Part are presented at the end of the Part to which they relate. They are numbered sequentially in each Part, e.g., Figure 1-1, 1-2, etc., for Part 1, and Figure 2-1, 2-2, etc., for Part 2. Volume 2 contains the large maps in the Map Folio and the Landslide Database. Maps in Appendix A relate to Part 1 and maps in Appendix B relate to Part 3. The maps in Appendix C relate to Part 4, maps in Appendix D related to Part 5, and the Landslide Database is in Appendix E.

1.4 Authorization

This study was performed in general accordance with the scope of work submitted on November 11, 1997, to Mr. Robert Chandler of SPU. Mr. Chandler orally authorized this study on October 9, 1997. Authorization by Mr. Chandler to proceed with Part 4 and Part 5 was received in January 1999.

1.5 Limitations

The database information was based on available records and brief, limited field observations. Differences between the data and actual conditions may exist. The database entries draw no conclusions regarding the extent to which reported contributing factors caused slope instability.

Based on the database and field observations, geotechnical evaluations were conducted to formulate remedial measures for improving stability. As a result of this effort, 43 Stability Improvement Areas were developed. These are areas where previous instability has been prevalent (i.e., a concentration of reported landslides). For each Stability Improvement Area, we evaluated the conditions that contributed to past instability or that may contribute to potential future instability, and have presented possible remedial measures to be considered by the City and private property owners for improving stability. The remedial measures presented are intended to be preliminary and are provided to give the City and others information that can be used to prioritize remedial efforts and to develop order-of-magnitude budgets for the work. The proposed remedial measures are also intended to be illustrative of potential solutions. Additional evaluation and, in some cases, field reconnaissance, are necessary to prioritize these proposed remedial projects. To determine final scopes of work and cost estimates, subsurface explorations and/or additional engineering studies are required.

The number of recommended stability improvements are extensive. It is, thus, obvious that a considerable length of time in years will be needed for conducting further studies, prioritizing the improvements, allocating funds, and implementing the work. The recommendations are general in nature and provide approximate locations where further analyses could take place for evaluating priorities and scheduling work. To be effective and to prevent overlapping of remedial measures, the prioritization process must be coordinated with the Needs Assessment of the Drainage Policy Study conducted for SPU by the consulting engineering firm of Black & Veatch.

Since landslides and potential areas of instability do not obey property boundaries, the improvements recommended in Parts 3 and 4 do not consider the location of property lines and would take place on City property, private properties, or both. The improvements are those that could be made by the City to protect utilities, drainage features, streets, and other City facilities; and also those measures to be coordinated between the City and private property owners to improve stability of an unstable slope. Some improvements would be made by the City, while other improvements or protection would be the responsibility of private property owners. It is anticipated that coordination between the City and private property owners would include expeditious processing of permits; granting of appropriate easements and variances to code requirements where needed to improve stability for private and/or public properties; shared costs, such as by Challenge Grants or Local Improvement Districts (LIDs); or other cooperative efforts. On private properties, the City may also facilitate the negotiation of easements for stabilizing measures.

The recommendations presented in this report are based on a technical evaluation, and are not intended to set City policy. The City's landslide responsibilities are a complicated blend of public policies involving public and private responsibilities and partnerships; therefore, the implementation of any or all of the recommendations are solely at the City's discretion.

Improvement of stability involves actions not only by the City, but actions by private property owners. Such actions by private property owners should include accepting existing conditions and the risks of slope instability, and accordingly controlling drainage, improving stability, providing protection for property and structures, and/or obtaining competent professional advice. Homeowners or potential property owners should also obtain competent professional advice regarding site selection, property purchase, site improvements, and/or new construction. The existing conditions to be accepted by private property owners include surface and subsurface drainage conditions, soil conditions and site geology, site topography, and other factors affecting stability as described throughout this report. The potential adverse conditions that may occur during times of very heavy and/or prolonged precipitation should also be considered. In addition, private property owners should avoid conducting site work that would jeopardize stability of adjacent property.

None of our studies have considered nor evaluated the specific contributing or predominant causes of any previous landslides. The stability improvements described in Parts 2, 3, and 4 of this report are general types of action that could be considered by the City and/or private property owners to improve stability and reduce landslide risks.

There are always risks of damage to property and structures involving landslides, for property located on or adjacent to a slope. Property owners need to accept those risks. Although the recommended improvements and homeowner education can lead to immediate or eventual improved slope stability conditions, the risks of damage cannot be completely eliminated. In addition to natural factors (soil, groundwater, heavy rainfall), other factors that may affect stability are excavations, fills, leaking or broken utility lines, improper drainage, lack of maintenance of drainage facilities or vegetative cover, unwise actions by adjacent property owners, or similar events or unknown conditions that may cause instability.

Property owners should also be aware of the advisability of obtaining insurance in addition to standard homeowner's insurance to specifically cover the risks posed by geologic hazards including earth and debris movement.

Shannon & Wilson has prepared the following "Important Information About Your Geotechnical Report" to assist you and others in understanding the use and limitations of our reports.

Important Information About Your Geotechnical/Environmental Report

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include: the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used: (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors which were considered in the development of the report have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the
ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland

PART 1. LANDSLIDE INVENTORY AND ANALYSIS

2.0 GEOLOGIC CONDITIONS

Landslides in Seattle are caused by a combination of steep slopes (topography), glacial and post-glacial soils (stratigraphy), and a pronounced wet winter season (typically November through March). It requires the interaction of all three to create landsliding in the city. With the exception of coastal California, Seattle suffers more damage from landslides than most other large cities in the United States.

2.1 Topography

Seattle is comprised of a series of linear ridges and broad plateaus with intervening river valleys and linear depressions that were shaped by the last glacial ice to reach this area. To the south of the Lake Washington Ship Canal, ridges and swales dominate the landscape. The major hills that crest at about 450 to 500 feet are Magnolia, Queen Anne Hill, Capitol Hill, Beacon Hill, West Seattle, and Mount Baker Ridge. They are separated by Interbay, Lake Union, the Duwamish River Valley, Rainier Valley, and Elliott Bay. Not all of the swales are water-filled. Some are naturally filled with glacial and nonglacial sediments and others are modified with artificial fill. With the exception of Longfellow Creek in West Seattle, the ground surface is drained by short and steep streams.

In the area north of the Lake Washington Ship Canal, the ground surface is a broad undulating plain. The ground rises up to the north gradually from the ship canal, nearly reaching elevation 500 feet near the north city boundary. It is broken by depressions, such as Green Lake, Haller Lake, and Bitter Lake. It has also been incised by Pipers Creek on the west and Thornton Creek on the east.

As shown on a topographic relief map of Seattle, Figure A-1 (Appendix A, Volume 2), the ridges and plateaus are surrounded on all sides by steep slopes. These slopes range in inclination from about 25 to 90 degrees with the horizontal. In general, the steeper slopes are those that border the shoreline of Puget Sound, particularly the rare ones that are not protected from wave erosion. The only remaining unprotected bluffs in the city are in Discovery Park and a short section of shoreline at the south end of Magnolia. Elsewhere, the shoreline is armored or otherwise protected by individual short bulkheads or by long bulkhead/embankments, such as the Burlington Northern Santa Fe Railroad (BNSF), north of the ship canal.

It is on these steep slopes that surround the ridges and plateaus of Seattle that Seattle's landslides recur on a regular basis. This process is particularly evident in the retreat of bluffs since the disappearance of the last glacial ice from the Seattle area about 13,500 years ago. It has been estimated, based on marine charts showing change in submarine topography, that the Puget Sound bluffs in Seattle have retreated at a rate of about 75 feet per century (Galster and Laprade, 1991). The rate was undoubtedly much greater in the first few millennia following glacial retreat; however, it is equally obvious that slope instability is still very active.

2.2 Stratigraphy

Seattle is underlain by bedrock of Tertiary age, glacial and interglacial soil deposits of the Pleistocene Epoch (2 million to 10,000 years ago), and nonglacial soil deposits of the Holocene Epoch (present-day). However, soils deposited during the most recent glaciation of the central Puget Lowland dominate the surface and subsurface geologic conditions in Seattle. These rock and soil deposits are very completely interwoven by repeated sequences of deposition and erosion. It is clear that each of the major ridges or uplands has a unique stratigraphic system.

2.2.1 Tertiary Bedrock

Bedrock, consisting primarily of sandstone and siltstone, outcrops sporadically to the south of the Seattle Fault (see Figure A-3); however, no bedrock outcrops to the north of this fault within the city limits because it is buried by 1,000 to 3,000 feet of glacial and nonglacial sediments. The bedrock does not play a significant role in the landslide history of Seattle. The only major area of bedrock instability occurred east of Boeing Field where large excavations were made for Interstate 5 (I-5) in the 1960s.

2.2.2 Pre-Vashon Deposits

Older nonglacial and glacial soils (pre-Vashon Stade) are present within the downtown business district and in the cores and flanks of most of the hillsides. However, these older soils have not produced much landsliding. Pre-Vashon glaciomarine deposits (Possession Drift) underlie the downtown business district, Beacon Hill, and Mount Baker Ridge. They are intermixed and chaotically stratified clayey till, glaciolacustrine silt/clay, and sand. Locally throughout the city, these deposits are overlain by a variety of sediments of the Olympia interglacial period. These sediments include sand, silt and clay layers with scattered organic fragments, peat pockets, and thin interbeds of gravel.

2.2.3 Vashon Glacial Deposits

The primary geologic units that are involved with landsliding in Seattle are those that were laid down during the Vashon Stade of Fraser Glaciation, between about 17,000 and 13,500 years ago (Waldron, 1962; Booth, 1987). Together, the four members (Lawton Clay, Esperance Sand, Vashon Till, and Vashon recessional outwash) comprise most of the ridges and uplands in the Seattle area (Figure 1-1). The lowest three members were overridden by approximately 3,000 feet of Vashon Stade ice. Recessional outwash was not overridden by this ice.

The Lawton Clay, a glaciolacustrine deposit, was laid down in a lake that formed as the glacial ice advanced southward from British Columbia and blocked the Strait of Juan de Fuca. The unit consists of laminated and massive silty clay and clayey silt with scattered fine sand lenses. It is hard, from having been glacially overridden. Because of its hard condition and fine-grained consistency, it is relatively impervious such that groundwater tends to perch on top of its upper surface. The Lawton Clay is typically interbedded with sands of the overlying Esperance Sand near the contact of the two units.

The Esperance Sand is a glaciofluvial advance outwash that was deposited by streams issuing from the Vashon glacier as it advanced southward. It is comprised chiefly of fine to medium sand that is locally gravelly. Locally, it also contains silt layers and pockets and discontinuous layers of gravel. It is very dense and pervious with groundwater normally flowing freely through this soil.

Vashon Till (lodgment till) was deposited beneath the Vashon Stade ice. Also known locally as "hardpan," it is normally a gravelly, silty sand or a gravelly, sandy silt with scattered cobbles and boulders. It is very dense (one of the most compact soils in the world) and relatively impervious. Water infiltrating through overlying deposits normally perches on top of the till; however, locally the till contains pervious water-bearing zones.

Vashon recessional outwash was deposited by streams issuing from the Vashon glacier as it receded or wasted. It is a slightly silty to silty sand with scattered gravel, and is the deposit that is found at the ground surface on most of the uplands. It is relatively pervious and loose to medium dense, not having been glacially overridden. Precipitation commonly infiltrates readily through this deposit and then perches on top of the Vashon Till.

2.2.4 *Holocene Deposits

Holocene (post-glacial) deposits are ubiquitous throughout Seattle. They include alluvium, beach deposits, depression fillings, and colluvium. These soils have not been glacially overridden. As such, they are typically loose to medium dense or soft to stiff. Alluvium is deposited along the major rivers and creeks, such as the Duwamish River and Longfellow and Pipers and Thornton Creeks. It is comprised of loose sand and gravel that is normally wet; however, because of its low slope gradient, it is not normally related to landsliding.

The beach deposits found along the shorelines are normally not landslide-prone because of a lack of relief. Landslides do deliver material to the shoreline that contribute to forming beach deposits.

Depression fillings commonly consist of soft clay and silt and organic materials, such as peat. They accumulate in low spots on the ground surface. They are normally found on the upland ridges and plateaus, although they can be included in the same areas as river alluvium. Depression fillings are not normally associated with landsliding.

Colluvium is very commonly associated with landsliding. Colluvium is the loose to medium dense or soft to stiff soil that mantles the sides and toes of slopes throughout the city. Because it was deposited by gravity processes such as soil creep, surficial sloughing, landsliding, and slope wash, grain-size can vary from clay and silt to boulder-size. The mode of accumulation ranges from slow creep (the imperceptible movement of only inches per year or less) to catastrophic landslides. Soil creep in the upper few feet of soil on a slope is commonly reflected in the bowing of trees on the slope. Colluvium is normally moist to wet, especially during the rainy season.

Another category of Holocene soil is fill placed by humans. Fill soils vary widely in grain-size, location, presence of debris, and size. Although many new fills have been compacted and engineered, most older fills were just dumped in place or nominally run over with the spreading equipment. These fills can be particularly unstable where they have been placed on or in close proximity to a steep slope.

2.3 Groundwater and Wet Weather

In addition to topography and stratigraphy, groundwater is the other factor that plays a significant role in the generation of landslides in Seattle. In spite of loose soils on steep hillsides, landslides very rarely occur in the dry summer months, although this sometimes happens under unusual conditions. It is the water pressures that build up in the ground, usually during the pronounced wet-weather season, that nearly always trigger the slide event. The source of water for any individual landslide can be natural or influenced in some way by human activity or a combination of these two factors. This section presents the various ways in which the groundwater interacts with the geologic units in Seattle.

The groundwater found closest to the ground surface is that perched atop the Vashon Till (Figure 1-1). In this case, precipitation or water related to human activities, such as improper drainage, infiltrates down through recessional outwash until it encounters the top of the Vashon Till. The perched water may flow until it emerges in a pond, creek, or a steep slope, where it forms a spring. Water at this contact normally dries up in the spring or summer and does not reestablish itself until the winter months. Because the contact between the recessional outwash and the Vashon Till is shallow (normally less than 10 feet deep), it is normally reachable with a backhoe for dewatering.

On the sides of hills, the undisturbed glacial soils are covered with a rind of colluvium. Water is commonly able to penetrate the semi-pervious colluvium because it is relatively loose and contains some fraction of sand; however, it cannot infiltrate easily into the very dense or hard underlying glacial soils. The water, therefore, travels along the inclined contact between the two materials of different permeability. Water at this contact also normally dries up in the spring or summer and does not reestablish itself until the winter months. Because the contact between the colluvium and the underlying undisturbed soil is shallow (normally less than 10 feet deep), it is generally reachable with a backhoe for dewatering.

The most prevalent groundwater aquifer in the Seattle area is the Esperance Sand. Precipitation infiltrates through "windows" or cracks in the Vashon Till and continues vertically down into the Esperance Sand until it encounters the top of the underlying Lawton Clay. Owing to the low permeability of the Lawton Clay, the groundwater perches on the clay and then moves laterally, eventually saturating near-surface colluvium and/or emerging in a spring on a hillside. Because of the residual lag travel time of this water, many of these springs are perennial. They are the most prolific springs throughout the city. It is fairly easy to trace the level of the sand/clay contact by locating the springs on a hillside. The source of water for an individual spring or group of springs is very difficult to define, as it probably has a large regional contributing area uphill from the spring.

The key stratigraphic marker for landslide location is the contact between the Esperance Sand and the Lawton Clay (Figure A-4). It is commonly termed "The Contact." In *Landslides in Seattle*, Tubbs concluded that "the landslide typically occurred along the trace of the contact between the Esperance Sand and either the Lawton Clay or pre-Lawton sediments." No experiences or collected data in the past 24 years have changed that conclusion.

3.0 METHODOLOGY

3.1 Data Sources

Three main sources were used to develop the historical database for the assessment of landslide hazards in the City of Seattle (City). The primary source of landslide information was Seattle Transportation Department (SEATRAN), which has landslide files dating to 1890. However, good records were not kept before the 1960s, when the landslide file was started by Mr. Finney of the Seattle Engineering Department. These files consist of several types of information including memorandums by field inspectors, court documents, photos, maps, subsurface data, geotechnical reports, mitigation plans, and cost estimates. The accuracy of the City landslide files is dependent on several factors. These factors include available staff levels, the amount of damaged or missing information, and, most importantly, the degree of landslide reporting by the public. In general, SEATRAN's files record landslides that primarily affected rights-of-way or utilities; not private properties.

The second source of information was the Department of Design Construction and Land Use (DCLU), which has maintained a landslide database since 1986. This database includes the reported locations of particular landslides with a brief description regarding the structural integrity of the affected structures. The DCLU landslide database contains a relatively complete record of landslides that occurred during severe storm periods. However, the database is relatively incomplete for other periods. Furthermore, the landslide dates included in the DCLU database reflect the time of inspection rather than the initiation of ground movement. Therefore, when the failure date could not be determined from the DCLU files, we assumed an approximate failure date that was close to a previous storm event.

The third source of information for the landslide database was the Shannon & Wilson, Inc. (Shannon & Wilson) files. Shannon & Wilson has maintained project files since 1954. The inventory includes projects that pertain to ground displacement performed by the company.

3.2 Data Description

The input data needed for the Seattle Landslide Study is subdivided into six main groups:

1. Landslide Identification
2. Landslide Characteristics
3. Stratigraphy (Geology)
4. Trigger Mechanism(s)
5. Roads and Public Utility Impact
6. Damage and Repair (Mitigation)

The following is a detailed description of the data comprising each of the attributes in the landslide database, which is presented in Appendix E. The Appendix also contains a legend that defines abbreviations and provides additional explanations for each data field.

3.2.1 Landslide Identification

Record Number

The Record Number field represents the unique identifying number for each documented landslide in the database. Each landslide was assigned a sequential record number when it was entered into the database. After data processing (refer to Section 3.3), several landslides were omitted (duplications, etc.) and, therefore, the record numbers are not in a continuous series.

Location

The Location field consists of an address representing one or more of the following: the address of the person or persons reporting the incident, the address of the closest property to the event, the address of a property affected by the event, or an approximate address specifically used for plotting on the Geographic Information System (GIS). Locations of landslides reported by BNSF along the railroad right-of-way are referenced by Milepost number (e.g., MP 8.5).

Date

The Date field contains the approximate date of initiation of ground displacement. In cases where the exact date was not known, we estimated precipitation year. This was accomplished by assigning the first day of January as the date. A higher percentage of older landslides were assigned this date because of the paucity of information in the older files. Note that many landslides occurred during the New Year's storm of 1997. The January 1, 1997, dates for these landslides are accurate. A note in the Comments field or the Date Confidence field includes the 1997 landslides where the exact date is not known. A precipitation year is defined for this study as beginning on July 1st and ending on June 30th of the following year.

File Number

The File Number field contains the source's file number where the information was obtained. DCLU file numbers consist of "J#", "J##", and "J###" (where "#" represents a number, e.g., J21) for events from 1986 to 1996 and "96-97 storm" for landslides occurring during the 1996-1997 winter. Other file numbers are as follows:

Data Source	File Number or Source Designation
Shannon & Wilson, Inc.	S&W
Observed During Field Reconnaissance	Field
Reported by source other than the City or Shannon & Wilson, Inc.	Citizen
SEATRAN	All other file numbers

Shannon & Wilson project files consist of "S&W." Landslide events discovered during field reconnaissance for this landslide study consist of "Field." Landslides reported by sources other than the City or Shannon & Wilson consist of "Citizen." All other file numbers represent SEATRAN file locations.

Consultant Report

The Consultant Report field shows a letter code if an engineering consulting company prepared a report regarding the landslide.

Field Checked

Field Checked shows the confidence in whether or not the landslide is properly located on the map. Landslides that are accurately located on the map and field checked are "True." Landslide locations shown on the map that were not found during field reconnaissance are marked "False."

Date Confidence

The Date Confidence field is "True" where the date is believed to be accurate and "False" when the date may be approximate. Where no determination could be made, the Date Confidence field was left blank.

3.2.2 Landslide Characteristics

Slope Height

The Slope Height field is an estimate of the approximate elevation difference, in feet, between the headscarp and the toe of the slide, as estimated from historical records and field verification. Differences between these estimates and actual conditions may exist.

Landslide Type

The Landslide Type field consists of a general classification of each landslide, even though more than one classification may have been involved at a specific location. The classification of landslide type recorded in the database was the predominant type based on our interpretation of the records and our site visits. There are four general landslide-type possibilities: high bluff peeloff (HBP), shallow colluvial (SC), deep-seated (DS), and groundwater blowout (BO). Please refer to Section 4.0 for detailed descriptions of landslide types.

Debris Flow

The Debris Flow field is "Y" if a debris flow with runout generally longer than 50 feet occurred and "N" if a debris flow did not occur or had a short runout. Where no determination could be made, the field was left blank.

Size

The Size field represents the approximate aerial extent of the ground displacement. Landslides covering an area greater than 10,000 ft^2 are denoted with an "L" and those equal to or less than 10,000 ft^2 are denoted with a "S." Note that some large landslides may cover a small area, but displace a large volume of material because of their depth. This type of landslide is not represented because of the difficulty in estimating depth to the slide plane. Differences between these estimates and actual conditions may exist.

Vegetation

The Vegetation field describes the vegetative ground cover contained within the landslide margins based on file pictures and/or field reconnaissance. There are four vegetation type possibilities: brush (B), wooded (T), sparse cover or bare ground (S), or grass (G). If even one tree was contained within the slide margins, the landslide was designated with a "T." When there were no pictures in the file and no determination could be made, the field was left blank.

Topography

The Topography field describes the approximate average slope angle. If the slope angle is greater than 40 percent, it was described as a steep slope (SS). Moderate slope (MS) was used for slope angles less than or equal to 40 percent.

3.2.3 Stratigraphy (Geology)

The next four fields of the attribute table indicate the geologic units involved in the ground displacement. The units are ordered, as they would appear in the headscarp, typically from youngest to oldest (top to bottom). The five designations used in this study were: fill (HF), colluvium (HC), glacial till (QT), glacial outwash sand (QS), and lacustrine clay/silt (QC). The list of geologic units involved in a particular event is estimated based on the type of landslide, the geographic and topographic location, and any subsurface information disclosed in the file. The geologic units were not field verified and differences between these estimates and actual conditions may exist.

3.2.4 Landslide Trigger Mechanisms

All four of these fields reflect documentation in files or reports regarding the trigger mechanism of ground displacement. These fields do not reflect the degree of contribution, nor do they necessarily represent a professional evaluation. Many landslides documented in the SEATRAN files are claims to the City and may contain some degree of bias.

Natural

The Natural field describes the trigger mechanism as being natural (Y) or human (N). For example, precipitation is considered a natural trigger, whereas pipe breaks and excessive lawn watering are not. Blank spaces indicate that no determination could be made.

Groundwater and Surface Water

The next two fields, Groundwater and Surface Water, indicate when groundwater or surface water may have been the possible triggering mechanism of the event. A "Y" indicates that groundwater and/or surface water may have triggered the landslide and an "N" indicates that these two trigger mechanisms probably were not involved. Improperly directed surface water by private parties and naturally occurring surface water were not differentiated in this field. Blank spaces indicate that no determination could be made.

Fill and/or Cut

The Fill and/or Cut field indicates when filling and/or cutting may have triggered the landslide event. Landslides resulting from inadequate shoring of an excavation, for example, were denoted by a "Y." An "N" indicates that filling and/or cutting was not involved in triggering the landslide. Blank spaces indicate that no determination could be made.

3.2.5 Roads and Public Utility Impact

All four of these fields reflect documentation in files or reports regarding the effect of roads and underground public utilities on ground displacement. These fields do not reflect the degree of contribution, nor do they necessarily represent a professional evaluation. Many landslides documented in the SEATRAN files are claims to the City and may contain some degree of bias.

Road Cut and/or Fill

The Road Cut and/or Fill field is similar in nature to the fill and/or cut field, but only pertains to public roads. Filling at the top of the slope is denoted by an "F", cutting near the toe of a slope is denoted by a "C", and in cases where both filling and cutting were factors in triggering the event, an "FC" was entered. Blank spaces indicate that no determination could be made.

Surface Drainage

The Surface Drainage field refers to the effect(s) of City-maintained drainage systems on ground failure. Landslides that may be affected by City-maintained drainage systems are denoted by a "Y" and landslides that may not be affected are indicated by an "N." Blank spaces indicate no determination could be made.

Pipe Leak

The Pipe Leak field refers to the presence of additional water introduced to a landslide as the result of a pipe leak or pipe rupture. A "Y" indicates the presence of a pipe leak or rupture and an "N" indicates no involvement. No differentiation was made between a pipe break resulting from ground displacement and ground displacement resulting from a pipe break. Blank spaces indicate no determination could be made.

Trench Fill

The Trench Fill field indicates the presence of trenches serving as conduits for groundwater that possibly contributed to instability. A "Y" indicates possible involvement and an "N" indicates no involvement. Blank spaces indicate no determination could be made.

3.2.6 Damage and Repair (Mitigation)

This group of fields pertains to landslide mitigation. It is important to note that the degree of damage or the type of mitigation does not necessarily refer to the address listed in the location field.

Damage

The Damage field is a numeric field referring to the degree of damage caused by a particular event. A value of "3" is equivalent to a "red tag" or severe damage to the property; a "2" is equivalent to a "yellow tag" or moderate damage; a "1" is equivalent to a "green tag" or some temporary damage; and a "0" indicates no observable significant damage to property. Blank spaces indicate no determination could be made.

Repair Type

The Repair Type field reflects repairs described in the files, reports, and/or observed in the field during field reconnaissance. Refer to the Legend in Appendix E for a description of the repair types. To the extent possible, we included only repairs that were constructed. However, some landslide records did not include enough information to determine if the repair was actually built. These cases may be included. Furthermore, the type of mitigation described in the table does not distinguish between structurally engineered repairs and non-engineered repairs.

Repair Effective

The Repair Effective field indicates whether the repair was effective in preventing further ground movement at that particular location thus far. It is not a warranty that the repair will remain stable. If repaired and no subsequent ground movement was documented, a "Y" was entered. If the repair subsequently failed regardless of design, an "N" was entered. If the integrity of the repair is questionable regardless of a failure, an "N?" was entered. If a repair was not necessary, no repair was accomplished, or if there was no information regarding repairs, the field was left blank.

3.3 Data Processing

Upon completion of the data collection phase of the study, duplicate addresses with the same failure date were eliminated. The data was plotted on a GIS using ArcView 3.0a desktop GIS software. Several City ArcInfo GIS coverages were loaned to Shannon & Wilson for purposes of map reproduction, discrete address-based geocoding, and analysis of the occurrence of landslides. These City ArcInfo coverages included, but were not limited to, orthophoto-derived topography, parcels, parks, streets, utilities (which includes water mains and laterals, and drainage and wastewater mains and laterals), buildings, potential slide areas, and steep slopes.

Each landslide data point was automatically geocoded using the location field and plotted on the center of the parcel. Note that each landslide is represented as a single point, regardless of size. The landslide locations were then edited based on the comments in the files regarding location, direction of sliding, type of damage, and/or existing topography. The point indicates the center of the point of landslide initiation (middle of the headscarp). Duplicate landslide events were eliminated based on date and their spatial relationship. For example, a property owner at the top of the slope may have reported a landslide to SEATRAN, while another property owner at the base of the slope may have contacted the DCLU because the same landslide affected their residence. Shannon & Wilson initially field checked landslides within the three study areas (West Seattle, Magnolia/Queen Anne, and Madrona) for accuracy. Shannon & Wilson subsequently field checked the balance of the landslides in the database for the analysis in Part 4. The map symbol was then moved to the proper location or deleted when necessary. The maps that portray the landslide locations are presented in the Map Folio, Volume 2.

4.0 LANDSLIDES

This section of the study discusses particular aspects of landsliding, including the types of landslides that occur in the city, the timing of landsliding, chronic landslide areas within the city and their characteristics, the causes or contributing factors of landsliding, and the coincidence of landslides with existing potential landslide areas and steep slope areas. To support this discussion, a series of figures and maps will be presented. The maps are presented in a separate volume (Volume 2) because of their size.

4.1 Landslide Types

In evaluating the landslide data compiled for this study, most of the landslides were found to fit into four generalized types. Those types, together with the figure numbers that illustrate a schematic profile view of each type, are as follows:

Generalized Landslide Type	Figure No.
High bluff peeloff	1-2
Groundwater blowout	1-3
Deep-seated	1-4
Shallow colluvial	1-5 and 1-6

There are various combinations of these generalized landslide types, as one type of mechanism may lead to another during the sliding, or the slide may be complex, exhibiting different modes of failure in different portions of the slide. Landslides involving fill material were classified as shallow colluvial landslides. The following sections describe each landslide type in greater detail.

4.1.1 High Bluff Peeloff

High bluff peeloffs (Figure 1-2) occur on the face of near-vertical bluffs where vegetation is absent or sparse. The soil at and near the bluff face, which has been loosened by the forces of weathering (freezing, thawing, root-wedging), slabs off or slides when it becomes wet during periods of heavy rainfall. This type of landslide commonly occurs following a period of freezing weather. Sometimes seepage from more pervious soils, such as recessional outwash, at the top of the bluff, or runoff over the edge of the bluff contributes to this type of instability. Also, water-bearing layers in the steep bluff could contribute to saturation of the face soils. Normally, the thickness of soil that slides off the face is only a few feet. The soil that comes off the bluff may or may not slide for a considerable distance, depending on the water content of the soil and the angle of the slope below the bluff. Alternative names for this type of landslide are earth fall and blockfall.

4.1.2 Groundwater Blowout

A profile of a groundwater blowout landslide is shown on Figure 1-3. This type of slide occurs where a pervious soil (sand) overlies a lower permeability soil (clay or silt). Groundwater collects in the pervious soil and becomes perched on the underlying, relatively impervious soil The lower permeability soil could be either a relatively thin silt or clay layer or a thick stratum of silt and clay. Seepage travels to the slope face immediately above the contact with the underlying, relatively impervious zone and causes instability where the sand essentially blows out and flows downslope (runout). Because of this blowout, the upper portion of the slope becomes undermined and also fails. Groundwater is more important in the development of this slide type than direct infiltration of precipitation and is commonly found at "The Contact," Figure A-4. Nevertheless, this type of slide normally takes place during or shortly after periods of heavy precipitation because of the added water near the spring exit. It should be noted that this mechanism for causing landslides (seepage at pervious/impervious contact) was probably involved in a number of slides that were categorized as shallow colluvial landslides in the database table and landslide maps. This categorization would occur where there was a lack of detailed data on a landslide, particularly in the older records.

4.1.3 Deep-Seated Landslides

In the database table, those landslides that were identified or estimated to involve a depth of movement greater than an estimated 6 to 10 feet were categorized as deep-seated (Figure 1-4). These landslides may involve higher density, in-place soil as well as colluvial soil. This type of slide normally consists of the block movement of soils where a mass of soil slides downhill on a failure surface that is often arc-shaped. Sometimes the surface of rupture parallels the ground surface. As blocks of soil move downhill, a setdown of the ground surface occurs at the upper edge of the blocks, thus forming a slide scarp. Such movement is commonly progressive; that is, a lower block of soil moves first, which takes away lateral restraint for higher blocks that, in turn, slide.

The deep-seated landslide is initiated by water coming into the slide mass, which takes place either from rising groundwater levels, direct infiltration of heavy precipitation, surface runoff, saturation by the discharge or leaking of pipes into or onto slope soils, or a combination of these sources of water. Where the soils subject to movement are relatively pervious, such as sand and/or gravel, the movement normally occurs rather abruptly (within minutes or hours). Where the soils are silt or clay, movements usually occur gradually, over days, weeks, or even months.

4.1.4 Shallow Colluvial (Skin Slide)

Shallow colluvial landslides occur when loose, heterogeneous soils on a steep slope become saturated and slide (Figure 1-5). The term "skin slide" is sometimes applied to this slide type because a relatively thin depth of soil is normally involved. They generally consist of rapid movements of the saturated soils, and commonly act like a thick fluid, flowing or running out over a considerable distance. In the database, they are noted as "debris flows" when the runout generally exceeded 50 feet. The saturation of soils that causes shallow colluvial landslides takes place by infiltration of surface runoff, direct infiltration of precipitation, groundwater seepage, discharge from pipes, or a combination of these sources of water.

Figure 1-6 illustrates a relatively shallow slide involving fill material. This type of landslide was categorized as shallow colluvial landslide in the database table and on landslide maps. If fill is placed at the top or the side of a slope without compaction and suitable drainage provisions (surface and subsurface), instability is likely inevitable.

4.2 Timing of Landslides

The timing of landslides is dependent on precipitation at three different scales. These are total annual rainfall, monthly rainfall, and a single storm event. The longest scale is that of annual rainfall. Although heavier rainfall can occur in years widely spaced or consecutively, the citywide pattern of landsliding, as shown on Figure 1-7, indicates that about every decade a higher than average amount of mass wasting (landsliding) occurs. For purposes of this study, the rainfall (landslide) year has been designated from July to June. In this way, all of the rainy winter season is tied together statistically.

The most notable landslide winters were 1933/34, 1955/56, 1959/60, 1960/61, 1966/67, 1968/69, 1971/72, 1973/74, 1985/86, 1995/96, and 1996/97. Of these eleven winters, three produced particularly large numbers of landslides: 1933/34, 1985/86, and 1996/97. The damage incurred during the winter of 1933/34 was responsible for the formation of the Works Progress Administration (WPA) drainage program in Seattle, administrated through the Seattle Engineering Department. The number of landslides during that winter may have been comparable to the 1985/86 or 1996/97 winters had there been comparable development in the city.

A map showing the distribution of landslides by decade is presented on Figure A-5 (refer to Figure A-3 for area locations described below). Figure A-5 indicates some interesting trends in the incidence of landsliding and/or changes in the recording of the landsliding. Many of the landslides on the west side of the Beacon Hill (east of Interstate 5 [I-5]) are older, probably because many of the larger landslides were stabilized by the construction of the freeway in the 1960s. The newer slides in this area are mostly smaller shallow colluvial landslides. This reduction in severity of landsliding illustrates how the construction of a major public works project can increase the stability of a hillside; the reason being that buttressing and drainage were widely incorporated into the project.

Two areas that appear to be dominated by new (post-1960) landslides are the Burke-Gilman Trail and Inverness. These locations may be prone to increasing numbers of landslides because the area had been only sparsely developed prior to 1950. However, it is also possible that older slides in this area were not reported.

The areas that have large numbers of landslides dispersed through time are chronic slide areas and they include: Beach Drive S.W., Alki, Pigeon Point, Madrona, Rainier Avenue S.E., Interlaken, Lakeview Boulevard, North Capitol Hill, Laurelhurst, East Queen Anne, Southwest Queen Anne, Southwest Magnolia, and Northwest Seattle.

An above-average winter rainfall punctuated by a large heavy storm on January 18, 1986, led to a rash of shallow colluvial slides throughout the city (refer to Figure A-6 for severe storm events). The most disastrous storm was the Holiday Storm of December 29, 1996, through January 2, 1997, during which heavy and prolonged rain melted an accumulation of about 12 inches of snow in a two-day period. The water equivalent of the snow and direct precipitation caused a total of 8.35 inches of water (as measured at SeaTac Airport) to run off and infiltrate the ground from December 29 to January 2.

Two other time periods were important, but did not produce landsliding on the level of 1933/34, 1986/87, and 1996/97. First, the landslides that occurred in the winter of 1971/72 were the basis for the statistical and geologic conclusions drawn by Tubbs in *Landslides in Seattle*. This publication was one of the major factors used as the basis for establishing the boundaries of the landslide-prone areas in the city. Then during the winter of 1995/96, the Northwest experienced a record winter-long rainfall; a four-month period from November through February. This exceptionally wet winter was a major contributing factor of numerous deep-seated landslides throughout the region, including Seattle.

A recent study (U.S. Army Corps of Engineers, 1997) of the relationship between landslide frequency and precipitation indicates that the most extensive landslide activity is related most closely to a 3-day storm event (lower bound of about 3.8 inches of precipitation) with an 8-year return interval. This statistic approximates a rule of thumb that has been used in the Seattle geotechnical professional community for many years. That rule of thumb says that landslides are likely to initiate whenever there is more than 2 inches of rain in one day or 3 inches in 2 days.

The time of year in which landslides occur is very closely related to the precipitation regime in Seattle, as shown on Figure 1-8 (2 sheets). The overwhelming number of slides occurs in January (45 percent); however, the landslide season typically encompasses a four-month interval: December through March (86 percent). Although November normally has more precipitation than March, it is likely that a certain threshold of antecedent groundwater is necessary to trigger landslides. In summary, although landslides are most likely to happen in January and February, it is not uncommon for landsliding to occur in December and March. However, for planning purposes, the landslide season could begin as early as November and end as late as April. Although slides can occur in the other months, the probability is low. Only 7 percent of the landslides in the database occurred during May through October, outside of the normal 6-month landslide season. These landslides are often not related to the normal factors that contribute to landsliding (precipitation, steep slope, high groundwater table). Examples include such things as overwatering, pipe breaks, and excavation slope failures.

4.3 Landslide Areas

The following sections discuss the distribution of landslides that have occurred in Seattle. These discussions and associated maps show the historical aspects of landsliding by decade, landslides that occurred without human influence, and the different types. The 1,326 landslides contained in the City database are presented on Figure A-5, where they are shown by decade. Figure A-7 shows the same events by type of landslide. The citywide map (Figure A-3) shows 22 specific areas in the city that have experienced landslides. They are as follows:

1. Northwest Seattle
2. Burke-Gilman Trail
3. Inverness
4. Laurelhurst
5. Southwest Magnolia
6. Southwest Queen Anne
7. East Queen Anne
8. Northwest Queen Anne
9. North Capitol Hill
10. Lakeview Boulevard
11. Interlaken
12. Madrona
13. Rainier Avenue S.E.
14. West Beacon Hill (I-5)
15. West Marginal Way
16. Alki
17. Admiral Way
18. Beach Drive S.W.
19. 47th Avenue S.W.
20. Seola Beach
21. Pigeon Point
22. Cheasty Boulevard S.

There are many other scattered areas of landsliding and singular landslides in the city; however, the areas listed above are those where densities and frequencies are the greatest. Five areas in particular appear to have the highest density of landslides: Southwest Magnolia, Southwest Queen Anne, Madrona, Interlaken, and Alki. The pattern for natural landslides (slide records that did not indicate human influence), as shown on Figure A-8, mimics the general map of landslide locations. Except for Southwest Queen Anne, the most dense areas of natural landslides appear to be the same as those that have the highest density, considering all landslides. This is no coincidence because those areas that are naturally unstable are more likely to continue unstable behavior when human activity disturbs the area than relatively stable areas. It has long been recognized that disturbance of the ground surface and improper drainage increases the frequency of landsliding.

The chart on Figure 1-9 indicates that only about 13 percent of the landslides recorded citywide were totally natural. For three percent of the events, there was not enough data to categorize if the slide was natural or influenced by human activity. About 84 percent of the landslides were determined to have some factor of human influence. This is consistent with other studies and estimates, including the estimated 80 percent in *Landslides in Seattle*, 1974.

Four maps (Figures A-9 through A-12) present the distribution of each of the four landslide types. In addition, a chart presented on Figure 1-10 indicates the percentage of each of the types of slides. The majority citywide (68 percent) were shallow colluvial slides, followed by deep-seated landslides at 20 percent. High bluff peeloff (3 percent) and groundwater blowout (6 percent) landslides were small percentages. A combination of shallow colluvial and deep-seated landslides accounted for 88 percent of the total landslides. Three percent of the landslides could not be categorized because of insufficient information in the records.

The distribution of high bluff peeloff landslides is shown on Figure A-9. This type of slope instability (illustrated on Figure 1-2) only occurs on precipitous cliffs, which are normally comprised of till or sand. Many, but not all, of these landslides are naturally triggered. They are either associated with the headscarps of deep-seated landslides or old sea bluffs that formed prior to the construction of shoreline protection. Such high bluffs are found in only a few locations: Perkins Lane, Northwest Seattle, and Southwest Queen Anne.

Groundwater blowout landslides (illustrated on Figure 1-3) are spread around the city, as shown on Figure A-10. They are a direct indicator of the sand/clay contact, where high groundwater pressures commonly exist. This landslide type may be more common than indicated by this database, but they are difficult to discern from the older records. Only those slides where an engineer or geologist noted the characteristics of this type of slide were placed in this category. Some of these slides are natural because they are related to groundwater, which is more likely than not from natural sources. Groundwater blowout landslides occurred in Northwest Seattle, Southwest Magnolia, Southwest Queen Anne, Alki, and West Beacon Hill (I-5).

The locations of deep-seated landslides are shown on Figure A-11. They are located in significant numbers in the following areas: Southwest Magnolia, Northwest and Southwest Queen Anne, East Queen Anne, Alki, Admiral Way, West Beacon Hill (I-5), Interlaken, Madrona, and Pigeon Point. Because deep-seated landslides (illustrated on Figure 1-4) are dependent on regionally recharged groundwater, these slides are mostly natural. This type of slide commonly encompasses several properties and sometimes one or more city blocks.

As shown on Figure 1-10 and Figure A-12, shallow colluvial landsliding is the most prevalent and widespread type in Seattle. The areas with the highest densities of shallow colluvial landslides include Burke-Gilman Trail, Laurelhurst, Madrona, Rainier Avenue S.E., Alki, Beach Drive S.W., East Queen Anne, Southwest Queen Anne, Southwest Magnolia, and 47th Avenue S.W. Although the distribution of this type of slide (illustrated on Figures 1-5 and 1-6) indicates that they follow the overall pattern of landslides in the city, they often occur outside of the areas where natural slides occur because a shallow colluvial slide is the type of landslide most likely to be caused by human activity.

4.4 Causes of Landslides

In its two most basic elements, a landslide can be categorized as natural or human influenced. Virtually all landslides in Seattle occur where natural factors are conducive to landsliding, but many are also influenced by human activity. It is normally difficult to discern the percentages of contribution between these two elements to landsliding. Likewise, it is very difficult to assign percentage of contribution among the many human-influenced contributing factors in a landslide. The natural factors that contribute to landsliding (geologic conditions, topography, freezing and thawing, heavy or prolonged precipitation, and natural groundwater seepage, among others) are conditions to be accepted. For sites or areas located on or near slopes, there is always a risk that instability can occur. Engineering solutions are generally available to reduce the risks to acceptable levels of safety; however, there will always be risks. As shown on Figure 1-9, totally natural slides only comprise about 13 percent of the total number of slides in the records reviewed. Their distribution is shown on Figure A-8. In general, totally natural slides are most likely deep-seated, groundwater blowout, or high bluff peeloff landslides. Deep-seated slides and groundwater blowout are influenced most often by regional groundwater sources. High bluff peeloffs are in mostly inaccessible locations that are not susceptible to human disturbance.

Some factor of human influence was reported in 84 percent of the landslides citywide in Seattle. The implications of this are that there are measures that can be implemented by the City and by private property owners to reduce the risk of damage to public and private properties. Some of the human reasons that contribute to landsliding in Seattle include improper drainage/subdrainage, broken or leaking pipes, excavation at the toe of a slope, fill placement at the top or side of a slope, imprudent cutting of vegetation, and the lack of maintenance of drainage facilities or vegetative cover. All of these factors have been chronicled in the files that were reviewed for this landslide inventory and study.

The percentage of reported landslides with some factor of human influence noted in this study may be high with respect to the total number of landslides that have taken place within the City. The actual percentage of landslides with such human influence may actually be lower. The reasons for this are as follow:

- Only the reported slides were included in the database that provided the basis for this study.
- Many of the reported landslides were those where people were making claims usually against the City.
- The reported landslides were generally in developed areas, and totally natural landslides in other areas may not have been reported.

Nevertheless, property owners and developers need to realize that human influences can be significant contributing factors to instability. It is thus imperative that competent professional advice be obtained to reduce the risks of landsliding and damage. On the other hand, the influence of geology in this area must also be recognized as a significant contributing factor in landslides since, with or without some factor of human influence, most of the instability occurs on the steep slopes that surround Seattle's ridges, near the sand-clay contact, or at other locations where adverse soil layering and groundwater conditions are present.

4.5 Potential Slide and Steep Slope Areas

The City of Seattle presently regulates public and private development in environmentally-critical areas by requiring special standards for design and construction in potential slide areas (and known slide areas) and steep slope areas. Potential and known slide areas are defined by historical landslides and by a zone encircling many of the hills and ridges based on the sand/clay contact as shown in Tubbs' *Landslides in Seattle*, 1974. Steep slopes are defined as slopes steeper than 40 percent, with a rise exceeding 10 vertical feet. These restricted areas are shown on maps prepared by the DCLU. If a proposed new development is within one of these zones, geotechnical evaluations must be completed to obtain a permit for construction.

Some of the benefits of accurately delineated potential slide areas in Seattle are for zoning, administration of construction permits, notification for landslide education and public meetings, and emergency notification.

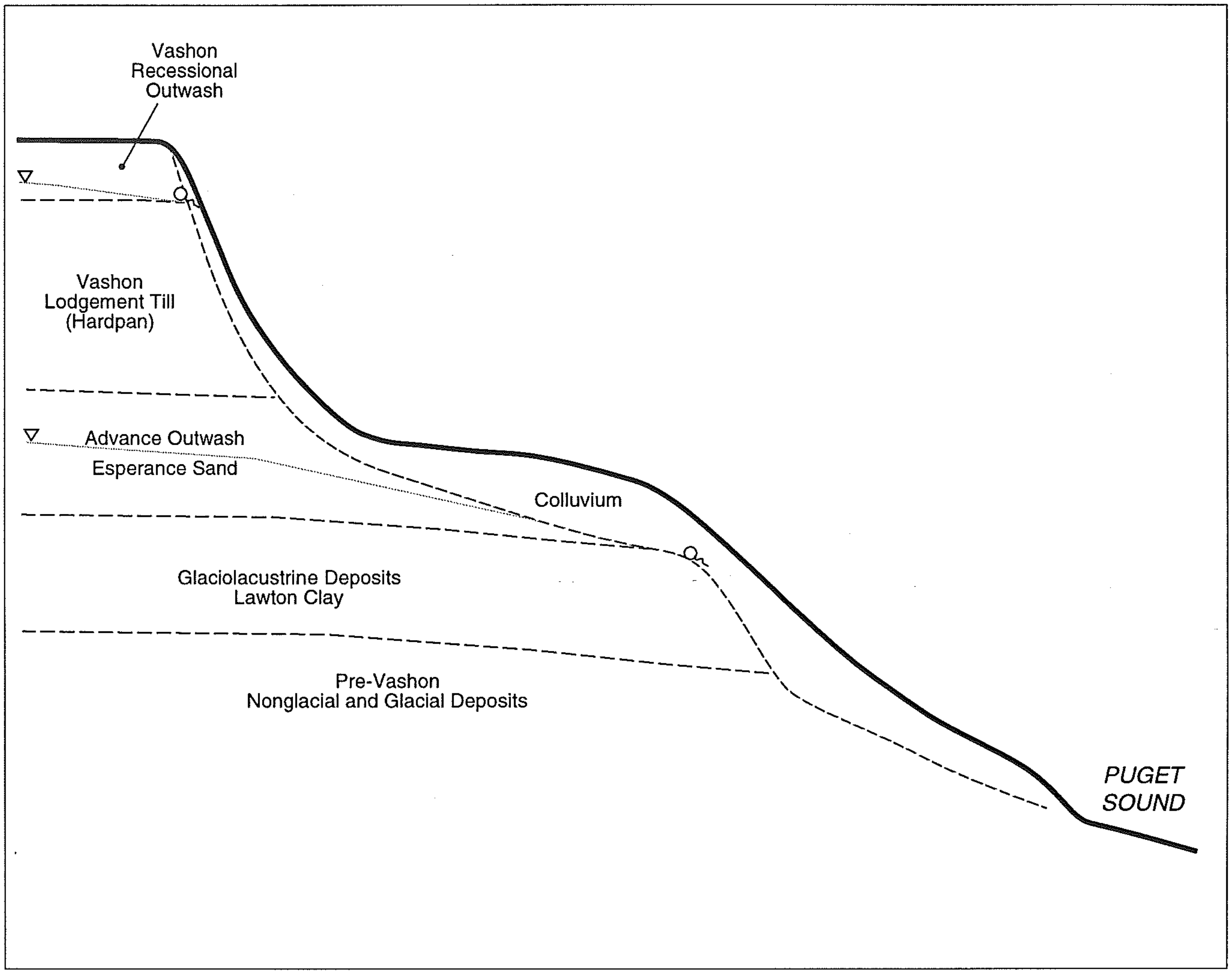
Some of the pitfalls for such accurate mapping may be complaints from property owners who did not want to be included within a restrictive zone, and the variation in accuracy of the data and geologic conditions on an individual property.

A common theme of homeowners who object to being included in a potential slide area is that their property values would be diminished. It has been our experience that property values can be reduced temporarily for one or two years when landsliding is active on a property; however, upon remediation of the instability, the property values revert again to the same or higher value as before the landslide occurred. Most of the property that is in a potential slide area is also view property that has risen steadily in value, unless an individual property is impacted by a landslide without suitable remediation. Therefore, it is beneficial for a public agency to accurately map and regulate construction in such sensitive lands to assist the public to prevent unwarranted reduction of property values. Knowledge of the potential landslide risk and education of property owners (including public owners) are the most effective methods to maintain property value in the long-term.

Of the total number of landslides in the database, 58 percent were within existing mapped potential slide areas and 76 percent were within the existing mapped steep slope areas. Note that 24 percent of the landslides occurred on slopes flatter than 40 percent. This shows the need for improving the potential slide area mapping. The percentage of landslides within either the steep slope or existing potential slide areas was 88. Nine areas in Seattle were identified where clusters of slides were outside of the potential slide areas. Figure A-13 is a map showing the landslides in Seattle in relationship to the potential slide areas. Those areas where significant groups of slides were outside the designated potential slide areas include: Interlaken, North Capitol Hill, Laurelhurst, Shilshole (south end of Northwest Seattle), the hillside west of Burke-Gilman Trail, Seola Beach, Rainier Avenue S.E., Mount Baker Ridge (south of I-90, at south end of Madrona), Admiral Way S.W. (southern end), Alki (high elevations), and 47th Avenue S.W. As shown on a map of landslides in relationship with steep slope areas (Figure A-14), the outliers are primarily scattered, isolated occurrences, but five areas have concentrations of slides outside the steep slope areas: Burke-Gilman Trail, Northwest Queen Anne, Admiral Way, east of Lincoln Park (in Beach Drive S.W. area), and the east side of Beacon Hill (Cheasty Boulevard S.).

A subset of shallow colluvial and groundwater blowout landslides is debris flows, those landslides defined herein as those that flowed more than 50 feet beyond the toe of the steep slope on which they fail. These slides are significant in that their runout zones do not normally confine themselves to the current potential slide areas, as presently mapped. As shown on Figure A-15, they are scattered around the city in the chronic landslide areas.

In our opinion, the above discussion points out the need for further definition of the hazard zones based on landslide prone characteristics and on runout zones that could impact downstream properties. Please refer to Part 5 for an evaluation of the existing Potential Slide Areas.



Not to Scale

LEGEND

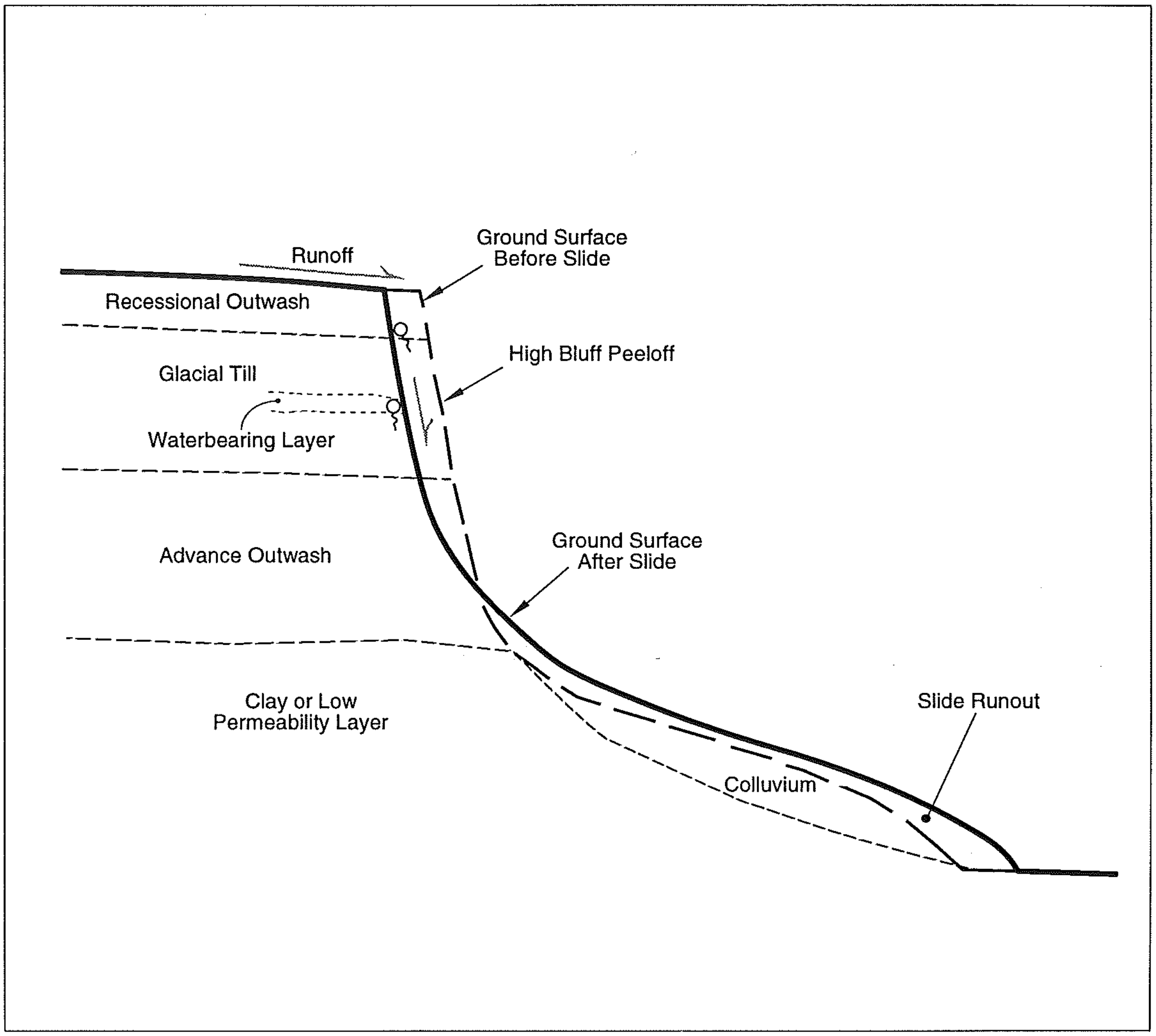
Seepage

Groundwater Table

NOTE

The thicknesses and elevations of these geologic units vary from place to place, and one or more units could be missing due to previous erosion.

Seattle Landslide Study Seattle Public Utilities Seattle, Washington	
IDEALIZED GEOLOGIC CONDITIONS TYPICAL SEATTLE HILLSIDE July 1999 W-7992-01	
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**FIG. 1-1**



Not to Scale

LEGEND

Slide Movement

Seepage

Seattle Landslide Study
Seattle Public Utilities
Seattle, Washington

HIGH BLUFF PEELOFF LANDSLIDE TYPE

July 1999 W-7992-01

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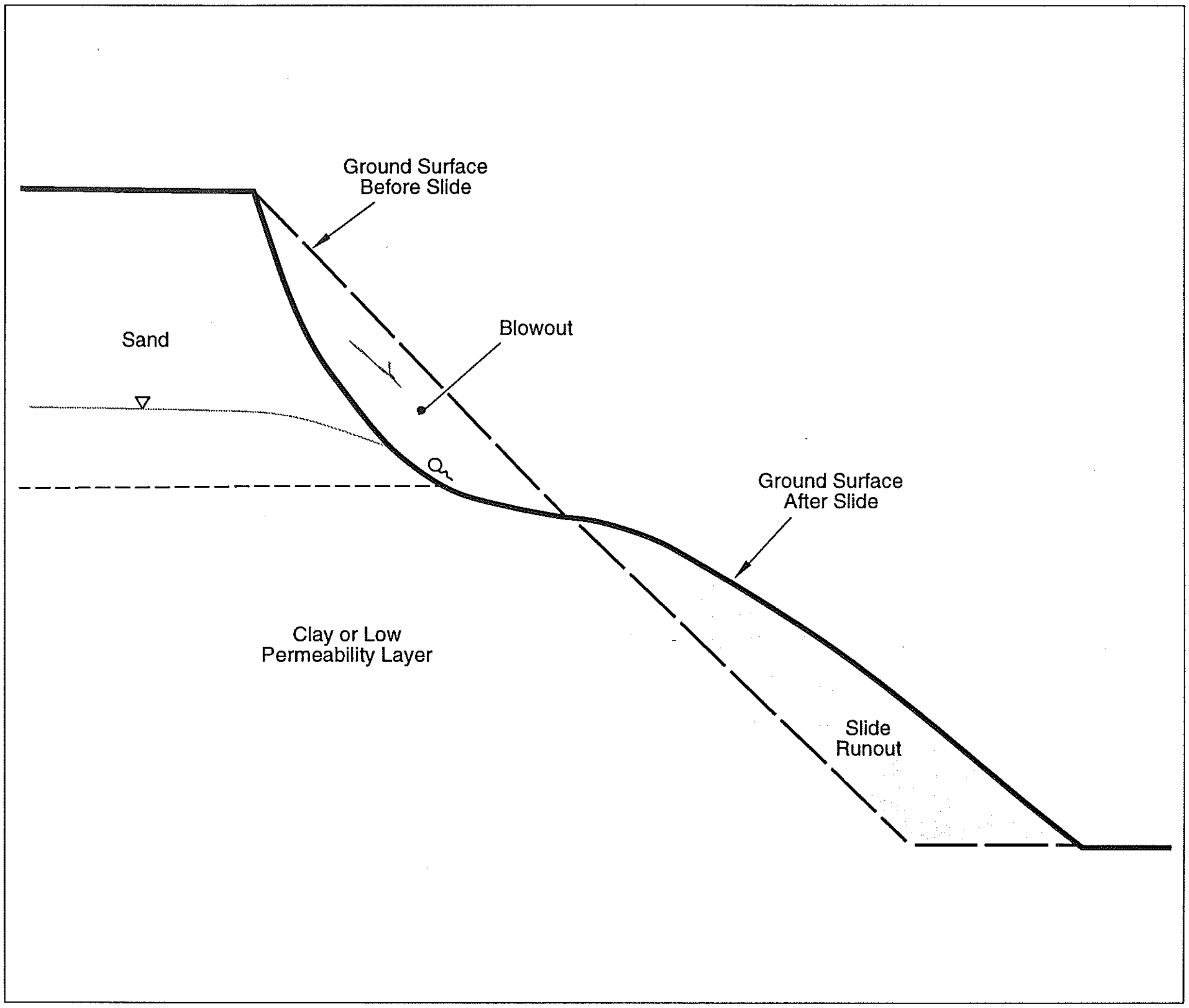
FIG. 1-2



Not to Scale

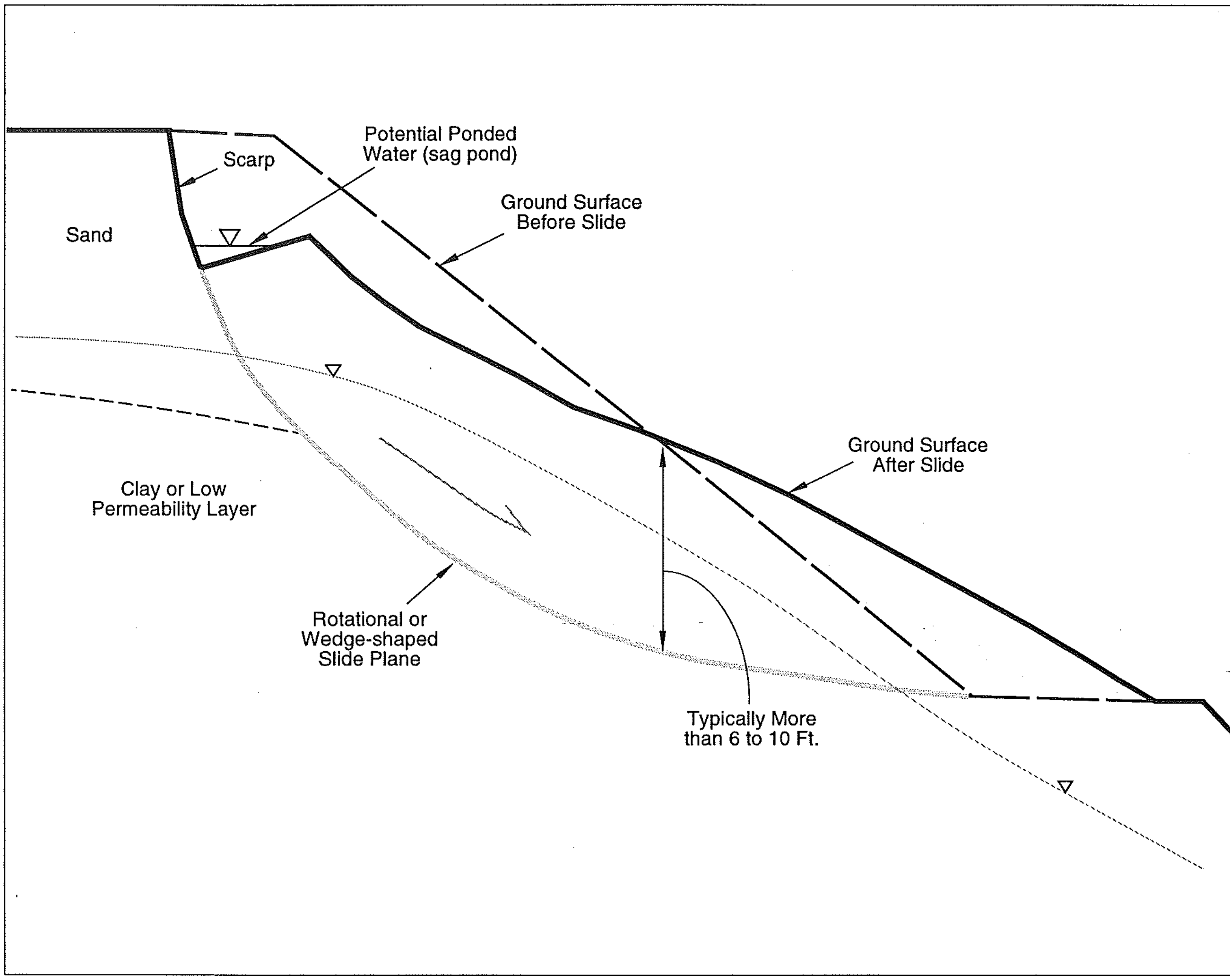
LEGEND

Perched Water

Slide Movement

Seepage

Seattle Landslide Study Seattle Public Utilities Seattle, Washington	
GROUNDWATER BLOWOUT LANDSLIDE TYPE July 1999 W-7992-01	
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**FIG. 1-3**



Not to Scale

LEGEND

Groundwater/Surface Water

Slide Movement

Seattle Landslide Study
Seattle Public Utilities
Seattle, Washington

DEEP-SEATED LANDSLIDE TYPE

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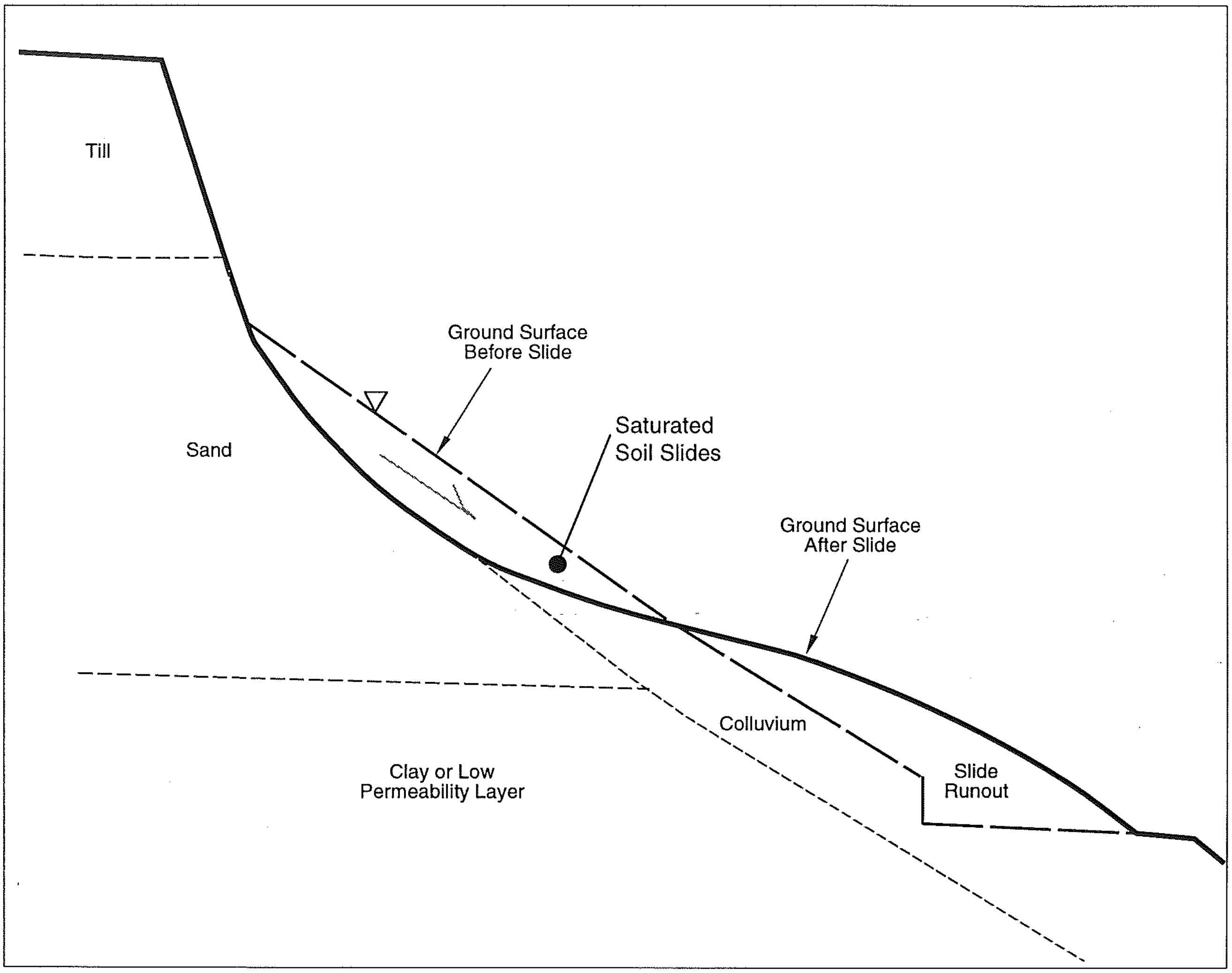
FIG. 1-4



Not to Scale

LEGEND

Slide Movement

Groundwater

Seattle Landslide Study
Seattle Public Utilities
Seattle, Washington

SHALLOW COLLUVIAL (SKIN SLIDE) LANDSLIDE TYPE

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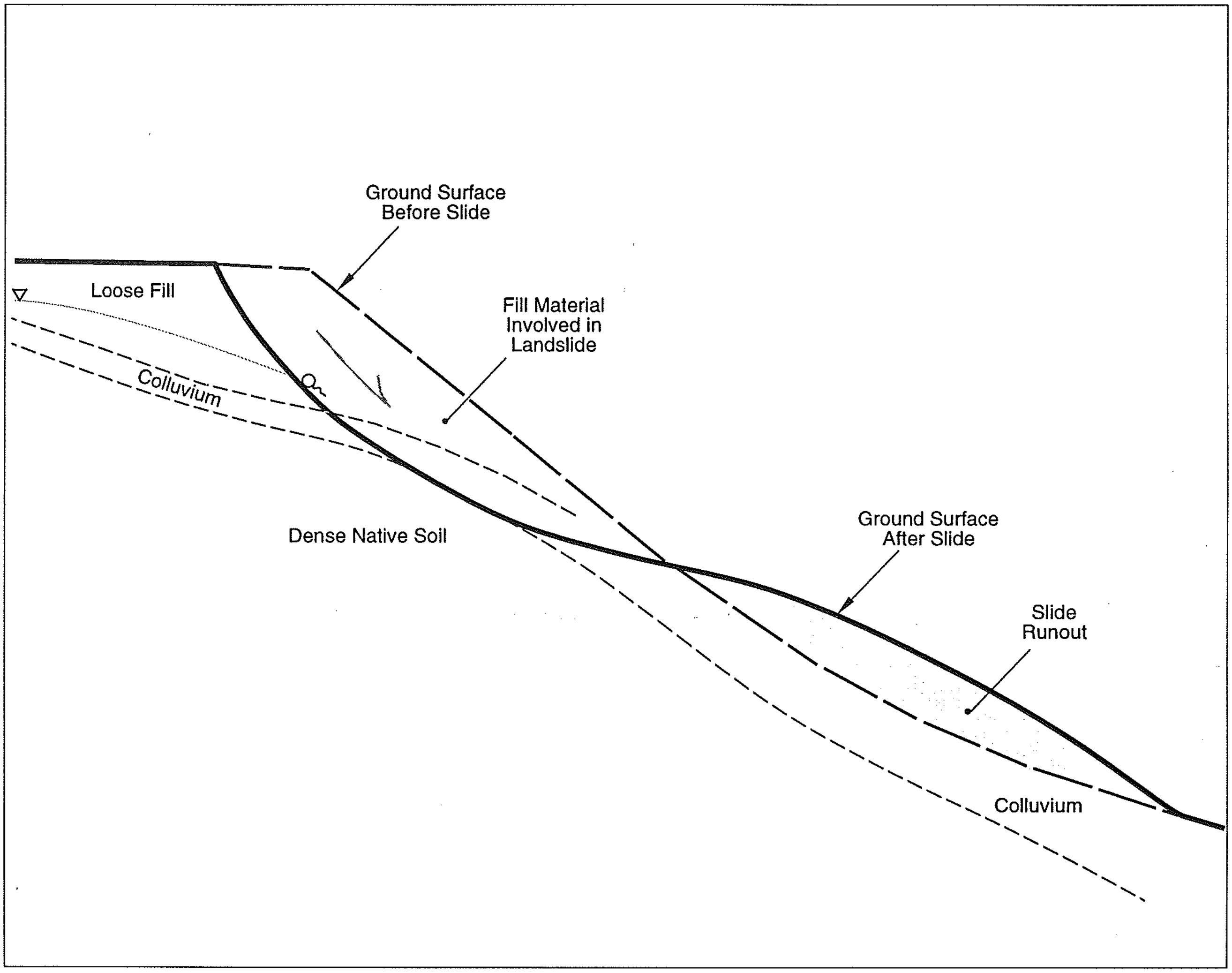
FIG. 1-5



Not to Scale

LEGEND

Slide Movement

Perched Water

Seepage

NOTE

This type of landslide was categorized as shallow colluvial landslide in the database table and landslide maps.

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LANDSLIDE IN FILL MATERIAL

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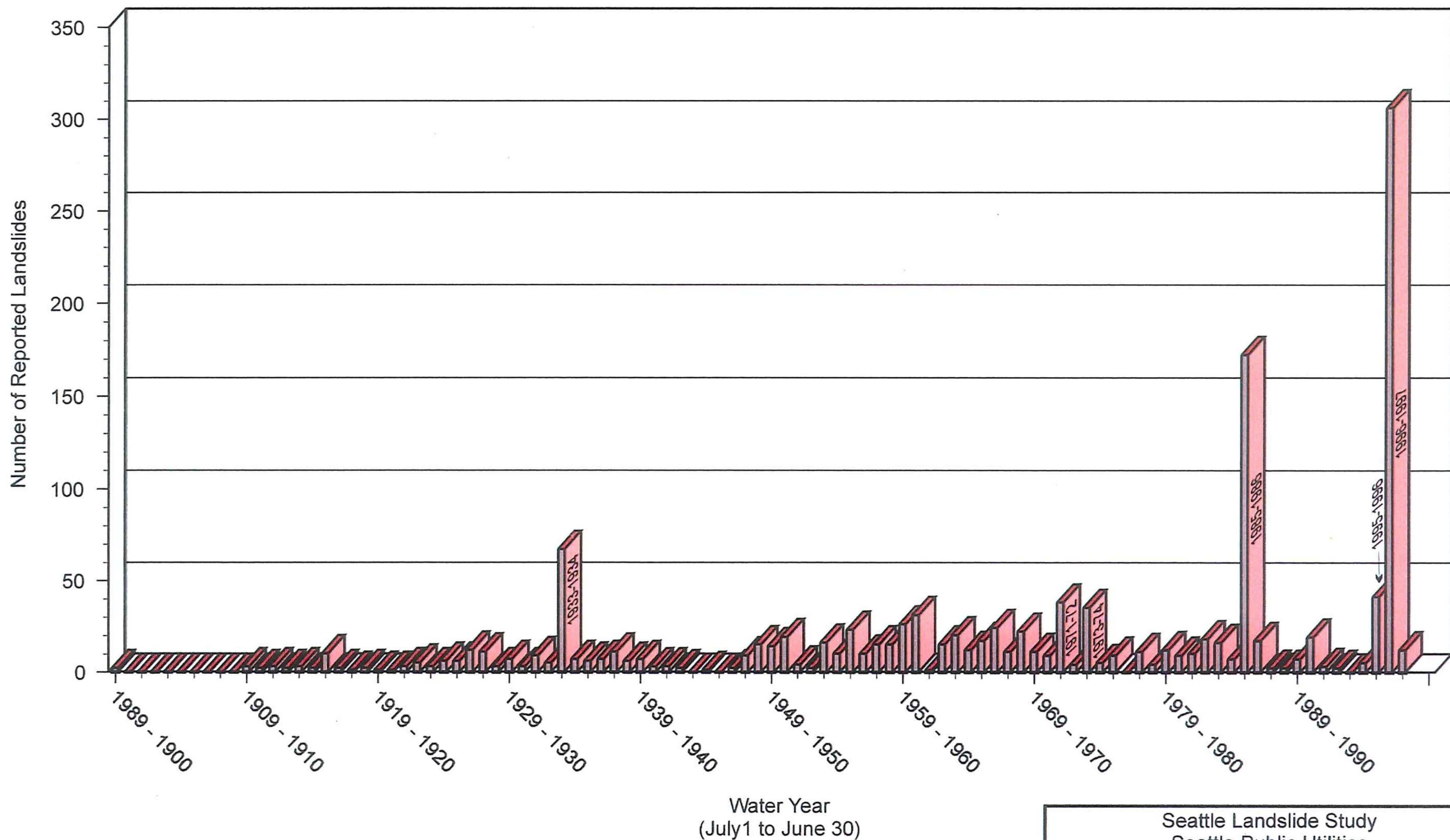
FIG. 1-6



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Seattle Public Utilities
Seattle, Washington

ANNUAL DISTRIBUTION OF LANDSLIDES

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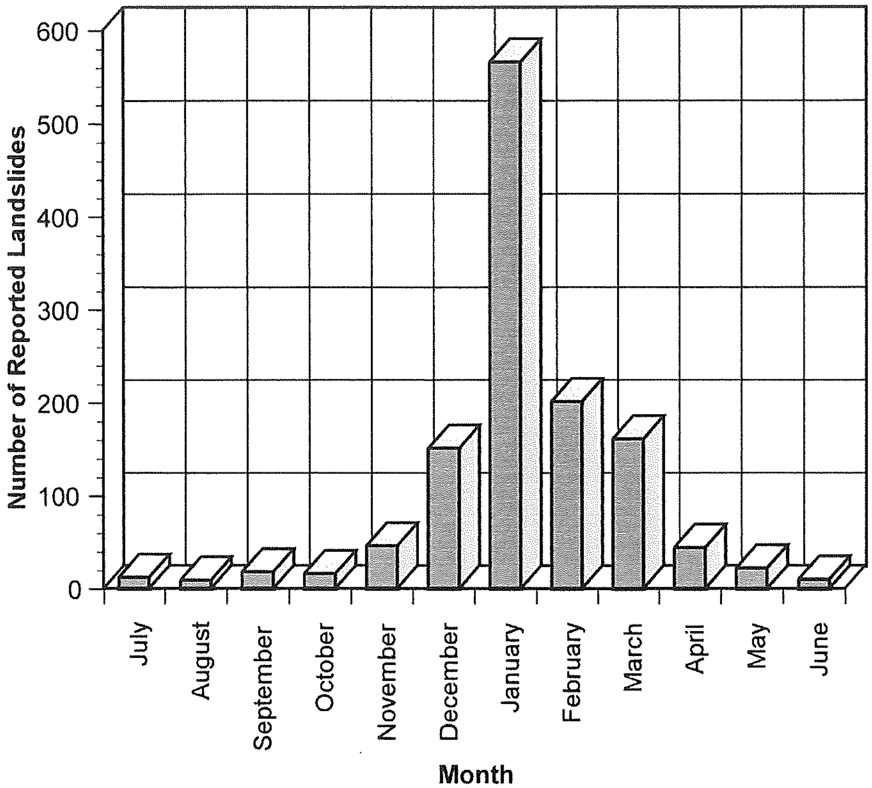
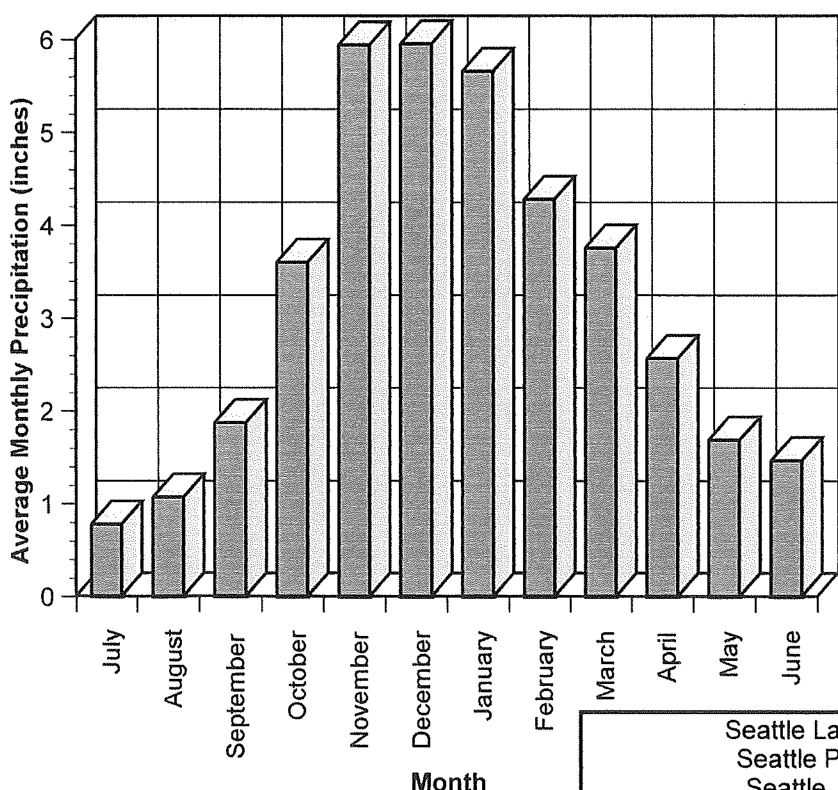
FIG. 1-7

FIG. 1-7





NOTES

1. Number of reported lanslides is for the period 1890 through 1999.

2. Reported average monthly precipitation is from the SeaTac station from January 1945 through June 1999.

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MONTHLY DISTRIBUTION OF LANDSLIDES

July 1999 W-7992-04

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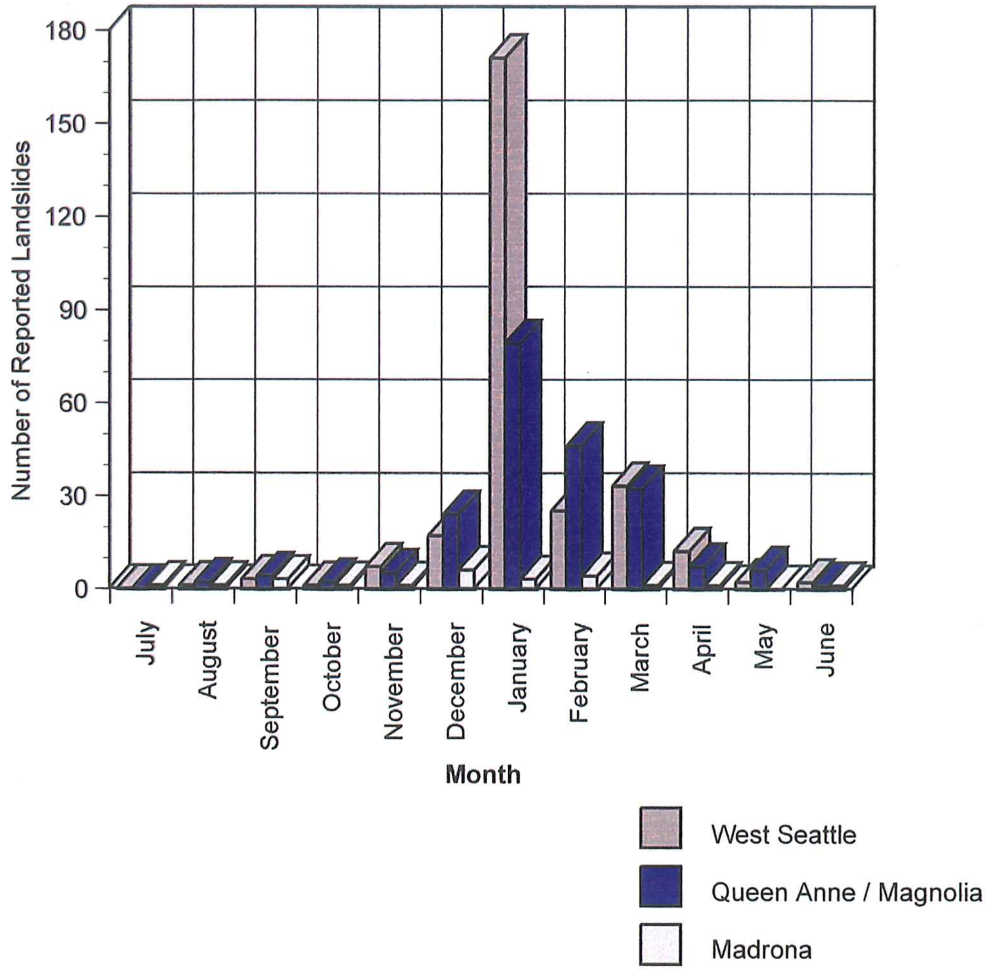
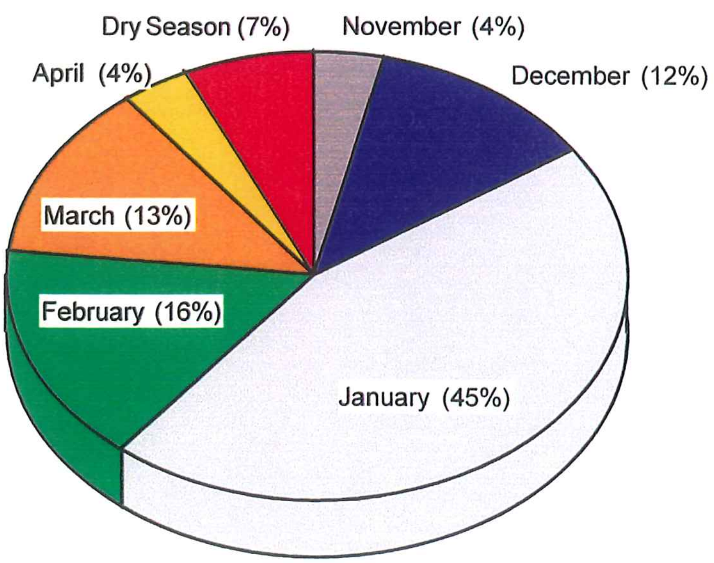
FIG. 1-8
Sheet 1 of 2





Seattle Landslide Study
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Seattle, Washington

MONTHLY DISTRIBUTION OF LANDSLIDES

July 1999 W-7992-04

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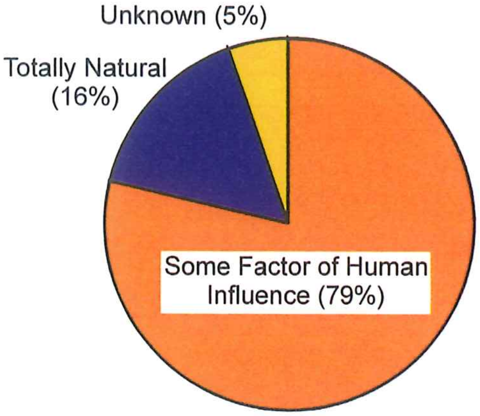
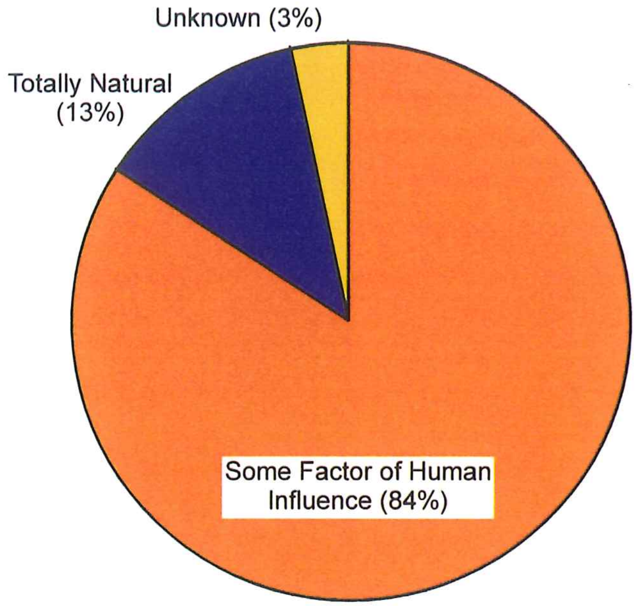
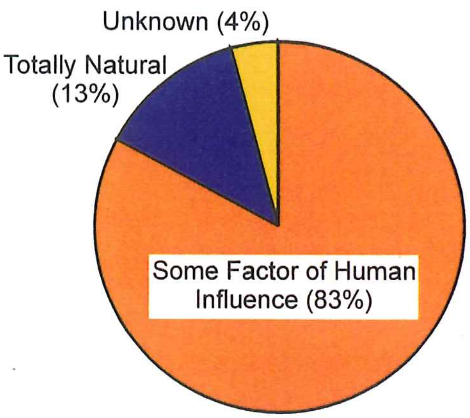
FIG. 1-8
Sheet 2 of 2



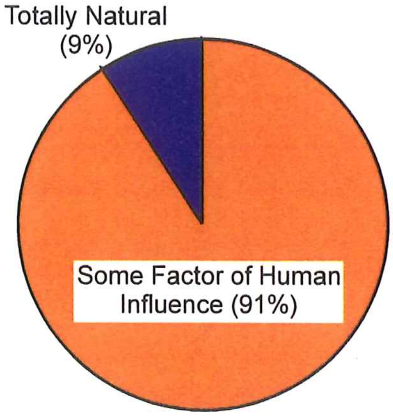
WEST SEATTLE



ALL CITY



QUEEN ANNE / MAGNOLIA



MADRONA

Seattle Landslide Study
Seattle Public Utilities
Seattle, Washington

HUMAN INFLUENCE ON LANDSLIDES

July 1999 W-7992-04

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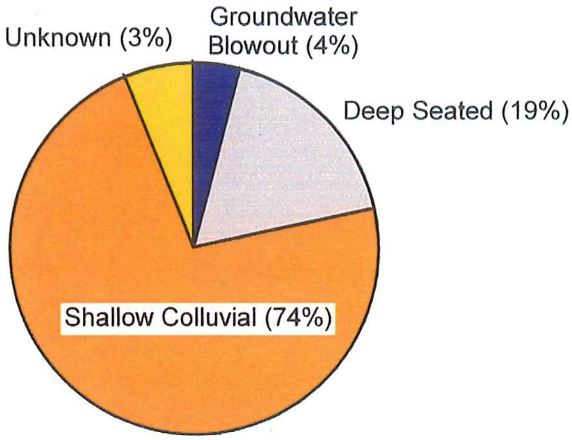
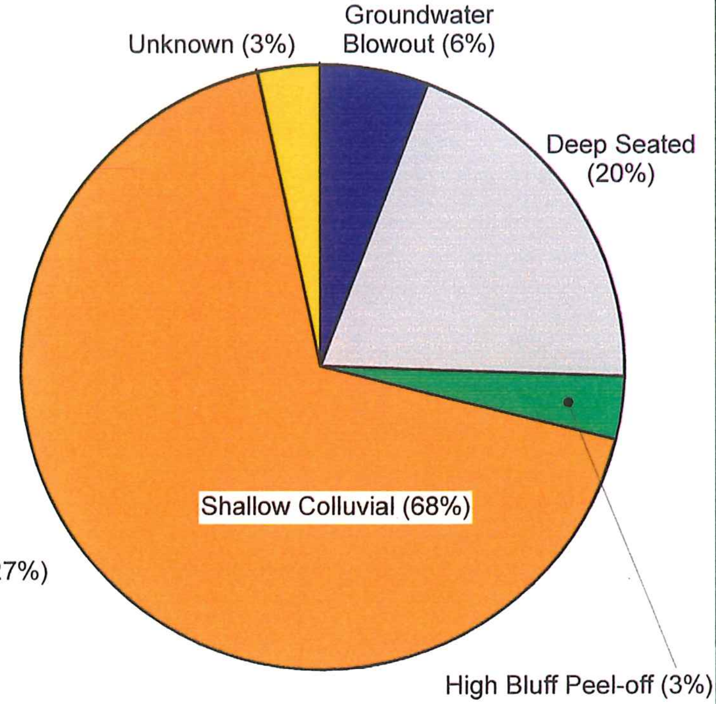
Geotechnical and Environmental Consultants

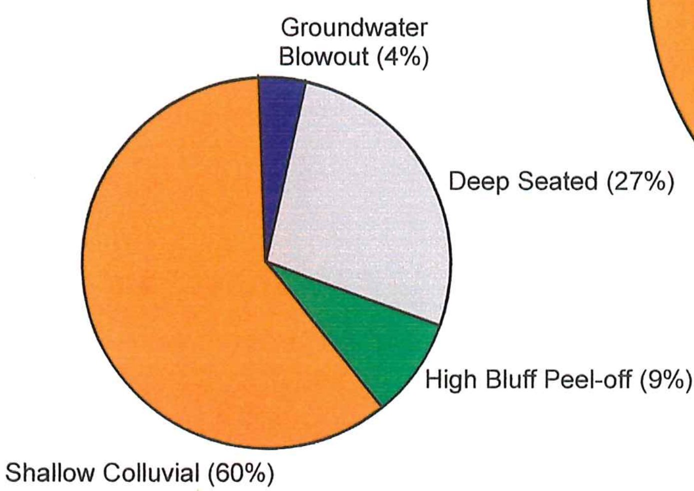
FIG. 1-9



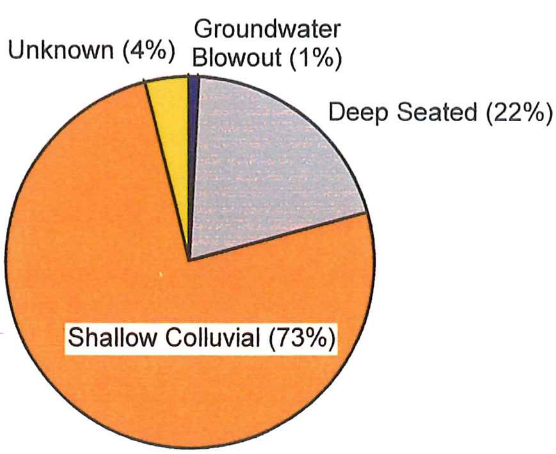
WEST SEATTLE



ALL CITY



MAGNOLIA / QUEEN ANNE



MADRONA

Seattle Landslide Study
Seattle Public Utilities
Seattle, Washington

TYPES OF LANDSLIDES

July 1999 W-7992-04

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FIG. 1-10

PART 2. GEOTECHNICAL EVALUATIONS

5.0 PURPOSE AND SCOPE

5.1 Purpose of Geotechnical Evaluations

Part 2 of this report presents a geotechnical engineering evaluation of the landslides that have occurred throughout the City of Seattle (City). In Part 3, engineering evaluations will be specifically related to three study areas: 1) West Seattle, 2) Magnolia/Queen Anne, and 3) Madrona. In Part 4, engineering evaluations will be related to additional study areas: Northwest Seattle, Northeast Seattle, Capitol Hill, and South Seattle. Based on these citywide and study area evaluations, typical measures will be presented to improve stability and reduce the risk of future landslides. In addition to preventive measures, remedial schemes will also be presented for landslides after they occur. Most of the stability improvements presented can be both preventive and remedial.

The purpose for our studies and recommendations regarding stability improvement is to inform both the public and representatives of the City of the factors that cause landslides and the steps that could be taken to improve stability. It is important for the City to protect utilities, drainage features, streets, and other City facilities; however, landslides do not obey property boundaries. Therefore, measures will be presented that could be made by the City and/or adjacent property owners to improve the stability of an entire landslide or unstable slope.

5.2 Scope of Geotechnical Evaluations

In order to meet the purpose described above, the following engineering evaluations have been made:

- We studied the landslide history described previously in this report (Part 1), with respect to topography, geologic and groundwater conditions, slide types, timing, City locations, and causes. This study has provided background for making engineering evaluations.

- For each type of landslide or potential landslide, we developed stability improvement measures consisting of surface and subsurface drainage, grading, and/or structures. Typical measures that could be applied citywide are presented in this report section (Part 2).

- We developed unit cost estimates for the typical measures applicable to citywide stability improvements (Part 2). These cost estimates can be extrapolated to provide budget figures for the stability improvements recommended in Parts 3 and 4 of this report.

- We studied the general implications of City utilities and streets as related to instability (Part 2).

- For the original three selected study areas (West Seattle, Magnolia/Queen Anne, and Madrona), we conducted detailed studies of the types and causes of landslides, including the effects of City utilities and streets, and we provide recommendations and cost estimate information for stability improvements. The results of these studies are presented in Part 3 of this report.

- For the four additional study areas (Northwest Seattle, Northeast Seattle, Capitol Hill, and South Seattle), we conducted studies generally similar to those accomplished for Part 3, and we provide recommendations and cost estimate information for stability improvements. The results are presented in Part 4 of this report.

6.0 TYPICAL IMPROVEMENTS RELATED TO LANDSLIDE TYPE

As presented in Part 1 of this report, most of the landslides were found to fit into four generalized types: 1) High Bluff Peeloff, 2) Groundwater Blowout, 3) Deep-Seated, and 4) Shallow Colluvial (includes landslides that involve fill material). There are various combinations of these generalized landslide types, as one type of mechanism may lead to another. This section describes approaches for repairing slopes with these types of landslides, improving the stability of slopes that could be affected by landslides, and reducing the hazard from debris flows to properties below landslides.

6.1 Geologic Conditions that Contribute to Landsliding and Instability

Part 1 of this report provides a detailed description of the geologic and hydrologic conditions that contribute to landsliding in the City. In general, the following factors affect the stability of a slope: topography, subsurface conditions, surface and groundwater conditions, and external loads, such as structures.

Factors that commonly trigger landslides include:

- Increased groundwater levels and surface runoff
- Removing support at the toe of the slope by erosion or by excavation
- Changes in the soil strength
- Loading the head of the slope with debris from another landslide or with manmade fills
- Seismic loading

Groundwater contributes to landslides in several ways. When saturated, a potential landslide block has more weight because of the water, which results in a larger driving force. Groundwater moving through the soil exerts seepage forces that further reduce stability. Finally, the presence of groundwater reduces the strength of the soil on a potential slide plane. Freezing weather can be an important process in reducing slope stability because frozen soil can impede groundwater seepage. When seepage is impeded at the surface, groundwater levels can build to cause an unstable condition.

Surface storm water runoff can reduce slope stability by infiltrating into the near-surface soils at critical locations, and by causing erosion. Where groundwater emerges at the surface, resulting in a spring or seep, the runoff can cause surficial erosion that can undermine and/or oversteepen a slope. Undermining and/or oversteepening and the consequent loss of support at the toe of the slope can trigger a landslide.

Prior to construction of seawalls along Puget Sound, the base of the bluffs and slopes were subject to continual shoreline erosion and oversteepening at the toe of the slope. Once undercut, the lower part of the slope would slide, thereby undercutting the slope at higher elevations. With the construction of seawalls and other shoreline protection measures, erosion has been arrested or greatly reduced. However, these slopes have not necessarily achieved a stable configuration, so landsliding may continue for the foreseeable future.

Development activities can result in undercutting and oversteepening slopes. This was more prevalent prior to modern building codes, such as the Department of Design Construction and Land Use (DCLU) Director's Rules 3-93, 3-94, and 3-97, regarding development in geologic hazard areas. Therefore, many oversteepened and improperly sloped or retained older cuts and excavations remain in the City, some of which contribute to instability. In general, modern cuts and excavations made under the guidance of a competent geotechnical engineer have achieved a suitable degree of stability.

Seasonal variations in moisture and temperature, combined with plant growth and decay, animal burrowing, and soil creep tend to reduce the strength of soil over time. This process particularly affects colluvial soil and glacially overridden soils that are exposed at the surface by an excavation or by a landslide.

Loads placed on or near the top of a marginally stable slope typically reduce the slope stability. These loads can be caused by debris from a landslide that occurred upslope, manmade fills or loads from structures. Modern fills that are designed and constructed under the direction of a geotechnical engineer generally are suitably stable. However, many older fills were built without proper subgrade preparation, adequate drainage or compaction. These fills include material that was loose-dumped without compaction in ravines and on slopes and loose sidecast road fills. A number of recent and older landslides in Seattle involve old uncontrolled fill material. Structures, heavy equipment, and material stockpiles that are built or placed on a steep slope or near the top of a steep slope can contribute to instability.

Many of the steep bluffs and slopes are susceptible to earthquake-triggered landslides. Both the 1949 Olympia and 1965 Seattle-Tacoma earthquakes caused a total of at least 41 landslides in the Puget Sound region. Recent geological and seismological research findings indicate that many of the large ancient landslides identified in the bluffs along Puget Sound were triggered by large prehistoric earthquakes.

6.2 Typical Approaches to Improve Stability

For each type of landslide, we evaluated potential stability improvements, which could be preventive and/or remedial. In general, the methods for achieving suitable stability for a site or project include: 1) avoiding the slope and 2) improving stability by reducing the forces that cause movement, increasing the forces that resist movement, or a combination of the two. These methods for improvement measures fall into several generalized categories, as presented in the following table.

APPROACH	PROCEDURE	EXAMPLES
Avoid the slope	Build structures, utilities and/or streets a safe distance from the landslide	Leave property undeveloped
	Build over the landslide, with supports on stable ground	Bridge over unstable area, build on deep piles or shafts
Reduce the driving forces	Remove weight from the upper, driving portion of the landslide	Flatten slope, remove material from the landslide top, move external loads away from the landslide top
	Remove the unstable material	Completely or partially remove unstable materials
	Drain surface water to reduce infiltration into the groundwater	Grading to promote drainage: ditches, swales, berms, storm sewers and tightlines, low permeability covers
	Drain groundwater to reduce the driving weight, seepage forces and erosion	Trench subdrains, springhead drains, finger drains, drainage blankets, drainage wells (horizontal, vertical and directionally drilled), drainage tunnels and adits.
	Build fills or replace existing soil with lightweight fills to reduce driving weight	Expanded polystyrene, sawdust, cinders, bottom ash.
Increase the resisting forces		
Apply external forces	Add weight to the resisting part of the landslide	Buttress, counterweights, toe berms
	Build structural retention systems to resist part of the driving forces	In situ walls (soldier pile, secant pile, tangent pile, etc.) and gravity walls (i.e., concrete cantilever, reinforced soil, gabion, crib, etc.)
	Install anchors that transfer driving forces into stable ground	Tieback anchors
Increase the soil strength	Drain the subsurface to increase the soil strength along the failure surface	Trench subdrains, springhead drains, finger drains, drainage blankets, drainage wells (horizontal, vertical and directionally drilled), drainage tunnels and adits.
	Install in situ reinforcement to increase the strength along the failure surface	Soil nails, anchors, piles, shafts
	Replace or modify the landslide soil to increase its strength	Excavation and replacement with high shear strength soil, improve soil by compaction or lime and cement stabilization, grouting, ground freezing
	Construct reinforced backfill that is stable on steeper slopes and has higher strength	Reinforced soil slopes and walls
	Use biotechnical stabilization to intercept rainfall and provide root reinforcement	Vegetation and vegetation combined with structural slope stabilization

In some cases, it may be difficult or not practical to improve the stability of a landslide; but structures, streets, and/or utilities located below the landslide could be damaged by slide debris. These circumstances could occur where a high bluff is present or where a potential landslide is on another property. In such cases, the areas below the landslide can be protected from slide debris with catchment or diversion structures designed for impact forces.

The following sections describe how these improvement measures or combination of measures could be applied to the four generalized landslide types described previously. For each landslide type, we present several sketches that diagrammatically show typical applications of different measures to improve stability. They are not intended to show all types of stability improvements, nor design details for improving stability of a slope or a landslide. Subsequent sections provide details regarding the common improvement measures. The details include a description of how each measure improves slope stability and general design requirements and details.

6.3 High Bluff Peeloff Landslides

The main factors that lead to high bluff peeloff landslides are nearly vertical slopes, groundwater seepage, and surface water runoff. In many cases, little can be done to prevent these landslides because of their height, steepness, and inaccessibility. The nearly vertical bluffs typically were formed by coastal erosion. Although this erosion may have been arrested or slowed by recent shoreline protection, the slopes have not achieved a stable slope through erosion and landsliding.

Figure 2-1 (3 sheets) shows simplified sketches of a high bluff peeloff landslide, together with several alternatives for reducing the likelihood of a high bluff peel off landslide, and protecting a structure, street, and/or utility below the bluff, as shown. Unless the bluff is low or otherwise accessible, remedial measures to reduce the likelihood of a landslide typically are limited to surface and groundwater improvements at the top of the bluff, as shown. Where structures, streets, and/or utilities are located below a bluff and in the likely landslide runout zone, measures can be taken to reduce damage from the landslide debris. These include building sufficiently far from the bluff that landslide debris should not affect the structure, building catchment, or diversion structures, and removing trees that likely would be incorporated into the landslide debris.

In cases where the bluff is accessible and/or where the consequences of a landslide are high, the slope could be retained with a wall. In general, this type of repair is costly and may not be economical or practical to build. Suitable wall types depend on the height of the bluff, and the topography and geologic conditions of the slope below the bluff. These wall types could include soil nails with reinforced shotcrete, as shown on Figure 2-1, Sheet 2 of 3, and soldier pile walls with lagging if the slope is low (Sheet 3 of 3). For a soil nail wall, reinforcing elements would be required into the bluff face. For a soldier pile wall with lagging, the soldier piles could be tied back or cantilevered. Installation of these soil nails or anchors would be expensive and access to the bluff would pose safety concerns for the workers.

We recommend removing hazard trees, other large vegetation, or structures that likely would be incorporated into a high bluff peeloff landslide. Such objects incorporated into landslide debris have damaged structures located below bluffs. Trees that are isolated or subjected to high winds that accelerate over bluffs are more likely to be uprooted. An uprooted tree can initiate a high bluff peeloff landslide.

6.4 Groundwater Blowout Landslides

Groundwater blowout landslides occur where a relatively permeable soil overlies a less permeable soil, resulting in perched groundwater and seepage towards the slope face. The high groundwater levels and seepage towards the slope face result in destabilizing seepage pressure and reduced soil strength. Seeps and springs that form where groundwater exits the slope face often cause erosion that can undermine and oversteepen a slope further reducing the stability.

Figure 2-2 (2 sheets) shows four simplified sketches of a groundwater blowout landslide, together with several alternatives for reducing the likelihood of a landslide. Because the primary driving force is groundwater seepage, suitable remedial measures usually include drainage to lower the groundwater level and to control seepage at the slope face. Drainage measures usually are most effective when they intercept groundwater at the contact between the relatively permeable soil and the underlying less permeable soil.

Sketch A on Figure 2-2 (Sheet 1) shows the application of an interceptor trench subdrain and a springhead drain. Both improve stability by lowering the groundwater level in a landslide or potentially unstable slope, thereby reducing the driving forces and increasing the soil strength. The springhead drain is used to collect water that emerges from the slope in a concentrated area, thereby reducing erosion potential and improving stability. Trench subdrains generally are applicable to slopes where the contact with the underlying low permeability material is relatively shallow. An interceptor trench subdrain is installed across the slope to intercept the groundwater before it reaches the slope face. Sketch B shows another type of trench subdrain, called a finger drain. It is similar in construction to an interceptor trench subdrain, except that it is installed along the slope fall line (perpendicular to slope contours).

Sketch C on Figure 2-2 (Sheet 2) shows two alternatives for drilled drains: horizontal drains and directionally drilled drains. Drilled drains are typically used to improve stability of slopes and landslides where the groundwater cannot be intercepted with trench subdrains, or where it is not practical to excavate trench subdrains. Drilled drains are commonly used to improve the stability of large deep-seated landslides. Horizontal drains are drilled from the slope face, which limits their application to sites that have suitable access near the toe of the landslide mass. Directionally drilled drains usually are installed from the top of the slope and can be aimed to intercept a specific zone where the drainage is needed. Vertical wells (not shown) can be used in special cases; however, their suitable application is limited. Vertical wells require continual pumping to maintain lower groundwater levels. As such, they incur the cost of electricity and are subject to power outages during critical rainy periods.

A replacement earth buttress is sometimes used to improve a marginally stable slope and more commonly to repair a landslide that has already occurred. As shown by Sketch D, the landslide mass or potentially unstable soil is removed and replaced with a well drained fill material. In some cases, the excavated soil can be recompacted to form the earth buttress, while in others a suitable imported backfill is compacted to form the earth buttress. In either case, an effective drainage layer and subdrain should be constructed under the earth buttress.

6.5 Deep-Seated Landslides

As described in Section 4.1.3, deep-seated landslides can occur in a variety of geologic settings. Most deep-seated landslides consist of a relatively large block of soil that may remain partially intact as it slides downhill on an arc- or wedge-shaped failure surface. The size of a deep-seated landslide can vary from a single backyard to a city block or more. Groundwater usually contributes to deep-seated landslides, although the source of the groundwater may not be clearly related to a contact between relatively high and low permeability soils. Because of the varied geologic and hydrologic conditions that contribute to deep-seated landslides, the alternatives for repairs are equally varied.

Figure 2-3 (3 sheets) shows five simplified sketches of a deep-seated landslide, together with several alternatives for improving the stability. Depending on the soil types and groundwater levels, the various schemes for dewatering that are shown on Figure 2-2 for the groundwater blowout landslide may be applicable. If the landslide failure surface is too deep to drain with trench subdrains, horizontal or directionally drilled drains could be used.

Deep-seated landslides often can be repaired by adding weight to the toe of the landslide and/or removing material from the upper, driving part of the landslide. Sketches A through F show various schemes for improving the stability of a deep-seated landslide or potential landslide.

Sketch A on Figure 2-3 (Sheet 1) shows an earth buttress constructed at the toe of an existing or potential landslide to add weight to the lower, resisting part of the landslide. In general, earth buttress, counterweights and toe berms should include a drainage layer beneath the main fill to reduce the potential for destabilizing groundwater conditions. Sketch B shows a similar situation, except that the toe of the fill is retained to accommodate a street, property line, or other construction-access limitations. The wall at the toe of the retained earth buttress fill could be a reinforced soil wall, as shown on the Sketch, or another type of wall. Both in situ and gravity wall types, as defined in the table with Section 6.2, could be suitable, with the choice of wall type depending on the geometry of the earth buttress fill.

Sketch C on Figure 2-3 (Sheet 2) shows an example where soil is removed from the upper, driving part of the landslide and removed from the site. The slope behind the landslide or marginally stable slope should be graded so it will be stable. Alternatively, a retaining wall could be used to reduce the loss of level ground above the landslide. In this instance, a soldier pile or other in situ wall probably would be most effective, although other types could be used if a suitable foundation can be provided. Sketch D shows an example where the stability is improved by a combination of removing soil from the upper driving part of the landslide and adding an earth buttress fill to the lower resisting part of the landslide. For the example shown, the excavation and fill quantities are approximately equal, so that most material would be derived and disposed of on site. Retaining walls could be used at the top or at the toe of the landslide to keep within rights-of-way or natural grade limitations.

Another alternative that reduces the driving weight is shown on Sketch C. After removing soil from the upper driving portion of a deep-seated landslide, lightweight fill material could be placed to restore grades. Wood chips, cinders, and expanded polystyrene (geofoam) are frequently used in this type of application.

Sketch A on Figure 2-3 (Sheet 1) also shows the use of an in situ wall to retain soil in the upper driving part of a marginally stable slope or landslide that has had incipient failure or small displacements. The piles also provide reinforcement across the landslide failure surface. In situ walls include soldier pile walls with lagging, secant piles and tangent piles (details given in Section 7.0). This type of repair is appropriate when other less expensive alternatives, such as drainage and regrading are not practical because of site limitations. Sketch E on Figure 2-3 (Sheet 3) shows an example where an in situ wall is used to retain the slope above the scarp after a landslide occurs. In this example, the wall retains the ground above the landslide and prevents progressive upslope failure. However, the ground below the scarp is not restored nor is the stability improved. Sketch F shows an example where a drained earth buttress is built at the bottom of the slope to improve stability and a slope fill is built to restore the grades. A reinforced soil wall or other type of gravity wall could be used to steepen the toe of the drained earth buttress if needed for property or site access limitations.

6.6 Shallow Colluvial Landslides

As described previously, colluvium is present on most slopes in Seattle. Because of the typically shallow depth, colluvium is particularly susceptible to rapid saturation from infiltration of surface runoff, direct infiltration of precipitation, groundwater seepage, discharge from pipes, or a combination of these sources of water. Shallow colluvial landslides are the most common type of landslide in Seattle.

Figure 2-4 (2 sheets) shows simplified sketches of a shallow colluvial landslide, together with several alternatives for reducing the likelihood of a landslide. They also show alternatives for protecting a structure, street, and/or utility below the slope, if it is not practical to stabilize the slope.

Sketch A Figure 2-4 (Sheet 1) shows a combination of surface drainage, subsurface drainage, removing hazards from the site, and a catchment wall. Depending on the circumstances, these measures could be used individually or in combination. As discussed previously, colluvium is particularly sensitive to rapid saturation from both surface water runoff and groundwater seepage. Therefore, all storm water runoff from roof drains, paved surfaces, foundation drains, etc., should be collected into a tightline and discharged to a suitable location. Ideally, storm drainage collected from the top of the slope should discharge to a storm sewer located at the top and back from the edge of the slope. If this is not possible, a tightline should convey this storm water runoff to the bottom of the slope or to a storm sewer located downslope. It is usually advisable that the tightline not be buried, to prevent breakage from soil creep or landslide movement.

Trench subdrains are often effective for providing subsurface drainage in colluvium. The depth to the underlying glacial soil is usually within the reach of a backhoe or even hand-excavated trenches. Both interceptor trench subdrains, as shown on Sketch A, and finger drains, as shown on Sketch B of Figure 2-2 (Sheet 1), can be effective. Drilled drains usually are not practical, except where site access precludes construction other than directionally drilled drains.

In some cases, it is not practical or possible to improve the stability of the slope. A catchment wall, as shown on Sketch A of Figure 2-4 (Sheet 1), can protect structures, streets, and/or utilities below the landslide. As stated previously, we recommend removing trees, other large vegetation, or structures that likely would be incorporated in a landslide. Trees and other large debris incorporated in a debris flow can cause as much or more damage than the moving soil. It is also possible to use the catchment wall as an in situ wall designed to retain potential sliding soil.

Sketch B shows a retaining wall to provide support for a marginally stable slope, or to allow an excavation at the toe of a slope. The type of wall will depend on the site conditions and access limitations. An in situ wall type, as shown on the sketch, often is needed to construct the wall within property lines. In addition, such walls can be built before making the excavation, so the slope is continually supported. Other improvements that could be made include an earth buttress and subsurface drainage.

Sketches C and D, on Figure 2-4 (Sheet 2), show two schemes for repairing a shallow colluvial landslide. Sketch C shows a case where site access limitations prevent extensive work on the slope above a bench where a structure, street, and/or utility is located. If springs or seeps are present on the slope, springhead drains could be installed to promote good drainage. Finger drains could be installed in the remaining colluvium at the base of the slope, as shown on Sketch B of Figure 2-2 (Sheet 1).

Sketch D shows a repair where the slope is restored by removing the landslide debris and unstable colluvium, and placing a well-drained structural fill or reinforced soil slope. Depending on the required slope, desired use, and property limitations, a gravity wall or other type of wall could be constructed at the toe of the new fill.

Landslides involving fill material typically are similar to shallow colluvial landslides. Therefore, the repairs are also similar. Figure 2-5 shows two sketches of stability improvements for landslides involving fill material that was placed at the top of a slope, or of a sidecast road fill. Both sketches show one or more walls to restore a level area damaged by a landslide. The sketches show that a gravity wall (e.g., reinforced soil, concrete cantilever, crib, etc.) or an in situ wall (e.g., soldier pile) could be effective. The number, type, and location(s) of the walls would depend on the slope geometry, the final grades desired, site access, and other site limitations. The fill material and underlying loose colluvium could also be replaced with a drained earth buttress, as shown on Sketch D, Figure 2-2 (Sheet 2), or a reinforced soil slope.

7.0 DETAILS REGARDING IMPROVEMENTS

Many different engineered systems are currently used to mitigate landslides in the Seattle area. Sometimes a single system is enough to provide the necessary level of slope improvement or property protection. Often a combination of several mitigation systems is required to adequately increase the stability of a landslide or a marginally stable slope. Site accessibility and the mitigation scheme required to improve stability are the primary factors that govern the total cost of a slope stability improvement project. Sections 6.1 and 6.2 describe the geologic conditions that contribute to instability and typical approaches to improving stability. Sections 6.3 through 6.6 show typical applications of these typical approaches to improving stability for each of the four landslide types. The following subsections discuss details regarding mitigation measures that are commonly used in the Seattle area. These include typical details for the following types of improvement measures:

1. Surface Water
2. Groundwater
3. Retaining Structures
4. Soil Reinforcement
5. Grading

Note that final design details are not provided in this report. Final details and the selection of the appropriate improvement measure or measures should be developed by a geotechnical engineer experienced in landslide repairs and based on site-specific explorations and engineering evaluations.

7.1 Surface Water Improvements

As described in Section 6.1, surface water runoff can contribute to landsliding by causing surficial erosion and/or rapid saturation of the ground. Surface water improvements generally are the least costly measures that can be implemented to reduce landslide potential or mitigate existing instability. These improvements can be effective where storm water runoff, including water from streets, other paved areas, and roofs flows onto or near steep slopes and potential landslide areas. In most cases, surface water improvements consist of capturing storm water runoff and redirecting it away from sensitive slope areas. Storm water can be captured in appropriately located ditches, swales, roof drains, curbs, and catch basins. Once collected, the runoff should be conveyed in a tightline (an unperforated pipe) to a suitable discharge location. A suitable discharge location includes a storm sewer with adequate capacity or the bottom of a slope. Where storm water runoff is discharged to the ground surface at the bottom of a slope, appropriate erosion control measures should be placed at the discharge point.

Runoff from roofs should be collected by gutters and conveyed with downspouts to a catch basin or other structure that permits periodic cleaning. From the catch basin, the runoff should be conveyed in a tightline to a suitable discharge location. The downspouts at some homes discharge into footing subdrain pipes. We strongly recommend against this practice. It introduces surface water rapidly into the ground, which can trigger landslides and can cause foundation settlement. In addition, poorly drained foundations are often the cause of wet basements.

Where surface water runoff occurs toward a potential landslide slope, we recommend constructing a paved or lined swale near the slope top to intercept runoff. The adjacent ground should be graded to drain into the paved swale. The swale should be sufficiently large to convey the design storm with some blockage from leaves, ice, and other debris. Water in the swale should be collected at a catch basin and then conveyed to a suitable discharge location.

Surface water management on roads and other City-owned pavement surfaces located on or near the top of the slope is discussed in detail in Section 9. In general, the concepts and need for controlling surface water runoff are the same for any property; however, City property does have some special implications because of streets that create large areas of low permeability surface that can generate considerable storm water runoff.

7.1.1 Tightlines

Tightline systems are an integral part of a surface water system. As such, it is essential that all tightline pipe systems are properly designed and durable. The primary functional design requirements include the inlet, pipe capacity (pipe size and slope), and outlet. An important design factor in landslide areas is that the tightline might be subjected to landslide ground motion. The location and type of inlet will depend on the storm water collection system. However, it should include some provision for preventing debris from entering the tightline and for periodic cleaning. For example, a catch basin allows large debris to settle before the water enters the tightline and provides for periodic cleaning.

The pipe size is a function of the anticipated runoff discharge and the pipe slope. All pipes should be continuously graded to prevent settlement accumulation in the pipe that could eventually block the pipe or reduce its capacity.

Tightlines can consist of a variety of materials including different types of plastic and metal pipe. Each one should be selected based on the particular project requirements. Tightlines that extend across steep terrain and unstable slopes should consist of durable plastic pipe, such as high-density polyethylene (HDPE). The joints should be durable and able to carry axial loads and accommodate flexural deformation of the pipe. Welded or through-bolted, flanged joints are examples of suitable joints. The pipe should be installed on the surface with an anchor system to prevent the pipe from being pulled apart by soil creep or by a landslide and to allow regular inspection. Figure 2-6 (2 sheets) shows examples of tightline anchoring systems.

Tightlines may consist of less expensive, jointed, flexible, corrugated plastic pipe in less critical stability areas, and depending on other project requirements. In addition, tightlines may be buried in stable areas.

As described previously, all tightlines should discharge to a suitable location. They should not be allowed to discharge at the top of a slope, directly into or onto a slope, or onto a mid-slope bench.

7.1.2 Surface Water Systems – Maintenance

All surface water systems should be regularly checked and maintained. Ideally, the City and residents on or near slopes should collectively implement and maintain the drainage features described above. We recommend designing maintenance programs in landslide prone-areas that use a partnership between the City and residents. The City could set up a regular maintenance program to:

- Clean catch basins and storm water runoff systems on a regular basis. Initially, the City could establish a frequent maintenance schedule in landslide hazard areas based on previous experience and current schedules. However, a record-keeping system should be implemented to identify an appropriate schedule for specific locations, and typical storm events that could block catch basins.

- Inspect streets in landslide-prone areas for significant cracking and surface wear. Repair significant cracks, as appropriate, where found to be needed to reduce storm water infiltration.

- Provide residents in landslide prone areas with information on measures they should implement to reduce surface water runoff and infiltration. These measures should include the issuance of publications such as the Washington Department of Ecology "Surface Water and Groundwater on Coastal Bluffs" (1995), and free inspection programs that might be similar in organization and implementation to the energy audits provided by Seattle City Light.

- In many cases, residents on steep properties cannot discharge storm water runoff into a storm sewer, either because there is no nearby storm sewer or because the grades are inappropriate. Depending on the property ownership, installing a tightline to the bottom of the slope may be an alternative. The City could work with these residents and their neighbors to identify storm water disposal alternatives. These might include installing tightlines that cross more than one property (including City property) to convey runoff to the bottom of the slope or to another suitable discharge point below the property.

- Form "Landslide Block Watch Groups," in which groups of residents would regularly inspect City storm water catch basins for debris. This group could perform some surficial cleaning when necessary and could alert the City when additional work is needed. The groups could provide annual or more frequent reminders to their neighborhood when regular maintenance, such as cleaning gutters and drains, should be performed.

Landslide Block Watch Groups could also assist the City with disseminating information on reducing residential surface water runoff and infiltration. This could include providing a liaison with City personnel, meeting with new homeowners, and identifying and helping to resolve neighborhood storm water runoff problems.

7.2 Groundwater Improvements

Intercepting groundwater upslope and from within the slope can reduce landslide potential and improve stability of existing landslides. Groundwater improvements can be effective on many slopes, and when used appropriately, they are often the most cost-effective approach. The primary goal is to remove groundwater in areas where groundwater reduces stability by adding weight to potentially unstable soils, causes seepage forces, and reduces the soil strength. However, capturing water flowing within the ground requires some different methods compared to surface water improvements. Common groundwater improvement methods include:

- Interceptor trench subdrains and finger drains
- Springhead drains

- Drainage blankets
- Drilled drains

All groundwater improvement schemes should be designed based on the site-specific subsurface conditions. To perform effectively, the system must lower the groundwater level near the landslide failure surface, which requires an understanding of the soil and groundwater conditions that cause instability at the location. The following sections provide a description of common groundwater improvements with some typical design details and requirements.

7.2.1 Interceptor Trench Subdrains and Finger Drains

Trench subdrains are relatively narrow trenches that contain a drainage pipe and permeable backfill. Figure 2-7 (2 sheets) shows typical trench subdrain cross-sections. Groundwater preferentially flows into the permeable trench backfill and then into the drainage pipe at the bottom of the trench. From there, it is conveyed into a tightline that discharges the groundwater to a suitable discharge location. Trench subdrains are most effective when they penetrate at least 1foot below the contact between the layer being drained and an underlying clay, silt, or less permeable layer. This contact is commonly also at or close to the slide plane. Two basic types of trench subdrains include interceptor trench subdrains and finger drains. An interceptor trench subdrain is usually oriented across the slope (parallel to the contours) to intercept groundwater as it flows downslope. Finger drains are frequently used to lower the water level within an active landslide mass by extending a trench subdrain from the toe of the slope up into the landslide debris. Other than their orientation (perpendicular to the contours), finger drains are constructed in the same manner as trench subdrains.

Trench Excavation

Most trench subdrains are excavated using a backhoe or a track-mounted excavator. Therefore, the practical depth for most trench subdrains is about 15 feet or less. Track-mounted excavators are available that can excavate 20 feet deep or more. However, deep trenches are often difficult and expensive to excavate because of the shoring required to maintain stable trench sideslopes. Where groundwater is shallow and site access is limited, hand dug trench subdrains may be practical.

The depth of the trench is generally determined by the maximum practicable depth of the excavating equipment, site conditions, shoring requirements, and other project limitations. As mentioned previously, trench subdrains are most effective when they penetrate though the layer being drained and at least 1 foot into an underlying less permeable soil.

Excavating open trenches in marginally stable soil is often difficult because of groundwater infiltration and the tendency for the trench sidewalls to collapse. Where practical, we recommend beginning the excavation at the outfall and proceeding upslope to allow water to drain away from the advancing trench excavation. It may be necessary to periodically stop work for a day or more to let the site drain before advancing the trench.

Drainage Pipe

The drainage pipe in an interceptor trench subdrain typically consists of a 6-inch (minimum) diameter slotted or perforated plastic pipe. The slots or perforations in the drainage pipe allow water to enter the pipe. However, the drainage pipe only conveys water when the groundwater level rises higher than the pipe invert. When lower water levels are present in portions of a trench subdrain system, the water flows through the surrounding permeable trench backfill. Therefore, the pipe may not need to be placed at precise grades. The pipe should be graded to drain continuously with no sags or depressions where water could infiltrate into the subgrade.

The drainage pipe at the bottom of the trench may consist of rigid or flexible and perforated or slotted pipe. Each type has its advantages and disadvantages. The rigid pipe is more durable and less susceptible to crushing during installation. However, a worker must be present in the trench to fit the pieces of pipe together and to prepare the bedding gravel. Having workers in a trench generally requires shoring, which results in a higher cost and a longer time for construction. The other alternative, flexible plastic pipe, is easily crushed if workers are not careful or if it is not properly bedded. The primary advantage of the flexible pipe is that it can be lowered down into a deep, unshored trench excavation without workers being in the trench.

The size of the perforations or slots should be compatible with the drainage backfill material around the pipe and the anticipated groundwater flow rates into the pipe. The following paragraph describes filter requirements for perforations and slots in more detail.

Trench Backfill

The trench backfill material depends on the anticipated groundwater inflow and the grain size of the surrounding soil. The backfill should be sufficiently permeable so that water easily flows from the surrounding soil into the trench subdrain. However, the backfill should also act as a filter to prevent migration of the surrounding soil into the subdrain trench. This migration process, known as piping, can eventually plug the subdrain pipe, the drainage backfill or both. In some cases, extensive piping can cause extensive settlement around the trench subdrain from the ground lost by piping. The backfill must also be compatible with the perforated or slotted drainage pipe. If the openings are too small, water cannot enter the pipe fast enough. However, if the openings are too large, the backfill will enter and plug the pipe. To meet the requirements of adequate permeability and filter characteristics, the backfill material should be designed for each specific situation. The following paragraphs describe examples of typical backfill materials that have been successfully used in Seattle soils.

In the example shown on Figure 2-7, Sheet 1 of 2, the perforated or slotted pipe at the bottom of the trench is bedded in washed pea gravel to provide a highly permeable material directly around the pipe. The pea gravel should be underlain by a geotextile where it rests on silt or clay soil. The remaining backfill can be less permeable, because the groundwater is flowing across a larger area. Therefore, a clean drainage sand or sand and gravel often is appropriate, as shown on the figure. Modified City Type 26 aggregate (Seattle Standard Specifications, 1989, Section 9-03.16) provides adequate permeability for many applications. It is also an adequate filter material for many Seattle soils. The example shown on Figure 2-7, Sheet 2 of 2, shows a slotted pipe with drainage sand and gravel used for both bedding and backfill.

Backfill is placed in the trench in layers and either tamped with a backhoe or systematically compacted. Generally, if the trench subdrain is located in a landscape area or an unused portion of property, the backfill can be moderately tamped in place with the backhoe bucket to reduce subsequent settlement. However, backfill in trench subdrains located where subsequent settlement of the backfill is not appropriate should be placed and compacted as structural fill material.

Tightline Connections

At the end of the trench subdrain and/or prior to daylighting the drainage pipe on the slope, the slotted or perforated pipe should connect to a tightline pipe. At this transition, the subdrain trench should be filled with concrete or clay to force water from the permeable subdrain trench backfill into the slotted or perforated pipe. Figure 2-8 shows an example of a drainage dam constructed with concrete or compacted clay. From the concrete or clay dam, the tightline should extend to a suitable discharge location.

Trench Cover

The upper 12 to 18 inches of the trench subdrain should be backfilled with a relatively low permeability material to prevent direct infiltration of surface water. Often the soil excavated from the trench is adequate because it should have a similar or lower permeability than the surrounding soil when recompacted in the trench. However, if the trench backfill must be compacted as structural fill, the trench excavation spoils may be too wet to achieve sufficient compaction without some drying and aeration.

In non-structural areas, where compaction is moderate, the backfill should be mounded slightly over the trench to prevent low areas from forming when the trench backfill settles. The surface should be graded to prevent water from ponding near the trench subdrain.

A paved or lined swale installed in conjunction with an interceptor trench subdrain can be used to limit infiltration into the trench subdrain. This remediation tactic is commonly used to control both surface and groundwater near the crest of a slope or close to the edge of a bluff, as shown on Figure 2-1, Sheets 1 and 2 of 3.

Geosynthetic Applications

Geosynthetic materials have been used in several trench subdrain applications. These include geotextiles that provide a filter for trench backfill materials and composite drainage materials that form both the drainage material and filter material.

Geotextiles can be used to separate the permeable trench backfill from the surrounding soil and prevent migration of fines into the trench subdrain. For this alternative, the drainage backfill could be a coarse-grained permeable soil that is a poor filter for the surrounding soil, such as uniformly graded gravel. The geotextile would be selected based on its ability to pass water and its filter characteristics to prevent migration of fines from the surrounding soil. The geotechnical engineer should specify this application on a case-by-case basis after careful consideration of the soil conditions. Note that geotextiles are made for many purposes. Therefore, not all geotextiles are appropriate for this application. In deep trenches where shoring boxes are required, the use of a filter geotextile can increase the time and labor costs of the project. Refer to Figure 2-7, Sheet 1 of 2, for the use of a geotextile to separate pea gravel from on-site soil.

One common misuse of geotextiles is wrapping the fabric directly around the perforated subdrain pipe. Because of the small area of the fabric around the pipe, it can quickly clog with fines, effectively blocking groundwater flow into the pipe.

7.2.2 Springhead Drains

Springhead drains are installed to intercept point-source springs, seeps, and shallow water-bearing zones in slopes or on existing landslides. They reduce the possibility for surficial erosion that can reduce stability by undercutting and oversteepening a slope. In addition, they reduce the amount of groundwater that can seep into the surficial colluvial and fill soils, which are often particularly susceptible to landsliding when saturated.

Springhead drains are placed at the point where springs and seeps emanate from the slope, to direct water through pipes to the base of the slope. Springhead drains have filter soils placed at the beginning of the drainpipe to reduce the potential of piping (migration) of soils into the springhead system. Figure 2-9 shows an example of a typical springhead drain installation.

The installation generally begins with an excavation to expose the seepage zone. The size of the excavation depends on the lateral extent of the seep or spring and on the practical size of a springhead drain. Difficult access on steep, wet slopes may require making excavations using hand tools. The excavation should extend at least 1 foot deeper than the seepage level to form a collection pool. A perforated or slotted 4-inch (minimum) diameter pipe is placed in the excavation perpendicular to the direction of the slope with the pipe ends capped. The pipe is connected to a tightline pipe and a dam of sandbags, concrete, or clay is placed around the connection to seal the leaks and force water into the tightline pipe. The installation must be completed in such a way that the entire seepage zone is backfilled with a free-draining aggregate that is sufficiently permeable to accommodate the anticipated seepage. Often the perforated or slotted pipe is backfilled with pea gravel or other more permeable clean granular aggregate to accommodate the increased flow rates as the collected seepage is concentrated near the collector pipe. Drainage sand and gravel, such as Seattle Type 26 Aggregate, may be suitable for the remainder of the backfill in the seepage zone. The selection of pipe diameter, perforation or slot size, and backfill materials depends on the amount of seepage and the grain size of the surrounding soil. As described in Section 7.2.1, the backfill material(s) must be adequate filters for the surrounding soil to prevent piping.

7.2.3 Drainage Blankets

When fills are constructed on a slope, a drainage blanket should be placed between the fill and the prepared subgrade surface to intercept seepage from the underlying soil and to improve drainage of water that infiltrates from the surface. Fills where a drainage blanket should be considered include toe buttresses, embankment fills, and slope fills placed to restore grades. A drainage blanket consists of a permeable layer of soil that is placed over the prepared subgrade before a fill is placed. Because it is designed to transmit groundwater, a drainage blanket should be designed as a filter for the subgrade and fill soils. Otherwise, piping of fines could plug the filter blanket and/or cause loss of ground.

The drainage blanket should be designed so it is capable of conveying the maximum anticipated seepage and infiltration water without saturating its full thickness. Figure 2-10 shows an example of a drainage blanket placed beneath an earth buttress fill. The design elements that need to be evaluated for each site include:

- The anticipated groundwater seepage and surface water infiltration rates.
- Permeability of the drainage blanket material and its thickness.
- The maximum distance to an interceptor trench subdrain or outlet.
- Seals to prevent direct surface water infiltration.
- If build on steep slopes, the drainage blanket should be built in benches or steps that penetrate into the natural slope. The drainage blanket should be continuous across the benches and should be graded to drain continuously.

7.2.4 Drilled Drains

Drilled drains consist of generally small-diameter drainpipes installed in drilled holes to a water-bearing soil layer. They are used to lower the groundwater level in a landslide or marginally stable slope where the depth to groundwater is too deep for dewatering using trench subdrains. The main advantage of drilled drains is that they can be installed at virtually any depth. Limitations include relatively high cost and the ability to intercept a sufficient amount of the permeable water-bearing zones to effectively lower the groundwater level. A thorough understanding of the subsurface soil and groundwater conditions is essential in planning a dewatering system using drilled drains. A geotechnical engineer and a hydrogeologist should explore the subsurface conditions, evaluate groundwater flow, and perform slope stability studies to develop an optimum drain configuration. The hydrogeologist should design the most appropriate drain spacing, well diameter, and well screen size. Pumping tests or other aquifer tests are commonly required to evaluate the effectiveness of proposed drilled drains. If drilled drains are selected as an element in improving the stability of a slope, groundwater monitoring wells should also be installed and monitored before and after drain construction to verify that the drains are achieving the degree of lowering in the groundwater levels desired. These groundwater monitoring wells can also be used to monitor the effectiveness of the system over time.

The three general categories of drilled drains include nearly horizontal drains (commonly called horizontal drains), directionally drilled drains, and vertical drains or wells. Horizontal and directionally drilled drains capture groundwater and drain it away from a sensitive slope area with gravity flow. Vertically drilled drains typically require pumping to remove groundwater, although gravity drainage is possible in certain circumstances, as subsequently described. Figure 2-11 shows a schematic of the various types of drains. If drilled drains are suitable, the site access limitations, the subsurface conditions, and construction costs typically dictate which system is feasible for a particular site.

Horizontal Drains

Horizontal drains are installed by drilling a nearly horizontal boring from a point at the bottom of a slope. Therefore, access to the bottom of the slope for a large, track-mounted vehicle must be possible for this option. Typically, two or more horizontal drains are radially drilled from one or more points to intercept the water-bearing stratum. The drilled holes extend as far into the hillside as necessary to intercept and lower the groundwater level. The length of drilled drains can be 200 feet or more. They are drilled straight at a constant upward inclination of 2 to 10 degrees from the horizontal, depending on the site access and elevation of the water-bearing zone. The installation technique generally consists of drilling a subhorizontal boring and concurrently placing a steel casing into the hillside. A slotted or screened plastic pipe is then placed inside the casing, which is then withdrawn leaving the plastic pipe in-place. A tightline pipe is attached to the end of the plastic pipe and a low permeability plug installed to force the water into the tightline for conveying the discharge water to a suitable location. Each pipe is generally fitted with individual valves for shutting-off and cleaning-out.

Directional Drains

Directional drains are similar to horizontal drains except that they are typically drilled from the top of the slope using a remotely guided drill to intercept a water-bearing soil layer at a predetermined location. Once the drilled hole reaches the target water-bearing layer, the drill bit continues until it exits the slope at the desired collection point. From there, the water is conveyed in a tightline to a suitable discharge location. The advantage to directionally drilled drains is that access to the bottom of the slope for heavy equipment is not needed. For many landslides or marginally stable slopes, access is not otherwise practical.

The drill rig is typically set up some distance away from the top of the landslide, with an initial drilling inclination on the order of 20 degrees from the horizontal. The position of the drill bit is monitored using an electronic tracking device. The drilling assembly can be steered using a specially tooled drill bit to direct it to the desired dewatering zone and exit point. The allowable radius of curvature of the drill steel limits the amount of steering. Once the hole is completed, it can be reamed if a larger diameter is needed. A slotted or screened plastic pipe, usually 2- or 4-inch-diameter polyvinyl chloride (PVC), is pulled through the drill hole from bottom to top to complete the drain. The discharge is captured at the lower end in a tightline pipe system and conveyed to a suitable discharge location. The upper end of the pipe is capped and encased in a monument at the surface to allow access for maintenance and cleaning.

Vertical Drains

Vertical drains consist of vertically drilled bore holes that extend into or through a water-bearing soil layer and remove the water either by constant pumping or in certain circumstances by gravity flow. Pumped vertical drains are essentially water wells and, as such, are designed and built in the same manner as water wells. Typically, the boring for a well is drilled through the permeable unit where dewatering is planned and into the underlying low permeability soil layer. The well consists of a screened section of well casing that extends through the permeable saturated soil and solid casing extending to the surface. A sand pack is placed between the screen and the native soil to increase the effective diameter of the well and to form a filter between the surrounding soil and the screened well casing. The filter prevents the well from piping fines from the surrounding soil that could cause loss of ground and impair the capacity of the well. Water is removed from the well using a submersible pump that is controlled with a switch activated by rising water level or hydrostatic pressure. Several different pumping system configurations are available. Dewatering wells can intercept water-bearing units that otherwise are not accessible by other types of subsurface drainage. However, they require continual pumping and maintenance, which can be costly. In addition, a reliable power source is essential because of the likelihood of power outages during wet stormy periods. Backup power systems require frequent maintenance and testing to ensure that they will function when the normal power system is interrupted.

A vertical gravity drain is installed in a similar manner as the pumping well, but instead of using a pump to remove water, the well drains groundwater to an underlying layer of permeable soil. This type of system requires specific subsurface conditions to be practical. These are:

1. The water-bearing layer that is reducing the slope stability must overlie a lower permeability layer (aquitard).
2. The aquitard must be underlain by a zone of permeable soil (e.g., sand or gravel).
3. The lower permeable zone must be below the level of slope instability.
4. The lower permeable zone must be able to drain the upper water-bearing layer, i.e., it must have sufficient permeability, thickness, and there must be a sufficiently large hydraulic gradient.

The design of this type of system requires detailed knowledge of the hydraulic characteristics of the entire system. A hydrogeologist is typically required to evaluate the soil parameters and design the well system. Another concern associated with vertical gravity drainage is the potential for cross contamination between upper and lower aquifers. If the groundwater in the upper soil layer is contaminated, vertical drainage into an underlying aquifer would be prohibited by environmental regulations. Even if the upper aquifer is not contaminated, the Washington State Department of Ecology or other local environmental regulatory agencies may require work to demonstrate that the underlying aquifer would not be degraded.

7.2.5 Other Subsurface Drainage Systems

Numerous other subsurface drainage systems have been used to lower groundwater levels in landslides and in marginally stable slopes. These other systems typically are appropriate for specific subsurface geologic and groundwater conditions and are not widely applicable. Some systems have largely been replaced because of technological advances. For example, during the Depression, several U.S. Works Progress Administration (WPA) projects were undertaken to install drainage in landslide areas. In many cases, the drainage consisted of hand-excavated tunnels that were subsequently backfilled with drain pipe and sand and gravel. Today, many of these drains would be installed by horizontal or directional drilling. Still drainage tunnels have specific, if limited, use for subsurface drainage. Other subsurface drainage systems include: electro-osmosis, vacuum dewatering, and siphoning. Because of the limited and site specific applications, these methods are not discussed in this report.

7.2.6 Monitoring and Maintaining Subsurface Drainage Systems

Subsurface drainage systems are only effective if they lower the groundwater level at least to the level assumed for design and if they maintain the lowered groundwater level. Therefore, we recommend performing regular maintenance and installing a monitoring system so that the effectiveness of a subsurface drainage system can be monitored. The type of monitoring depends on the site conditions, the type or types of subsurface drainage system(s) used, and the degree of reliability required. Maintenance includes clearing vegetation from outlet pipes and tightlines, inspecting and repairing damage to surface installations, removing accumulated sediment from catch basins, and jetting pipes to remove sediment and encrustation.

Subsurface drainage systems are usually monitored by measuring the groundwater level in one or more monitoring wells and measuring the discharge rates from drain outlets. The continuity of a drain line can also be evaluated by adding water at an uphill cleanout location and observing the flow at a downhill discharge location. However, this type of test should only be performed during the dry summer season. The groundwater level in monitoring wells can show that the drainage system is lowering the groundwater to the levels assumed in design. Monitoring wells should be installed before the subsurface drainage system is installed to establish pre-construction groundwater level(s). Often the monitoring wells that were installed during the initial site explorations can be used for long-term monitoring. After the subsurface groundwater drainage system is installed, the groundwater levels should be monitored on a regular basis to evaluate the performance of the drainage system, including its response to seasonal rainfall events. The measurement and data recording interval should be determined for each site. Depending on the complexity and criticality of the subsurface drainage system, an automated data recording system may be justified. Once the groundwater response to seasonal and rainfall events is established, groundwater level monitoring should be conducted at least once on an annual basis thereafter. Unanticipated changes in groundwater levels typically show the need for cleaning or other maintenance. The discharge rates from subsurface drains should also be measured and recorded. Declines in the discharge rate may indicate buildups of encrustation or sediment that reduce the effectiveness of the system.

Subsurface drainage systems require regular maintenance to perform as designed. Maintenance should start by designing surface installations that are protected from damage. For wells, this could be accomplished by installing guard posts and steel monuments to prevent vandalism and accidental damage. All surface installations, such as wells, drains, and tightlines, should be placed in locations where they can be easily found. Vegetation around these installations should be regularly trimmed to allow inspection for deterioration, breaks, leaks, and other damage. If groundwater monitoring and/or discharge rates indicate a decline in the performance of the subsurface drainage system, it should be cleaned by flushing, jetting, or other appropriate means.

7.3 Retaining Structures

Structures can be built to increase the stability of marginally stable slopes or existing landslides by:

- Retaining fills that add weight to the resisting part of the landslide
- Retain part of the driving forces
- Transfer driving forces into stable ground
- Increase the resisting forces of the soil along the failure surface
- Retain oversteepened scarp areas to prevent progressive upslope landsliding

Structures are also used to limit the runout of debris and to protect specific areas or structures upslope or downslope of the landslide. Debris catchment and diversion structures are described in Section 7.6. The appropriate type of structure for a given landslide or marginally stable slope will depend on many factors such as:

- Access to the site
- Stability of the slope
- Magnitude of slide forces
- Availability and cost of materials
- Future risk to life and safety
- Intended use of the stabilized slide zone

This section describes in situ and gravity walls. In situ walls include various types of pile walls, such as cantilevered or tied-back soldier piles with lagging, tangent piles, secant piles, and sheet pile walls. Gravity walls include concrete cantilever walls, mass concrete walls, crib and gabion walls. Reinforced soil walls and slopes are described in Section 7.4.

7.3.1 In Situ Walls

In situ walls are structures that are built in place, without removing large volumes of soil to form a footing. They are well suited for many landslide repairs where access limitations or stability concerns prevent excavations needed to construct other wall types or to place earth buttress fills. They can also be used for catchment and/or diversion walls to reduce the debris flow hazard to structures below landslide-prone slopes. In general, this type of wall is built with piles, drilled shafts, and/or anchors. The common types of in situ walls include soldier pile walls with lagging, secant pile walls, tangent pile walls, and sheet pile walls.

Pile walls and tieback installations require a specialty contractor with equipment capable of drilling deep shafts and installing tiebacks into a hillside. Because landslide zones frequently consist of soft, unstable ground with uneven terrain and limited access, it is sometimes necessary to construct pile walls with limited-access drilling or pile driving equipment. As shown in Section 9.0, the equipment and labor costs for this type of wall construction are relatively high.

In situ walls are typically built with drilled shafts or driven piles. Drilled shafts are cast-in-place concrete structures that usually contain a steel rebar cage or a steel H-beam for reinforcement. Drilled shafts for retaining walls have diameters that typically range from about 2 feet to more than 4 feet. In situ walls can also be constructed of driven piles including steel pipe piles, timber piles, steel H-beam piles, and sheet piles. Wood or concrete lagging typically is placed between the piles when they are spaced more than 2 to 4 feet apart to retain the soil between piles. Lagging is not necessary when the piles touch (tangent pile) or intersect (secant pile wall).

An in situ wall can be designed as a cantilever structure when the height of the cantilevered portion of the pile or drilled shaft is relatively low (typically less than about 15 feet) and the active (driving) earth pressures are moderate. A cantilevered pile structure resists lateral movement acting on the upper part of the pile (e.g., in the landslide mass) by transmitting the lateral loads into the portion of the pile embedded in hard or dense soil. The cantilever is the height of the pile or shaft that is above the slide plane or competent soil. Pile walls that are higher or must retain large landslide forces could also contain tieback anchors to further resist sliding forces. A tieback anchor consists of single or multiple steel wires, strands, or bars that are installed at a shallow inclination from the face of the pile, through the landslide mass, and into undisturbed soil. The tieback is anchored into stable, dense or hard soils with a cement grout or a mechanical end such as a helical plate or a swivel plate that expands into the soil when pulled. The tieback anchor is usually post tensioned, although not always. They are typically load tested. These anchors transmit sliding forces exerted by the landslide mass into the underlying stable soil.

Soldier Pile Walls

Figure 2-12 shows an example of a soldier pile wall. Most soldier pile walls in Seattle consist of steel H-beams with wood or concrete lagging between the piles. The piles are placed in predrilled holes, which are then backfilled with concrete. The diameter of the predrilled hole depends on the size of the steel H-beam, but typically is about 2 feet. Piles walls can also be constructed with augercast piles and driven piles, including timber piles; however, driven piles are more difficult to align to facilitate construction of the lagging. In addition, it can be difficult to drive the piles to the required depth in some soils. Other materials can be used for piles, including steel pipes. The distance between the piles depends on a number of factors, including the lateral forces acting on the wall, the size of the piles, the height of the piles, and if tieback anchors are used. Typically, soldier piles are spaced 4 to 10 feet apart. Larger, deep-seated landslides typically involve driving forces that are relatively high, and resisting (passive) earth pressures that are low or subject to reduction due to potential slope movement downslope. Consequently, pile walls for landslide mitigation frequently require use of tiebacks. Wood or concrete lagging is placed between the piles to retain the soil between piles.

Figure 2-12, Sheet 1 of 3, shows typical earth pressure diagrams for cantilevered soldier piles walls or walls with a single row of tiebacks, and also for multiple rows of tiebacks. The actual design earth pressures will depend on the wall height, soil type(s), slopes above and below the wall, and groundwater. These factors typically are different at every site.

The presence of groundwater behind any wall causes large destabilizing forces. Therefore, proper wall drainage is necessary to maintain stability. Figure 2-12, Sheets 2 of 3 and 3 of 3, show two examples of drainage. In the second, drainage is achieved by backfilling behind the wall with a well-drained fill material that is similar to a trench subdrain. If it is not practical to excavate behind the wall to install drainage, then a drainage board can be placed on the retained side of the lagging, as shown on Sheet 2 of 3. The drainage board should discharge into a tightline pipe that is sloped to drain and that discharges to a suitable location.

Other In Situ Walls

As described previously, walls can also be constructed using secant piles, tangent piles, and sheet piles. The design of these wall types is similar to the design of a soldier pile and lagging wall, except that lagging is not used. Drainage is important for all in situ wall types.

Occasionally, the stability of a landslide is improved using discrete piles and/or anchors rather than constructing a wall. The concepts are similar, in that the landslide forces acting on the piles or anchors are transmitted into stable soil that is present beneath the landslide. Instead of a linear wall that may also retain a change in grade, discrete piles and/or anchors are typically installed solely to increase the forces resisting sliding. They are commonly installed on a grid or along one or more lines across the landslide. Anchors that are installed as discrete retention elements in a landslide usually require a concrete reaction block to transfer the forces exerted by the sliding mass into the anchor. The size of the concrete reaction block depends on the size of the anchor and the soil properties of the landslide mass..

7.3.2 Gravity Walls

Gravity walls are structures that resist sliding forces with their weight and internal stability alone. They include walls constructed with mass concrete, concrete cantilever walls, rock-filled gabion baskets, rock-filled or soil-filled concrete, metal or timber cribs (Figure 2-13), and interlocking concrete blocks (Figure 2-14), which are commonly known as ecology blocks. Gravity walls resist sliding forces by the friction developed along the bottom of the wall and passive resistance where the wall is embedded into dense or hard soil. They resist rotation and overturning by being constructed at a batter, i.e., leaning towards the retained landslide mass and/or by having sufficient mass. Gravity walls are typically keyed into stable foundation soils at the toe of a small landslide and then constructed up to the appropriate height to resist slide forces.

Gravity walls are commonly appropriate to provide toe support for landslides where the horizontal slide forces are relatively small. Another common application includes a wall to retain an earth buttress fill where property limitations require a steep fill. Gravity walls are generally less costly than in situ walls (such as soldier pile and lagging walls) and often can be constructed in difficult access areas. It is important that gravity walls be properly constructed by qualified contractors using proper methods and materials to achieve the required internal shear strength while also being flexible and tolerant of deflections.

7.4 Soil Reinforcement

Soil can be reinforced by adding materials that have high shear strength. When reinforced, soil can be built into steep slopes and walls. Soil reinforcement has a number of applications for landslide repairs and for improving the stability of marginally stable slopes. These include reinforced soil walls, in situ reinforcement, and replacement of the landslide mass with a stronger material.

7.4.1 Reinforced Soil Walls

Reinforced soil walls, which are also referred to as mechanically stabilized earth (MSE) walls, consist of compacted soil with intervening layers of manmade material that is placed to improve the shear strength of the soil mass. Adding such internal reinforcement to a soil fill provides shearing resistance against landslide forces. Most reinforced soil walls are inherently flexible because they do not contain rigid elements such as concrete. Therefore, they can tolerate relatively large settlements of the foundation soils while retaining their structural integrity. Typical reinforcing materials consist of synthetic polymer materials (geotextiles and geogrids), welded wire fabric, or metallic strips. The geotextiles are generally the least costly material; however, geogrids and metallic strips can provide higher strengths that may be required for some projects. A sketch of a typical geotextile wall section is shown on Figure 2-15. Figure 2-16 presents an illustration of a typical geogrid-reinforced soil wall.

The finished face of a reinforced soil wall can be vertical or sloped. Vertical wall faces must be finished with erosion resistant facing such as sprayed-on concrete (shotcrete) or concrete masonry units (CMU) blocks. There are large varieties of CMU blocks locally available that are suitable for use in conjunction with soil reinforcing. Depending on the slope, sloping wall faces may be planted for vegetative erosion resistant facing. Figure 2-17 shows an example of a reinforced soil slope.

The design of reinforced soil walls for slope stabilization is based on the external loading demands due to the slide mass behind the wall and the internal strength capabilities of the wall and strength of underlying soils. Geotechnical parameters for the foundation soil, the reinforced fill soil, and the retained soil must be evaluated and incorporated into the design. External as well as internal stability analyses must be completed to arrive at an appropriate wall dimension and reinforcement design. As shown on Figures 2-15, 2-16, and 2-17, reinforced soil structures must have adequate drainage.

7.4.2 Soil Nail Walls

Soil nails and reinforced shotcrete can be placed on the exposed slope soils to increase the stability of marginally stable slopes and bluffs. Soil nails consist of metal bars that are installed to reinforce the native soil. They reinforce the soil by the frictional resistance that occurs between the relatively inextensible bars and the adjacent soil. In many respects, soil reinforced with soil nails is similar to other types of reinforced soil, except that the soil is reinforced in situ. Unlike post-tensioned tieback anchors, soil nails do not exert an external force on the retained face.

Typically, soil nails are installed either by driving a steel bar into the soil, or by grouting a bar in a 4- to 6-inch-diameter drilled hole. For most applications in the Puget Sound area, nails are installed by grouting bars into pre-drilled holes. The annulus between the soil nail and borehole is then backfilled with cement grout. The nails are typically installed at a slight angle (5 to 20 degrees) from horizontal. The nail spacing depends on the soil characteristics, but typically varies between about 4 and 6 feet, measured center to center. Their lengths are determined based on the estimated thickness of the failure wedge, the height of the bluff, and the engineering characteristics of the soils. Typically, the length varies between about 3/4 and 1 1/4 times the bluff wall height.

Reinforced shotcrete is placed in between the soil nails and on the exposed soil slope surface to protect the face from erosion and to prevent it from raveling, as shown on Figure 2-1 (Sheet 2 of 3). Shotcrete consists of concrete that is sprayed onto the surface. The shotcrete is typically reinforced with wire mesh that is installed before the shotcrete is applied. The soil nails and shotcrete act in concert to form a reinforced membrane. Provisions must be included for drainage behind a soil nail wall. The base of the reinforced shotcrete should be protected from erosion at the toe. As shown on Figure 2-1 (Sheet 2 of 3), such erosion control measures might include embedding the shotcrete into competent material, constructing a toe drain, and providing a lined drainage swale.

7.5 Grading

Grading improvements to mitigate landslides involve making changes to surface topographic features on, above, or below slopes. Grading includes any changes made to the ground surface by excavating, filling, or a combination of excavating and filling. These changes can be made to accomplish one or more of the following:

- Improve drainage to reduce the amount of rainfall infiltration and runoff on or above a landslide or marginally stable slope.
- Decrease the driving weight of a landslide mass.
- Increase the weight of soil that acts to resist sliding.
- Increase the soil strength to resist sliding.
- Remove unstable soil.

Before any grading is accomplished, a geotechnical slope stability study that includes subsurface explorations should be accomplished to evaluate the effects of the proposed grading. The geotechnical study should consider the effects of the proposed construction on the landslide or marginally stable slope and the slopes above, below, and adjacent to the proposed construction. For example, if the proposed stability improvements include excavating soil from the top of a landslide, the geotechnical study should include stability analyses of the slope(s) remaining above the proposed excavation. In an urban environment, grading improvements are commonly limited to relatively small changes in line and grade. Large-scale excavations and fills often are not practical because of property limitations and high property values.

7.5.1 Drainage Improvements

Drainage can be improved by grading the ground surface to direct water away from a slope or other areas where infiltration can reduce stability. Section 7.1 presents related surface water drainage improvements. Grading to improve surface water drainage is commonly accomplished at the top of the slope to prevent overland surface runoff from flowing onto the slope. Grading can also improve drainage for water that accumulates in closed swales, ditches, ponds, and other low lying areas, which would otherwise infiltrate and raise the groundwater level. Section 6.1 describes how groundwater can adversely affect slope stability.

Landslide surfaces are commonly irregular with sag ponds (depressions in a landslide surface that fill with water) and other poorly drained areas. Therefore, after a landslide occurs, grading to smooth the surface can promote stability by improving runoff and reducing the opportunity for rapid infiltration. Old landslide surfaces can also be regraded to a generally smooth constant slope that promotes runoff. If regrading the entire slope or hummock portions of the slope is not practical, specific areas of ponded water can be drained with ditches and tightlines. Slope stability analyses should be accomplished before any grading occurs to evaluate the effects, if any, from changes in the slope geometry.

Water should not be allowed to flow over the top of a slope onto a landslide area or marginally stable slope. In addition, poorly drained areas near the top of a slope should be regraded to prevent ponding. Ideally, the tops of slopes should graded so that the ground surface slopes away from the landslide zone or marginally stable slopes. Alternatively, runoff can be directed into a drainage swale or ditch that is parallel to the top of the slope. The swale or ditch should be continuously graded to prevent ponding and infiltration. Where a drainage swale or ditch cannot be directed to a suitable discharge location, the water should be collected in a catch basin. A tightline should convey the water from the catch basin to a suitable discharge location. In permeable soil, swales and ditches should be lined with asphalt or compacted silt and clay to reduce infiltration.

Infiltration into permeable soil can be reduced by constructing a low-permeability cover that promotes runoff to a suitable location. This type of improvement would be practical mainly in areas where grading or other surface drainage improvements are not practical. In most applications, a low-permeability cover could be constructed by compacting 2 feet of clay soil. Whenever grading or land clearing occurs on a steep slope, the disturbed ground surface should be compacted to promote runoff and reduce infiltration. Dense vegetative cover reduces erosion potential and also reduces infiltration by increasing evapotranspiration. Therefore, we recommend reestablishing suitable vegetation after clearing or regrading a slope or area adjacent to a slope.

7.5.2 Decrease Driving Weight

Grading can be accomplished to remove weight from the portion of a slope that provides a driving force for a landslide or marginally stable slope. Usually, the driving portion of a landslide is the upper steep portion of the slope. Therefore, the driving weight can usually be reduced by flattening the slope and/or removing soil from the top of the slope. If changes in line and grade are not acceptable, the driving weight can be reduced by replacing soil that is causing a driving force with a lightweight fill material.

When permanent removal of soil weight at the top of a slope is a viable alternative, conventional earthwork equipment can be used to excavate and haul soil from the site. The soil should be excavated in a manner that improves stability. Temporary soil stockpiles should not be allowed on or adjacent to the slope. The final surface should be graded to a smooth and stable configuration that also promotes runoff to a suitable location. The final surface should be seeded and/or planted to provide permanent erosion control.

Lightweight fills include materials such as fly ash, bottom ash, expanded polystyrene (geofoam), sawdust, wood chips, cinders, and cellular concrete. The particular lightweight fill material selected for a given application depends on the required fill characteristics, availability, and project budget. Fill materials such as sawdust can result in substantial settlement over the life of a project. Expanded polystyrene is commonly used because of its very light weight, strength, and workability. However, it is soluble in gasoline such that expanded polystyrene fills must be protected from fuel spills. If the unit weight of a lightweight fill material is less than that of water, the fill can float if not weighted down when high groundwater conditions occur or if the site floods. Chipped tires have been used for several lightweight road fills around the country; however, chipped tires can combust in situ, depending on the fill thickness and other environmental conditions. Chipped tire fills that have "burned" resulted in the loss of the fill and also caused soil and groundwater contamination. We, therefore, recommend against using chipped tires as fill.

Soil and vegetative debris derived from clearing and grading activities should not be sidecast over the top edge of a slope. This practice tends to load the top of slopes and is a cause of many landslides in the City.

7.5.3 Increase Resisting Weight

Grading to increase resistance to driving forces in landslides generally involves placing a fill near the toe of the landslide. Buttresses, counterweight fills, and toe berms improve the stability by their dead weight in the resisting part of the landslide. The dead weight over the toe of a landslide increases the shear strength of the soils along the slide plane. Depending on their geometry, fills placed near the toe of the landslide can extend the length of the landslide failure surface and additional shear strength from the new fill can improve stability. Buttresses are typically keyed into underlying dense or hard soil to increase sliding resistance, while toe berms (also called counterweight fills) not so keyed still improve stability by the increased dead weight.

The above-described fills can be constructed with any type of inorganic soil fill, provided the new fill itself is stable. In the Seattle area, fine-grained soils can be difficult to compact if the water content is too high and during wet weather. Often relatively clean crushed rock or sand and gravel is used to facilitate construction; however, other fill materials can be used provided they can be compacted to a relatively dense condition. Fill slopes generally are built at 2 Horizontal to 1 Vertical (2H:1V) or flatter for constructability and to facilitate maintenance. Steeper slopes usually require reinforcement or a retaining structure. Sometimes, crushed rock slopes are constructed to 1.5H:1V.

The height, width, and length of the buttress or toe berm will depend on the size of the landslide and the forces involved. The design should evaluate the effect of the proposed fill on improving the stability of the landslide or unstable slope. It should also consider the stability of the fill itself. The fill should be stable with respect to sliding, overturning, and bearing failure of the underlying soils. As mentioned above, buttresses are typically keyed into the underlying dense or hard soil to provide sliding resistance. The fill should have a subdrain system that includes a drainage blanket beneath the fill, unless the entire fill is pervious, as well as interceptor subdrains. The placement of subdrains depends on the amount and location of groundwater seepage expected, including groundwater encountered during construction. Therefore, it is advisable to have the geotechnical engineer provide recommendations for additional drainage based on the seepage conditions exposed during excavation.

Subgrade preparation on active landslides requires care and planning to avoid reactivating or accelerating the landslide movement. In particular, excavations made at the toe of a landslide to place a fill keyed into stable subgrade material or to replace landslide debris with structural fill can remove a substantial portion of the resisting landslide mass. Therefore, it may be necessary to complete earthwork in relatively small sections. Each section should be backfilled with structural fill material before excavating the adjacent section. The area of each section depends on the site-specific conditions. Therefore, earthwork should be monitored on a full-time basis by a geotechnical engineer who can provide field recommendations if unanticipated movements occur.

7.5.4 Increase Soil Strength

The stability of a slope can be increased by replacing the soil that is marginally stable or that has already slid with a relatively strong fill material. Strong fill materials include well-compacted sand and gravel, gravel, quarry spalls, and riprap. Typically, angular aggregate has higher shear strength than well-rounded aggregate. Regardless of the fill material, the fill should be well drained. A drainage blanket and associated subdrains should be incorporated into the design as appropriate.

Figure 2-18 shows an example of a typical replacement fill buttress. In the example, the majority of the landslide debris is removed and replaced with a stronger, granular backfill material. The replacement fill material consists of a well-graded sand and gravel or crushed rock that meets the gradation for Seattle Type No. 17 aggregate. When compacted, this fill material provides relatively high shear strength. In the example, approximately half of the failure surface is replaced with the stronger fill material. The drainage layer shown on the figure consists of drainage sand and gravel that would be an effective filter for the granular backfill and the native soil that underlies the fill. The drainage prevents groundwater from saturating the fill material and, consequently, reducing the shear strength along potential failure surfaces. The fill must be embedded below the ground surface and to a sufficient depth so that a new failure surface will not develop below the replacement fill buttress.

Fills constructed to increase the resisting weight typically change the surface lines and grades in a manner that tends to lengthen potential failure surfaces. Therefore, if the likely failure surface following the repair extends through the new fill, the resistance to sliding can be increased by using a relatively strong soil for the fill material.

7.5.5 Remove Unstable Soil

Landslides that involve uncontrolled fill material or other loose or soft soil over a hard or dense substrate can often be repaired by removing all or part of the unstable soil. The soil should be excavated in a manner that improves stability. The final surface should be graded to a stable configuration that promotes surface water runoff. Following final grading, the surface should be revegetated to reduce erosion and surface water infiltration. In an urban environment, removal of unstable soil commonly is limited to old uncontrolled fills that tend to destabilize a slope.

7.6 Catchment or Diversion Structures

A catchment or diversion structure can be used to limit runout of debris and protect specific areas downslope of potential landslides. Catchment structures consist of a barrier to stop and contain landslide debris. Diversion structures are not intended to stop a debris flow, but to divert it away from a specific area. In either case, once a landslide begins, the landslide debris must accumulate somewhere. Catchment and diversion structures only change the location where the landslide debris is deposited. Following a landslide, catchment areas must be cleaned to prevent a future landslide from overtopping the catchment structure.

Catchment structures are typically built to protect a specific structure or road and are oriented across the slope, i.e., at right angles to the landslide debris path. Catchment structures must be designed to withstand the impact and contain the volume of the landslide debris. Therefore, their design requires a site-specific study to determine the likely size of the landslide, the zone of debris runout, and the velocity of the landslide debris at the desired catchment location. Catchment walls can consist of structures normally used for retaining walls provided they can be free standing. Examples include soldier piles and lagging, concrete cantilever walls, gabion baskets, and riprap or soil berms. The wall should include a drainage layer on the upslope side to promote dewatering of the landslide debris after it is deposited. This drainage will reduce the static loads on the wall, and will facilitate excavation of the landslide debris from behind the wall. Because the landslide debris must be removed following every landslide, the wall design should include permanent access for earthmoving equipment after the wall is completed. Depending on the location of a catchment wall, it could also be designed to provide support at the toe of a slope.

Diversion structures are not intended to stop landslide debris, but rather to direct the debris away from specific areas or structures. They can consist of walls, berms, or grading to direct landslide debris to an undeveloped area. The design criteria include the anticipated size of the landslide, its flow velocity, and radius of any curves in the diversion structure. The diversion structure must be sufficiently high to prevent overtopping and structurally capable of withstanding impact loads. Landslide debris should not be diverted without permission onto adjacent property. If berms are used, they should be sufficiently armored to prevent erosion and breaching. Permanent access to the depositional area should be provided to remove the landslide debris following landslides. Diversion structures are often successfully applied in areas where the debris can be diverted into natural channels. However, before this strategy is employed, the effects of additional sediment loading in streams that could receive the landslide debris should be evaluated. The Endangered Species Act and other habitat restrictions could prevent or limit this type of diversion. This scheme is not advisable unless there are no other options available.

Section 7.3 provides recommendations for selecting and designing specific types of retaining walls. These recommendations are generally applicable for designing catchment and diversion structures, with the additional requirement of the impact loads imposed by a debris flow on the wall. Figure 2-19 shows a typical pressure diagram for designing a catchment wall that includes the impact loads exerted on the wall by a debris flow.

7.7 Vegetation

Vegetative cover can contribute to the stability of steep slopes by reducing erosion, reducing direct infiltration from rainfall, and increasing the strength of the near-surface soil. Dense vegetation intercepts direct rainfall before raindrops impact the soil surface, thereby reducing or eliminating rainsplash erosion. With dense vegetative cover and thick forest litter, the likelihood of overland flow (sheetwash) is also reduced or eliminated. If overland flow does occur, the flow velocity will be reduced by the vegetation. Without overland flow or with reduced flow velocities, surficial erosion will be eliminated or reduced.

Thick vegetation, forest litter, and thick organic soil horizons typically retain moisture from direct precipitation. After a rainstorm, plant leaves retain water that is available for evaporation back into the atmosphere. The plants also transpire water that is absorbed by root systems. Water that does not runoff or return to the atmosphere by evapotranspiration eventually infiltrates into the ground. However, thick forest litter, organic soil, and heavy vegetation root systems can reduce the rate at which excess water is released into the groundwater.

Root systems can increase the strength of the soil they penetrate. The increased strength occurs as an apparent cohesion, but it does not appear to affect the angle of internal friction of the soil. The amount of apparent cohesion depends on the plant type and density. The increase in apparent cohesion that results from root strength ranges from about 20 to 250 pounds per square foot (Turner and Schuster, 1996). The effects of root reinforcement are limited to relatively shallow soil. Therefore, root reinforcement can help reduce the likelihood of shallow landslides, but will provide little improvement on slopes where deep-seated landslides are likely.

Certain types of vegetation can have an adverse effect on slope stability. Unstable trees can initiate a landslide if they are toppled during high wind conditions. Therefore, trees that pose a safety hazard (rotting, dying, or excessively leaning trees) should be removed from tops of bluffs and on slopes; however, stumps should be maintained. Slopes vegetated with dense, low-lying, deeply-rooted plants or shrubbery provide better protection from erosion and shallow landsliding than shallow-rooted vegetation. For example, grasses tend to provide a relatively small amount of protection. Generally, native vegetation is desirable because it can be maintained without irrigation during the dry season. Ideally, the vegetation should require no more moisture than what typically occurs in the region to reduce the need for watering on the slope. Publications such as "Slope Stabilization and Erosion Control Using Vegetation" (Washington State Department of Ecology Publication 93-30) provide guidance in selecting plant species. As described previously, yard debris, or any other debris or fill, should not be placed on the slope as the additional loading adversely affects slope stability and inhibits plant growth.

8.0 COST ESTIMATE

Representative unit costs of improvement measures are presented in Table 2-1, Typical Improvement Unit Costs. This table can be used for calculating a preliminary budget of the remedial work contemplated. For a specific project, a more definite cost should be based on a more accurate cost breakdown that includes labor, materials, equipment, and engineering and administrative project costs.

The unit costs presented are based mostly on recent contractor bids on work which Shannon & Wilson, Inc., was involved within the Seattle area. Some prices were obtained from telephone interviews with specialty wall manufacturers and contractors. In some cases, unit prices from bids in other States and Means Cost Data were used. Reference projects were both small and large and included residential and commercial work as well as public and private projects. The prices are representative of the prevailing cost for the 1997-1998 period. Budget estimates based on this table should be adjusted for inflation in the following years.

The unit prices are shown as a range and as an average. In some cases, we did not have sufficient information to provide a range of cost. In these cases, we provided only the average unit price. Typically, the greater unit price should be used in smaller residential projects and the lesser unit price should be applied for larger projects. Other costs that should be added include:

- Incidentals and contingencies.
- A mobilization fee of approximately 10 to 15 percent of construction cost should be added to the unit cost items.
- Engineering costs for design that typically range from 10 to 15 percent, depending on the size and complexity of the job.
- Contract administration and construction observation costs.
- City administrative costs.
- State sales tax.

A number of other factors should be considered in making a preliminary cost estimate. These include:

- Anticipated weather conditions
- Access difficulties
- Availability of staging areas

- Environmental constraints such as wetlands and erosion control
- Availability of qualified contractors
- Union or non-union labor wages
- Required traffic control
- Noise constraints imposed by neighbors
- Available work hours

The effect of these and other factors specific to the site need to be included in the final budget estimate. Total project costs can increase by a factor of approximately two or three when all the above factors and cost additions are included.

The cost items considered in Table 2-1 follow in general the outline of improvements discussed in Section 7.

9.0 UTILITIES AND STREETS

Buried utilities and streets can affect the stability of the slopes they are built on, into, or adjacent to. The presence of a buried utility or a street can act either to enhance or reduce stability. For example, a utility trench could be designed and built to act as a trench subdrain that would remove groundwater from a slope, thereby improving the stability. The same utility trench, if not properly graded, covered, or drained, could provide a conduit to rapidly convey surface and/or groundwater to a critical portion of a slope, and then infiltrate the water at that location. For another example, low permeability street pavements typically inhibit infiltration of surface water into the groundwater, thereby improving stability. However, if the storm water system is inadequate or not present, then uncontrolled, concentrated surface water runoff can discharge onto a slope and reduce its stability. This section presents recommendations and typical design concepts for using buried utilities and streets to improve stability. For instances where stability improvements are not practical, this section makes recommendations for reducing possible destabilizing effects.

9.1 Streets

Streets can be used in a number of ways to improve the stability of the slopes they are built on or adjacent to, which include the following:

- Reduce infiltration
- Storm water runoff control

- Subsurface drainage
- Structures and grading improvements

We recommend that the City consider these types of improvements when building a new street, performing maintenance, or when rebuilding an existing street. The following sections provide typical details and design recommendations regarding these improvements.

9.1.1 Reduce Infiltration

Most streets are paved with low permeability asphalt concrete or Portland cement concrete. As such, streets tend to reduce infiltration into the subsurface. Where reducing infiltration could improve stability, we recommend adopting the following measures:

1. Use low permeability pavements. Do not use pavement materials that are designed to allow rapid infiltration of surface water, such as Class F asphalt concrete.

2. Design the pavement section for a high degree of reliability and long service life to reduce deterioration and cracking that would increase the permeability of the pavement surface.

3. The performance of a pavement depends largely on the condition of the subgrade. Therefore, subgrade improvements should be made where practical, such as with new streets or major renovations and repairs. Subgrade improvements include overexcavating soft, loose, and compressible soil until undisturbed, firm, and unyielding native soil is exposed. Any backfill or embankment fill materials should be placed and compacted in accordance with the Seattle Standard Specifications, except that all fill material should be compacted to at least 95 percent of the maximum dry density (American Society for Testing and Materials [ASTM] D 698).

4. Perform regular inspections and maintenance to detect and seal significant cracks, if necessary. Evaluate the subgrade in areas of chronic cracking. Correct soft or loose subgrade conditions that lead to poor drainage and/or cracking, if found to be needed.

5. Provide adequate storm water drainage system. Grade pavement surfaces as may be found needed to promote rapid runoff and to prevent ponding.

9.1.2 Storm Water Runoff Control

Storm water runoff from streets and other low permeability surfaces, including areas of low-permeability soil, is sometimes a contributing factor to landslides. Developers and private property owners must assess existing conditions and take steps to protect their property and comply with existing drainage codes. In general, storm water drainage is beyond the scope of this study. We understand that another study has been commissioned by the City to study storm water drainage citywide, which will address issues such as adequacy of storm drainage collection in landslide-prone areas and storm sewer capacity.

The following measures can be used to reduce the flow of storm water from streets onto or adjacent to slopes:

1. Provide curbs and/or lined storm water ditches to prevent runoff onto or adjacent to slopes. Curbs or ditches should be designed to contain and convey all runoff to a storm sewer or other appropriate facility. In some cases, curbs that are higher than normally built could be effective in controlling runoff in landslide-prone areas. The capacity of the ditch or curb should take into account the design storm, and reasonable allowances for reductions in capacity from debris and/or melting snow or ice.

2. We do not recommend constructing unlined ditches to convey storm water runoff in landslide-prone areas.

3. Grade streets to drain into storm sewer catch basins. Provide curbs and berms as needed to ensure proper runoff into the catch basins.

4. Regularly inspect and maintain curbs, ditches, and storm drains.

5. Educate and enlist the assistance of neighborhood organizations or individual residents regarding storm drainage facilities. Residents could perform simple surface cleaning of debris and/or could notify the City when maintenance is needed. Communication lines to the City need to be open, accessible, and made known to the public.

9.1.3 Subsurface Drainage

Subsurface drainage can be incorporated during construction and/or renovation of streets and adjacent storm water ditches. In general, subsurface drainage associated with streets would fall into two general categories: trench subdrains built under or adjacent to a street and a drained pavement base course.

A drained pavement base course can intercept shallow groundwater and surface water that infiltrates through the pavement surface. It can improve stability of slopes below the road to the extent that groundwater is intercepted and infiltration is reduced. Usually, this type of shallow drainage will have the greatest benefit in improving the stability of roadway embankment fills. However, in areas of shallow groundwater, drainage in the base course can effectively drain natural slopes. While not related to slope stability, well-drained pavements generally perform better and have a longer service life. A drained pavement base course is constructed in the same manner as a normal base course, with the following exceptions:

1. Grade the pavement subgrade to drain into a perforated or slotted collector pipe. The collector pipe should be constructed in the same manner as a trench subdrain, described in Section 7.

2. The collector pipe should be graded to drain to a suitable discharge point, such as a storm sewer. It should not be allowed to discharge directly onto the surface.

3. Cleanout points should be provided for the collector pipe, and a regular cleaning and maintenance program adopted.

4. The base course aggregate should meet the requirements listed in Section 4-04.2 of the Seattle Standard Specifications, with the following additional requirements.

   a) The aggregate should have not more than 3 percent passing the No. 200 mesh sieve, based on the minus 3/4-inch fraction in a wet sieve analysis (ASTM D 422).

   b) The aggregate should also meet filter criteria with respect to the underlying subgrade soil. A non-woven filter fabric could be placed between the subgrade and the drainage base course layer in lieu of using a base course aggregate.

Trench subdrains associated with streets would not be substantially different in their application and construction from those described in Sections 6.0 and 7.0. Streets in landslide-prone areas are generally parallel to the slope. As such, they are well suited for constructing a groundwater cutoff trench subdrain, either in or adjacent to the street. A groundwater cutoff trench subdrain could be particularly effective in improving the stability of an embankment fill that was placed over a soil with relatively low permeability, such as Lawton Clay or a fine-grained colluvium. It could also effectively dewater relatively permeable colluvium that overlies Lawton Clay or other low permeable soil.

A trench subdrain could be built either in the roadway, and then paved over, or on the upslope side of the roadway. If a storm water drainage ditch is being excavated next to the roadway, a trench subdrain could be incorporated. The trench would be excavated to the depth needed for the subdrain. The trench subdrain materials would be placed and then covered with a low permeability liner material for the storm water ditch.

As with all subsurface drainage, the collector pipes should discharge to a suitable location, such as a storm sewer. The system should include provisions for periodic inspection and cleaning. The City should adopt a regularly scheduled program for inspecting, cleaning and maintaining subsurface drainage.

9.1.4 Structures and Grading Improvements

New street construction or a major street renovation provides the opportunity to incorporate structures and grading improvements that would improve stability. In general, these types of improvements are described in Sections 6.0 and 7.0. Specific applications of structures and grading improvements that could be applied when building or renovating a street include the following:

1. All proposed structures, embankment fills, and excavations, whether retained or not, should be evaluated to determine the effects, if any, they will make on the stability of the slope. Both the slopes above and below the proposed improvements should be evaluated.

2. Streets constructed near the top of a slope could make use of retained or sloped excavations to remove load from the upper portions of a marginally stable or potentially unstable slope.

3. Where fills are required near the top of a slope or midslope, lightweight fill materials can be used to reduce new loads imposed on the slope, or to reduce the loads currently imposed on the slope. A common lightweight fill material used in roadway construction is expanded polystyrene (geofoam). Other types of lightweight fill material are discussed in Section 7.5.2.

4. Retaining structures can be used to retain cuts and fills to improve the overall stability of the slope. Walls designed to retain excavations on the upslope side of a street could also be designed as catchment walls. This "double duty" would be relatively inexpensive, yet it could provide considerable protection of streets in areas where debris flows are likely.

5. Streets constructed near the toe of a slope could be built on an embankment fill that also serves as a toe buttress to improve the stability of the slope above.

9.2 Buried Utilities

Buried utilities, such as water, sewer, and storm drainage pipes and electrical and communication lines, could be used to improve the stability of a slope by providing subsurface drainage. In some cases, grading changes could be made when a buried utility is installed that could also improve stability. In some cases, buried utilities have triggered landslides. These cases include pipe leaks and breaks, and possibly when groundwater is conveyed to a marginally stable slope in the trench backfill. The following sections describe methods to improve slope stability associated with buried utilities, including:

- Subsurface drainage
- Groundwater control methods
- Old buried utilities
- Grading improvements

9.2.1 Subsurface Drainage

A buried utility trench could also be used as a trench subdrain. In general, the use and design of a trench subdrain that is associated with a buried utility is the same as presented in Sections 6.0 and 7.0. The utility location limits where drainage can be installed. Therefore, the potential effectiveness of such drainage as well as the possibility for conveying groundwater into an inappropriate location should be carefully evaluated. In addition to the recommendations presented in Sections 6.0 and 7.0, we recommend that trench subdrains constructed in conjunction with a planned buried utility include the following elements:

1. The trench subdrain should extend all the way through saturated or potentially saturated soil. Portions of the utility trench that are excavated in permeable, unsaturated soil should be backfilled with clay or another low permeability material. The collector pipe should be connected to a tightline through such permeable trench sections. If a perforated pipe was placed in unsaturated permeable soil, water could flow from the perforated pipe into the soil it was intended to drain. Under these circumstances, the trench subdrain could actually reduce the stability of a slope.

2. A slotted or perforated collector pipe should be included as part of the trench subdrain system. It should be designed with sufficient capacity to convey the anticipated groundwater inflow.

3. Concrete or clay dams should be built wherever perforated pipes are connected to tightlines. The concrete or clay dams will force the water into the tightline and prevent water from moving along the outside of the tightline. Section 7.2.1 provides additional information on tightline connections.

4. The trench and the collector pipe should be continuously graded to drain so there are no low spots where water would tend to pond.

5. The trench backfill, collector pipe and native soil should be compatible with respect to filter criteria to prevent piping of fines that could cause loss of ground or clogging of the collector pipe.

6. Provide cleanouts and provisions for maintaining the collector pipe.

7. Install groundwater monitoring wells along the utility trench to verify that the trench subdrain is functioning as intended. The monitoring wells should be used to determine when maintenance is required.

9.2.2 Groundwater Controls

Some landslides have been attributed to buried utilities. Usually, these instances involve a pipe leak or break in a water, sewer, or storm drainage line. The following section provides recommendations for constructing pipes in landslide prone areas. The utility bedding and/or trench backfill can also provide a path for groundwater to migrate to a landslide area. In such cases, bedding and trench backfill for utilities should be made in a manner that either does not change the drainage characteristics of the soil, or in a way that inhibits groundwater migration to the slope. This section provides recommendations for constructing buried utilities to prevent groundwater migration to potential landslide zones.

Buried utilities can provide an adverse path for groundwater migration under the following circumstances:

1. The backfilled trench passes through saturated soil, i.e., a groundwater source, and then into an area of unsaturated permeable soil that is marginally stable.

2. The pipe bedding and/or trench backfill is more permeable than the native soil, but is not sufficiently well-drained to maintain groundwater levels in the backfill that are below the groundwater level in the adjacent native ground.

3. The trench is not continuously graded, so there are low areas where water can infiltrate from the pipe bedding or trench backfill into the native ground.

4. The trench backfill is not covered with a low permeability soil at the surface, thereby allowing surface water to infiltrate into the permeable backfill.

Water can migrate along a buried utility either in a permeable backfill material, or along small voids between the pipe and the backfill material. The latter process, piping, can also result in ground loss by erosion of the backfill material around the pipe. The following measures can reduce the potential for undesirable groundwater migration along a buried utility.

1. When practical, backfill the trench with compacted native soil. This should result in a trench backfill that is hydraulically similar to the undisturbed ground.

2. Backfill the upper 1 to 2 feet of a utility trench with low permeability soil to reduce surface infiltration. In landscaped areas, mound the backfill soil over the trench and grade the surrounding area to promote runoff away from the trench and to reduce the possibility of ponding.

3. Install concrete or clay dams at intervals along the pipe to prevent groundwater flow in the pipe bedding and/or backfill. Concrete or clay dams can also reduce the potential for piping.

4. Pervious granular bedding material is often required for certain types of pipes. In these cases, consider alternate pipe materials or install a sufficient number of concrete or clay dams to prevent groundwater migration into sensitive areas. If possible, collect water from behind the concrete or clay dams with a tightline.

5. Install utilities above ground.

6. Provide subsurface drainage at key points. For example, a trench subdrain could be installed where saturated soil is encountered in the utility excavation. A concrete or clay dam should be installed at the end of the trench subdrain section to force the groundwater into the collector pipe and to prevent further groundwater migration along the buried utility.

9.2.3 Old Buried Utilities

Numerous existing pipelines, and especially sewers, were constructed by bedding the pipe in pea gravel. This pea gravel bedding material has a relatively high permeability that provides the capacity to convey potentially large volumes of water. If water is conveyed out of a potentially unstable slope, the stability of the slope is improved. However, as noted in Section 9.2.2, the opposite can also occur. That is, pea gravel pipe bedding can act as a conduit to rapidly convey water into an unstable slope, thereby reducing the stability of the slope. We recommend establishing a program to evaluate buried utilities that are in or adjacent to landslide-prone areas. Those that may have pervious bedding and/or backfill material, and especially pea gravel pipe bedding, should be further evaluated to determine if they have the potential to adversely affect slope stability. For buried utilities that could adversely affect slope stability, we recommend the following:

1. If the utility is old and close to its design life, consider early replacement. The replacement utility should be designed to improve subsurface drainage, as described in Section 9.2.1, or to prevent adverse groundwater migration, as described in Section 9.2.2. If this alternative is selected, the old buried utility should be excavated to remove pervious bedding and/or backfill materials.

2. Install concrete or clay dams at key locations to prevent groundwater migration along the pervious bedding and/or backfill material. If possible, drainage should be installed at each concrete or clay dam location to collect and convey the groundwater to a suitable discharge location.

3. Install adjacent drainage to intercept water that the buried utility may convey into a marginally stable slope. Such drainage could include trench subdrains that are located downgradient from the buried utility.

4. Grout the pipe bedding and/or backfill to reduce the permeability. While this alternative may be technically feasible, it is also relatively expensive. Therefore, we anticipate that it would be used only for relatively short sections where other alternatives are not practical.

9.2.4 Grading Improvements

In most cases, buried utilities are placed in trenches that are subsequently backfilled to restore the original grades. For these cases, slope stability improvements are mostly limited to subsurface drainage as described in the previous section. However, in certain circumstances, grading improvements could be made in conjunction with placing utilities. In most cases, grading improvements would be made when desirable for maintaining stability of the proposed utility installation. For example, excavations could be made at the top of a slope to reduce the driving forces of a marginally stable slope in conjunction with installing a pipeline. Lightweight fill materials can be effective in improving stability or reducing adverse effects when a fill is needed midslope or at the top of a slope. As mentioned previously, the stability of any fills or excavations should be evaluated to demonstrate that the stability both above and below the proposed grading is not adversely affected.

Large utility excavations that extend below a landslide failure surface or potential failure surface could be backfilled with compacted angular aggregate to form a shear key.

9.2.5 Other Considerations

Landslide prone areas pose a breaking or rupture hazard to buried utilities. Water lines, sewers, and storm drains that are damaged by ground movement can cause leaks that further exacerbate the unstable conditions. Therefore, before installing new buried utilities, the utility route should avoid areas where ground movement is likely. Where these areas cannot be avoided, several alternatives could be considered as may be appropriate to reduce the likelihood of damage. These include:

1. Install the utilities above ground. Above ground installations generally are less susceptible to damage from relatively small ground movements. Also, they can be readily inspected for damage. Storm drainage and communication lines are particularly well suited to above ground installations.

2. Use materials that are more tolerant to ground motion. For example, bell and spigot concrete pipe is sensitive to relatively small movement as compared to HDPE pipe that has fused joints.

3. Install flexible connections and joints that also allow for some extension or compression.

4. Use pumped sewer lines in landslide areas instead of gravity drainage. A pumped sewer line does not need granular bedding material to set the pipe at the grades required for drainage. Also, if small movements cause grade changes, a pumped line would not be affected, whereas a gravity line may not function as designed.

TABLE 2-1
TYPICAL IMPROVEMENT UNIT COSTS

Method	Description	Unit	Range $/unit	Average $/unit
Surface Water Improvements				
	Tightlines	LF	10-30	20
	Tightline Anchor	EA	500-1,000	750
	Paved Swales	LF		6
	Machine Formed Concrete Curbs	LF	10-25	15
	Catch Basins	EA	1,200-3,600	2,200
	Sealing Cracks In Pavement	LF	0.50-1.00	1
	Paving (Asphalt Concrete)	SY	85-150	120
Groundwater Improvements				
	Trench Subdrains			
	5 ft deep, complete	LF	15-70	35
	10 ft deep, complete	LF	50-150	100
	15 ft deep, complete	LF	110-380	220
	15 ft deep with Trench Box, complete	LF	140-440	260
	Trench Excavation	CY	15-60	30
	Trench Box Shoring	SF	1-2	2
	Backfill: Common Backfill	CY	5-20	10
	Backfill: Seattle Type 26 Aggregate	CY	20-70	45
	Dam: Concrete	EA	300-1,700	1,150
	Dam: Clay	EA	250-500	350
	Subdrain Cleanout	EA	125-800	360
	Finger Drain: Typically 20 ft long 5 ft deep	EA	550-1,800	1,000
	Springhead Drains	EA		1,000
	Drainage Blanket	CY	20-70	45
Drilled Drains				
	Horizontal Drains			
	3-in-dia Sand	LF	14-20	17
	3-in-dia Sand/Gravel	LF	14-20	18
	Cleaning Horizontal Drains	LF		1
	Directional Drains			
	3-in-dia	LF	25-85	50

TABLE 2-1
TYPICAL IMPROVEMENT UNIT COSTS (CONT.)

Method	Description	Unit	Range $/unit	Average $/unit
	Vertical Drains			
	Dewatering Well: 24-inch-dia	LF	9-35	22
	Pump and Power	Week		140
Structures (excluding drainage provisions)				
	Pile Walls			
	Cantilever Soldier Pile – Concrete Facing	SF	45-140	105
	Cantilever Soldier Pile – Timber Lagging	SF	30-90	75
	Soldier Pile & Tieback – Concrete Facing	SF	70-190	140
	Soldier Pile & Tieback – Timber Lagging	SF	55-140	110
	Tangent Drilled Shafts	SF	60-100	80
	Sheet Pile – Permanent	SF	16-24	20
	Sheet Pile – Removed/Salvaged	SF	11-13	12
	Gravity Walls			
	Reinforced Concrete Cantilever	SF	30-60	50
	Gabion Baskets	SF	25-55	40
	Rock-filled Concrete	SF	32-37	35
	Metal Cribs	SF	33-55	45
	Concrete Cribbing, excluding backfill	SF	20-35	30
	Timber Cribs, excluding backfill	SF	15-25	20
	Ecology Blocks	SF	15-30	25
	Catchment Walls			
	Soldier Pile and Concrete Lagging	SF		185
	Reinforced Concrete Cantilever	SF	30-60	42
	Gabion Baskets	SF	25-55	40
	Riprap Berms	CY	36-40	38
	Soil Berms	CY		25

TABLE 2-1
TYPICAL IMPROVEMENT UNIT COSTS (CONT.)

Method	Description	Unit	Range $/unit	Average $/unit
Soil Reinforcement				
	Geotextile Reinforced Soil Slope, including backfill	SF	15-30	25
	Segmental Block (MSE) 10-30 ft high	SF	20-55	35
	Panel Facing Systems (MSE) 10-30 ft high	SF	25-55	40
	Rockery Facing	SF	15-30	25
	Soil Nail Walls: Permanent	SF	20-75	35
	Soil Nail Walls: Temporary	SF	20-25	22
Grading and Fills				
	Clearing and Grubbing	SF	1-3	2
	Excavation	CY	20-50	30
	Hauling (one mile)	CY	5-10	8
	Riprap Backfill	TON	20-40	35
	Crushed Rock Backfill	CY	30-60	45
	Sand and Gravel Backfill (Seattle Type 17)	CY	20-60	35
	Soil Compaction	CY	2-4	3
Lightweight Fills				
	Expanded Polystyrene	CY	50-70	60
	Cellular Concrete	CY	700-200	
Vegetation				
	Hydro-seeding	SY	2-8	4
	Hand-seeding	SY	5-10	8
	Jute Mesh	SY	1.50	
City of Seattle Retaining Wall Construction Costs*				
	Cantilever Soldier Pile (H-pile)	LF	707-3,917	2,137
	Soldier Pile (H-pile) with Tiebacks	LF	2,541-7,341	3,752
City of Seattle Retaining Wall Total Costs*				
	Cantilever Soldier Pile (H-pile)	LF	--	3,500
	Soldier Pile (H-pile) with Tiebacks	LF	--	6,100

TABLE 2-1
TYPICAL IMPROVEMENT UNIT COSTS (CONT.)

Legend:

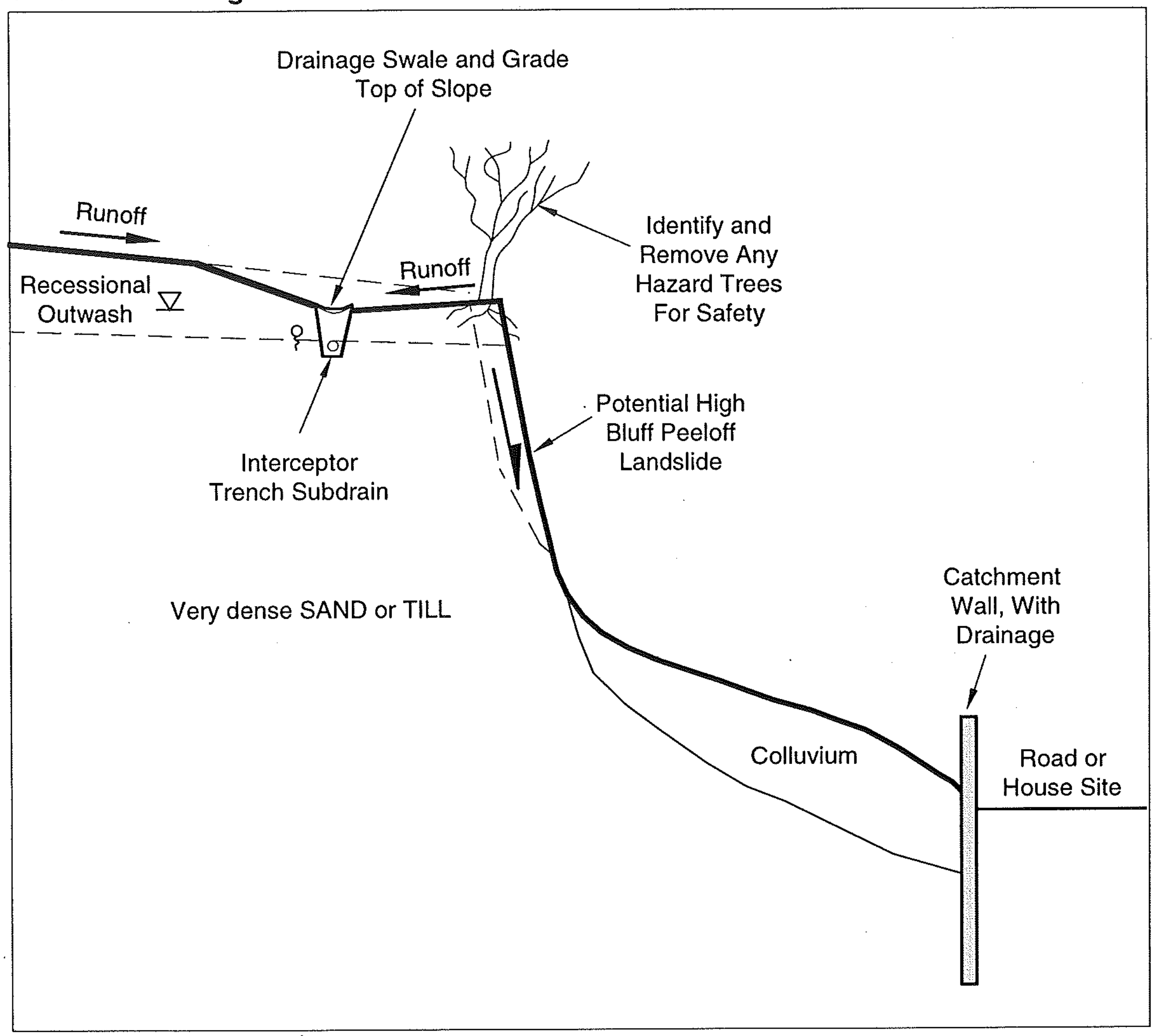
CY	=	cubic yard
dia	=	diameter
EA	=	each
ft	=	foot
in	=	inch
LF	=	linear foot
MSE	=	Mechanically Stabilized Earth Wall, including excavation and backfill
SF	=	square foot
SY	=	square yard

* City experience based on cost of walls built in last 15 years, with amounts adjusted to 1997 dollars. Exposed wall heights typically range from approximately 8 to 15 feet.

NOTE:

The unit costs in this table should be used in conjunction with the information provided in Section 8.0 of this report.

Sketch A - Drainage and Catchment



NOT TO SCALE

LEGEND

Potential Movement

Perched Water

Seepage

Seattle Landslide Study
Seattle Public Utilities
Seattle, Washington

TYPICAL HIGH BLUFF PEELOFF LANDSLIDE STABILITY IMPROVEMENTS

July 1999

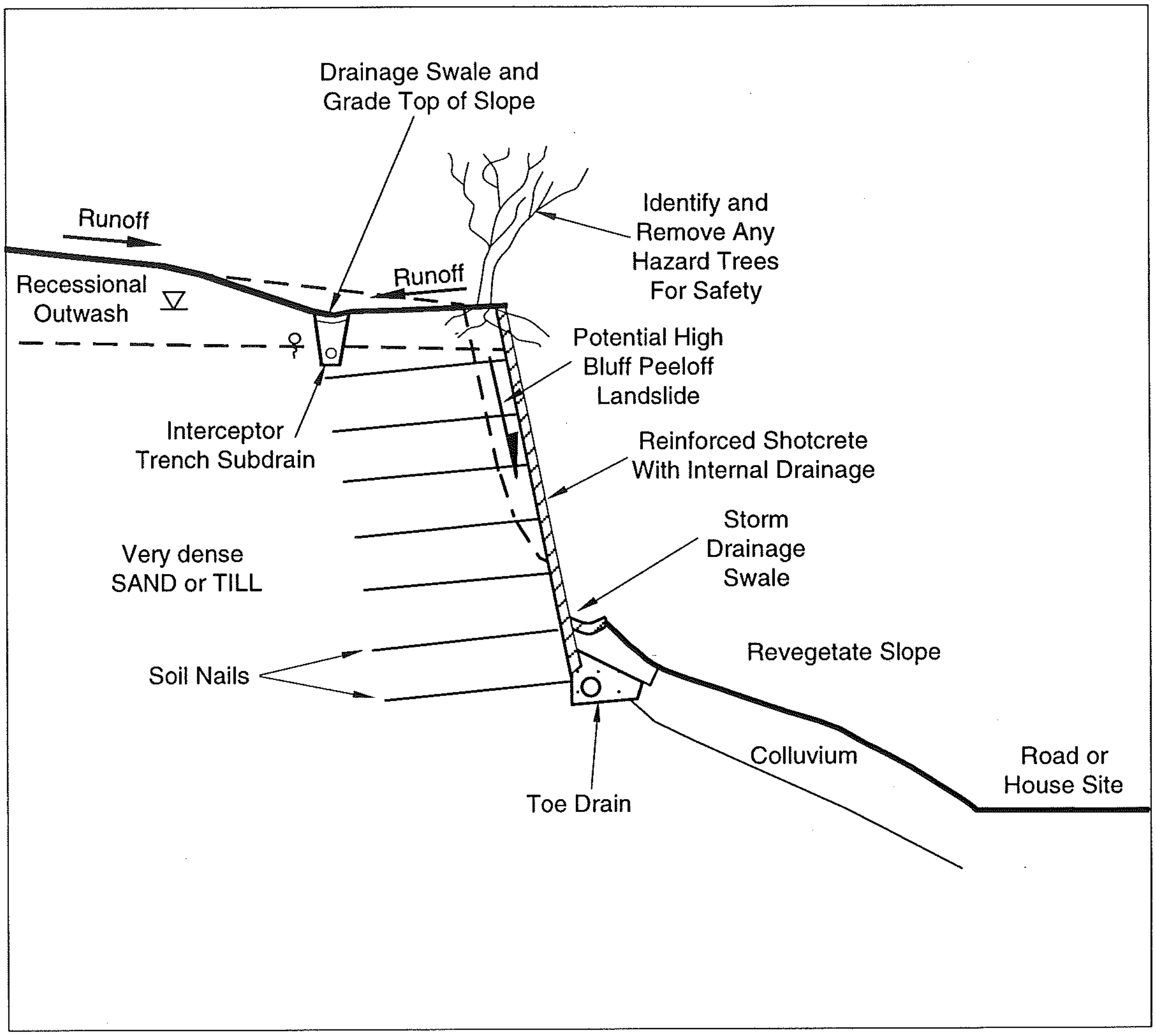
W-7992-01

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 2-1
Sheet 1 of 3

I/Documents/Mac/W Jobs/W-7900s/W-7992/01/W-7992-01 Fig. 2-1

Sketch B - Shotcrete and Soil Nails



NOT TO SCALE

LEGEND

Potential Movement

Perched Water

Seepage

Seattle Landslide Study
Seattle Public Utilities
Seattle, Washington

TYPICAL HIGH BLUFF PEELOFF LANDSLIDE STABILITY IMPROVEMENTS

July 1999 W-7992-01

SHANNON & WILSON, INC.
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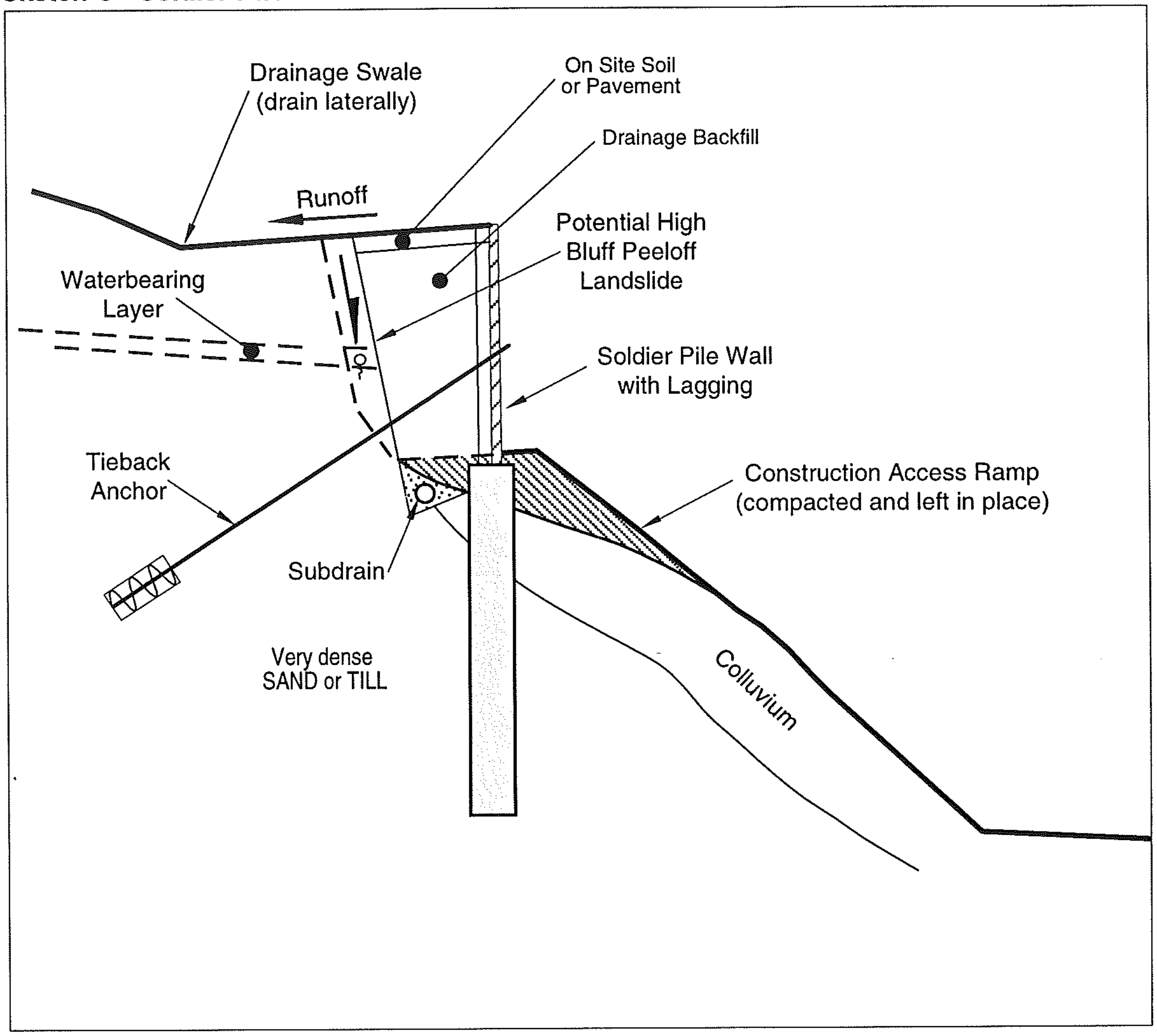
FIG. 2-1
Sheet 2 of 3

I/Documents/Mac/W Jobs/W-7900s/W-7992/01/W-7992-01 Fig. 2-1 Sheet 2 of 3

Sketch C - Soldier Pile Wall



NOT TO SCALE

LEGEND

Potential Movement

Seepage

Seattle Landslide Study
Seattle Public Utilities
Seattle, Washington

TYPICAL HIGH BLUFF PEELOFF LANDSLIDE STABILITY IMPROVEMENTS

July 1999 W-7992-01

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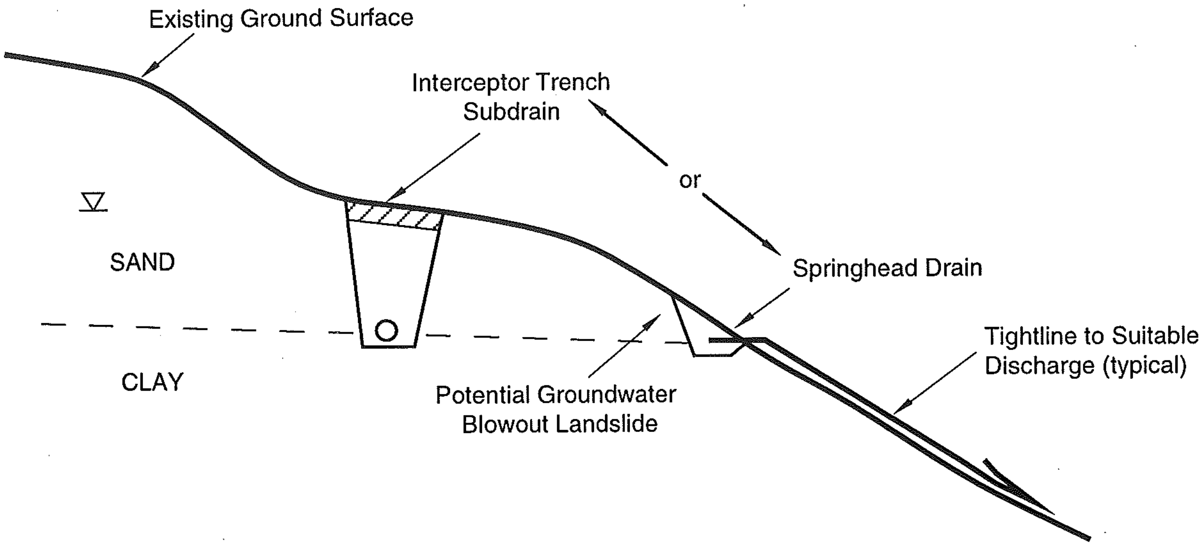
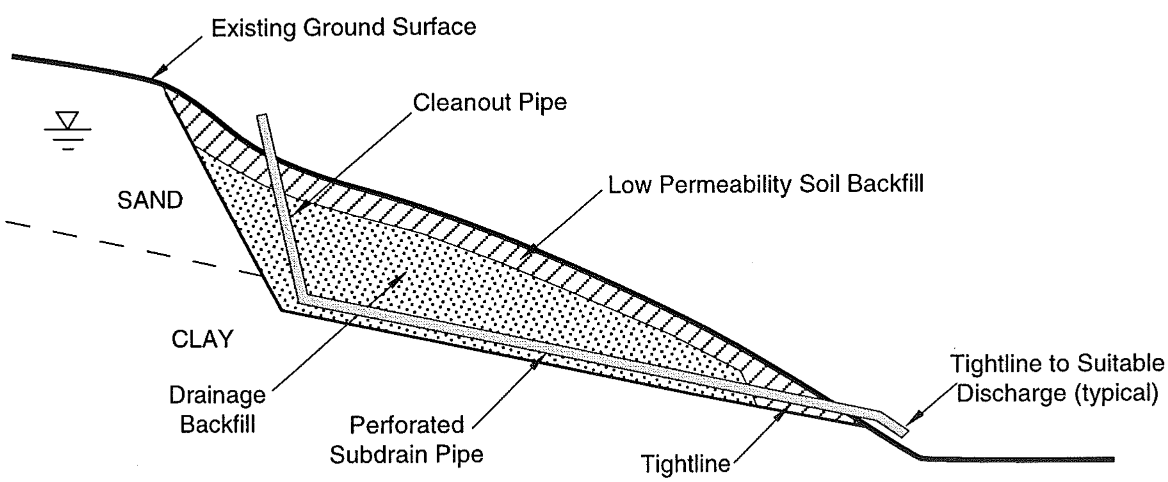
FIG. 2-1
Sheet 3 of 3

Sketch A - Interceptor Trench Subdrain and Springhead Drain



Sketch B - Finger Drain



SKETCHES NOT TO SCALE

LEGEND

▽ Perched Water

♀ Seepage

Seattle Landslide Study
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Seattle, Washington

TYPICAL GROUNDWATER BLOWOUT LANDSLIDE STABILITY IMPROVEMENTS

July 1999 | W-7992-01

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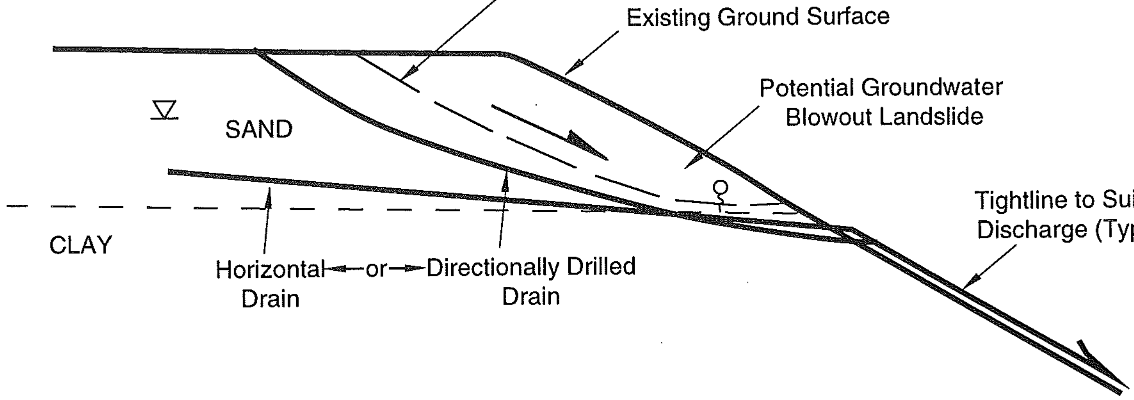
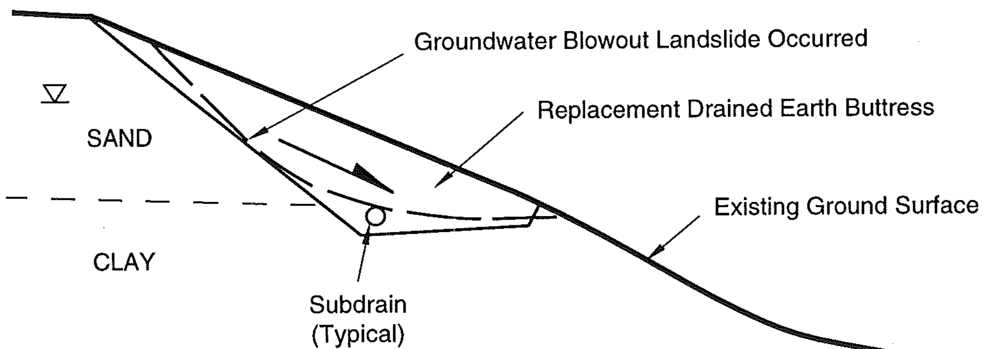
FIG. 2-2
Sheet 1 of 2

Sketch C - Drilled Drains



Sketch D - Replacement Earth Buttress



SKETCHES NOT TO SCALE

LEGEND

Potential Movement

Perched Water

Seepage

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Seattle Public Utilities
Seattle, Washington

TYPICAL GROUNDWATER BLOWOUT LANDSLIDE STABILITY IMPROVEMENTS

July 1999 W-7992-01

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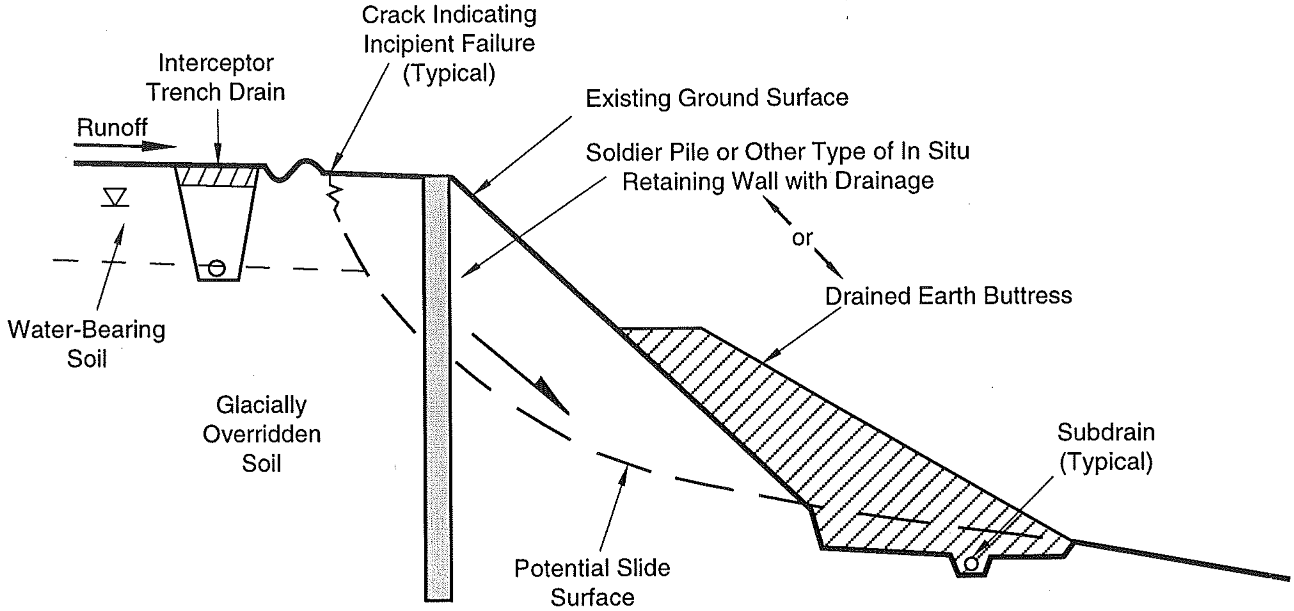
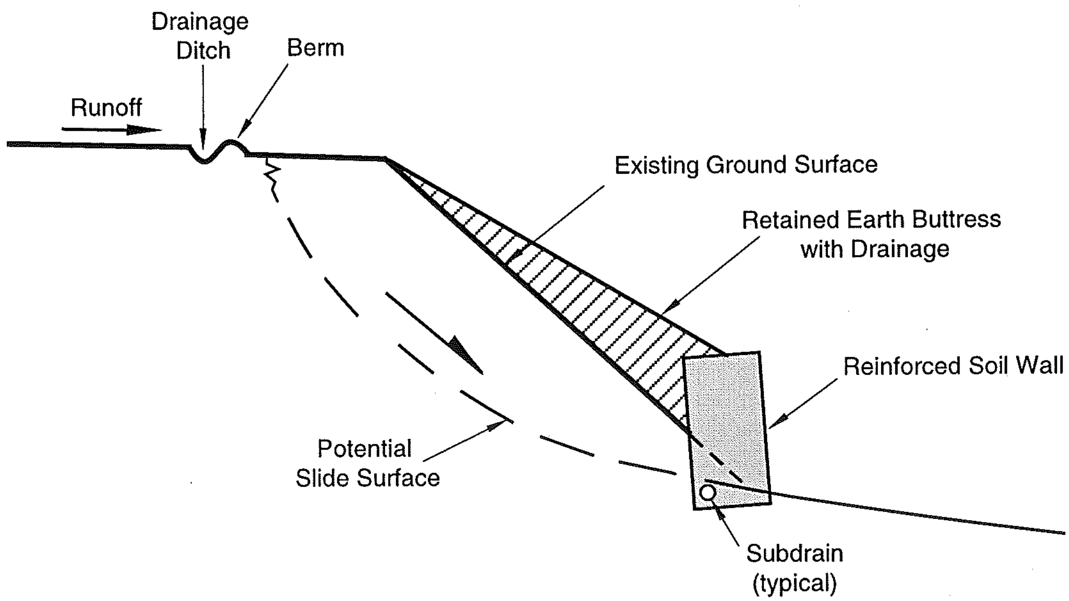
FIG. 2-2
Sheet 2 of 2

Sketch A - Earth Buttress and In Situ Wall Alternatives



Sketch B - Retained Earth Buttress



SKETCHES NOT TO SCALE

LEGEND

Potential Movement

Perched Water

Seepage

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Seattle Public Utilities
Seattle, Washington

TYPICAL DEEP-SEATED LANDSLIDE STABILITY IMPROVEMENTS

July 1999 W-7992-01

SHANNON & WILSON, INC.
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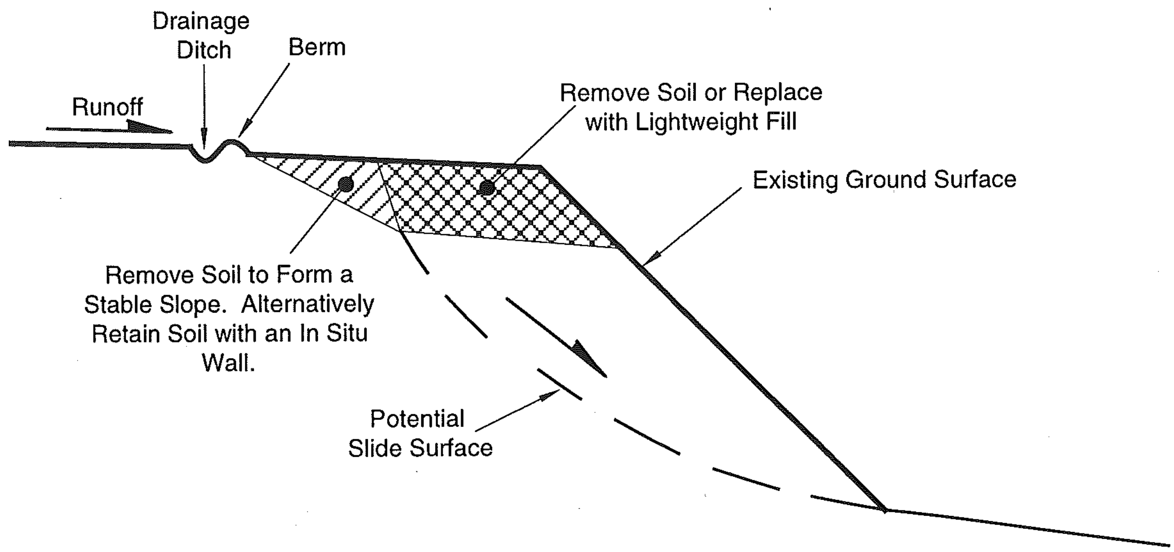
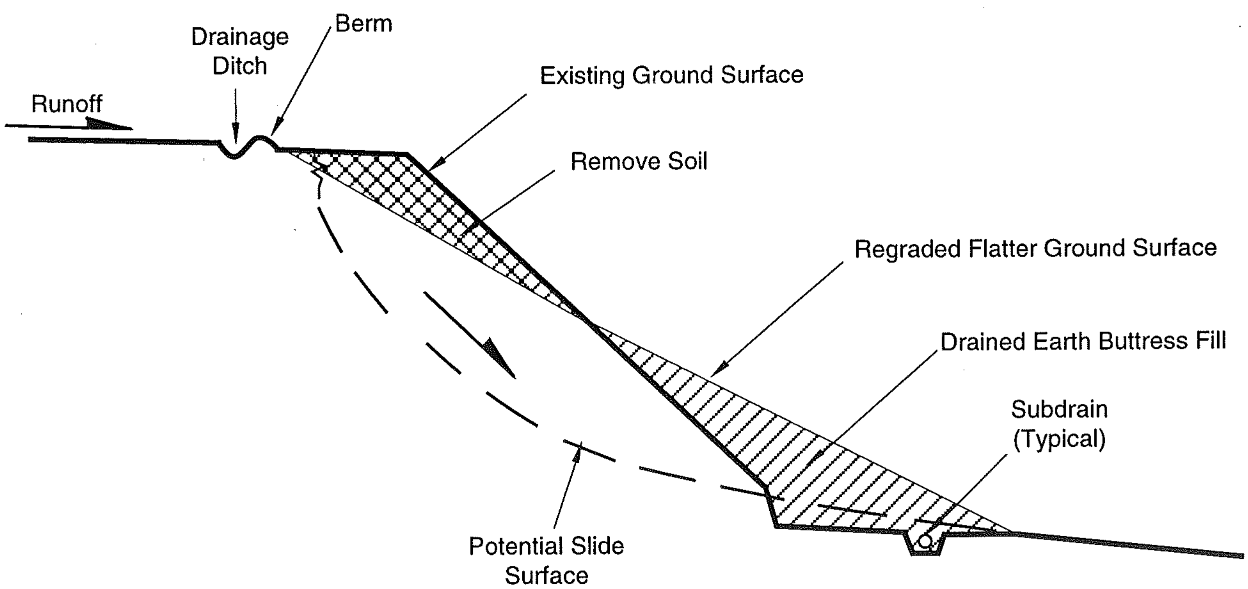
FIG. 2-3
Sheet 1 of 3

Sketch C - Remove Driving Weight



Sketch D - Flatten Slope



SKETCHES NOT TO SCALE

LEGEND

Potential Movement

Perched Water

Seepage

Seattle Landslide Study
Seattle Public Utilities
Seattle, Washington

TYPICAL DEEP-SEATED LANDSLIDE STABILITY IMPROVEMENTS

July 1999 W-7992-01

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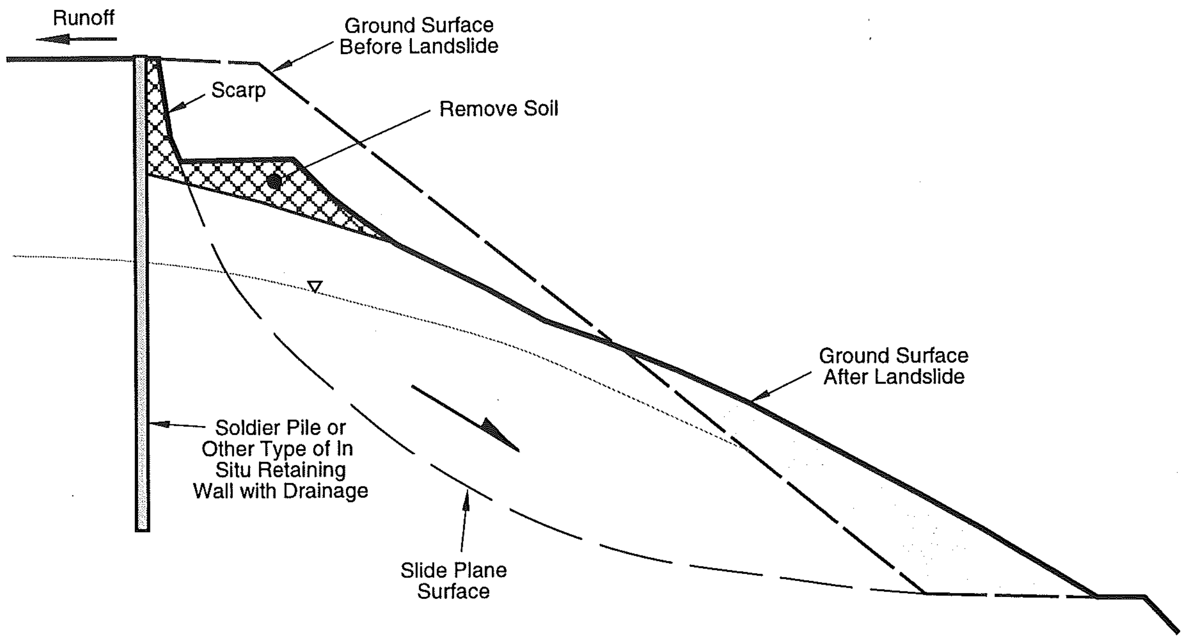
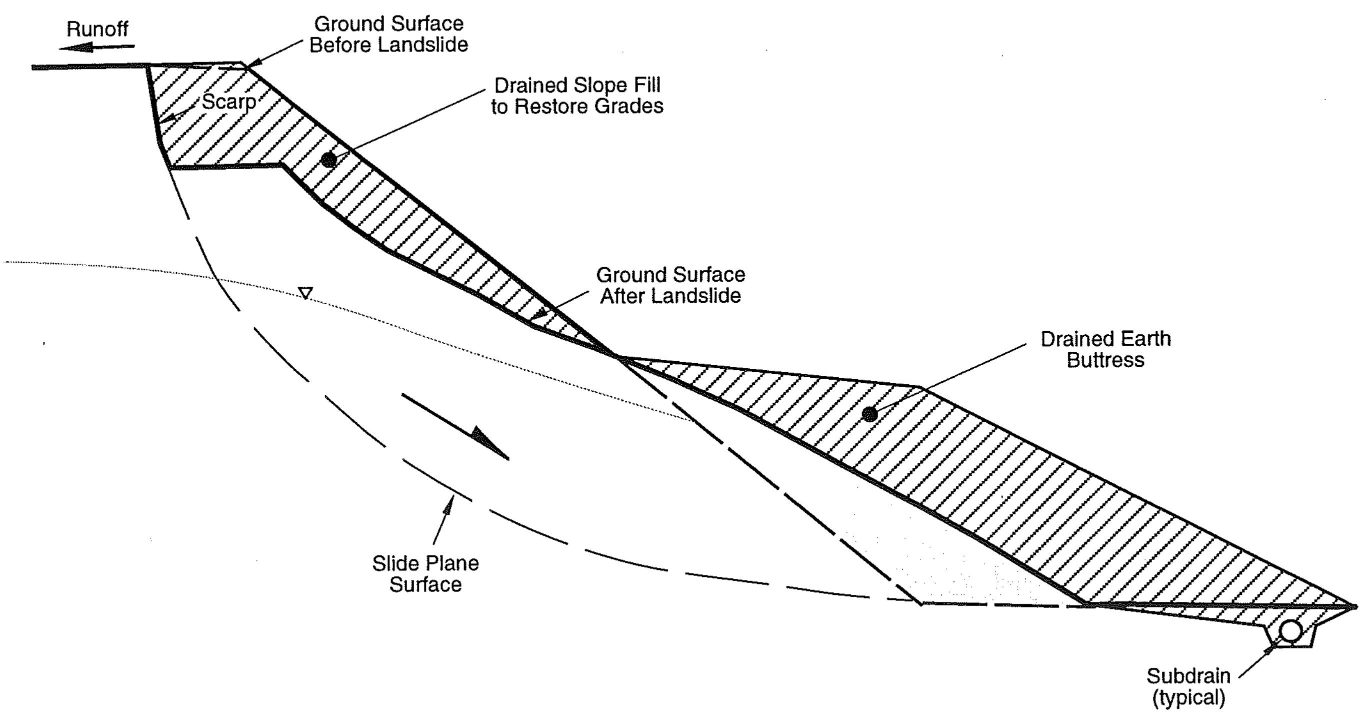
FIG. 2-3
Sheet 2 of 3

Sketch E - Retain Scarp



Sketch F - Restore Slope and Earth Buttress Fill



SKETCHES NOT TO SCALE

LEGEND

Groundwater Table Before Landslide

Landslide Movement

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Seattle Public Utilities
Seattle, Washington

TYPICAL DEEP-SEATED LANDSLIDE STABILITY IMPROVEMENTS

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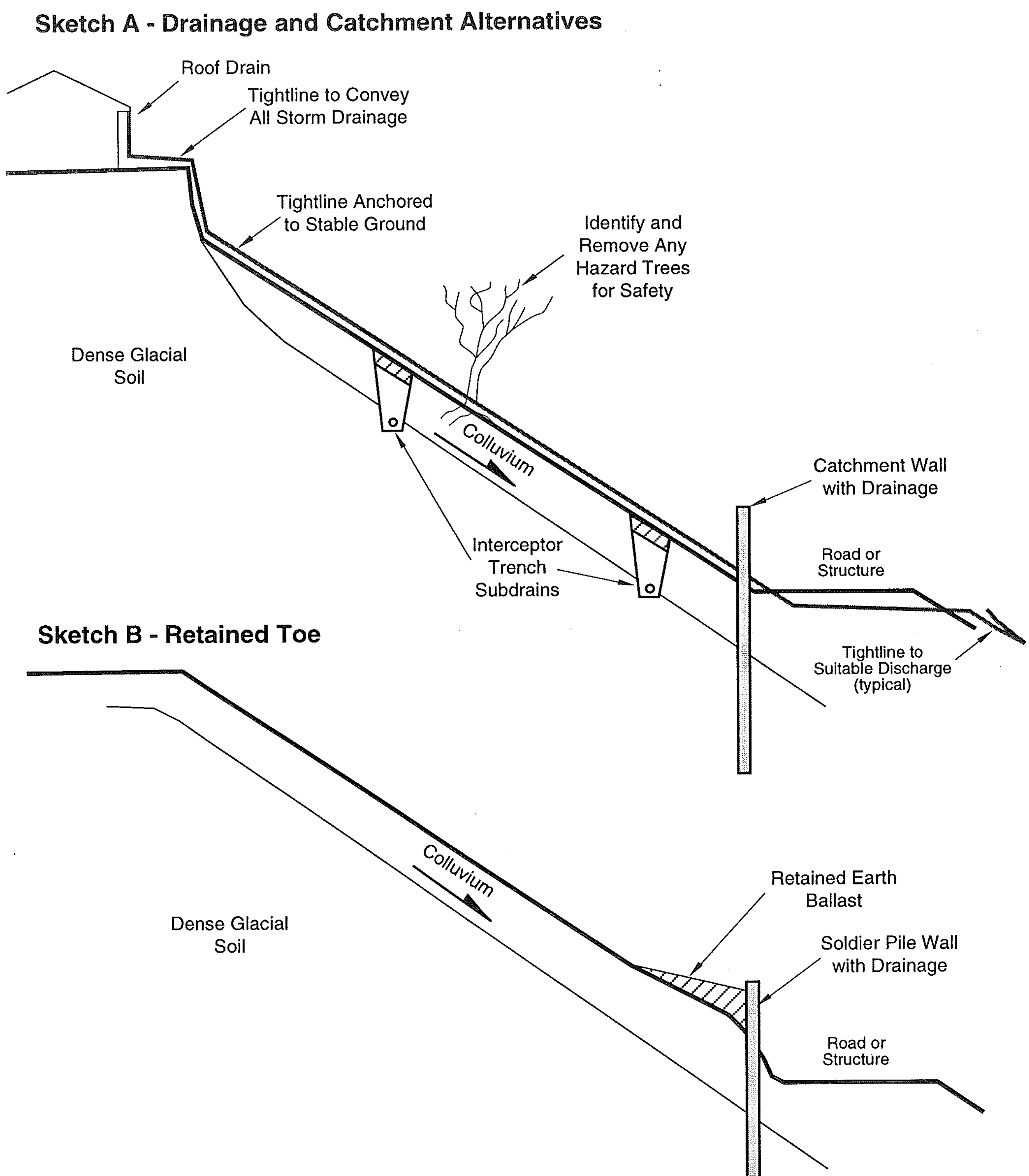
FIG. 2-3
Sheet 3 of 3



SKETCHES NOT TO SCALE

LEGEND

Potential Movement

NOTE:

The catchment wall (in situ wall) can also be designed to retain the potential sliding soil (colluvium).

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Seattle, Washington

TYPICAL SHALLOW COLLUVIAL LANDSLIDE STABILITY IMPROVEMENTS

July 1999 W-7992-01

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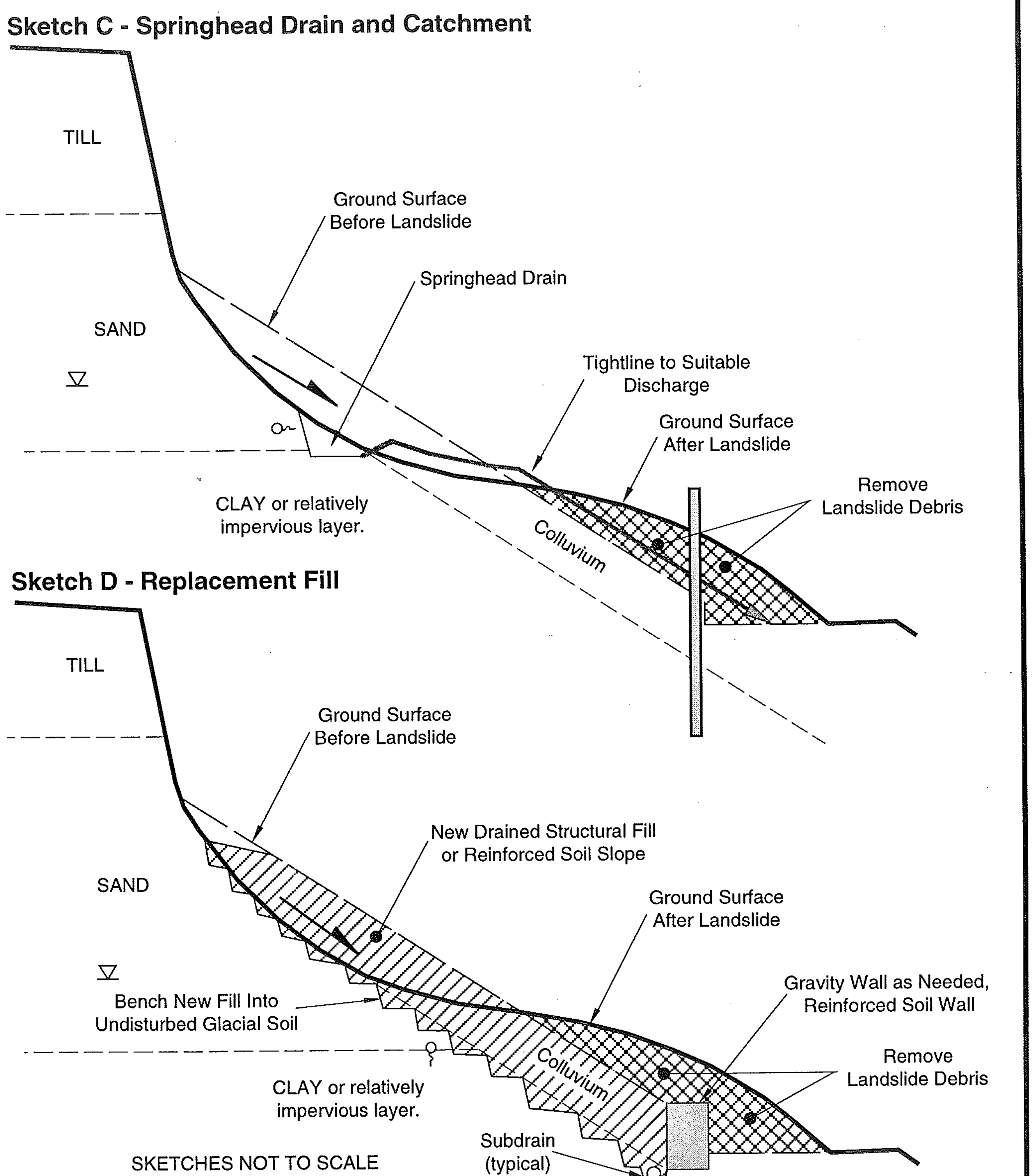
FIG. 2-4
Sheet 1 of 2

I/Documents/Mac/W Jobs/W-7900s/W-7992/01/W-7992-01 Fig. 2-4 Sheet 1 of 2



SKETCHES NOT TO SCALE

LEGEND

Potential Movement

Seepage

Groundwater Table

NOTE:

The catchment wall (in situ wall) can also be designed to retain the potential sliding soil (colluvium).

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Seattle, Washington

TYPICAL SHALLOW COLLUVIAL LANDSLIDE STABILITY IMPROVEMENTS

July 1999 | W-7992-01

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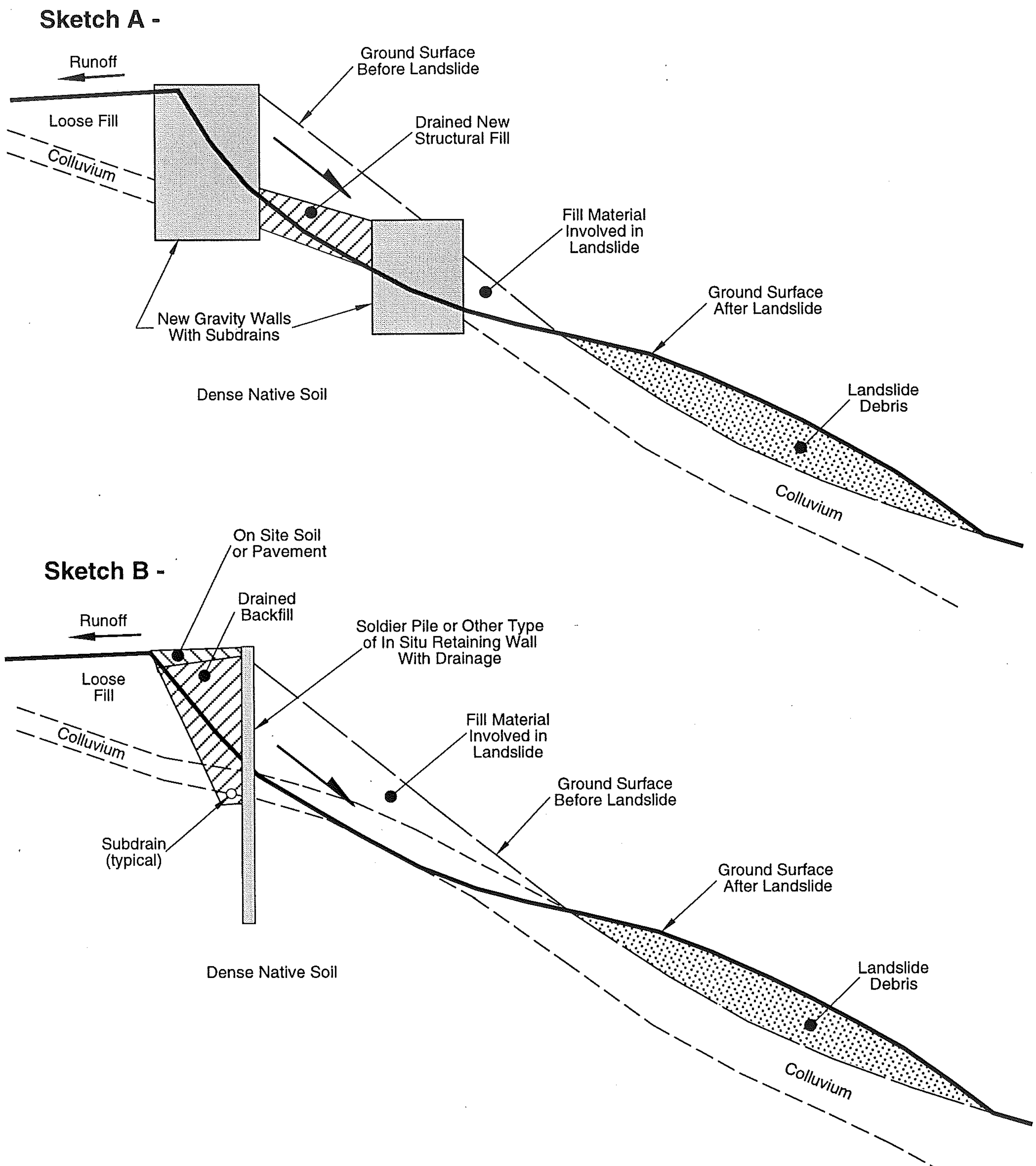
FIG. 2-4
Sheet 2 of 2



SKETCHES NOT TO SCALE

LEGEND

Landslide Movement

NOTE:

This type of landslide was categorized as shallow colluvial landslide in the database table and landslide maps.

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TYPICAL LANDSLIDE IN FILL MATERIAL STABILITY IMPROVEMENTS

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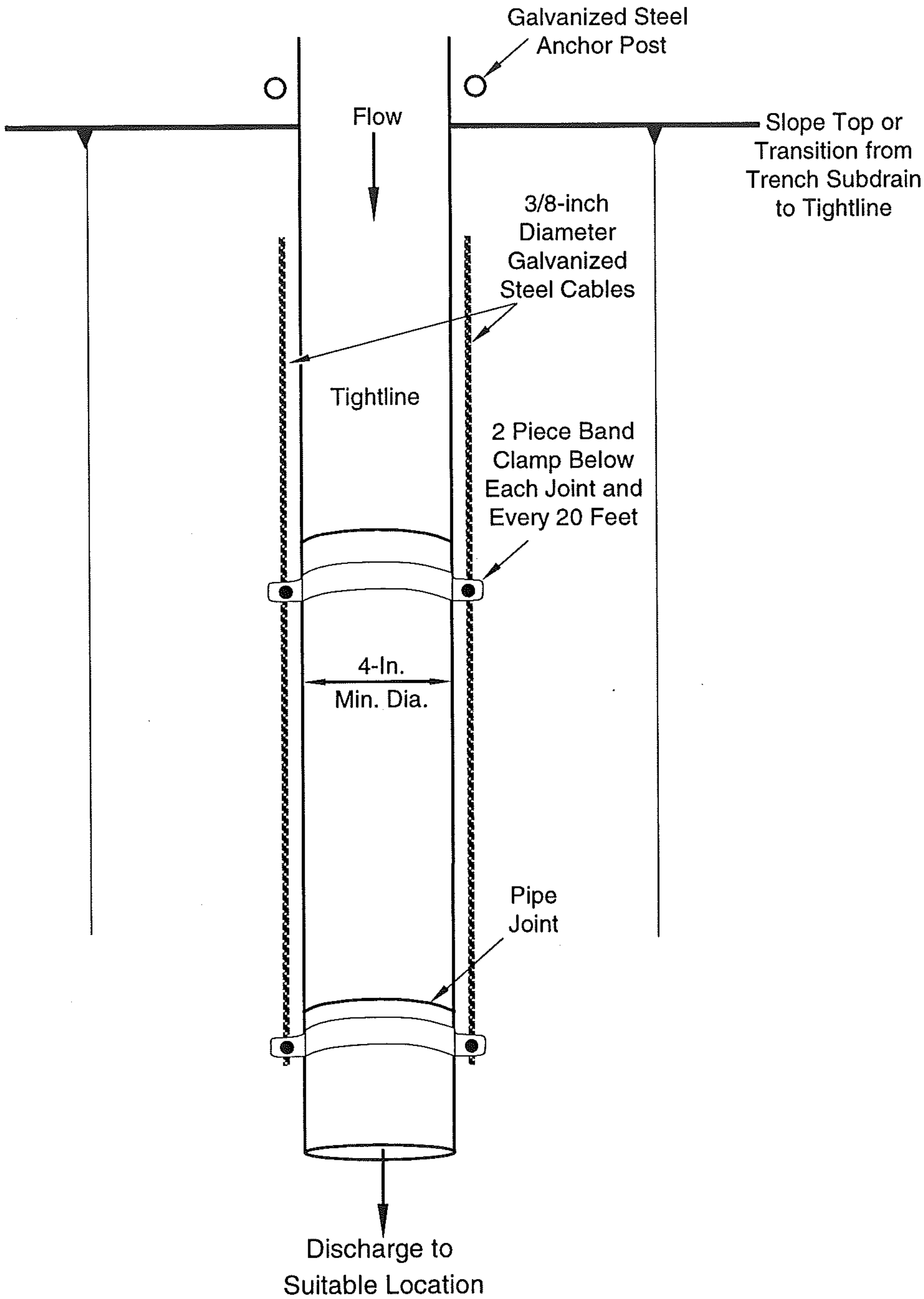
FIG. 2-5



TYPICAL PLAN VIEW

NOT TO SCALE

NOTES

1. This figure is not for construction. it should only be used for information pertaining to potential design concepts. Final design should be based on site-specific conditions and accomplished by a geotechnicl engineer licensed as a professional engineer.

2. Preferably, the tightline should discharge into a storm sewer.

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TYPICAL TIGHTLINE ANCHORING DETAIL

July 1999 W-7992-01

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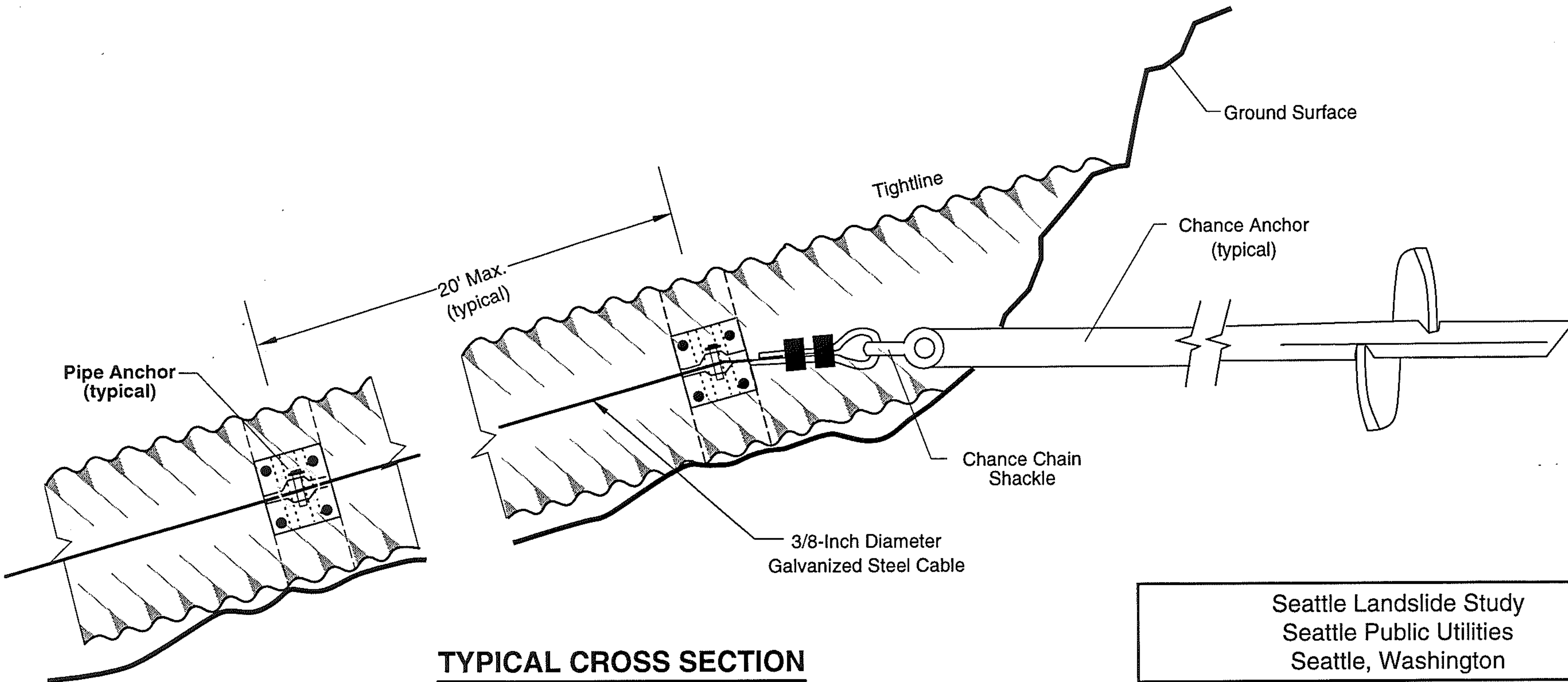
FIG. 2-6
Sheet 1 of 2

NOTES

1. This figure is not for construction. It should only be used for information pertaining to potential design concepts. Final design should be based on site-specific conditions and accomplished by a geotechnical engineer licensed as a professional engineer.
2. Place anchors below each pipe joint and not more than 20 feet apart.
3. Galvanize pipe anchor components in accordance with AASHTO M36.
4. Design ground anchors for the anticipated axial load, but not less than 5000 lb.
5. Contractor may propose an alternate anchor system for approval by the engineer.



TYPICAL CROSS SECTION

NOT TO SCALE

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Seattle Public Utilities
Seattle, Washington

TYPICAL TIGHTLINE ANCHORING DETAILS

July 1999 W-7992-01

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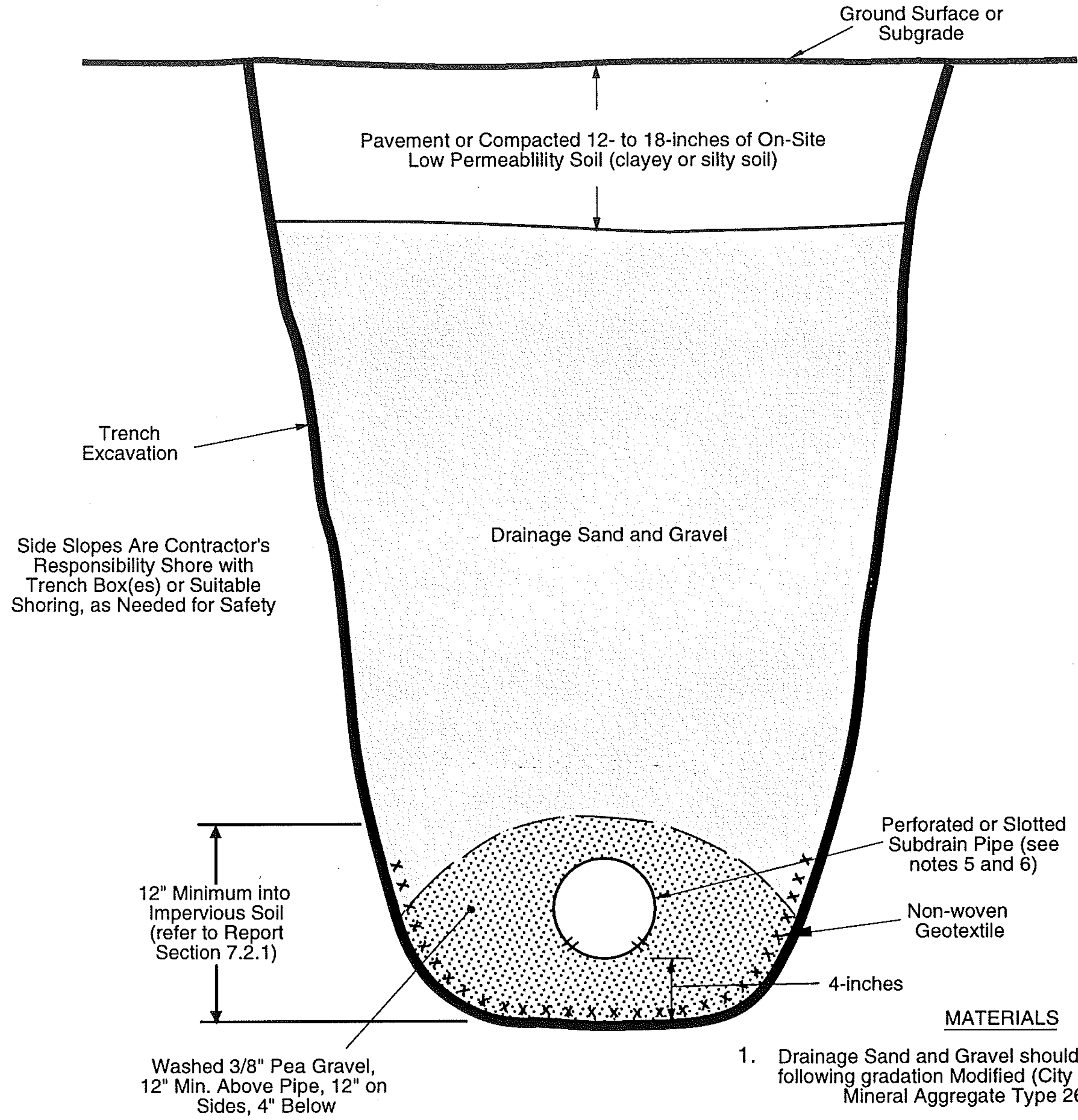
Geotechnical and Environmental Consultants

FIG. 2-6
Sheet 2 of 2

FIG. 2-6
Sheet 2 of 2



TYPICAL CROSS SECTION

NOT TO SCALE

NOTES

1. This figure is not for construction. It should only be used for information pertaining to potential design concepts. Final design should be based on site-specific conditions and accomplished by a geotechnical engineer licensed as a professional engineer.
2. Possible caving soil conditions may require that the subdrain pipe and backfill be placed concurrently with the trench excavation.
3. Extend pipe by means of a tightline to a suitable discharge point. Where subdrain pipe changes to a tightline, provide impervious dam (concrete or clay) so as to force all water into the tightline (see Figure 2-8).
4. Drain backfill should be compacted to a relatively dense condition (see Report Section 7.2.1).
5. Perforated or slotted subdrain pipe; tight joints; sloped to drain (6"/100' min. slope); provide clean-outs; min. diameter: 6 inches.
6. Perforated pipe holes (1/8-in. to 3/8-in. dia.) to be in lower half of pipe with lower quarter segment unperforated for water flow. Slotted pipe to have 1/8" maximum slot width.

MATERIALS

1. Drainage Sand and Gravel should meet the following gradation Modified (City of Seattle Mineral Aggregate Type 26):

Sieve Size	% Passing by Weight
1-inch	100
3/4-inch	85 to 95
1/4-inch	30 to 60
No. 8	20 to 50
No. 50	3 to 12
No. 200	0 to 1
(by wet sieving)	(non-plastic fines)

An alternative to drainage sand and gravel is City of Seattle Mineral Aggregate Type 6 (washed sand).

2. Washed 3/8" pea gravel to meet City of Seattle Mineral Aggregate Type 9.

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Seattle Public Utilities
Seattle, Washington

TYPICAL TRENCH SUBDRAIN INTERCEPTOR TRENCH AND FINGER DRAIN

July 1999 W-7992-01

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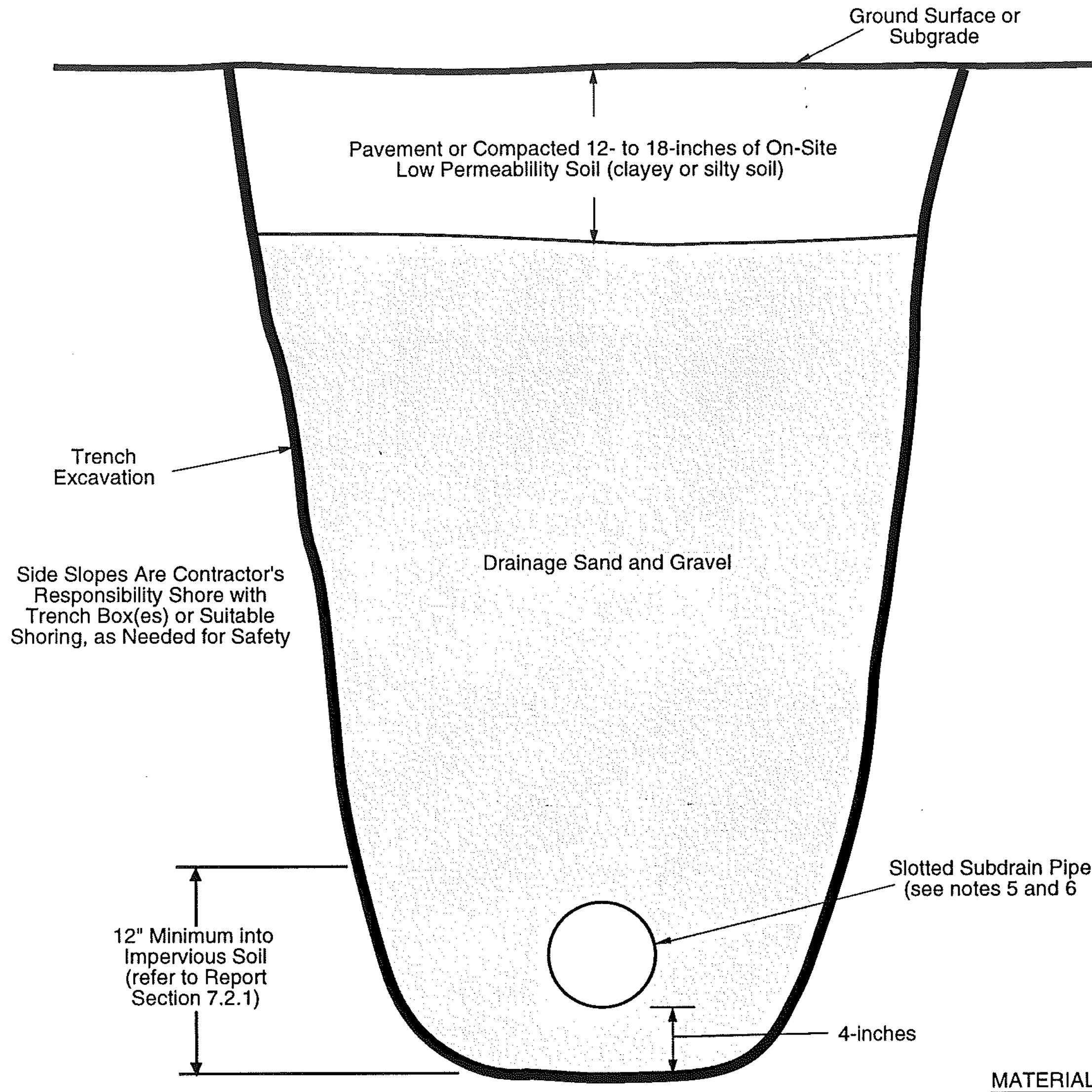
FIG. 2-7
Sheet 1 of 2



TYPICAL CROSS SECTION

NOT TO SCALE

MATERIALS

Drainage Sand and Gravel should meet the following gradation (Modified City of Seattle Mineral Aggregate Type 26):

Sieve Size	% Passing by Weight
1-inch	100
3/4-inch	85 to 95
1/4-inch	30 to 60
No. 8	20 to 50
No. 50	3 to 12
No. 200	0 to 1
(by wet sieving)	(non-plastic fines)

An alternative to drainage sand and gravel is a 50-50 mixture of washed pea gravel (Mineral Aggregate Type 9) and washed sand (mineral aggregate Type 6).

NOTES

1. This figure is not for construction. It should only be used for information pertaining to potential design concepts. Final design should be based on site-specific conditions and accomplished by a geotechnical engineer licensed as a professional engineer.
2. Possible caving soil conditions may require that the subdrain pipe and backfill be placed concurrently with the trench excavation.
3. Extend pipe by means of a tightline to a suitable discharge point. Where subdrain pipe changes to a tightline, provide impervious dam (concrete or clay) so as to force all water into the tightline (see Figure 2-8).
4. Drain backfill should be compacted to a relatively dense condition (see Report Section 7.2.1).
5. Slotted subdrain pipe; tight joints; sloped to drain (6"/100' min. slope); provide clean-outs; min. diameter: 6 inches.
6. Slotted pipe to have 1/8" maximum slot width.

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Seattle Public Utilities
Seattle, Washington

TYPICAL TRENCH SUBDRAIN INTERCEPTOR TRENCH AND FINGER DRAIN

July 1999 W-7992-01

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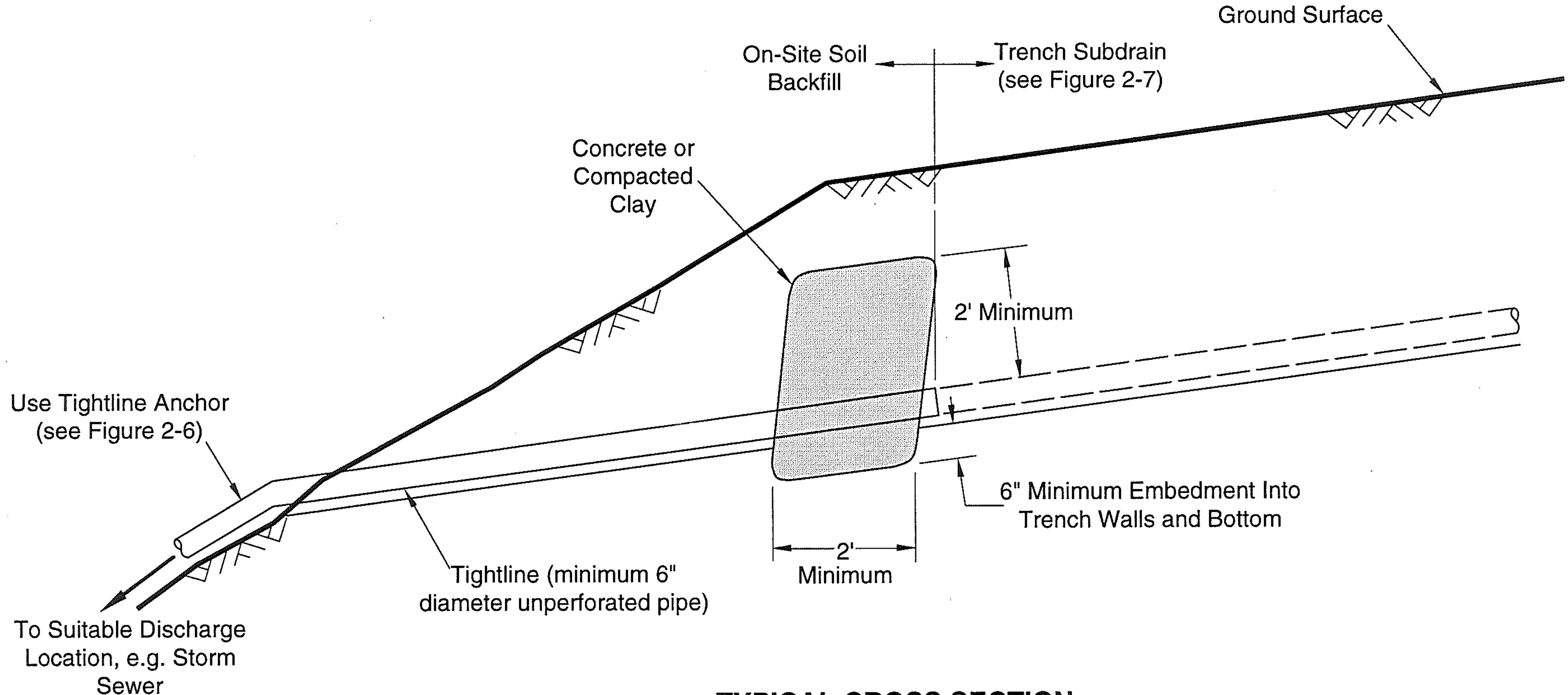
FIG. 2-7
Sheet 2 of 2



TYPICAL CROSS SECTION

NOT TO SCALE

Note:

This figure is not for construction. It should only be used for information pertaining to potential design concepts. Final design should be based on site-specific conditions and accomplished by a geotechnical engineer licensed as a professional engineer.

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Seattle, Washington

TYPICAL DRAINAGE DAM DETAIL

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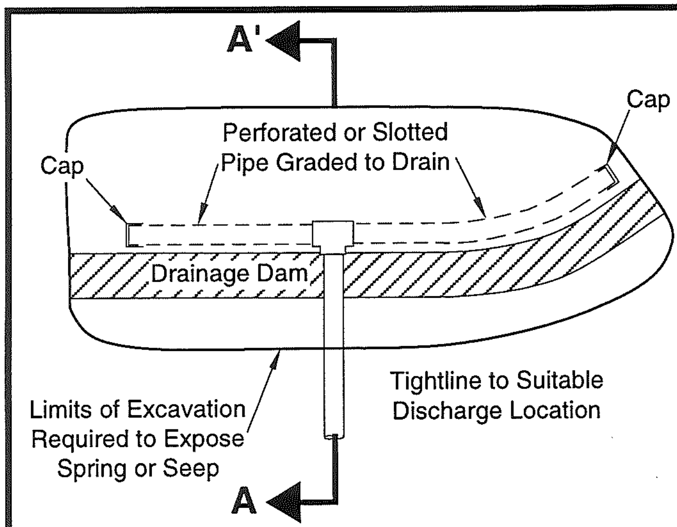
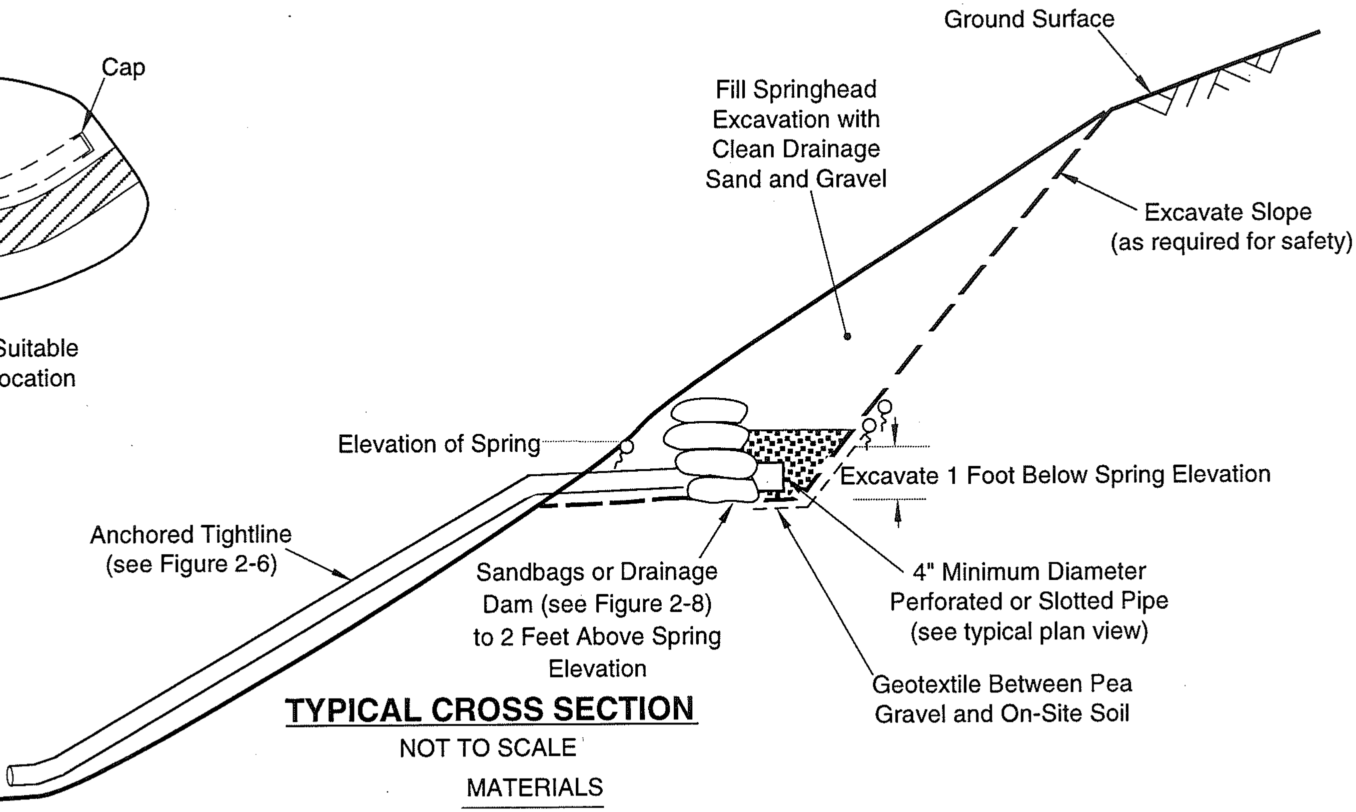
FIG. 2-8

FIG. 2-8



TYPICAL PLAN VIEW

NOT TO SCALE



TYPICAL CROSS SECTION

NOT TO SCALE

MATERIALS

1. Drainage Sand and Gravel should meet the following gradation (Modified City of Seattle Mineral Aggregate Type 26):

Sieve Size	% Passing by Weight
1-inch	100
3/4-inch	85 to 95
1/4-inch	30 to 60
No. 8	20 to 50
No. 50	3 to 12
No. 200	0 to 1
(by wet sieving)	(non-plastic fines)

An alternative drainage sand and gravel is a 50-50 mixture of washed pea gravel and washed concrete sand.

2. Washed 3/8" Pea Gravel to Meet City of Seattle Mineral Aggregate Type 9.

NOTES

1. This figure is not for construction. It should only be used for information pertaining to potential design concepts. Final design should be based on site-specific conditions and accomplished by a geotechnical engineer licensed as a professional engineer.

2. Perforated pipe holes (1/8-inch to 3/8-inch diameter) to be in lower half of pipe with lower quarter segment unperforated for water flow. Slotted pipe to have 1/8" maximum slot width.

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TYPICAL SPRINGHEAD DRAIN DETAIL

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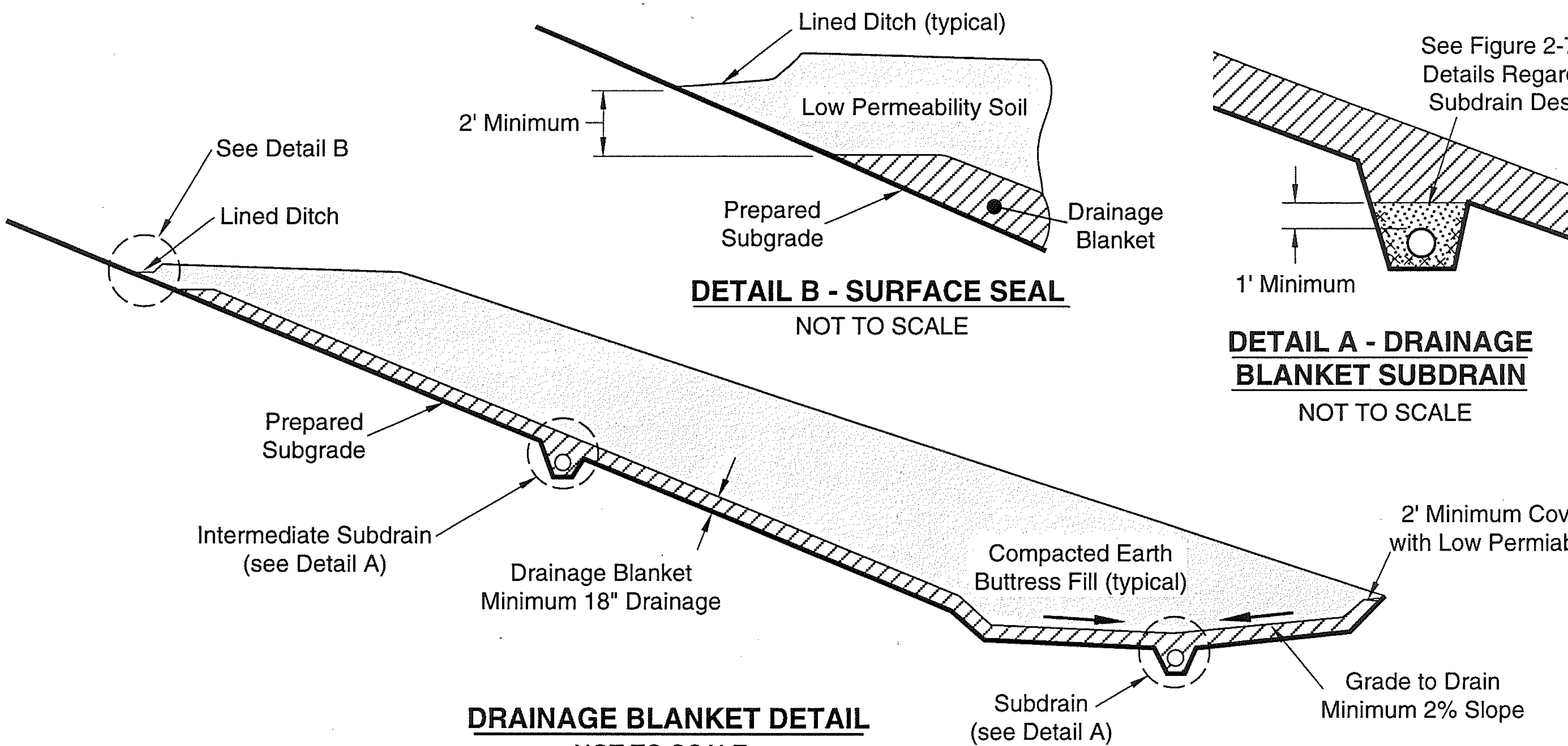
FIG. 2-9

FIG. 2-9



DETAIL B - SURFACE SEAL
NOT TO SCALE

DETAIL A - DRAINAGE BLANKET SUBDRAIN
NOT TO SCALE

DRAINAGE BLANKET DETAIL
NOT TO SCALE

MATERIALS

Drainage Sand and Gravel should meet the following gradation (Modified City of Seattle Mineral Aggregate Type 26):

Sieve Size	% Passing by Weight
1-inch	100
3/4-inch	85 to 95
1/4-inch	30 to 60
No. 8	20 to 50
No. 50	3 to 12
No. 200	0 to 1
(by wet sieving)	(non-plastic fines)

An alternative drainage sand and gravel is a 50-50 mixture of washed pea gravel and washed concrete sand.

Note:

This figure is not for construction. It should only be used for information pertaining to potential design concepts. Final design should be based on site-specific conditions and accomplished by a geotechnical engineer licensed as a professional engineer.

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TYPICAL DRAINAGE BLANKET DETAIL

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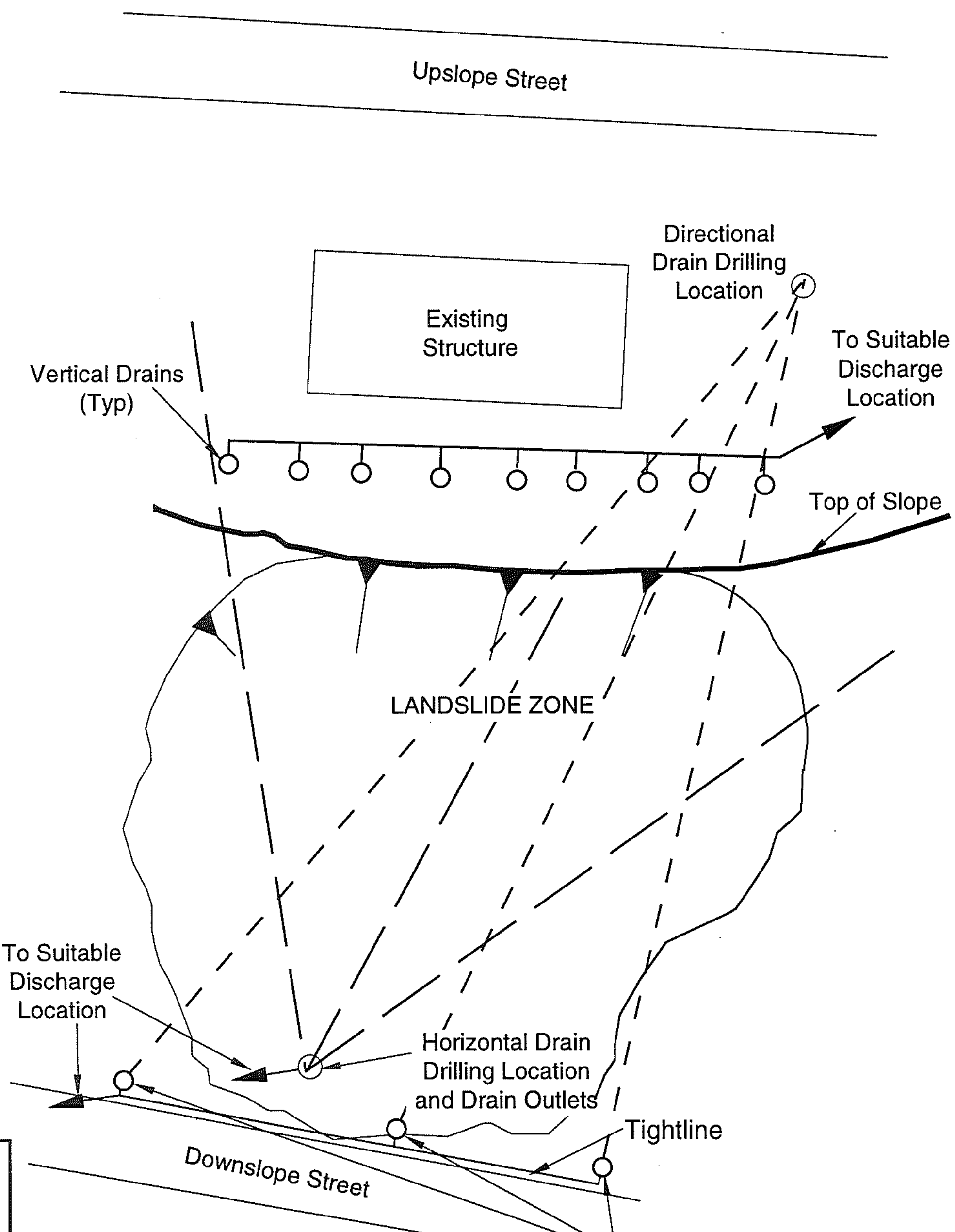
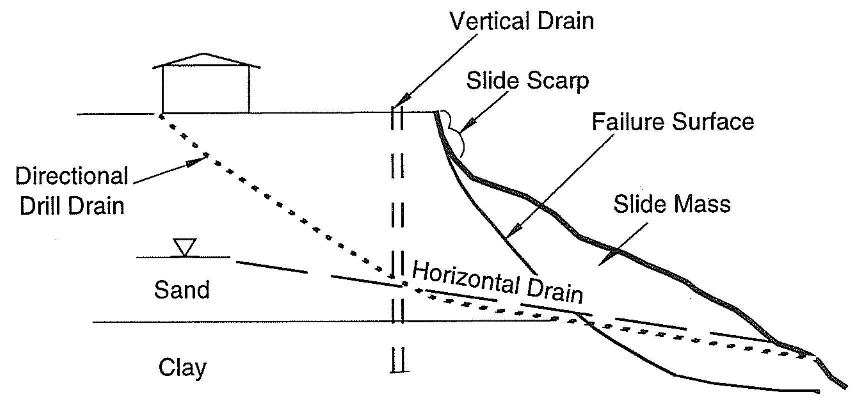
FIG. 2-10

FIG. 2-10



TYPICAL PLAN VIEW
NOT TO SCALE



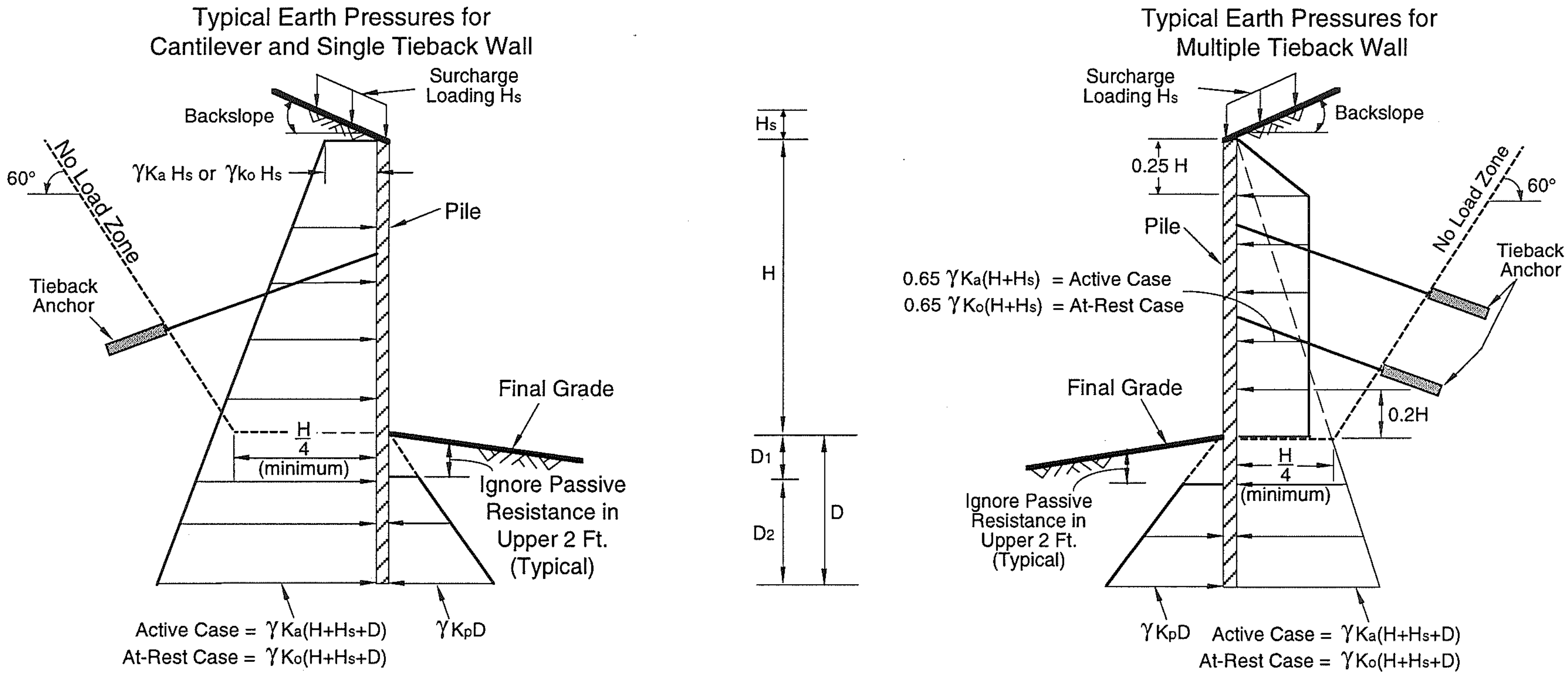
TYPICAL CROSS SECTION
NOT TO SCALE

Note:

This figure is not for construction. It should only be used for information pertaining to potential design concepts. Final design should be based on site-specific conditions and accomplished by a geotechnical engineer licensed as a professional engineer.

Seattle Landslide Study Seattle Public Utilities Seattle, Washington
TYPICAL HORIZONTAL, DIRECTIONAL AND VERTICAL DRAIN SKETCH July 1999 W-7992-01
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants — **FIG. 2-11**

FIG. 2-11



NOTES

1. This figure is not for construction. It should only be used for information pertaining to potential design concepts. Final design should be based on site-specific conditions and accomplished by a geotechnical engineer licensed as a professional engineer.
2. Wall Embedment (D) should satisfy both force and moment equilibrium. The minimum embedment should be based on the site conditions.
3. The pressure diagrams shown are based on uniform soil conditions. Typically, piles are embedded in dense soil. For this case, the active pressure can be reduced accordingly in the dense soils.
4. These typical pressure diagrams are based on a continuous wall system. If soldier piles with lagging are used, apply active pressure over the width of the soldier piles below the lagging and apply passive resistances over twice the width of the piles or the spacing of the piles, whichever is smaller.
5. The passive pressure should include a factor of safety of 1.5 to reduce wall deflection.
6. Free drainage assumed behind the wall.
7. For lagging design, the above design pressures can be reduced for soil arching.
8. Determine allowable vertical pile capacity for piles backfilled with lean concrete using: Skin Friction = f_s; End Bearing = q_p
9. The no load zone should also include any soil above any potential slide surface.

LEGEND

H = Excavation Height (Ft.)
H_s = Equivalent Surcharge Height (Ft.)
D, D_1, D_2 = Embedment Depths (Ft.)
K_a = Active Earth Pressure Coefficient
K_o = At-Rest Pressure Coefficient
K_p = Passive Earth Pressure Coefficient
γ = Unit Weight of Soil

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TYPICAL SOLDIER PILE WALL DESIGN CRITERIA

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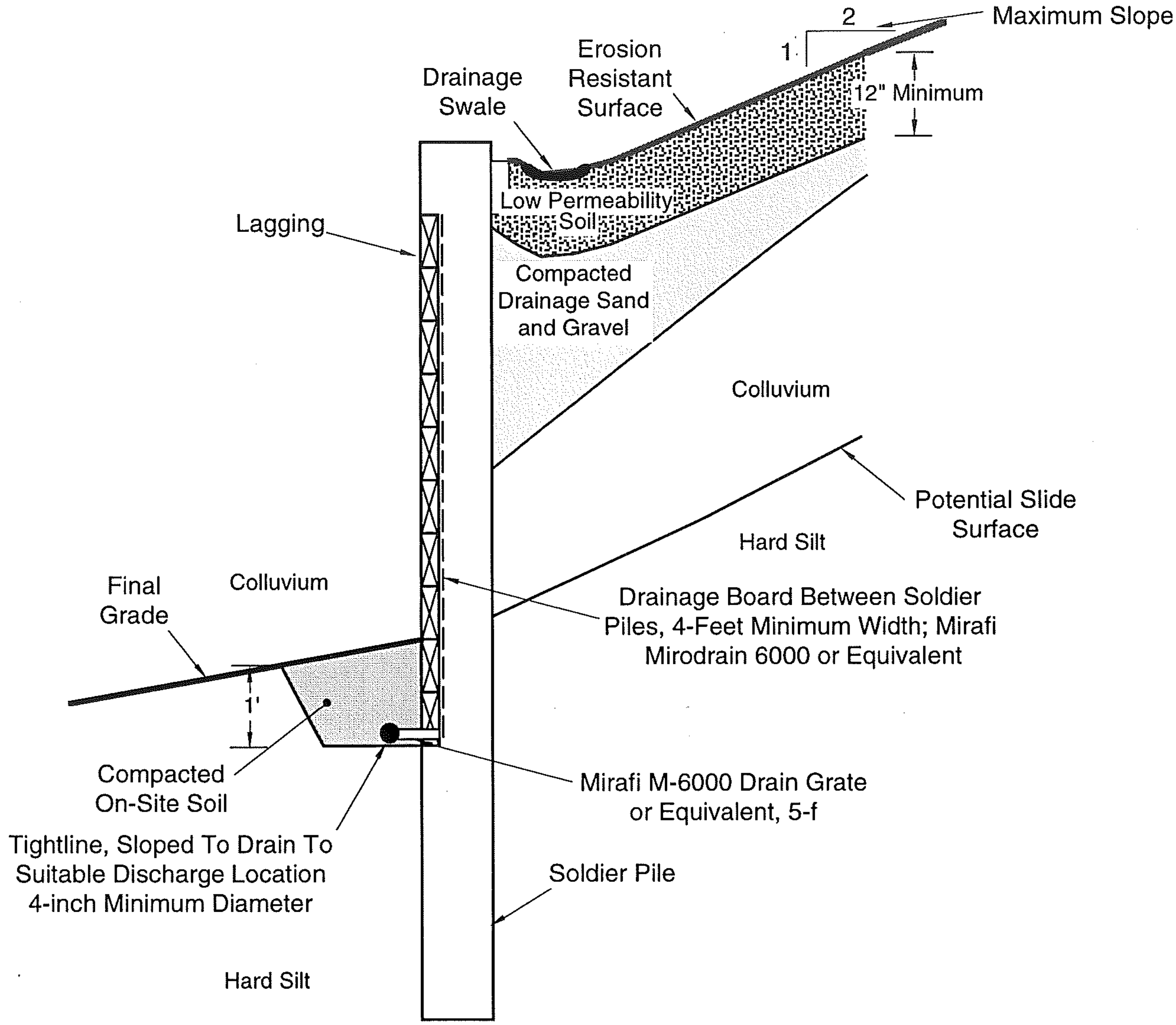
FIG. 2-12
Sheet 1 of 3

FIG. 2-12
Sheet 1 of 3



NOT TO SCALE

NOTES

1. This figure is not for construction. It should only be used for information pertaining to potential design concepts. Final design should be based on site-specific conditions and accomplished by a geotechnical engineer licensed as a professional engineer.
2. Refer to Fig. 2-12, Sheet 3 of 3, for applicable notes and materials.

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TYPICAL SOLDIER PILE WALL
BACKFILL AND DRAINAGE

July 1999 W-7992-01

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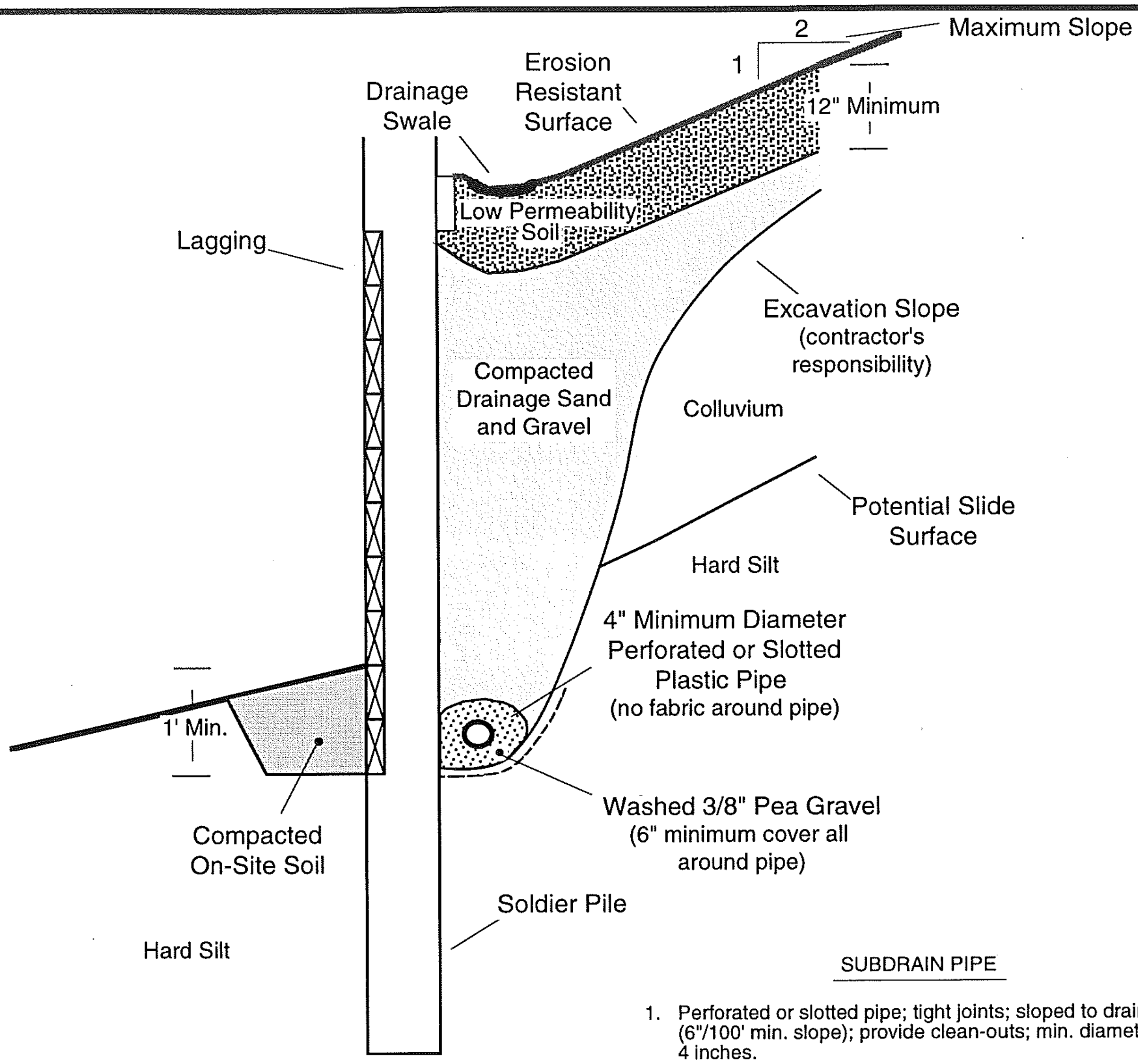
FIG. 2-12
Sheet 2 of 3



NOT TO SCALE

SUBDRAIN PIPE

1. Perforated or slotted pipe; tight joints; sloped to drain (6"/100' min. slope); provide clean-outs; min. diameter: 4 inches.
2. Perforated pipe holes (1/8-in. to 3/8-in. dia.) to be in lower half of pipe with lower quarter segment unperforated for water flow.
3. Slotted pipe to have 1/8-in. max. width slots.

NOTES

1. This figure is not for construction. It should only be used for information pertaining to potential design concepts. Final design should be based on site-specific conditions and accomplished by a geotechnical engineer licensed as a professional engineer.
2. Compact drainage sand and gravel behind wall to at least 92% of Modified Proctor maximum dry density (ASTM: D1557); where settlement is to be minimized compact to at least 95% of Modified.

MATERIALS

1. Drainage Sand and Gravel should meet the following gradation (Modified City of Seattle Mineral Aggregate Type 26):

Sieve Size	% Passing by Weight
1-inch	100
3/4-inch	85 to 95
1/4-inch	30 to 60
No. 8	20 to 50
No. 50	3 to 12
No. 200	0 to 1
(by wet sieving)	(non-plastic fines)

An alternative to drainage sand and gravel is a 50-50 mixture of washed pea gravel and washed concrete sand.

2. Washed 3/8" Pea Gravel to Meet City of Seattle Mineral Aggregate Type 9.

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TYPICAL SOLDIER PILE WALL BACKFILL AND DRAINAGE

July 1999 W-7992-01

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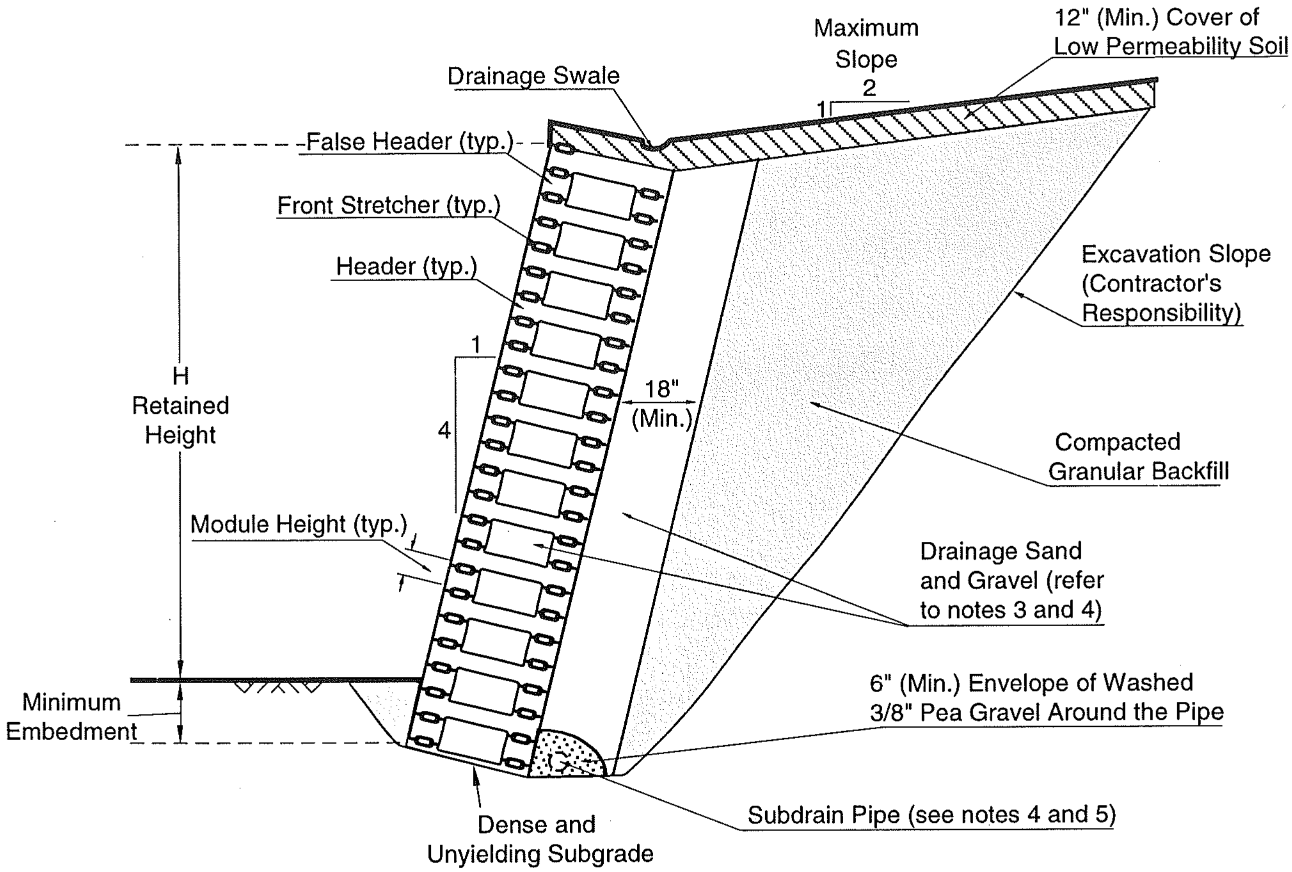
FIG. 2-12
Sheet 3 of 3



NOT TO SCALE

NOTES

1. This figure is not for construction. It should only be used for information pertaining to potential design concepts. Final design should be based on site-specific conditions and accomplished by a geotechnical engineer licensed as a professional engineer.

2. Crib retaining wall to be designed by a licensed structural engineer and reviewed by the geotechnical engineer.

3. Drainage sand and gravel and granular backfill should be placed into layers not exceeding 6 inches loose thickness and compacted to at least 95 percent of its Modified Proctor maximum density (ASTM: D 1557), except within the cribs where it should be compacted to at least 92 percent. Crib units, drainage sand and gravel, and granular backfill should be built up together.

4. An alternative would be to use excavated granular soil as backfill inside cribs.

5. Perforated or slotted pipe; tight joints; sloped to drain (6"/100' min. slope); provide clean-outs; min. diameter: 4 inches.

6. Perforated pipe holes (1/8-in. to 3/8-in. dia.) to be in lower half of pipe with lower quarter segment unperforated for water flow.

MATERIALS

1. Drainage Sand and Gravel should meet the following gradation (Modified City of Seattle Mineral Aggregate Type 26):

Sieve Size	% Passing by Weight
1-inch	100
3/4-inch	85 to 95
1/4-inch	30 to 60
No. 8	20 to 50
No. 50	3 to 12
No. 200	0 to 1
(by wet sieving)	(non-plastic fines)

An alternative to drainage sand and gravel is a 50-50 mixture of washed pea gravel and washed sand (Mineral Aggregate Type 6).

2. Washed 3/8" Pea Gravel to Meet City of Seattle Mineral Aggregate Type 9.

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TYPICAL CRIB WALL WITH SUBDRAINAGE AND BACKFILLING

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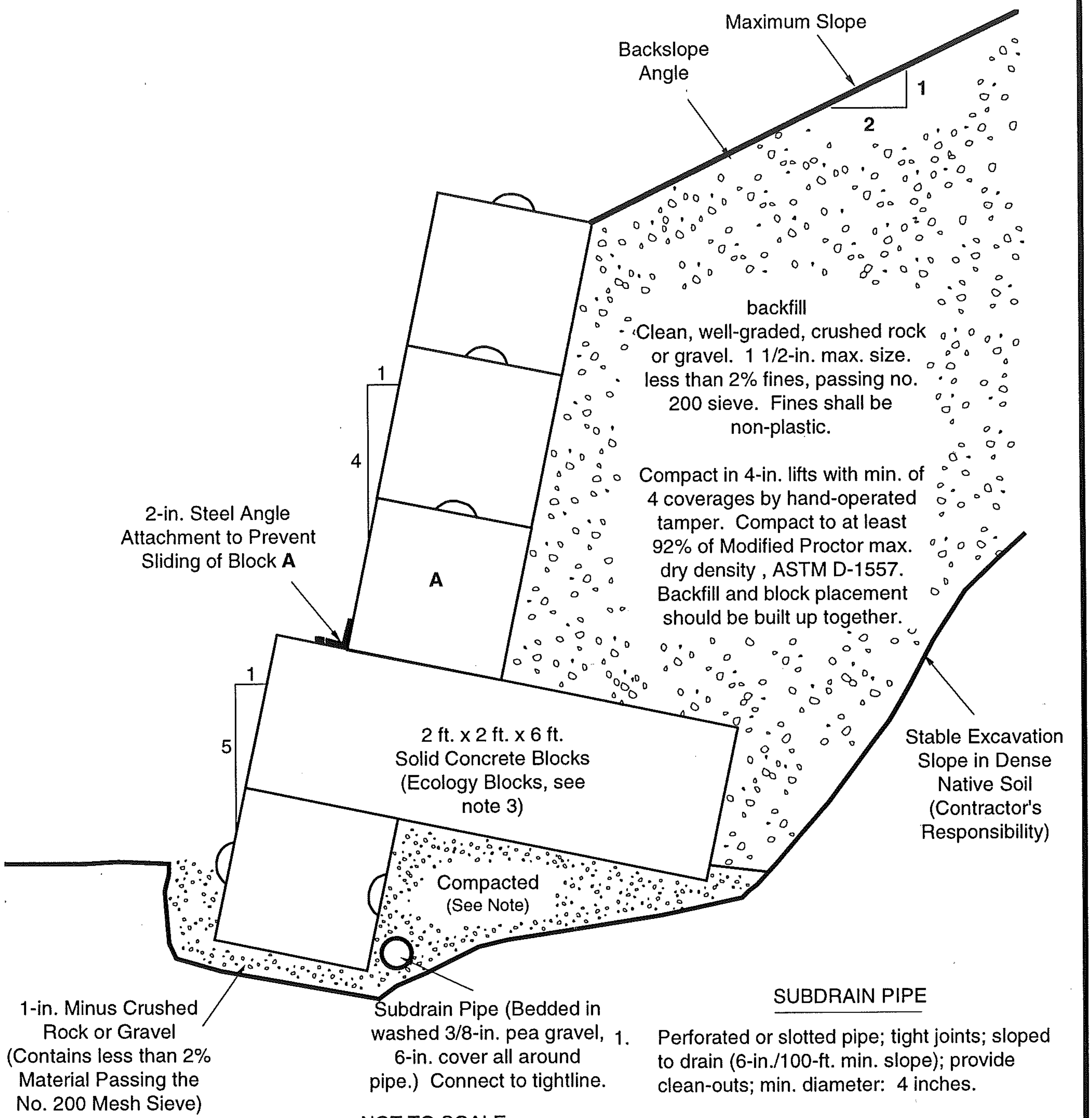
FIG. 2-13



NOT TO SCALE

NOTE

1. This figure is not for construction. It should only be used for information pertaining to potential design concepts. Final design should be based on site-specific conditions and accomplished by a geotechnical engineer licensed as a professional engineer.

2. All loose to medium dense soil at block foundation should be overexcavated down to dense soil and replaced with compacted backfill as described above. The excavation shall be kept free of water. The prepared block foundation shall be evaluated by a geotechnical engineer prior to placement of blocks.

3. Other block sizes and types are available.

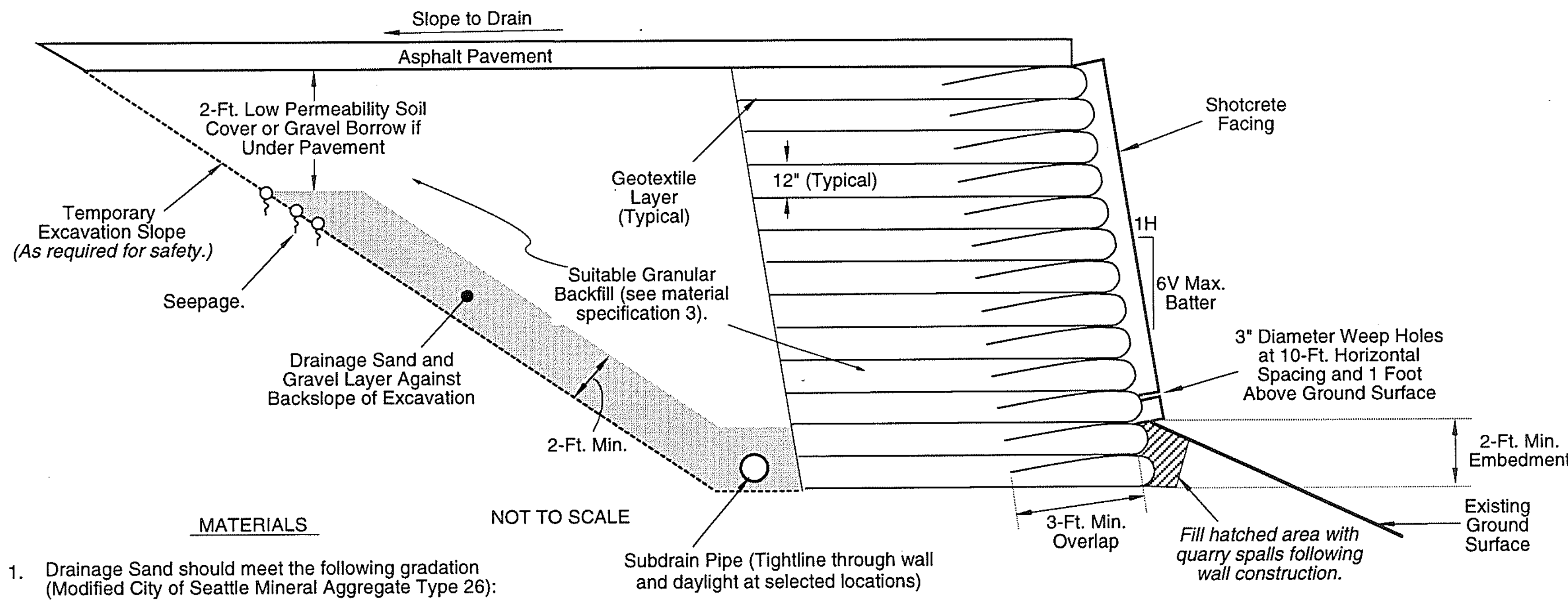
SUBDRAIN PIPE

1. Perforated or slotted pipe; tight joints; sloped to drain (6-in./100-ft. min. slope); provide clean-outs; min. diameter: 4 inches.

2. Perforated pipe holes (1/8-in. to 3/8-in. dia.) to be in lower half of pipe with lower quarter segment unperforated for water flow.

3. Slotted pipe to have 1/8-in. max. width slots.

Seattle Landslide Study Seattle Public Utilities Seattle, Washington	
TYPICAL SECTION ECOLOGY BLOCK WALL	
July 1999	W-7992-01
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**FIG. 2-14**



NOT TO SCALE

MATERIALS

1. Drainage Sand should meet the following gradation (Modified City of Seattle Mineral Aggregate Type 26):

Sieve Size	% Passing by Weight
1-inch	100
3/4-inch	85 to 95
1/4-inch	30 to 60
No. 8	20 to 50
No. 50	3 to 12
No. 200	0 to 1
(by wet sieving)	(non-plastic fines)

An alternative to drainage sand and gravel is a 50-50 mixture of washed pea gravel and washed sand (Mineral Aggregate Type 6).

2. Washed 3/8" Pea Gravel to Meet City of Seattle Mineral Aggregate Type 9.

3. Granular backfill to consist of suitable on-site soil or imported, clean, well-graded sand and gravel or crushed rock; material must meet the following gradation criteria (City of Seattle Mineral Aggregate Type No. 17);

Sieve Size	% Passing by Weight
3-inch	95-100
1/4-inch	25 -75
No. 200	0 to 5
(by wet sieving)	(non-plastic fines)

4. SUBDRAIN PIPE

a Perforated or slotted pipe; tight joints; sloped to drain (6"/100' min. slope); provide clean-outs; min. diameter: 4 inches.

b Perforated pipe holes (1/8-in. to 3/8-in. dia.) to be in lower half of pipe with lower quarter segment unperforated for water flow.

c Slotted pipe to have 1/8-in. max. width slots.

d Surround Subdrain Pipe with a minimum of 6 inches of washed 3/8" pea gravel.

NOTES

1. This figure is not for construction. It should only be used for information pertaining to potential design concepts. Final design should be based on site-specific conditions and accomplished by a geotechnical engineer licensed as a professional engineer.

2. Compact backfill in 6-in. maximum loose lifts to at least 95% of Modified Proctor maximum dry density (ASTM D-1557). Backfill, drainage sand and gravel, and geotextile placement should be built up together.

3. Keep the excavation free of water. A geotechnical engineer should evaluate the prepared subgrade before placing fill.

4. If soft or loose materials are present in the subgrade, they should be removed and replaced with compacted granular backfill.

5. Wall system to be designed by a professional engineer.

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TYPICAL GEOTEXTILE SOIL WALL SECTION

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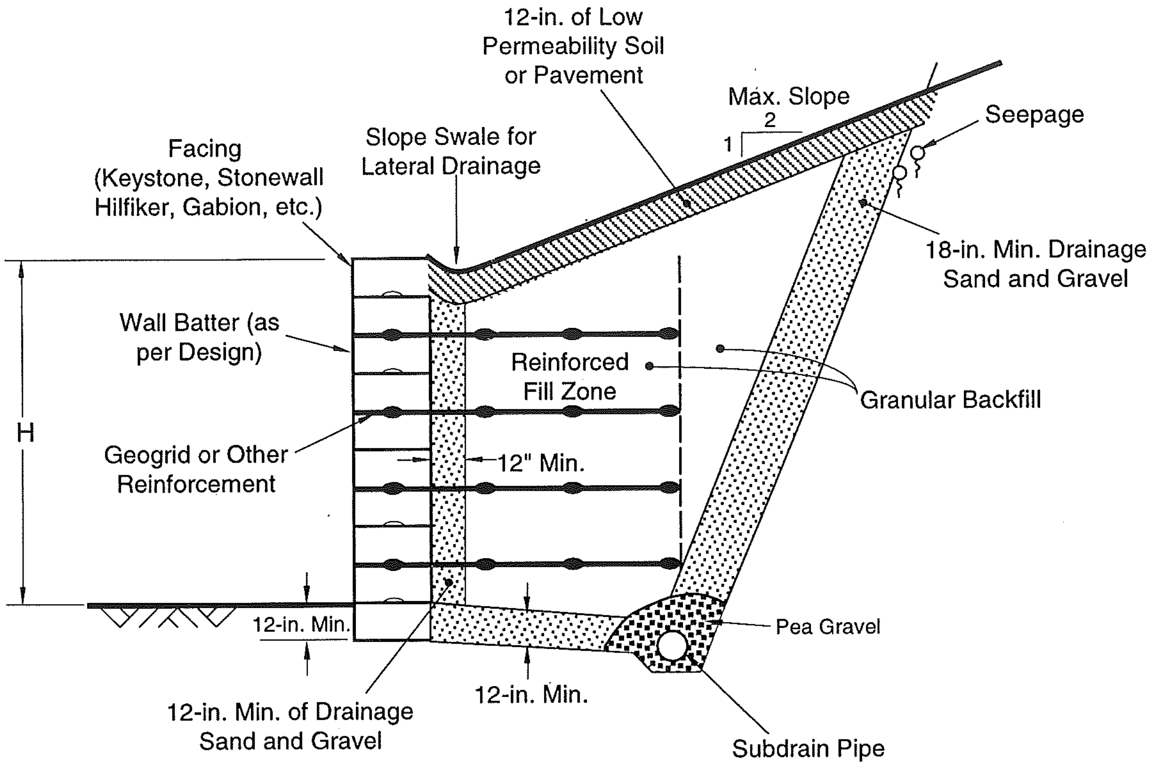
FIG. 2-15

FIG. 2-15



Not to Scale

MATERIALS

1. Drainage Sand And Gravel Should Meet The Following Gradation (Modified City Of Seattle Mineral Aggregate Type 26):

Sieve Size	% Passing by Weight
1-inch	100
3/4-inch	85 to 95
1/4-inch	30 to 60
No. 8	20 to 50
No. 50	3 to 12
No. 200	0 to 1
(by wet sieving)	(non-plastic fines)

An alternative to drainage sand and gravel is a 50-50 mixture of washed pea gravel and washed sand (Mineral Aggregate Type 6.)

GRANULAR BACKFILL

2. Compact granular backfill to consist of suitable on-site or imported clean, well-graded sand and gravel or crushed rock; either material must meet the following gradation criteria (City of Seattle Type No. 17);

Sieve Size	% Passing by Weight
3-inch	95-100
1/4-inch	25 -75
No. 200	0 to 5
(by wet sieving)	(non-plastic fines)

3. SUBDRAIN PIPE

a. Perforated or slotted pipe; tight joints; sloped to drain (6"/100' min. slope); provide clean-outs; min. diameter: 4 inches.

b. Perforated pipe holes (1/8-in. to 3/8-in. dia.) to be in lower half of pipe with lower quarter segment unperforated for water flow.

c. Slotted pipe to have 1/8-in. max. width slots.

NOTES

1. This figure is not for construction. It should only be used for information pertaining to potential design concepts. Final design should be based on site-specific conditions and accomplished by a geotechnical engineer licensed as a professional engineer.

2. Compacted backfill in 6" maximum loose lifts to at least 95% of Modified Proctor maximum dry density (ASTM D-1557).

3. Wall system to be designed by professional engineer.

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TYPICAL GEOGRID REINFORCED SOIL WALL SECTION

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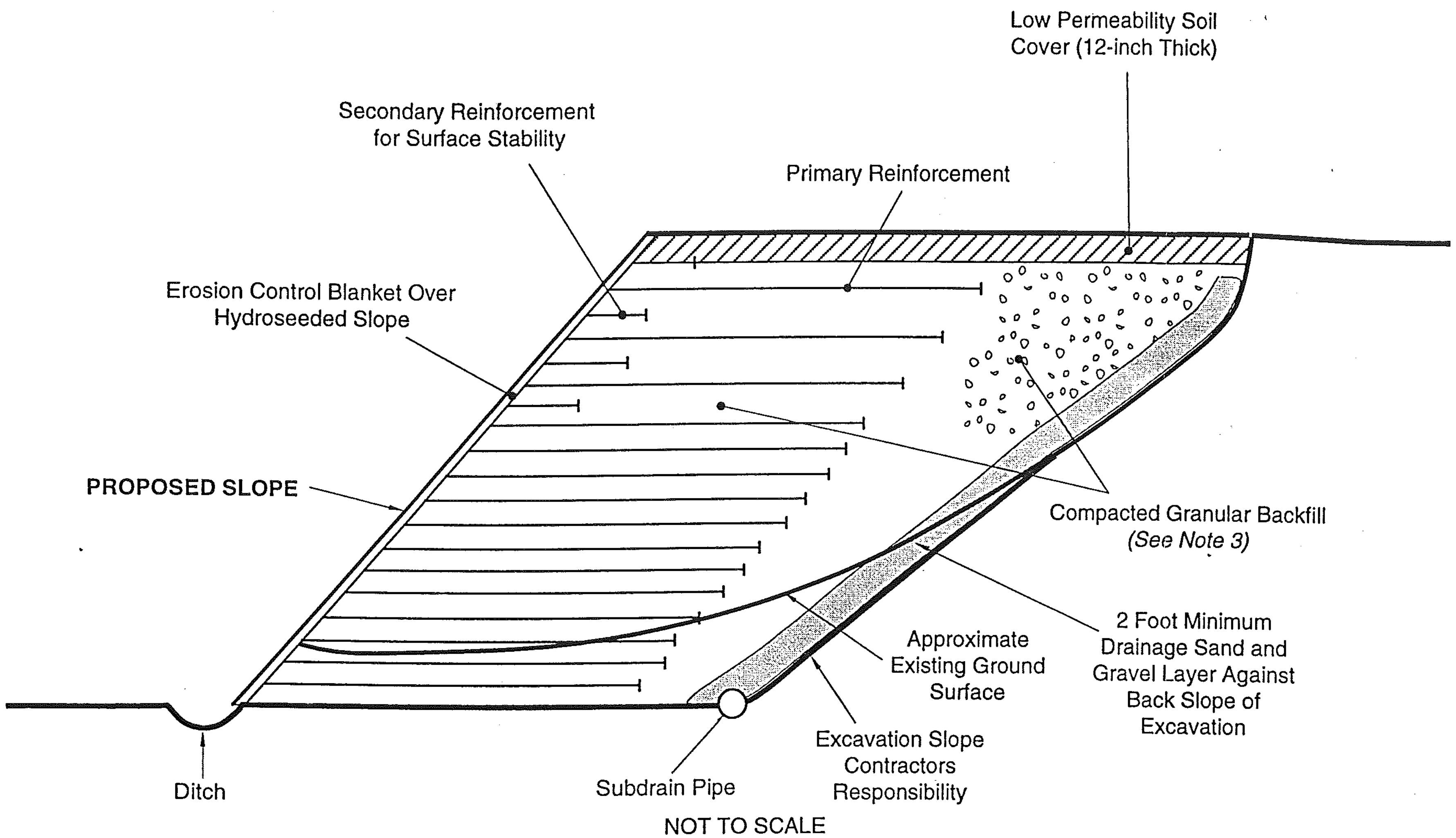
FIG. 2-16



MATERIALS

1. Drainage Sand and Gravel should meet the following gradation (Modified City of Seattle Mineral Aggregate Type 26):

Sieve Size	% Passing by Weight
1-inch	100
3/4-inch	85 to 95
1/4-inch	30 to 60
No. 8	20 to 50
No. 50	3 to 12
No. 200	0 to 1
(by wet sieving)	(non-plastic fines)

An alternative to drainage sand and gravel is a 50-50 mixture of washed pea gravel and washed sand (Mineral Aggregate Type 6).

2. Washed Pea Gravel to Meet City of Seattle Mineral Aggregate Type 9.

3. Granular backfill to consist of suitable on-site or imported clean, well-graded sand and gravel or crushed rock; either material must meet the following gradation criteria (City of Seattle Mineral Aggregate Type No. 17);

Sieve Size	% Passing by Weight
3-inch	95-100
1/4-inch	25 -75
No. 200	0 to 5
(by wet sieving)	(non-plastic fines)

4. SUBDRAIN PIPE

a. Perforated or slotted pipe; tight joints; sloped to drain (6"/100' min. slope); provide clean-outs; min. diameter: 4 inches.

b. Perforated pipe holes (1/8-in. to 3/8-in. dia.) to be in lower half of pipe with lower quarter segment unperforated for water flow.

c. Slotted pipe to have 1/8-in. max. width slots.

d. Envelope subdrain with 6" minimum of washed pea gravel. Place suitable filter fabric (non-woven geotextile) between pea gravel and on-site soils.)

NOTES

1. This figure is not for construction. It should only be used for information pertaining to potential design concepts. Final design should be based on site-specific conditions and accomplished by a geotechnical engineer licensed as a professional engineer.

2. The following construction sequence is typical.

(a) Mobilize and prepare the site including placing erosion control measures, excavating the landslide debris in the proposed reinforced zone and hauling this material temporarily off-site, and recompacting the subgrade.

(b) Construct slope including geotextile placement, place backfill and compact.

(c) Place 12 inches of low permeability soil cover at the top of the reconstructed bank.

(d) Hydroseed the slope face and the top of the bank.

(e) Place geosynthetic erosion control blanket on the slope face.

(f) Demobilize, including removing erosion control measures and cleanup.

3. Compact backfill in 6" maximum loose lifts to at least 95% of Modified Proctor maximum dry density (ASTM D-1557).

4. Keep the excavation free of water. A geotechnical engineer should evaluate the prepared subgrade before placing fill.

5. If loose or soft materials are present in the subgrade, they should be removed and replaced with compacted granular backfill.

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TYPICAL SECTION
GEOTEXTILE-REINFORCED SLOPE

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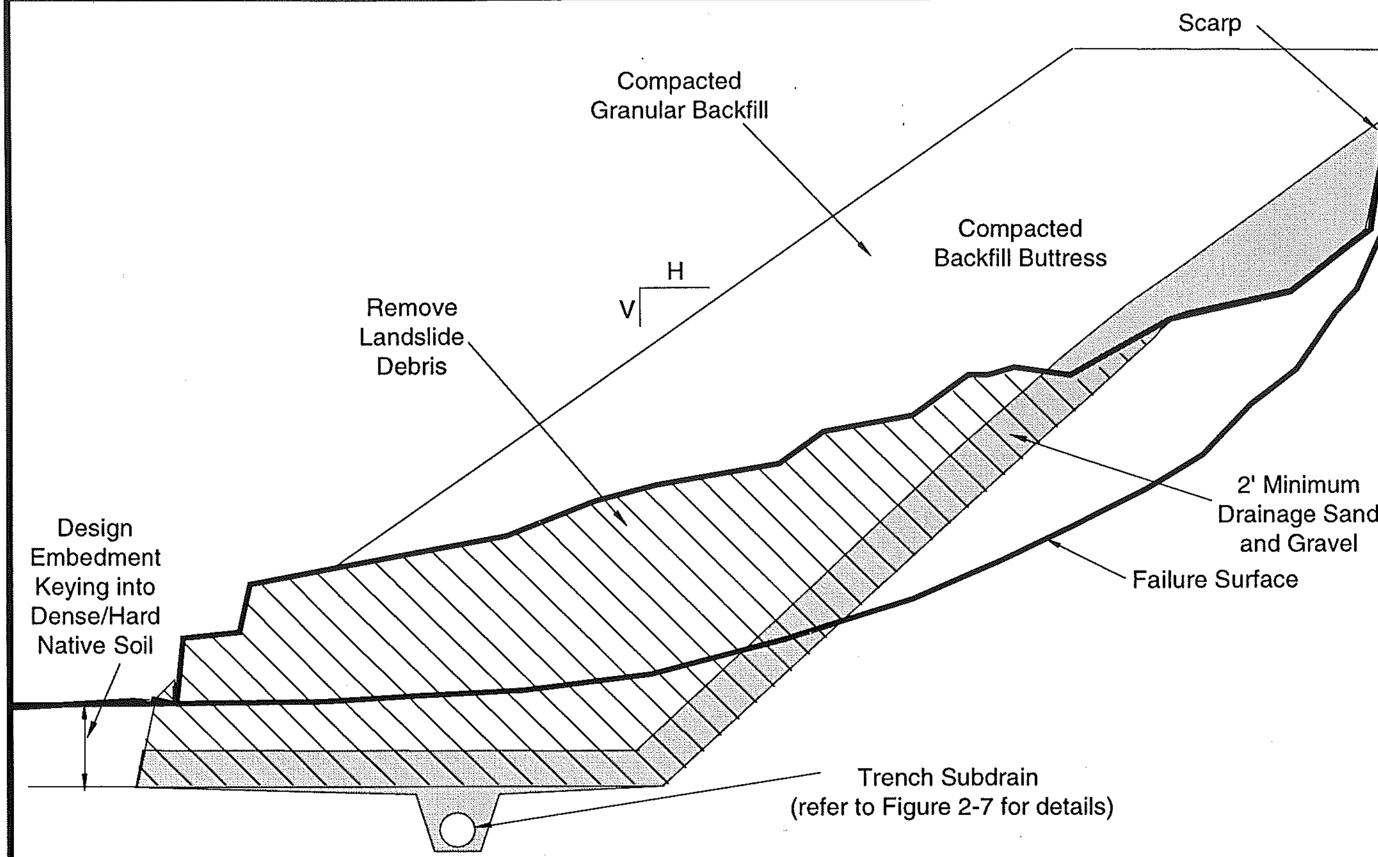
FIG. 2-17



MATERIALS

1. Drainage Sand and Gravel should meet the following gradation (Modified City of Seattle Type 26):

Sieve Size	% Passing by Weight
1-inch	100
3/4-inch	85 to 95
1/4-inch	30 to 60
No. 8	20 to 50
No. 50	3 to 12
No. 200	0 to 1
(by wet sieving)	(non-plastic fines)

An alternative drainage sand and gravel is a 50-50 mixture of washed pea gravel and washed concrete sand.

2. Granular backfill to consist of suitable on-site or imported clean, well-graded sand and gravel or crushed rock; either material must meet the following gradation criteria (City of Seattle Type No. 17);

Sieve Size	% Passing by Weight
3-inch	95-100
1/4-inch	25 -75
No. 200	0 to 5
(by wet sieving)	(non-plastic fines)

NOTE

This figure is not for construction. It should only be used for information pertaining to potential design concepts. Final design should be based on site-specific conditions and accomplished by a geotechnical engineer licensed as a professional engineer.

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TYPICAL FILL BUTTRESS

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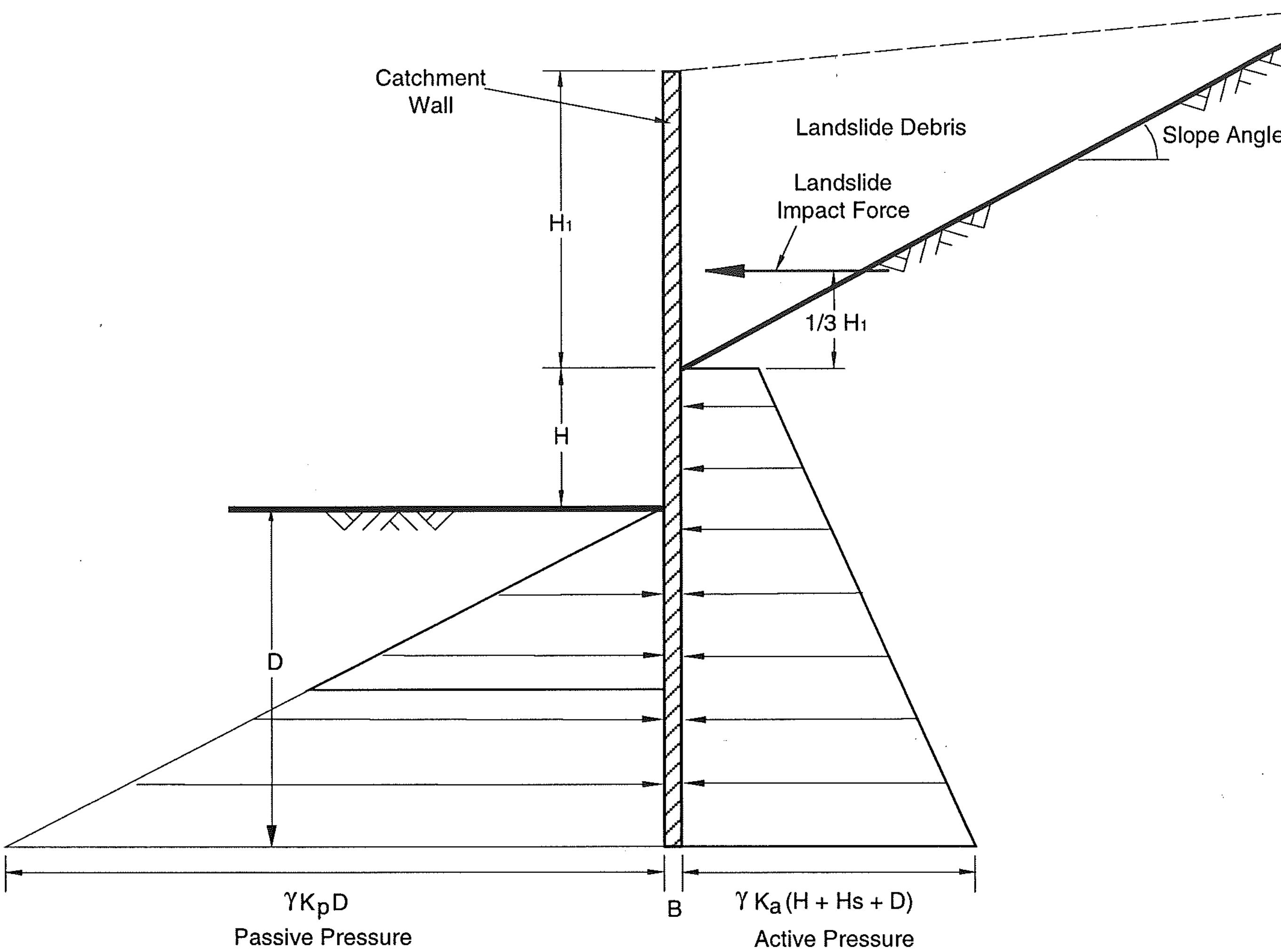
FIG. 2-18



LEGEND

H_1 = Landslide Catchment Height (ft)

H = Retained Height (ft)

Hs = Equivelent Surcharge Height (ft)

D = Embedment Depth (ft)

Ka = Active Earth Pressure Coefficient

Kp = Passive Earth Pressure Coefficient

γ = Unit Weight of Soil

B = Soldier Pile Width (ft)

NOT TO SCALE

NOTES

1. This figure is not for construction. It should only be used for information pertaining to potential design concepts. Final design should be based on site-specific conditions and accomplished by a geotechnical engineer licensed as a professional engineer.

2. The landslide impact force should be computed using the anticipated speed of the landslide debris and H_1.

3. Increase allowable stresses in structural members as appropriate for transient loading.

4. Use 50% of active pressures for lagging design.

5. See additional design recommendations for soldier pile walls on Figure 2-12 (sheet 3 of 3).

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TYPICAL SOLDIER PILE CATCHMENT WALL

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FIG. 2-19

PART 3. LANDSLIDES IN THREE STUDY AREAS WEST SEATTLE, MAGNOLIA/QUEEN ANNE, MADRONA

10.0 GENERAL

10.1 Purpose and Scope

Part 3 of the report presents a general geologic and geotechnical evaluation of the original three specific study areas previously mentioned in the Preface of this report. The emphasis is on evaluating factors that influence soil stability, and presenting general remedial measures for the types of slope instability found in the West Seattle, Magnolia/Queen Anne, and Madrona study areas.

The purpose for our studies and recommendations regarding stability improvements in these study areas is to provide the City of Seattle (City) with information that can be used to prioritize remedial efforts and to develop order-of-magnitude budgets based on the cost data given in Part 2, Section 8.0 of this report. The remedial measures presented are intended to be preliminary, with final scopes of work and corresponding cost estimates based on additional engineering studies and subsurface explorations.

The purpose described above has been accomplished in accordance with the following scope of services:

- We field checked the location of the reported landslides in the original three study areas. During this effort and an additional field visit, we evaluated the alternatives for stability improvements in the areas based upon the conditions observed (slide type, groundwater and surface water conditions, soil stratigraphy, etc.).
- For each study area, we prepared a description of the topography, geologic and groundwater conditions, slide types, timing, and slide locations.
- We divided each study area into smaller Stability Improvement Areas where landslide activity has been prevalent. For each smaller area, we evaluated the conditions contributing to current instability and/or potential future instability.
- Based on the above, we formulated stability improvements for consideration in the Stability Improvement Areas. The types of improvements recommended are described in Part 2, which also presents unit costs relative to the various types of improvements.

- The above scope of work is presented in this part of the report and is summarized in Table 3-1. The table provides preliminary estimates of quantities (length, square footage, etc.) related to improvements in the various areas.

In general, two site visits were made to each Stability Improvement Area, as indicated above. The first site visit, actually made prior to formulating the improvement areas, was primarily to field check the database locations and make appropriate changes in the database. The second site visit was for the purpose of formulating general types of measures that could be considered by the City and/or private property owners to improve stability and reduce landslide risk. Specific sites were not evaluated. The stability improvements listed on Table 3-1 include homeowner education; existing storm drainage facilities maintenance; storm drainage facilities improvement, as may be indicated by future observations or studies; subdrainage systems; fill stabilization; and retaining wall construction. The number, length, square footage, etc., listed on the table are rough estimates presented only to formulate order-of-magnitude budgets. Upon further studies needed to prioritize improvements, such studies may conclude that the extent or type of recommended improvements may or may not be needed, or that changes and/or additions may be advisable.

It should be mentioned here that some landslides have occurred outside the designated Stability Improvement Areas. These are usually isolated cases and the improvement areas were selected for locations where instability was prevalent. For landslides outside the designated areas, the stability improvement methods described in Part 2 of this report would apply, including homeowner education and drainage control.

The stability measures recommended do not consider the location of property lines and relate to improvements made on City property, private properties, or both. Since landslides and areas of potential instability do not obey property boundaries, improvements are sometimes necessary on both public and private land to suitably improve stability in an area. Therefore, the improvements recommended in Part 3 are those that could be made by the City to protect utilities, drainage features, streets, and other City facilities; and also those measures or actions to be taken by the City and/or adjacent property owners to improve stability of an unstable slope. In the latter case, the City and private property owners should coordinate efforts to improve stability and/or provide protection (such as catchment walls) should instability take place. It is anticipated that some improvements will be made by the City, while other improvements or protection will be the responsibility of private property owners.

It should be noted here that there are always risks of damage to property and structures involving landslides, for property located on or adjacent to a slope. Property owners need to accept those risks. Although the recommended improvements and homeowner education can lead to immediate or eventual improved slope stability conditions, private property owners should also obtain professional geotechnical advice to reduce current risks for their properties.

The analyses and recommendations presented in Part 3 of this report must be considered only in conjunction with the Limitations Section 1.5 presented in the Preface of this report.

10.2 Actions by City

In the succeeding sections of Part 3, various improvement measures and other actions are presented that we recommend be considered by the City. These actions include:

- Providing homeowner education materials regarding actions private property owners can take to reduce instability.
- Maintaining and/or improving storm drainage facilities.
- Conducting further detailed engineering studies in areas of prevalent landslides, including subsurface explorations.
- Implementing stability improvements.
- Coordinating stability improvements with private property owners.

Homeowner education is important so that the public is made aware of the factors that cause landslides and the steps homeowners should take to improve stability. Information should be provided to homeowners relative to prudent construction practices and obtaining professional advice for improving stability for existing homes, additions, or new construction. It is particularly important that homeowners learn that filling on a slope (especially at the top of a slope), or cutting into a slope (especially at the toe), can lead to instability and should only be undertaken with proper advice and consultation with competent geotechnical engineers or engineering geologists. Even the placement of yard waste on a slope decreases stability and, therefore, should be properly composted on flat ground or taken off-site. Homeowners should also be required to properly maintain and control their on-site drainage systems and to discharge drainage in accordance with applicable regulations, since improperly channeled water decreases slope stability, particularly when concentrated

In addition to the above, we recommend that the City continue to conduct neighborhood informational meetings to facilitate two-way discussion regarding stability matters. Valid concerns of homeowners should be taken into account in planning and implementing improvements. We also recommend that the general public be made aware of a telephone "hot line" that can be readily reached to report locations of poor drainage, landslides, or potential instability.

In areas of potential landsliding, it is important that existing storm drainage facilities be maintained. In addition, storm drainage improvements could be considered when indicated by subsequent observations and studies. In this regard, the City has retained a consulting engineering firm (Black & Veatch) to evaluate surface drainage systems throughout the city. The scope of this "Needs Assessment" included visual observation of the roadway runoff where it had potential to impact landslide-prone slopes. Their studies are to be coordinated with the landslide studies presented herein, with the goal of improving stability conditions. In the succeeding sections of this report, recommendations regarding maintaining and/or improving storm drainage facilitates are subject to the evaluations and recommendations to be made by Black & Veatch. Therefore, prioritizing and budgeting relative to surface drainage improvements are beyond this current landslide study.

As stated previously, the stability improvements presented in Part 3 are preliminary and for the purpose of providing the City with information they can use to prioritize remedial efforts and develop "ballpark" budgets. Further detailed studies, including subsurface explorations, should be undertaken by the City to determine final scopes and design of remedial measures, and more accurate cost estimates. Geotechnical and other consultants should be used as appropriate. Implementing stability improvements by the City would consist of preparing plans and specifications using the data presented in Part 2 of this report, and observing actual construction to verify suitable conformance with project requirements.

Since landslides and potential instability cut across property boundaries, a cooperative effort between property owners is advisable in obtaining the greatest benefits of stability improvements. In addition to homeowner education, previously discussed, the City should facilitate the processing of permits submitted by private property owners so remedial work can take place expeditiously to improve stability. Variances to code requirements should be allowed where needed to improve stability for private and/or public properties. Temporary and/or permanent easements on or across City property could be granted, where allowed by ordinance, such as when needed to construct protective structures or to allow gravity flow, in lieu of pumped drainage, for suitably designed drainage facilities on private properties. Coordination between the City and private property owners may also include shared costs, such as by Challenge Grants or Local Improvement Districts (LIDs).

10.3 Actions by Private Property Owners

Improvement of stability involves actions not only by the City, but actions by private property owners. Such actions by private property owners should include accepting existing conditions and the risks of slope instability. Measures should accordingly be implemented on private properties as may be needed to protect and improve stability for existing property, structures, additions, or new construction. Those measures to be taken by private property owners are the same types of improvements presented in Part 2 of this report, and professional advice should be obtained from geotechnical and other appropriate consultants regarding the improvements. Such advice should also be obtained by prospective buyers of property in slide potential areas.

Stability improvements would include proper drainage of surface water, including suitable discharge of roof gutter downspouts. Surface water should not be improperly channeled to or concentrated on slopes and particularly not onto adjacent property. Other remedial measures would consist of properly designed subdrains, site grading, soil retention systems (walls, soil reinforcement, tieback anchors, etc.), drilled drains, or other measures as conditions may dictate.

Of particular concern are structures located above or at the bottom of a potentially unstable slope. Private property owners should seek professional advice regarding such measures as underpinning walls and/or tieback anchors near the top, or catchment/retaining walls at the bottom of a slope.

Private property owners should take advantage of the homeowner education materials prepared by the City or other entities. Cooperation with the City and with adjacent property owners is also important so that remedial measures can be coordinated to achieve the greatest benefits of stability improvement. Private property owners should also notify the City regarding areas observed with poor drainage, landsliding, or potentially unstable ground, so that drainage and stability improvements can be coordinated between City and private property owners as appropriate.

10.4 Additional Considerations

The contributing factors to instability, as described for the Stability Improvements section of this report, include terms such as surface drainage, runoff, storm water runoff, surface water runoff, etc. Such drainage or runoff includes that from pavement areas as well as from soil or vegetated areas. The more pervious the soil, such as sand and/or gravel, the more that rainfall will infiltrate the ground, which reduces the amount of runoff. Conversely, for more impervious soils like silt or clay, runoff will be greater. Runoff also takes place from vegetated slopes, being greater for areas of sparse vegetation than for slopes with heavy vegetation.

Cuts at or near the toe of a slope, or fills on or near the top, are also contributing factors to instability. Such factors, particularly where cuts or fills took place years ago, may still have some influence on the stability of an area; however, such a factor may or may not be the predominant cause of recent or future instability. For example, a road cut area may remain stable for years, yet experience instability as the direct result of such things as a leaking or broken pipe, improper drainage from adjacent property, new filling or excavation on a slope, or other unwitting actions by owners or adjacent property owners. Each occurrence of instability requires evaluation to assess the predominant factor or factors leading to slope failure.

In describing some of the Stability Improvement Areas, we noted remedial measures of landslides that had recently been completed or were taking place. However, there are probably other remedial measures being planned, in progress, or completed by the City or private property owners that are not mentioned. Furthermore, we have not mentioned specific locations where surface drainage improvements have recently been undertaken or are being planned in conjunction with the "Needs Assessment" portion of the surface drainage studies by Black & Veatch.

11.0 WEST SEATTLE

The West Seattle area contains the most documented landslide events of the three study areas and of the whole city, as well as one of the two specific areas with the highest density of landslides, i.e., the Alki Avenue S.W. area. (The other area with the highest density of landslides is the Perkins Lane W. area in Magnolia.) In the early part of this century, West Seattle consisted primarily of summer homes that Seattle residents used only seasonally. Initially, Alki Avenue Southwest was constructed on piles around the Duwamish Head to provide access to the summer beach houses at the base of the Duwamish Head bluff. Later, Alki Avenue Southwest was filled to create a permanent roadway, which eliminated shoreline erosion at the base of Duwamish Head. The City of Seattle annexed the Arroyo Heights and Seola Beach areas, south of Lincoln Park, in the 1950s; therefore, instability south of Lincoln Park prior to the 1950s is not recorded in the City files.

11.1 Site Description

West Seattle is comprised of two linear ridges separated by Longfellow Creek (refer to Figure B-1). These north-south ridges and parallel depressions were shaped by the last glacial ice to occupy this area. West Seattle is bounded on the east by the Duwamish Waterway and on the west by Puget Sound. The eastern longitudinal ridge (Puget Ridge) is bounded on the east by West Marginal Way aligned between the base of the slope and the Duwamish Waterway. West of Puget Ridge is Longfellow Creek, which is one of Seattle's longest and lowest gradient streams. Pigeon Point represents the northern-most extension of this lineal ridge. West of Longfellow Creek the ground surface rises to a maximum elevation of 425 feet (High Point) atop a broad plateau representing the second longitudinal ridge. The margins of this broad ridge are steep and drained by several short and steep streams including Fairmount Gulch, Schmitz Creek, Fauntleroy Creek, and Seola Creek. This west ridge is bounded on the west by Puget Sound. Both longitudinal ridges extend farther south, beyond the city limit.

11.2 Soil Stratigraphy

Soils deposited during the most recent glaciation of the central Puget Lowland dominate the surface geologic conditions in West Seattle. Because West Seattle is south of the Seattle Fault (an east-west-trending reverse fault, dipping to the south), Tertiary bedrock is shallower in depth south of the fault, relative to those areas north of the fault. Tertiary bedrock outcrops sporadically near Alki Point along the beach and just east of the Alki Point lighthouse. For the most part, the bedrock is not landslide prone. One shallow colluvial landslide occurred on the west slope of one of these topographic bedrock highs.

The primary geologic units in West Seattle are the Vashon glacial deposits, although older, glacially deposited and nonglacial soils are present in stream cuts and at lower elevations. The glacially transported soils consist of all ranges in particle size, from clay to boulders. They can be divided into six broad categories based on the environment in which they were deposited: pre-Vashon glacial deposits, pre-Vashon nonglacial deposits, glaciolacustrine deposits (Lawton Clay), advance outwash (Esperance Sand), lodgement till (Vashon Till), and recessional outwash.

A seventh geologic unit in West Seattle is colluvium, which is a by-product of the weathering, erosion, and movement of the previously deposited soils. Colluvium is an accumulation of eroded soils and landslide debris on moderate or steep slopes. At some locations, it exists as a thin rind of soft or loose soil on very steep slopes such as the Duwamish Head bluff area. When direct precipitation and/or groundwater seepage saturate colluvium (generally soft or loose), it can lose strength and fail. Resulting failures occur typically as either a shallow or deep-seated colluvial slide. Colluvium mudflows (debris flows) commonly travel for a significant distance (greater than 50 feet) beyond the toe of the steep slope and are common throughout the landslide history of West Seattle. Colluvial landslides also occur where colluvium on a bench becomes unstable due to water pressures and moves over the top and down the face of steep bluffs.

11.3 Groundwater

Groundwater plays an important role in slope instability in West Seattle. There are three general types of groundwater present in this study area:

- Groundwater perched atop the lodgement till after percolating down through the relatively permeable recessional outwash near the highest elevations of West Seattle. (This source of groundwater has not contributed to instability in West Seattle to the same extent as the other two types of groundwater identified below.)
- Groundwater perched atop glaciolacustrine deposits after percolating through "windows" or cracks in the overlying lodgement till, and through the relatively permeable advance outwash sand.
- Groundwater perched on slopes at the contact between the overlying loose or soft colluvial soils and the glacially overridden soils.

As mentioned earlier, a key stratigraphic marker for landslide location is the contact between the advance outwash sand (Esperance Sand) and the underlying glaciolacustrine silt and clay (Lawton Clay), i.e., "The Contact" (Tubbs, 1974). The contact includes interbedded layers of silt, clay, and sand, which transition between the two geologic units. West Seattle has the longest trace of this sand-clay contract of any neighborhood within the City of Seattle (refer to Figure B-2). Although this contact is pronounced and well exposed in the Alki area, it extends continuously southward along the west-facing slope to the Arroyo Heights area. This contact is also present on the east-facing slope west of West Marginal Way and on the slope west of Longfellow Creek. This hydrologic discontinuity produces springs on the flanks of all of the West Seattle hills.

11.4 Landslide Types

11.4.1 High Bluff Peeloff Landslides

Please refer to Section 4.1.1 for a detailed description of high bluff peeloff type landslides.

There are no documented high bluff peeloff type landslides in West Seattle. The main reason for the absence of high bluff peeloff landslides in this area is the presence of Harbor Avenue Southwest, Alki Avenue Southwest, and Beach Drive Southwest along the shoreline of Puget Sound. These roadways protect the base of the steep slopes against shoreline erosion along West Seattle, thereby eliminating undercutting of the bluffs.

South of Lincoln Park, no high bluff peeloff landslides are documented in this area; however, the absence of documented landslides may be a reflection of the relatively recent (1950s) annexation of this area by the City of Seattle.

11.4.2 Groundwater Blowout Landslides

As previously described, a key stratigraphic marker for landslide location is the contact between advance outwash sand and an underlying glaciolacustrine silt and clay. Groundwater blowout landslides occur at this contact or other locations where pervious soil zones with high groundwater pressure influence the ground displacement. Therefore, the initiation point of earth movement, also referred to as the headscarp, generally lies on or near the contact between the pervious soil and the underlying less permeable soil. Because colluvium is usually involved in groundwater blowout landslides, it is common to classify them merely as shallow colluvial landslides. For this reason, there is an anomalously low incidence of reported groundwater blowout landslides throughout West Seattle. Figure B-2 illustrates the locations of documented groundwater blowout type landslides in West Seattle. For reference, the sand-clay contact (Tubbs, 1974) is also shown on the map. Nearly all of the groundwater blowout landslides were initiated at the sand-clay contact. A high percentage of shallow colluvial landslides were also initiated at or near this contact, and some may be improperly classified in the historical records.

11.4.3 Deep-Seated Landslides

Deep-seated landslides were identified in the database as ground displacement deeper than about 6 to 10 feet. The plane of movement may be arcuate or relatively planer and may involve glacially overridden soils as well as the surficial colluvial soils.

A map illustrating the distribution of deep-seated landslides in West Seattle is presented on Figure B-3. The highest densities of deep-seated landslides in West Seattle occur along Alki Avenue S.W., Delridge Way S.W. (23rd Avenue S.W.), S.W. Jacobsen Road, 5900-block of Beach Drive S.W., S.W. Admiral Way, and the intersection of Chilberg Avenue S.W. and Boyd Place S.W. With the exception of Delridge Way, all of the densest concentrations of deep-seated landslides occur at or near the sand-clay contact, similar to the groundwater blowout and shallow colluvial landslides. Grading of roadways by either cutting material from the toe of a slope or placing fill at the top of a slope may be one of the influences of the deep-seated failures.

The deep-seated landslides shown on Figure B-3 in the Alki Avenue area occurred on a topographic bench formed at the contact between the Esperance Sand and the underlying Lawton Clay; refer to Figure 3-1. The bench was formed by the erosion, sliding, and gradual regression of the upper portion of the bluff composed of Esperance Sand. The mechanism for this type of landslide is as follows:

1. Landslides from the upper slope deposit thick colluvium on the bench.

2. As colluvium accumulates on the bench, it becomes unstable due to groundwater pressure at the contact between the colluvium and the clay/silt (Lawton Clay), decreasing the frictional resistance to sliding.

3. The thick wedge of unstable material then translates along the lower portion of the bench, depositing debris (trees, vegetation, and colluvium) over the top of the bluff and onto the lower slope. Deep-seated, rotational sliding predominates on the bench with the slide planes reaching depths as much as 50 to 60 feet into the thick wedge of colluvium and slide debris on the bench.

4. The added material on the lower slope becomes unstable because of several factors, including: abundant groundwater emerging along the sand-clay contact, the steep slope angle, and the relative lack of vegetation on the lower slope. Shallow colluvial landslides predominate along the steep, lower slope, and trees from the bench move downslope with the colluvium.

11.4.4 Shallow Colluvial Landslides

Shallow colluvial landslides occur when loose, mostly heterogeneous soil on a moderate to steep slope becomes saturated. Commonly, these landslides result in rapidly moving saturated soil acting as a viscous fluid that can travel significant distances. In cases where the travel distance of the slide mass is greater than 50 feet, it was termed a debris flow for purposes of this study (refer to Section 4.1.4).

A map showing the distribution of historical shallow colluvial landslides in West Seattle is presented in Figure B-4. Shallow colluvial landslides make up 74 percent of the total reviewed landslides in West Seattle. Although they typically result from short duration heavy precipitation, regional groundwater also can be a factor. This is illustrated by the frequent occurrence of shallow colluvial landslides in West Seattle close to the sand-clay contact.

The highest concentrations of shallow colluvial landslides occur along 47th Avenue S.W., Atlas Place S.W., S.W. Jacobsen Road, and along the Alki area of West Seattle. The conspicuous double row of landslides on the northwest-facing slope of the Alki Avenue area represents shallow colluvial landslides occurring on two distinct topographic levels. The southeastern-most row of landslides is located along the upper bluff, which is composed primarily of overridden Esperance Sand (advance outwash) (refer to Figure 3-1). The lower or northwesternmost row of shallow colluvial landslides is located along the lower slope below the bench. Between the two distinct slopes is the bench created by ongoing deep-seated landsliding in the thick accumulations of reworked Esperance Sand (colluvium). Few of these landslides are reported because they are on forested, undeveloped property.

11.5 Landslides with Debris Flows

Debris flows are shallow colluvial landslides and generally consist of rapid movements of saturated soils that act as a fluid and travel considerable distances. As mentioned previously, landslides that have runout distances of greater than 50 feet are considered debris flows in this study.

Figure B-5 presents a map showing the distribution of debris flows in West Seattle. The Alki Avenue area of West Seattle has the highest density of debris flows in the City of Seattle. A debris flow typically begins as either a groundwater blowout or shallow colluvial landslide on a steep slope. These slides may also initiate as an earth fall of saturated colluvial debris from a bench onto the lower slope. Debris flows include not only mud but wood debris and other objects that can act as projectiles that may cause structural damage to structures in their path. Structures situated at the toe of the slope along Alki Avenue are susceptible to this type of landslide because of their close proximity to the steep slope.

In the vicinity of the 1300 block of Alki Avenue S.W., an area with several debris flow landslides (1956, 1983, 1997), colluvial landslides on the upper slope near Sunset Avenue S.W. flowed into a confined, short, steep gully and down to Alki Avenue S.W. (refer to Figure B-5).

Once in the gully, the slide debris mixed with additional water in the intermittent stream channel, decreasing the viscosity and increasing the volume and runout distance of the debris flow.

11.6 Timing of Landslides

A map showing the historical distribution of landslides by decade is presented in Figure B-6. Areas where the landslides are chronologically dispersed through time in West Seattle include Beach Drive S.W., Alki Avenue S.W., and Delridge Way (23rd Avenue S.W.). More recent (post-1960) landslides dominate the 47th Avenue S.W. and Seola Beach Drive S.W. areas. As previously discussed, these two areas were not significantly developed until after 1960, so it is likely that older landslides occurred in these two areas but were not reported.

11.7 Severe Storm-Related Landslides

A map illustrating the distribution of landslides in West Seattle during the four most notable landslide winters (1933/34, 1971/72, 1985/86, 1996/97) is presented in Figure B-7. The most notable trend in the quantity and distribution of the severe storm-related landslides in this area is the high proportion of 1996/97 landslides in West Seattle. The scarcity of 1933/34 and 1971/72 reported landslides may be a function of lesser urban development during those time frames rather than the relative magnitude of the earlier severe storms.

11.8 Potential Slide Areas

The City of Seattle presently regulates development in steep slope and potential slide areas. Historical landslide locations and the location of the sand-clay contact were used by the Department of Design Construction and Land Use (DCLU) to define the Potential Slide Areas, as described in Section 20.0 of this report. Figure B-8 illustrates the location of the potential slide areas with all of the landslides in the database for West Seattle. About 63 percent of the reviewed landslides in West Seattle fall within the existing Potential Slide Areas. The obvious areas where landslides occur outside of the potential slide areas are the 47th Avenue S.W. area, Seola Beach Drive S.W. area, and the upper slope along Alki Avenue S.W.

11.9 Stability Improvements

This section presents possible stability improvements that could be made by the City to protect utilities, drainage features, streets, and other City facilities. It also presents measures that could be made by the City and adjacent property owners to improve the stability of an unstable slope. We present further comments regarding educating private property owners on steps they may take to improve stability.

The West Seattle area has been divided into ten smaller Stability Improvement Areas, where landslide activity has been prevalent. As shown on Figure B-9 (Appendix B, Map Folio), the ten areas are as follows:

1) 23rd Avenue S.W.
2) Admiral Way
3) Fairmount Gulch
4) Harbor Avenue
5) Alki Avenue
6) Boyd Place/Chilberg Place
7) Jacobsen Road
8) Beach Drive/Atlas Place
9) 47th Avenue S.W.
10) Seola Beach

For each area, we summarize the general subsurface conditions, landslide types and causes, and present actions that could be considered for improving slope stability. Table 3-1, located following the text in Part 3 of this report, presents a summary of this information.

11.9.1 23rd Avenue S.W.

In the 23rd Avenue S.W. Stability Improvement Area, as designated on Figure B-9, 24 landslides were recorded. Both deep-seated and shallow colluviual landslides occurred, and a number of landslides were not identified as to the type. The landslides in this area have taken place along the west-facing slope generally between 21st Avenue S.W. and Delridge Way S.W. at the toe of the slope. Instability in this area was reported as early as 1914. The most recent landsliding took place in January 1997, which damaged 23rd Avenue S.W. (one block east and uphill of Delridge Way) at S.W. Dakota Street. As a result of the January 1997 landslide, several properties on the downhill (west) side of 23rd Avenue were also damaged by the earth movement.

The landslides that occurred in this area prior to 1960 reportedly were related to grading of 22nd and 23rd Avenues, presumably caused by cutting into the slope on the east side of these streets. The instability that took place following 1960 was generally related to filling on private properties on the west side of 23rd Avenue, or cutting into the slope on private properties east of Delridge Way.

The subsurface conditions in this area consist of a silt-clay colluvium that is up to 25 feet thick, located over stiff to hard clay. Groundwater levels are typically high because this area is at and near the toe of a slope. The sand-clay contact (Tubbs, 1974) has not been mapped in this area. The contributing factors to instability are the soil conditions on this slope (colluvium over stiff to hard clay), undercutting or filling on the slope, and high groundwater levels/seepage. The landslides were triggered by heavy rainfall that resulted in surface runoff and infiltration into the slope soils.

To improve stability for 23rd Avenue S.W. at the Dakota Street right-of-way (not a through street), a buried, drained, secant-type soldier pile wall was constructed along the west edge of the street. The wall length was about 110 feet. Repaving the street east of the new wall included provisions to control surface drainage. With wall construction, stability was improved for 23rd Avenue; however, instability could still occur downhill from the wall, particularly on private properties where owners should obtain professional advice for improving stability on their sites.

Recommended actions in this area would include homeowner education and storm drainage systems maintenance and/or improvement, including the improvement of storm drainage from private properties uphill from 23rd Avenue. Finger drains could also be considered to improve stability for the toe of the hillside upslope of 23rd Avenue.

11.9.2 Admiral Way

The Admiral Way Stability Improvement Area is the east-facing slope situated as shown on Figure B-9. In this area, a total of 26 shallow colluvial and deep-seated landslides have been recorded for this area, beginning in 1917. Some of the landslides occurred on the steep slope uphill from S.W. Admiral Way, and others took place on the steep slope downhill. The most recent instability occurred uphill from Admiral Way in early 1998, which resulted in the City constructing remedial measures including a rock buttress near the top of the slope, and a 120-foot-long, drained soldier pile and concrete lagging wall (6 to 8 feet high) along the toe of the slope on the west side of Admiral Way.

The subsurface conditions consist of colluvium on the steep slopes overlying glacially overridden native soils. In some areas, fill may be present, such as for backyards. The original construction of Admiral Way likely included some fills along the east side and cutting along the west. A 6- to 8-foot-high rail (trolley) and concrete lagging toe wall is present along much of the west side of Admiral Way. A portion of this wall failed at the time of the 1998 landslide, and other portions of the wall are bulging or have been overtopped by slope erosion debris or previous landslides. The slopes both west and east of Admiral Way exhibit active signs of creep.

Based on available subsurface information, the colluvium on the slopes is 10 or more feet deep. The sand-clay contact (Tubbs, 1974) extends through this area.

The factors that contribute to instability in this area are steep topography, relatively deep colluvium on the slope, high groundwater levels/seepage, storm water runoff, and cutting and filling. The triggering mechanism is generally heavy rainfall with surface water runoff and infiltration.

Stability improvements that could be considered are trench subdrain installations, wall construction, storm drainage systems maintenance and/or improvement, and homeowner education. Subsurface drainage is probably the most cost-effective method for improving slope stability in the area. Interceptor trench subdrains parallel to contours uphill from Admiral Way, or trench subdrains at intervals perpendicular to contours (finger drains) could be effective. Such subdrains should extend through the colluvium and into the glacially overridden soils. Stronger, higher walls for toe support and increased catchment area for slide debris could also be considered to protect Admiral Way. Refer to Table 3-1, for estimated lengths of subdrains and wall that could be considered for budgeting purposes in this area. A comprehensive study and improvement to storm drainage is recommended for east of 35th Avenue S.W. and north of S.W. Spokane Street. The instability downhill of Admiral Way occurred mostly on private properties where homeowner education and prudent development practices should be followed.

11.9.3 Fairmount Gulch

The Fairmount Gulch Stability Improvement Area consists of a large, steep-sided ravine that extends from Harbor Avenue to the southwest where Admiral Way crosses the ravine on a high bridge; refer to Figure B-9. Eleven landslides have been recorded for this area, mostly on the east-facing slope of the ravine. Ten landslides were listed as shallow colluvial with three of them debris flows, and one was not identified as to type, although it was likely also a shallow colluvial landslide based on the database comment. The earliest recorded landslide occurred in 1937, and instability has been reported through the years. Only one event involving instability (tension cracks in backyard) was reported due to the 1996/1997 storm.

The subsurface soils in this area, based on geology mapping and our experience in this area (no explorations reviewed), consist of colluvium overlying glacially overridden soils. The overridden soils consist of sand over clay, and the sand-clay contact (Tubbs, 1974) is present at lower elevations in the ravine The landslides reported in this area are primarily failures in colluvium and/or yard fills placed by private property owners. One landslide was reported in

1995 to be related to road fill placed for Belvidere Avenue S.W. This street is located in the ravine near the sand-clay contact. The factors contributing to instability are steep topography, loose fill and/or colluvium on the slope, high groundwater levels with associated seepage particularly near the sand-clay contact, and heavy rainfall (triggering cause) that saturates the loose soil.

It is recommended that work by the City to improve stability include maintaining existing storm drainage facilities and improving them when indicated by future observations in this area. Springhead drains installed at known seepage points could reduce infiltration and saturation of colluvial soils by groundwater springs and seeps. Homeowner education is recommended to include providing information regarding prudent construction and site drainage practices, and obtaining professional advice for improving stability for existing property, additions, or new construction.

11.9.4 Harbor Avenue

Sixty-one landslides have been recorded for the Harbor Avenue Stability Improvement Area. Most of the landslides were reportedly of the shallow colluvial type (48), while a few were listed as deep-seated (8) and groundwater blowouts (5). These landslides have generally occurred on the easterly- and northerly-facing steep slopes in this area; refer to Figure B-9. The earliest recorded landslide occurred in 1916 and instability has occurred continually through the years, including 1998. A number of landslides occurred in this area during the 1996/97-winter storm (13) including a large deep-seated landslide at California Way S.W. and Ferry Ave S.W., which closed California Way S.W. for several months.

There are three general areas of instability in this area: the east-facing slope between Victoria Avenue S.W. and Harbor Avenue S.W., the east-facing slope between Palm Avenue S.W. and California Way S.W., and the north to northwest-facing slope between California Lane S.W./California Way S.W. and Alki Avenue S.W./Harbor Avenue S.W. Shallow colluvial landslides and debris-flows dating back to about 1933 have impacted structures at the toe of the slope, east of Victoria Avenue. The slope below Palm Avenue exhibits abundant groundwater seepage near the sand-clay contact and is the location for two relatively large deep-seated landslides that occurred in early 1996 and in early 1997: the 1300-block of California Way, and California Way/Ferry Avenue, respectively. The pavement along the east side of Palm Avenue was cracked and had settled at the time of our visit in 1998. The City improved the stability of the slope in the 1300-block of California Way by constructing a drainage blanket retained by a 45-degree earth slope reinforced with geogrids. This repair was the first use of reinforced slopes with geosynthetics in Seattle. The City improved the stability of the California Way/Ferry Avenue landslide using subsurface drainage, crib walls, grading, and buttressing. A buried, soldier pile wall approximately 110 feet long, was also constructed along the east side of California Way to improve roadway stability. The structures at the base of the northerly-facing slope below California Lane and California Way have been impacted by gradual bluff regression and sloughing since 1955. Structures on the bench, in the vicinity of California Lane, have been impacted by at least two deep-seated landslides.

The subsurface soils in this area, based on geologic mapping and our experiences in this area, consist of colluvium overlying glacially overridden soils. The glacially overridden soils consist of slightly silty sand (outwash) over silty clay (glaciolacustrine). The sand-clay contact (Tubbs, 1974) is present at approximately elevation 100 feet (±20 feet) throughout the Harbor Avenue Stability Improvement Area. A bench of variable width exists at the top of the clay unit with roughly 10 to 60 feet thickness of colluvium accumulated from up-slope sources. The factors that contribute to instability in the Harbor Avenue area include steep topography, loose colluvium over glaciolacustrine clay, high groundwater levels/seepage, and cutting at the toe of the slopes. The landslides reported in this area typically initiate at or near the bench with debris traveling down the lower clay slope.

Recommended improvements that could be considered by the City and private landowners in this area consist of storm drainage systems maintenance and/or improvement, road fill replacement, springhead drain installation at identified seepage points, retaining/catchment wall installation, trench subdrain installation, and homeowner education. Surface drainage in the vicinity of California Lane could be improved in order to reduce infiltration into the thick colluvial soils along the bench. We recommend that existing and new drainage facilities installed in the area by the City or private landowners be checked and maintained on a regular basis for proper functioning. Interceptor trench subdrains may be appropriate along the bench area in the vicinity of California Lane and downslope of Palm and Victoria Avenues. Consideration could also be given to removing the fill portion of Palm Avenue and replacing it with compacted material to reduce settlement and pavement cracking and resulting surface water infiltration into downslope soils. Retaining/catchment walls would be effective along the toe of the lower slope below Victoria Avenue and at the northernmost tip of the Stability Improvement Area. Furthermore, installation of springhead drains could be considered in discrete areas of acute groundwater seepage along the steep slopes to prevent saturation of the colluvial soils by spring water. We accordingly recommend that property owners in this area obtain geotechnical advice regarding precautions to reduce the risk to properties, including catchment walls at the base of the slope and surface drainage at the top of the slope.

11.9.5 Alki Avenue

The Alki Avenue Stability Improvement Area is a northwesterly-facing slope situated as shown on Figure B-9. In this area, a total of 106 landslides (deep-seated, groundwater blowout, and shallow colluvial) have been recorded in this area since 1916. Approximately one-third of the landslides occurred along the upper bluff, just west of Sunset Avenue S.W. The others occurred along the lower bluff behind the properties along Alki Avenue S.W. While approximately 33 slides were reported in this area due to the 1996/97 winter storm, the most notorious landslide occurred in the spring of 1974 where a large-scale deep-seated event threatened properties along the 1400-1700 blocks of Alki Avenue S.W.

The subsurface conditions consist of thick accumulations of colluvium (up to 50 to 60 feet thick) on a midslope bench and thin rinds blanketing the steep slopes, as shown on Figure 3-1. Underlying the colluvium is an upper unit of glacially overridden outwash sand (Esperance Sand) with glaciolacustrine clay (Lawton Clay) and older, pre-Vashon silt/clay and sand below. Some fills placed for roads and residential construction may be present along the upper sand bluff just west of Sunset Avenue and in the vicinity of California Lane and Bonair Drive, where the construction of these streets likely included fills along the west margins of the roads. Near the 1300-block of Sunset Avenue, the City installed a buttress fill and a drained soldier pile and lagging wall to mitigate landslides that occurred on the upper steep slope just below Sunset Avenue. Several other remedial measures in this area included crib walls and soldier pile and lagging walls to protect structures along the upper bluff, and catchment walls and surface drainage behind structures at the toe of the lower slope.

The contributing factors to instability in this area are steep topography, colluvium on the slope, high groundwater levels/seepage, cutting and filling, and heavy rainfall and associated infiltration (triggering mechanism).

With respect to the instability during the spring of 1974, Shannon & Wilson, Inc. (Shannon & Wilson) was retained by the City to conduct a geotechnical study of the area. Based on geologic reconnaissance and subsurface borings, a report dated July 1975 recommended two conceptual preliminary design alternatives. One alternative was to design a large earth buttress (including subdrains) on the bench. The other alternative was to construct a large, tied-back cylinder pile wall on the bench in conjunction with trench subdrains. Because of the great depth of colluvium on the bench, such measures to improve stability would be extensive and expensive. Upon further exploration and evaluation in 1999, a scheme of horizontal drains and deep trench drains was chosen to increase stability of this slope and particularly the bench area. To help fund this work, the City applied for and received a Federal Emergency Management Agency (FEMA) grant. Although some improvement in stability conditions is anticipated above and below the bench area, some risks of instability would still be present. Property owners above and below the bench area would still need to seek geotechnical advice and take precautions to reduce the risk to their properties.

Other improvements that the City could consider consist of storm drainage systems maintenance and/or improvement, subdrain and springhead drain installations along the bench area outside the project area described in the preceding paragraph, and homeowner education. Homeowner education is probably the most cost-effective method for improving slope stability in this area. Property owners in the Alki Avenue Stability Improvement Area should avoid making improper cuts and fills, maintain existing drainage systems, seek geotechnical advice, and take precautions to reduce risk to their properties.

It is recommended that the City consider evaluating, repairing, and maintaining existing City-owned drainage pipes that have been installed over the years in this area (a drainage map is available in City files). Furthermore, it is recommended that the City coordinate efforts (expeditious processing of permits or other cooperative effort as described in Sections 1.5 and 10.2 in this report) with private property owners along Alki Avenue, relative to building catchment walls along the toe of the slope for protection of the structures.

11.9.6 Boyd Place/Chilberg Place

The Boyd Place/Chilberg Place Stability Improvement Area consists of a west-facing steep slope, as indicated on Figure B-9. Seven landslides are recorded for this area, mostly in the vicinity of the Boyd Place S.W. and Chilberg Place S.W. intersection. Three landslides were listed as shallow colluvial and four, at the Chilberg/Boyd intersection, were listed as deep-seated. The earliest recorded landslide occurred in 1964 and consisted of a setdown along the Boyd Place right-of-way. Other landslides in the vicinity of this intersection, along Boyd Place, have occurred in 1971, 1974, and 1997. During the 1997 earth movement, and probably as a result of this instability, a water main ruptured, exacerbating the situation. Remedial measures included an 85-foot-long wall installed along the west side of Boyd Place to retain the road fill and a 55-foot-long wall was installed along the east side of Boyd Place to retain the cut slope. These two walls consisted of soldier piles with concrete lagging. An 83-foot-long reinforced concrete retaining wall was also constructed along the downhill side of Chilberg Place to provide support for that roadway. The City also installed catch basins and other drainage improvements in the vicinity.

The subsurface conditions in this stability improvement area generally consist of colluvium overlying glacially overridden native soils. In some areas, existing fill is present, such as for backyards and roads. The glacially overridden soils consist of outwash sand overlying glaciolacustrine silt and clay. The sand-clay contact is located in the vicinity of the intersection of Chilberg and Boyd Place. Associated groundwater seeps and springs exist in this area.

The factors that contribute to instability in this area are steep topography, abundant groundwater seeps and springs associated with high groundwater levels, storm water runoff, and cutting and filling. The triggering mechanism is generally heavy rainfall with surface water runoff and infiltration into downslope soils.

It is recommended that actions by the City in this area consist of maintaining and/or improving storm drainage systems, particularly in areas outside of the 1997 Chilberg/Boyd Place project area. Homeowner education is also recommended.

11.9.7 Jacobsen Road

The Jacobsen Road Stability Improvement Area is a west-facing slope situated as shown on Figure B-9. In this area, a total of 18 landslides (deep-seated, shallow colluvial, and groundwater blowout) are recorded in the database since 1933. Some of the landslides occurred on the steep slope on the east side of S.W. Jacobsen Road, and others, including several deep-seated landslides, have occurred on the steep to moderate slope along the west side of Jacobsen Road. The most recent instability occurred downhill from Jacobsen Road in early 1997, which resulted in severe structural damage to two residences west of Jacobsen Road. Remedial measures have been planned by private property owners to improve slope stability and repair the damaged structures. The City placed an asphalt curb to prevent surface water from infiltrating the slope soils west of Jacobsen Road.

The subsurface conditions consist of colluvium on the steep slopes overlying glacially overridden native soils. In some areas, existing fills may be present, such as for residences along the west side of Jacobsen Road. The original construction of Jacobsen Road likely included some fills along the west side and cutting along the east. The sand-clay contact with its associated groundwater seepage exists along the downslope side of the southern portion of

Jacobsen Road, and crosses to the uphill side of Jacobsen Road to the north. The slopes on both sides of Jacobsen Road exhibit signs of soil creep. Based on available subsurface information, we estimate that the colluvium on the slopes is 10 or more feet deep.

The factors that contribute to instability in this area are the steep topography, relatively deep colluvium on the slope, high groundwater levels/seepage, and cutting and filling. The triggering mechanism is generally heavy rainfall with surface water runoff and infiltration.

Stability improvements that the City could consider consist of surface drainage maintenance and/or improvement, interceptor trench subdrain installation, and homeowner education. Surface drainage is probably the most cost effective method for improving slope stability along the west side of Jacobsen Road. An interceptor trench subdrain along the east (upslope) side of Jacobsen Road may be appropriate, unless a suitably functioning subdrain is already in place. Installation of curbs and gutters to prevent surface water from Jacobsen Road from flowing onto and infiltrating the downslope areas west of the roadway could also be considered. Information should be provided to property owners regarding proper cutting and filling, and controlling their on-site drainage systems.

11.9.8 Beach Drive/Atlas Place

The Beach Drive/Atlas Place Stability Improvement Area, as shown on Figure B-9, consists of the following: 1) an upper, west-facing steep slope between 49th and 50th Avenue S.W. (east of Atlas Place) and Atlas Place S.W.; 2) a bench approximately 300 feet wide (upon which Atlas Place is constructed); and 3) a lower, west-facing moderate slope west of Atlas Place that extends down to Beach Drive S.W. Twenty-five landslides have been recorded for this area. Six landslides were listed as deep-seated and the others were the shallow colluvial type. The earliest recorded landslide occurred in 1927, and instability has been reported through the years. Four landslides occurred during the winter storm of 1996/97, including a deep-seated event in the 5900-block of Atlas Place.

The subsurface soils in this area, based on geologic mapping and our experience in this area (no explorations reviewed), consist of colluvium overlying glacially overridden native soils. The overridden soils consist of sand overlying clay, and the sand-clay contact (Tubbs, 1974) is present at roughly the same elevation as the bench. The slope instability reported in this area is located along the steep slope west of Atlas Place, along the steep slope east of the 6500-block of Beach Drive, and along the west shoulder of Beach Drive.

The landslides that have been reported in this area occurred primarily in colluvium and/or road cuts and fills for both Beach Drive and Atlas Place. Instability along the west margin of Beach Drive appears to result from fills placed during the construction of the roadway. Ponding water and road-settlement were observed along Beach Drive during our field reconnaissance. Shallow colluvial landslides along the east (uphill) side of Atlas Place appear to be the result of cutting into the slope without any slope retention measures. Surface water is also contributing to instability between Beach Drive and Atlas Place. The City placed an asphalt curb along the west side of Atlas Place to prevent surface water from infiltrating the downslope areas. In summary, the factors contributing to instability are the steep topography, cutting and filling, surface water, and high groundwater levels/seepage.

Actions the City could consider consist of improvement of the Atlas Place street grade with curbs, gutters, and storm drainage facilities; removal and replacement of existing loose soils along the west side of Beach Drive, and education of property owners in this area. Improvement of the Atlas Place street grade would include the retention of the cut-slopes, a possible interceptor trench subdrain along the centerline of the roadway, and provisions for surface drainage along the full length of the roadway. Springhead drains would be effective in capturing groundwater seeps and springs along the cut slope east of Atlas Place. Improving stability of Beach Drive could include removal of the existing fill soils and replacement with lightweight, structural fill material. It is recommended that homeowner education include proper methods for controlling on-site drainage systems and discharging drainage in accordance with City regulations.

11.9.9 47th Avenue S.W.

The 47th Avenue S.W. Stability Improvement Area is a steep, west-southwest-facing slope situated as shown on Figure B-9. In this area, one deep-seated, one groundwater blowout, and 19 shallow colluvial landslides have been recorded since 1955. Some of the landslides occurred on the steep slope uphill from 47th Avenue and others took place on the steep slope downhill of 47th Avenue. Others occurred in the vicinity of Maplewood Place S.W. (private road) located near the southern edge of this stability improvement area The most recent instability took place at the intersection of 47th Avenue and Maplewood Place during the 1996/97 winter storms. This resulted in the City constructing remedial measures, including a gabion wall on the east side of 47th Avenue, a soldier pile and lagging wall along the west side of Maplewood Place, and drainage improvements.

The subsurface conditions consist of colluvium on the very steep slopes overlying glacially overridden native soils. Fills are present in some areas such as for residential backyards, based on the landslide descriptions. The overridden native soils consist of limited occurrences of glacial till overlying outwash sand with glaciolacustrine clay below. The sand-clay contact (Tubbs, 1974) is present east of 47th Avenue at elevation 170 feet (±30 feet). The landslides that have been reported in this area are primarily failures in colluvium resulting from surface water runoff and groundwater seepage near and downslope from the contact. Numerous groundwater seeps and hydrophitic vegetation exist along the east (uphill) side of 47th Avenue.

The factors that contribute to instability in this area are steep topography, improper cutting and filling, high groundwater levels with associated seepage particularly near and downslope from the sand-clay contact, and improperly directed surface water. For example, improper cutting into the toe of the slope on both private and public properties, or private utility failures (water and sewer lines) reportedly influenced approximately five of the recorded slides in this Stability Improvement Area.

Stability improvements that we recommend the City and private property owners consider are surface drainage systems maintenance and/or improvement and homeowner education. The City could consider placing a curb/gutter along the west side of 47th Avenue S.W. to prevent infiltration of surface water into downslope areas, particularly upslope of Maplewood Place, a private road. Furthermore, it is recommended that the City facilitate the processing of permits regarding design, access, and construction efforts with private property owners along Maplewood Place, with respect to catchment wall construction along the toe of the cut slope for protection of the structures. A soldier pile retaining wall could also be considered along the west margin of 47th Avenue upslope of the Maplewood Place dead-end to improve stability for the street. It is recommended that homeowner education emphasize the need to obtain professional advice before cutting and/or filling along any slopes. Private property owners in this area should control their on-site drainage systems and discharge drainage in accordance with regulations, since improperly channeled water decreases slope stability.

11.9.10 Seola Beach

The Seola Beach Stability Improvement Area consists of a moderately steep to steep-sided ravine that extends from Puget Sound to the north-northeast for approximately one mile; refer to Figure B-9. All of the landslides recorded in the database for this area are on the west side of the ravine. The east side of the ravine is outside of the Seattle City Limits. Along the west side, a total of six landslides (shallow colluvial and deep-seated) have been recorded, beginning in the spring storm of 1986.

The subsurface conditions based on geologic mapping and our experience in the area (no explorations reviewed) consist of colluvium overlying glacially overridden outwash sand and gravel. There is no lacustrine clay exposed in this area below the outwash sand and gravel. Therefore, the sand-clay contact is not mapped in this area. The landslides that have been reported in this area primarily occurred in colluvium and/or yard fills placed by private property owners. One landslide/debris flow was reported in 1986 to be related to the rupture of a sewer main on the upper plateau, north of the south end of Seola Beach Drive S.W. The runout of debris reached Puget Sound.

The factors contributing to instability are moderately steep to steep topography, private backyard fills and/or colluvium on the slope, and heavy rainfall (triggering cause) that saturates the loose soil and causes failure.

In the long-term, there do not appear to be practical remedial measures that the City could take to prevent the natural occurrence of landslides in this area other than homeowner education.

12.0 MAGNOLIA/QUEEN ANNE

While Magnolia and Queen Anne are two distinct topographic highs, they share similar geology conditions and, therefore, are treated as a single study area. Perkins Lane West, located along the southwestern margin of Magnolia, is similar to Alki Avenue in West Seattle in that it contains a very high density of historical reported landslide events.

12.1 Site Description

Magnolia and Queen Anne are two distinct topographic highs separated by Interbay, a north trending linear depression (refer to Figure B-10). Magnolia, offset north with respect to Queen Anne, reaches a maximum elevation of 375 feet. Queen Anne, similar in size to Magnolia, reaches a maximum elevation of 400 feet. The area is bordered by Puget Sound and Elliot Bay to the west and southwest, the Lake Washington Ship Canal to the north, Lake Union to the east, and downtown Seattle to the south. Steep slopes surround both Magnolia and Queen Anne. The bluff along the west side of Magnolia, extending from Smith Cove to the Lake Washington Ship Canal, is locally armored against wave action and is the steepest slope in Magnolia. Kinnear Park and the slope west of Aurora Avenue are among the steepest slopes on Queen Anne.

12.2 Soil Stratigraphy

Soils deposited during the most recent glaciation of the central Puget Lowland dominate the surface and subsurface geologic conditions in the Magnolia and Queen Anne study area. Because Magnolia and Queen Anne lie north of the Seattle Fault, Tertiary bedrock is buried below roughly 3,000 feet of glacial and non-glacial sediments.

The primary geologic units involved with landsliding in Magnolia and Queen Anne are the Vashon glacial deposits. The glacial soils consist of all ranges in particle size from clay to boulders and may be divided into four broad categories: glaciolacustrine deposits (Lawton Clay), advance outwash (Esperance Sand), lodgement till (Vashon Till), and recessional outwash.

Colluvium is also present along the lower portions of the hillsides in the Magnolia and Queen Anne study area. Particularly thick accumulations of colluvium occur along the Perkins Lane West area of Magnolia. Colluvium also forms a thin rind on steep slopes all around Magnolia and Queen Anne.

12.3 Groundwater

Groundwater plays a key role in slope instability in Magnolia and Queen Anne. The contact between advance outwash sand and underlying glaciolacustrine silt and clay is exposed in slopes around both Magnolia and Queen Anne. Prominent springs associated with this contact occur throughout these areas including Perkins Lane W., Kinnear Park, 15th Avenue W., and Westlake Avenue N. Figures B-11 through B-19 illustrate the location of the sand-clay contact.

12.4 Landslide Types

12.4.1 High Bluff Peeloff

High bluff peeloff-type landslides occur in only a few discrete areas in the City of Seattle. A map showing the distribution of high bluff peeloff landslides in the Magnolia and Queen Anne study area is presented in Figure B-11. Areas where the slopes are near vertical resulting from either wave action at the base of the slope or the presence of resistant lodgement till, or both, are present in Kinnear Park, Lawtonwood, and along the southwestern shoreline of Magnolia. With the exception of Kinnear Park and portions of Perkins Lane W., there is little or no armoring along the toe of the slope below the high, steep bluffs. The high bluff peeloff landslide located at the northern tip of Magnolia likely occurred as a result of undercutting by wave action at the base of the bluff. In 1997, a high bluff peeloff landslide occurred along a short section of steep bluff east of the northern portion of Perkins Lane. The high bluff peeloff landslides along the southwest margin of Magnolia occurred on the steep bluff above (east of) Perkins Lane W. The very steep, bare bluff south of the southern end of Perkins Lane West has a long history of high bluff peel-off type landslides, but the City files do not have information on these events because they generally have little effect on structures or transportation routes.

12.4.2 Groundwater Blowout Landslides

A map showing the distribution of groundwater blowout landslides in the Magnolia and Queen Anne area is presented in Figure B-12. The contact between the advance outwash (Esperance) sand and the glaciolacustrine silt and clay (Lawton Clay) is also shown. As stated previously, without accurate reporting and analysis of the landslide event, it is difficult to distinguish between a shallow colluvial landslide and a groundwater blowout landslide. Several landslides described in the historical records as shallow colluvial landslides may, in fact, be groundwater blowout landslides. In Magnolia, the Works Progress Administration (WPA) completed several projects designed to capture and redirect groundwater for slope stabilization purposes. The WPA projects are marked on Figure B-10 with a pick and shovel symbol.

12.4.3 Deep-Seated Landslides

A map illustrating the locations of deep-seated landslides in the Magnolia and Queen Anne areas is presented on Figure B-13. The highest density of deep-seated landslides is located along the west side of Queen Anne and Perkins Lane W. Along the west side of Queen Anne, several deep-seated landslides were reported, including a very large area of instability that was active from 1951 to 1956 in the vicinity of 12th Avenue W. and W. Blaine Street. The Perkins Lane W. landslides generally occur below the bluff, in a relatively thick colluvial wedge as shown on the Idealized Geologic Conditions West Magnolia profile, Figure 3-2. The colluvial wedge overlies a hard surface of Lawton Clay that commonly slopes toward Puget Sound. This contact between the colluvial wedge and the hard Lawton Clay creates groundwater conditions conducive to landsliding.

12.4.4 Shallow Colluvial Landslides

Figure B-14 shows the distribution of shallow colluvial landslides in Magnolia and Queen Anne. It is our opinion that groundwater along southwest Magnolia and southwest Queen Anne significantly contributes to shallow colluvial landslides as well as groundwater blowout landslides. Furthermore, based on the spatial distribution of shallow colluvial and groundwater blowout landslides along the southwest margins of Magnolia and Queen Anne, the flow direction of groundwater perched atop the glaciolacustrine silt and clay may be toward the southwest.

Landslides plotted from the database are conspicuously absent from the north margins of both Magnolia and Queen Anne even though the sand-clay contact surrounds both hills. Many landslides have occurred in Discovery Park, but these were not reported because they have little to no affect on structures or transportation routes.

The east side of Queen Anne represents an area where proper development can increase the stability of a hillside by incorporating proper buttressing and consequent drainage improvements. For example, the undeveloped steep slope west of Westlake Avenue N. is susceptible to landsliding resulting from uncontrolled drainage. Where several condominiums were recently built along Westlake Avenue N., Dexter Avenue N., and Aurora Avenue North, the potential for shallow colluvial sliding has been reduced substantially because of the incorporation of tied-back retaining walls, subsurface drainage, and surface drainage improvements.

12.5 Landslides with Debris Flows

A map showing the distribution of debris flow landslides in the Magnolia/Queen Anne area is presented in Figure B-15. Areas where debris flows are common include Kinnear Park, Perkins Lane W., and along Magnolia Way W. Near Perkins Lane W., the landslides with debris flows generally originate in the depressions along the undulating slope crest of the upper lodgement till bluff. Near Kinnear Park and Magnolia Way, steep slopes with relatively unimpeded runout zones dominate these areas.

12.6 Timing of Landslides

A map of Magnolia and Queen Anne showing the distribution of landslides by decade is presented in B-16. It illustrates that both the west and east sides of Queen Anne and the Perkins Lane W. area of Magnolia are chronic landslide areas.

12.7 Severe Storm-Related Landslides

A map illustrating the distribution of landslides in Magnolia and Queen Anne during the four most notable landslide winters (1933/34, 1971/72, 1986/87, and 1996/97) is presented in Figure B-17. The most notable trend in the quantity and distribution of the severe storm-related landslides in this area is the large number of 1996/97 landslides in Magnolia. Conversely, while the 1933/34 precipitation year is believed to be comparable to that of 1996/97 (based on information received from the City), very few 1933/34 landslides are documented in the database in Magnolia. The severity of the 1933/34 storm was partially responsible for the large number of

WPA projects in Seattle (notice the proximity of the 1933/34 events to the WPA project locations along Perkins Lane W. and Westlake Avenue N. on Figure B-10); therefore, the landslides resulting from this storm may not be sufficiently documented.

12.8 Potential Slide Areas

A map illustrating the coincidence of historical landslides in the Magnolia/Queen Anne study area with the potential slide areas is presented in Figure B-18. Approximately 81 percent of the historical landslides in the Magnolia/Queen Anne area fall within the currently mapped Potential Slide Areas, as described in Section 20.0 of this report. Landslides outside of the Potential Slide Area occurred along the upper and lower slopes along the northern portion of Perkins Lane W. in Magnolia and along the east flank of Queen Anne, west of the existing Potential Slide Areas as indicated in City documents.

12.9 Stability Improvements

This section presents possible stability improvements that could be made by the City to protect utilities, drainage features, streets, and other City facilities in the Magnolia/Queen Anne area. Furthermore, this section includes measures that could be made by adjacent property owners in conjunction with the City to improve the stability of an entire landslide or unstable slope. We further present comments regarding educating private property owners on steps they may take to improve stability.

The Magnolia/Queen Anne area has been divided into nine smaller Stability Improvement Areas where landslide activity has been prevalent, in order to describe various improvements and homeowner education suggestions. As shown on Figure B-19 (Appendix B, Map Folio), the nine areas are as follows:

1) Perkins Lane North
2) Perkins Lane South
3) 32nd Avenue W.
4) W. Galer Street
5) Magnolia Way
6) Kinnear Park
7) West Queen Anne
8) Northwest Queen Anne
9) East Queen Anne

For each area, we summarize the general subsurface conditions, landslide types and causes, and present actions that could be considered for improving stability.

12.9.1 Perkins Lane North

The Perkins North area, located as shown on Figure B-19, is notorious for instability. It consists of those properties in and north of the 1900-block of Perkins Lane W. Properties and instability south of the 1900-block are presented subsequently under the Perkins South Stability Improvement Area.

In the Perkins Lane North area, 111 landslides have been reported. They have occurred throughout the years, being first recorded in 1930 and extending through January 1998. All four types of landslides have been recorded: high bluff peeloff (11), groundwater blowout (4), deep-seated (40), and shallow colluvial (56). The high bluff peeloffs and the groundwater blowouts have been recorded primarily on the uphill side (east) of Perkins Lane. The other two types of landslides (deep-seated and shallow colluvial) have reportedly occurred on both sides of Perkins Lane. On a number of occasions, landsliding has damaged the roadway and frequently debris has come down onto the lane. A number of houses on both sides of Perkins Lane have been destroyed by landslides.

In the 1900-block along Perkins Lane (south end of this designated area), a 110-foot-long portion of the lane was rebuilt with lightweight fill material (bottom ash from Centralia, Washington) in 1983. This work was contracted by homeowners in this area in order to repair a landslide that destroyed a portion of Perkins Lane and prevented vehicle access to properties to the south. A deep subdrain trench was incorporated into this repair effort.

Perkins Lane is located at the western edge of Magnolia, overlooking Puget Sound (refer to Figure B-19). The lane, reportedly constructed in 1926 and 1927, is situated on an uneven midslope bench. From the top of Magnolia Bluff to the east, near the location of Magnolia Boulevard W., the ground surface slopes steeply to precipitously down to the west. The midslope terrace on which Perkins Lane is built slopes moderately to steeply down to the Puget Sound shoreline on the west. A majority of the shoreline beaches are protected by rock seawalls or concrete bulkheads. The right-of-way of Perkins Lane is normally 40 feet wide (locally 60 feet); however, the asphalt-paved lane is rarely wider than 20 feet and no sidewalks are present. Drainage ditches and catch basins are commonly included in the paved section. To the south of W. Raye Street (approximate center of this designated improvement area) a rail and concrete lagging toe wall, about 4 feet high, is locally present.

The subsurface conditions in this area are illustrated on Figure 3-2, Idealized Geologic Conditions, West Magnolia Bluff. As shown, there are five geologic units; however, not all the units are present everywhere and may not be of similar thickness as indicated. Near the south end of this improvement area, Vashon till is exposed in the bluff, but toward the north, the till is absent and advance outwash sand dominates the hillside. The elevation of the sand-clay contact varies along Perkins Lane. Draped over much of the hillside is colluvium (relatively loose), which is commonly thicker at and to the west of Perkins Lane. On the steep, unvegetated portions of the bluffs, soil loosened by weathering is present.

With respect to groundwater seepage, that which occurs at the contact between the recessional outwash (not always present) and Vashon till is minor. The more prolific springs emanate from the sand/clay contact. Seepage can also occur from pervious sand layers within the till or clay units.

The primary factors that contribute to instability in this area are steep to moderately steep slopes, colluvium or weathered soil on the slopes, and high groundwater levels with associated seepage near the sand/clay contact. The predominant triggering mechanism is heavy rainfall with storm/surface water runoff and infiltration.

As a result of the 1996/1997 storms, 16 landslides were identified by the City between the 1900 and 3400 blocks of Perkins Lane W. As a result of these landslides, the City contracted for design and construction of remedial measures, which were made in the latter part of 1997. The stability improvements consisted of drainage improvements, rock buttresses, and catchment/retaining walls. The drainage improvements included finger drains, intercept trench subdrains, springhead drains, and directional drains. The improvements apparently are generally performing as anticipated, although some additional effort is recommended below.

Additional stability improvements that could be considered to protect the lane are catchment/retaining walls at two locations: 3400-block and 2800-block south of W. Barrett Street. Additional finger drains may be appropriate in the 2800-block and 2300-2400 blocks. It is recommended that existing storm drainage facilities be maintained, possibilities for improving drainage explored, and homeowner education take place. In particular, drainage from private properties should be suitably controlled so as not to reduce stability.

12.9.2 Perkins Lane South

This Stability Improvement Area consists of the 1700 and 1800 blocks along Perkins Lane W., extending from Magnolia Boulevard W. (to the east) down to the shore of Puget Sound; refer to Figure B-19. In this area, 17 landslides have been recorded in the Seattle

Landslide Database: listed as high bluff peeloff (7), deep-seated (6), and shallow colluvial (4). The earliest recorded landslide occurred in 1934. Since then, landslides have been reported in the 1960s through 1990s. The high bluff peeloff landslides have occurred primarily from the high bluff on the east side of Perkins Lane. The recorded deep-seated landslides have generally occurred in colluvium located at and west (downhill) of Perkins Lane. One of these deep-seated landslides (1972) damaged the west shoulder of the roadway and was repaired by the City with pit-run sand and gravel backfill. The deep-seated landslides that were recorded in 1996 and for the 1996/1997 storm also involved the movement of bluff soils uphill from the lane. The shallow colluvial landslides also occurred in colluvium located on the downhill side of the residences west of Perkins Lane.

The site topography is generally similar to that described for the Perkins North area. In this Perkins South Stability Improvement Area, Perkins Lane is also situated on a midslope bench. This area slopes from Magnolia Boulevard steeply to precipitously some 75 to 100 feet downward to Perkins Lane on the west. The private properties to the west of Perkins Lane slope moderately to steeply downward an additional 80 to 90 feet (vertical measurement) to the Puget Sound shoreline. Most of the shoreline, except toward the north, is protected by some type of seawall (rocks or timber piles).

The subsurface conditions in this area consist of Vashon till or till-like soils exposed in the bluff to the east of Perkins Lane, and a relatively thick layer (up to 25 feet or more) of colluvium beneath and downslope of Perkins Lane. Both the till/till-like zone, which contains sand layers, and the colluvium overlie hard clay. The Esperance Sand that normally overlies the clay is absent, based on those borings made in this area. Groundwater is present in sand layers within the till and clay, and is also present in the relatively loose colluvium.

Recent instability in this area was first reported to the City in February 1996. The movement detected at that time involved City property at and uphill (east) of Perkins Lane and the southernmost four private properties on Perkins Lane, although subsequent evaluation indicated that slope movements originated in the colluvium to the west of Perkins Lane. As a result of this instability that exhibited slope movement through June of 1996, the City graded the bluff back to a flatter inclination and began to install pumping wells along Perkins Lane in order to reduce groundwater pressures. Before the completion of pumping well installation and additional remedial work, the 1996/1997 storms occurred and slope movement again took place in this area. Separate movement then took place to the north to include two more residential sites at the south end of Perkins Lane. Six houses in the 1700-block of Perkins Lane (no houses are in the 1800-block) have now been destroyed by the landsliding. In general, the slope at and west of Perkins Lane in the 1700 and 1800 blocks has dropped 20 or more feet.

The primary factors that contribute to instability in this area are the loose nature of colluvium at and downslope of Perkins Lane, the steep bluff above (east of) Perkins Lane, and possibly preexisting planes of weakness behind the bluff face. The movements were triggered by heavy precipitation and high groundwater levels. The available data on movement in this area indicate that movement in the colluvium removes lateral restraint for the bluff soils which, in turn, move.

There is litigation in progress with respect to landsliding in this area. Thus, even preliminary improvement recommendations would not be appropriate at this time; however, as a general statement, improvements to stability likely would include subdrainage installations, lightweight fills, retaining walls, etc.

12.9.3 32nd Avenue W.

In the 32nd Avenue W. Stability Improvement Area, as designated on Figure B-19, eight landslides are indicated. High bluff peeloff and shallow colluvial type landslides were reported. The landslides in this area have taken place along the east- and south-facing slope of the ravine, upslope (west) of 32nd Avenue W. and north (upslope) of Logan Avenue W, along the toe of the slope. Instability in this area was reported as early as 1965. The most recent instability took place during the 1996/97 winter storm, when two shallow colluvial and one high-bluff peeloff type landslides reportedly occurred. The high-bluff peeloff landslide was located upslope of Logan Avenue (undeveloped) and impacted the back of a residence along the shoreline of Elliot Bay. Both January 1997 shallow colluvial landslides took place along the east-facing slope uphill of 32nd Avenue.

The topography in this Stability Improvement Area generally dictates the distribution of landslide types. Three high-bluff peeloff landslides occurred (two in 1968 and one in 1997) on the south-facing, near-vertical bluff just north of Logan Avenue. The instability that took place west of 32nd Avenue was typified by shallow colluvial landslides generally resulting from groundwater seepage, surface water runoff, and filling along the top of the slope, east of Magnolia Boulevard W.

The subsurface conditions in this area consist of colluvium on the moderate to steep slopes west of 32nd Avenue, overlying glacially overridden native soils. In some areas, fill material is present, such as for residences along the east side of Magnolia Boulevard, upslope and west of 32nd Avenue. The overridden soils consist of dense to very dense lodgement till over glaciolacustrine silt and clay. Although the sand-clay contact (Tubbs, 1974) is mapped in this area, there is no outwash sand present above the glaciolacustrine silt and clay. Instead, glacial lodgement till directly overlies the silt and clay. In the vicinity of Logan Avenue, there is little to no colluvium on the south-facing steep bluff.

The landslides reported in this area primarily occurred in colluvium, weathered bluff soils, and/or fills placed by private property owners. Two shallow colluvial landslides upslope (west) of 32nd Avenue reportedly were related to saturated, loose soil and triggered by heavy rainfall. Three high-bluff peeloff landslides were reported along the south-facing slope behind the beach houses along Logan Avenue. Landslides resulting from residential fills in the 1500-block of Magnolia Boulevard (1965, 1967, 1986) comprise the remaining three landslides reported in this area. The factors contributing to instability are steep bluffs, loose fill and/or colluvium on the slope, and high groundwater levels/seepage in sand and gravel lenses within the till and lacustrine silt and clay soils. Heavy rainfall that saturates the loose soil generally triggers the failures.

It is recommended that action by the City include surface drainage maintenance and/or improvement and homeowner education. Furthermore, springhead drains could be considered along the east-facing slope west of 32nd Avenue to prevent groundwater seepage from infiltrating the downslope colluvial soils. With respect to the south-facing bluff north of Logan Avenue, there is little the City can do to prevent instability; however, educating the homeowners on the dangers for property and structures at the toe of a steep and unstable bluff should be done. Homeowner education should emphasize the risks of making fills on slopes and present prudent practices, such as maintenance of private drain systems and construction of catchment walls to protect against landslide debris damage.

12.9.4 W. Galer Street

The W. Galer Street Stability Improvement Area consists of a south-facing, steep slope situated as shown on Figure B-19. This slope is the eastern extension of the steep bluff north of Logan Avenue W., as described in the previous section. In this area, ten landslides representing all four types have been recorded, beginning in 1928. Six landslides were listed as shallow colluvial, two as high-bluff peeloff, one as deep-seated, and one as groundwater blowout type. All of the landslides occurred between Galer Street (at the toe of the slope) and Magnolia Blvd (north and uphill of Galer Street). One event, a deep-seated landslide in 1969, undercut the south shoulder of Magnolia Boulevard, which resulted in the City constructing remedial measures. These measures included a 200-foot-long rail and wood lagging wall along the south shoulder of Magnolia Boulevard to protect the street and placement of rubble along the toe of the slope to protect against beach erosion. The most recent instability (March 1997) blocked Galer Street with landslide debris. Five landslides were reported in this area during the 1996/97 winter storm.

The subsurface soils in this area consist of variable thicknesses of colluvium (thinner in the steep bluff areas and thicker at the toe of the slope) overlying glacially overridden native soils. In some areas, existing fill may be present, such as along the downslope margin of Magnolia Boulevard. The overridden soils consist of dense to very dense lodgement till overlying interbedded silt and fine sand, in turn overlying glaciolacustrine clay. Although the sand-clay contact (Tubbs, 1974) is mapped in this area, there is no outwash sand present above the silt and clay. During our field reconnaissance in March 1997, we noted seepage coming out of the interbedded silt and fine sand. Large streams of water were also seen issuing from cracks (joints) in the till about 10 feet below the top of the bluff at the easterly side of the Stability Improvement Area. Furthermore, during the March 1997 field reconnaissance, we noted that surface water at the top of the slope appears to flow toward the southwest corner of Magnolia Park; however, surface water does not flow over the top of the bluff.

The factors that contribute to instability in this area are a combination of groundwater seepage occurring 10 to 20 feet below the top of the slope, and cutting at the toe of the slope (along Galer Street, and by beach erosion). Heavy precipitation appears to be the triggering mechanism for the instability in this area.

It is recommended that the City consider providing drains for the springs located along the slope. Such drainage could consist of springhead drains and a deep trench subdrain north of the top of the slope. It is recommended that homeowner education emphasize maintenance of shore protection along the beach area south of the residences along W. Galer Street, and that the City maintain and/or improve storm drainage facilities along Galer Street and Magnolia Boulevard.

12.9.5 Magnolia Way

The Magnolia Way Stability Improvement Area is the east-southeast-facing slope situated as shown on Figure B-19. In this area, a total of ten shallow colluvial and groundwater blowout landslides have been recorded, beginning in 1940. All of the landslides occurred on the steep slope north of the Magnolia Bridge and east of Magnolia Way W. The most recent instability occurred during the 1996/97 winter storm. The largest was a deep-seated type landslide that destroyed some of the supports for the Magnolia Bridge. This landslide resulted in the City constructing remedial measures including a 30-foot-high, permanent tieback soldier pile wall with concrete lagging located near the top of the original hillside extending for 260 feet north of the bridge. Another, slightly smaller landslide occurred farther north in the vacated W. Blaine Street right-of-way (an east-west street located near the center of the Stability Improvement Area). Stability for the head-scarp of this landslide was improved with a large soldier pile and wood lagging retaining wall, yet surficial sloughing still exists along the north and south margins of the landslide scar (observed during field reconnaissance in July 1998).

The subsurface conditions consist of loose to medium dense colluvium and recessional outwash sand overlying glacially overridden native soils. In some areas, fill may be present, such as for backyards. The overridden soils consist of glacial lodgement till overlying interbedded sand and silt which, in turn, overlies glaciolacustrine silt and clay. The sand–clay contact (Tubbs, 1974) is mapped in this area. Groundwater levels are typically high and groundwater seeps and springs exist throughout this Stability Improvement Area.

The contributing factors to instability are the steep topography, high groundwater levels and associated seepage and springs, and surface water and roof runoff. The landslides were generally triggered by periods of heavy rainfall that resulted in heavy runoff and infiltration into the slope soils.

Recommended action in this area includes homeowner education that strongly emphasizes the proper control of on-site drainage systems, particularly downspout discharge, and discharge of drainage in accordance with government regulations. The City could also consider improving storm drainage systems (curbs and gutters, etc.).

12.9.6 Kinnear Park

The Kinnear Park Stability Improvement Area is the southwest-facing steep slope situated along the southwest side of Queen Anne Hill, as shown on Figure B-19. Twelve landslides have been recorded for this area, generally on the steep slope/bluff above Elliott Avenue W. (located at the toe of the slope) and downslope of 9th Avenue W. and W. Olympic Place (near the top of the slope/bluff) along the east margin of the Stability Improvement Area. All four types of landslides are recorded in this area: deep-seated (2), groundwater blowout (2), shallow colluvial (7), and high bluff peeloff (1). The earliest recorded instability occurred in 1933 (two landslides). The high bluff peeloff-type landslide occurred during the 1996/97 winter storms toward the southern portion of the Stability Improvement Area where the slope is steepest. Debris from this landslide as well as from two other landslides along the southern steep bluff area (downslope of Olympic Place) impacted several structures along Elliott Avenue W. Other areas of instability include several landslides that occurred along the downslope side of 9th Avenue (in the northern portion of the Stability Improvement Area); and along the W. Prospect Street right-of-way (near the center of the Stability Improvement Area). Older landslides (1950 and 1933), reportedly influenced by sewer breaks, occurred along the lower portion of the slope in the vicinity of the VanBuren Avenue W. right-of-way. Previous landslide repairs conducted by the City in this area ranged from simple removal of landslide debris to bio-engineered vegetation mats with interceptor drains (in the vicinity of W. Prospect Street).

The subsurface conditions consist of variable thicknesses of colluvium overlying glacially overridden native soils. In some areas, existing fill may be present, specifically in the northern portion of the Stability Improvement Area. The glacially overridden soils consist of an upper layer of lodgment till (jointed) overlying interbedded silt and gravelly, fine sand which, in turn, overlies glaciolacustrine clay. Abundant groundwater seepage occurs from the interbedded silt and sand zone. The moderate to steep slope west of 9th Avenue exhibits active signs of creep and appears to be an ancient landslide scar. Development in this area has included various amounts of fill material that appears to contribute to the instability west of 9th Avenue.

The factors that contribute to instability in this area are steep topography (specifically in the southern portion of the Stability Improvement Area), high groundwater levels/seepage, and cuts and/or fills (particularly in the northern portion of the area). Based on field reconnaissance shortly after the 1996/97 winter storms, there were little to no signs of surface water runoff over the top of the steep bluff area.

Stability improvements that the City could consider consist of interceptor-trench subdrain installation, storm drainage systems maintenance and/or improvement, and homeowner education. These improvements are discussed further in the following paragraphs.

Measures the City could consider to reduce the rate of bluff regression include constructing an MSE wall, a geotextile-reinforced soil slope, or flattening the slope face in combination with an interceptor trench drain. Constructing an MSE wall or reinforced soil slope would be a long-term solution to bluff regression above the wall base elevation, and would be less expensive than a concrete pile wall. An alternative solution would be to flatten the slope face in conjunction with installing an interceptor trench north of the slope crest. This latter alternative should provide sufficient groundwater drainage while reducing the volume of excavated and imported material and, thus, is less expensive than the other two alternatives. With respect to reducing the risk of damage when landslides occur, the City could consider removal of selected trees that may impact structures along Elliott Avenue and removal of precariously perched soil blocks on the slope (coordinate with private property owners). Private parties planning new construction or stability improvements for existing homes in the northern portion of the Stability Improvement Area (west of 9th Avenue) should obtain professional advice. Retaining/catchment walls could also be considered along the toe of the slope to protect downhill properties adjacent to Kinnear Park and along Elliott Avenue. It is recommended that homeowner education emphasize proper control of on-site drainage systems, particularly in the north half of the Stability Improvement Area.

12.9.7 West Queen Anne

The West Queen Anne Stability Improvement Area is the west-facing slope situated north of the Kinnear Park Stability Improvement Area, as shown on Figure B-19. The hillside has a steep upper and lower slope separated by a mid-slope bench. The hillside above the bench is about 100 feet high and has slopes between 20 and 50 degrees with the horizontal. The steepest part of the slope is at the top, just below the houses along 11th Avenue W. and 12th Avenue W. The hillside below the bench is also about 100 feet high and has slopes between 25 and 45 degrees.

In this area, a total of 23 landslides have been recorded, beginning in 1909. Three types of landslides are documented in this area, consisting of: 4 groundwater blowout, 7 deep-seated, and 12 shallow colluvial landslides. Some of the landslides occurred on the steep slope just below 11th and 12th Avenues, while others took place on the steep slope below the mid-slope bench. Instability during the 1996/97 winter storms occurred on both the upper and lower slopes in this Stability Improvement Area. In particular, 1996/97 landslides affected the City (Seattle Parks Department) maintenance facility along 15th Avenue W., the intersection of W. Galer Street and 11th Avenue, and a soldier pile retaining wall behind a residence along 11th Avenue (south of the intersection of Galer Street). In 1989, several horizontal drains were drilled and connected to a catch basin located mid-slope in the vicinity of 12th Avenue to improve stability of the slope for new residences near W. Blaine Street (near the center of the Stability Improvement Area). The City performed repairs to 12th Avenue (at W. Garfield Street), where a reactivation of the 1951 deep-seated landslide occurred during the 1996/97 storm.

In 1998, deep-seated instability occurred between Garfield Street and Galer Street, and extends down to the Magnolia (Garfield Street) Bridge on-ramp. A combination of subsurface drainage improvements, retaining walls, and grading has been implemented by the City in this area.

The subsurface soils in this area consist of colluvium and recessional outwash sand overlying glacially overridden native soil. The thickness of the colluvial layer varies considerably over the slope; it is as thin as 1.5 feet along the steep slopes and as thick as 13 feet at the midslope bench and at the toe of the lower slope. The overridden soils consist of lodgment till and outwash sand in the upper steep slope, which overlies glaciolacustrine silt and clay. The top of the clay unit occurs at the mid-slope bench. Abundant groundwater seeps and springs exist at the top of the lower slope at the edge of the bench. Water also ponds along portions of the mid-slope bench, the overflow of which was reported to have contributed to several landslides along the lower slope.

Contributing factors to instability are steep topography, loose soil conditions, high groundwater levels/seepage and springs, pond overflow from the bench onto the lower slope, residential and road fills along the top of the upper slope, and heavy precipitation (triggering cause). Furthermore, based on a review of previous Shannon & Wilson reports, abandoned pipes and tightlines discharging storm-water runoff onto the mid-slope bench from upslope sources may also contribute to instability.

Stability improvements that the City could consider in this area consist of interceptor trenches, control of surface water runoff onto downslope areas, and homeowner education. Several springhead drains could be installed at points of known seepage, although the installation of a deep interceptor trench along the outer edge of the midslope bench would be a more positive method of improving long-term stability of the lower slope area. The City could consider installation of a catchment/retaining wall at the toe of the lower clay slope to protect City utilities and other structures from damage by landslide debris originating from the bench or along the lower slope. Improperly directed surface water runoff, including discharge from tightlines and other abandoned utilities, should be eliminated if found. Ponded water on the midslope bench should also be eliminated. Homeowner education should emphasize proper control of on-site drainage systems, specifically eliminating downspout discharge onto downslope soils.

12.9.8 Northwest Queen Anne

The Northwest Queen Anne Stability Improvement Area is the north-facing slope located at the northern tip of Queen Anne Hill, as shown on Figure B-19. In this area, a total of 14 shallow colluvial and deep-seated landslides have been recorded, beginning in 1922. Some of the landslides occurred on the moderate to steep slope between W. Emerson Street (uphill and south of W. Nickerson Street) and W. Nickerson Street (at the toe of the slope). Others took place on the northwest-facing slope between 13th Avenue W. (generally uphill and east of 15th Avenue W.) and 15th Avenue W. (west and near the toe of the slope). The most recent instability occurred along Nickerson Street in October 1997 along the edge of an unimproved alley south of a residence. Approximately 10 of the recorded landslides in this Stability Improvement Area were reported to be related to improper fills and/or cuts by both public and private property owners. For example, the 1922 landslide (located at the northern tip of the Stability Improvement Area) was reportedly related to the grading of Nickerson Street, which resulted in the City constructing a rail (trolley) and concrete lagging toe wall along the south margin of Nickerson Street. Other reported landslides include a rockery failure behind an apartment building, instability related to the grading of Emerson Street without proper slope retention, and several reactivations of a large deep-seated landslide in the alley west of 13th Avenue.

The subsurface conditions consist of colluvium on the moderate to steep slopes overlying glacially overridden outwash sand and glaciolacustrine silt and clay. In many areas, fill may be present, such as for roads and private structures. The sand-clay contact (Tubbs, 1974) is mapped in this area. The construction of Nickerson Street and Emerson Street likely included several cuts along the south margins of the roadways, and Emerson Street may have some fills along the northern margin. The factors that may contribute to instability in this area are cuts and fills made by both public and private property owners, and high groundwater levels and associated seepage.

Homeowner education would be appropriate to emphasize obtaining professional advice for improving stability for existing homes, additions, or new construction, and controlling on-site drainage.

12.9.9 East Queen Anne

The East Queen Anne Stability Improvement Area consists of an east-facing steep slope generally between Dexter Avenue N. (uphill and west of Westlake Avenue N.) and Westlake Avenue N. located at the toe of the slope (east of Dexter Avenue N.); refer to Figure B-19 for location. Twenty landslides have been recorded for this area, all of which are located on the steep slope above Westlake Avenue N. Nineteen landslides were listed as shallow colluvial and one was deep-seated. The earliest recorded landslide was in 1926, and instability has been reported through the years. Five shallow colluvial landslides occurred during the 1996/97 storm, and at least two of them deposited debris onto the southbound lanes of Westlake Avenue. Three landslides along Dexter Avenue (2500-block) were reported in 1933, 1954, and 1969 to be related to the grading of Westlake Avenue by the City in 1920. Subsequent instability along the downslope side of the 2500-block of Dexter Avenue was reported in 1978, 1982, 1986, and 1997.

The subsurface conditions in this area consist of a silt-clay colluvium overlying stiff to hard clay. Groundwater levels are typically high because this area is at or near the toe of the slope. The sand-clay contact (Tubbs, 1974) is mapped west (upslope) of this Stability Improvement Area. The contributing factors to instability are the steep topography, the soil conditions on this slope (colluvium over stiff to hard clay), undercutting or filling on the slope, and high groundwater levels/seepage. The landslides were triggered by periods of heavy rainfall that resulted in surface runoff and infiltration into the slope soils.

Stability improvements the City could consider consist of a deep interceptor-trench subdrain installation (locally), catchment/toe wall installation, and homeowner education. For example, installation of a deep interceptor-trench subdrain extending south along portions of 8th Avenue and 9th Avenue (rights-of-ways that are closest to the slope crest), keyed into the stiff to hard, overridden silt and clay, may be effective in reducing groundwater levels downslope locally. In addition to reducing groundwater levels, a method for decreasing the landslide risk to Westlake Avenue would be a catchment/toe wall along the west side of Westlake Avenue in areas most frequently impacted by sliding. It is recommended that homeowner education emphasize minimizing cuts and fills along the slope as well as properly controlling on-site drainage, including downspouts and surface water runoff.

13.0 MADRONA

Madrona represents one of the oldest neighborhoods in Seattle. Therefore it has some of the oldest recorded landslides in the database.

13.1 Site Description

The Madrona study area is located along Lake Washington in east Seattle. The study area extends from E. Madison Street to Coleman Park, located south of Interstate 90. Madrona lies on the east-facing slope of Mount Baker Ridge, which is an elongated ridge that extends south from Madison Park to Mt. Baker Park (south of Colman Park). The Madrona area is moderately incised by several short, steep gullies that are occupied by intermittent streams in Colman Park, Madrona Park, Leschi Park, and Frink Park. The slopes of Madrona are flatter than the West Seattle and Magnolia/Queen Anne study areas. Because Madrona is located along the relatively placid Lake Washington, there are no high bluffs or near-vertical slopes resulting from shoreline erosion at the base of the slopes. Furthermore, both Lake Washington Boulevard and Lakeside Avenue protect the west shoreline of Lake Washington from erosion. Portions of these roads and at least one row of residences or a strip park are located on the former lake bottom that was exposed when the lake level was lowered about 10 feet in 1916 for the Lake Washington Ship Canal project. Refer to Figures B-20 through B-28 for Site Plans of the Madrona Area.

13.2 Soil Stratigraphy

The soils that underlie the Madrona study area are products of the most recent glaciation of the central Puget Lowland. Because the Seattle Fault extends through the Madrona study area, the depth to Tertiary bedrock increases from less than 300 feet on the south side of the fault to 1,000 to 3,000 feet on the north side of the fault. There are no bedrock outcrops in the Madrona study area. The primary geologic units involved with landsliding in the Madrona area are the pre-Vashon glacial and nonglacial deposits, Vashon glacial deposits, and colluvium.

13.3 Groundwater

Groundwater plays a key role in slope instability in the Madrona study area. The contact between advance outwash (Esperance) sand and glaciolacustrine silt and clay (Lawton Clay) is between elevations 160 feet and 250 feet and extends roughly from Madison Park, south to Colman Park (refer to Figures B-21 through B-28). Because most slopes in Madrona are moderately inclined and have thick accumulations of colluvium, groundwater from the sand-clay contact generally does not flow to the surface, but rather moves downslope within the colluvium.

13.4 Landslide Types

13.4.1 High Bluff Peeloff Landslides

Because the Madrona study area has no steep bluffs, there are no high bluff peeloff landslides documented in the database for this area.

13.4.2 Groundwater Blowout Landslides

Only one groundwater blowout landslide is documented in the Madrona study area (refer to Figure B-21). Although this landslide does not occur near the sand-clay contact, its elevation is consistent with the elevation of the top of the impermeable clay/silt unit. Furthermore, groundwater does not necessarily emerge from a single stratigraphic horizon due to gradational changes between advance outwash and lacustrine units. Based on the spatial distribution of the shallow colluvial landslides in the Madrona study area with respect to the sand-clay contact (see Figure B-23), the effect of groundwater on landsliding is significant, in our opinion. An explanation for the lack of documented historical groundwater blowout landslides in the Madrona area may, therefore, be that springs and seeps are rarely observed emerging from discrete points along the sand-clay contact. Water from the sand-clay contact apparently travels along the colluvium-clay contact, contributing to landsliding of the colluvium. The location of the resulting landslide may therefore be some distance downslope of the sand-clay contact. Without exposures of the glacial soils or other evidence of groundwater seepage, it is difficult to identify a groundwater blowout landslide.

13.4.3 Deep-Seated Landslides

A map illustrating the distribution of deep-seated landslides in the Madrona study area is presented in Figure B-22. The highest densities of deep-seated landslides in the Madrona study area occur in the vicinity of South Dose Terrace and the 1100 block of Lake Washington Boulevard S. With the exception of a deep-seated landslide that occurred in 1959 on S. Judkins Street, all the deep-seated landslides in these two areas occurred prior to 1936. The conspicuous absence of deep seated landslides after 1940 may be a result of increased development and related surface and subsurface drainage improvements.

13.4.4 Shallow Colluvial Landslides

Shallow colluvial landslides in the Madrona study area are shown on Figure B-23. This type of slope instability is the most common in Madrona. The highest densities of shallow colluvial landslides are located near the 1200 block of Lakeside Avenue S., Lake Dell Avenue E., Madrona Drive, and McGilvra Boulevard E.

Based on "The Preliminary Geologic Map of Seattle and Vicinity, Washington" (Waldron, and others, 1962), nearly all of the historical shallow colluvial landslides in the Madrona area occur on slopes underlain by glaciolacustrine silt and clay. In other words, nearly all of the shallow colluvial landslides documented in the Madrona area lie at, or topographically below, the sand-clay contact, as shown on Figure B-23.

13.5 Landslides with Debris Flows

Because the slopes in the Madrona study area are flatter than those in the other two study areas, the occurrence of debris flows is limited. A map showing the distribution of landslides with debris flows in the Madrona study area is presented on Figure B-24. The highest concentration of debris flows in the Madrona study area occurs in the 200-block of Lake Dell Avenue E. where the undeveloped slopes are among the steepest and longest in Madrona.

13.6 Timing of Landslides

A map displaying the distribution of all landslides by decade in the Madrona area is shown on Figure B-25. It suggests that the likelihood of landsliding in the Madrona area has not changed over time. Shallow colluvial landsliding is the primary mode of ground failure in Madrona in more recent years.

13.7 Severe Storm-Related Landslides

A map showing the distribution of landslides resulting from the four major storm events occurring in the Madrona study area in the past century (1933/34, 1971/72, 1986/87, and 1996/97) is shown on Figure B-26. Landslides from the winter 1986 storm account for nearly 50 percent of the total number.

13.8 Potential Slide Areas

A map displaying the distribution of historic landslides in the Madrona study area with respect to the existing City maps showing Potential Slide Areas, as described in Section 20.0 of this report, is shown on Figure B-27. It shows that most of the landslides occurred within the existing Potential Slide Areas (79 percent). Exceptions include landslides in the vicinity of the 200-block of Lake Dell Avenue E., south of Colman Park, and near the 2000-block of Lake Washington Boulevard S.

13.9 Stability Improvements

This section presents possible stability improvements that could be made by the City to protect utilities, drainage features, streets, and other City facilities. Measures are also presented that could be made by the City and adjacent property owners to improve the stability of an entire landslide or unstable slope. We present further comments regarding educating private property owners on steps they may take to improve stability.

In order to describe various improvements and homeowner education suggestions, the Madrona area has been divided into seven smaller Stability Improvement Areas, where landslide activity has been prevalent. As shown on Figure B-28 (Appendix B, Map Folio), the seven areas are as follows:

1) Hillside Drive
2) 32nd Avenue E.
3) Madrona Drive
4) Madrona Park
5) Lake Dell
6) Lakeside North
7) Lakeside South

For each area, we will summarize the general subsurface conditions, landslide types and causes, and present actions that could be considered for improving stability.

13.9.1 Hillside Drive

For the Hillside Drive Stability Improvement Area, as designated on Figure B-28, five landslides are indicated. Deep-seated (2), shallow colluvial (2), and groundwater blowout (1) landslides have been recorded in this area since 1946. The landslides in this area have taken place on the east-facing slope between 36th Avenue E. (uphill and west of Hillside Drive E.) and Hillside Drive E., as well as along the east shoulder of Hillside Drive E. The most recent instability took place approximately 50 feet downslope of the 600-block of 36th Avenue and flowed across Hillside Drive in the spring of 1997. The debris from this event also impacted several properties on the downhill (east) side of Hillside Drive.

The landslides that occurred in this area prior to 1950 were related to instability along the downhill (east) side of Hillside Drive, possibly in fill either placed for residences or during the grading of Hillside Drive. During our field reconnaissance for this phase of the study, we observed settlement and areas of ponded water in the same location as the pre-1950 landslides along the east portion of Hillside Drive.

The subsurface conditions in this area consist of a silt-clay colluvium over glacially overridden native deposits. The glacial deposits consist of an outwash sand unit and a lower glaciolacustrine clay unit. Although the sand-clay contact is not mapped in this area, several groundwater seeps and springs exist along the west (uphill) side of Hillside Drive E. The factors contributing to instability are the soil conditions on the slope (as much as 25 feet of silt-clay colluvium), high groundwater levels with associated seeps and springs, settlement of fill, and ponded water along the east side of Hillside Drive E.

To improve stability for the Hillside Drive area, we recommend that the City consider improving the surface drainage systems along Hillside Drive, installing springhead and/or finger drains, and promoting homeowner education. Replacement and compaction of existing fill areas and installation of a new curb along the east side Hillside Drive may be the most cost-effective way to reduce ponding, runoff, and infiltration of surface water on downslope areas. We further recommend that the City consider recording the location and amount of paving placed along Hillside Drive in order to evaluate specific areas along Hillside Drive where fill settlement is a potential problem. Springhead and/or finger drains could be effective in reducing instability associated with groundwater seeps and springs along the uphill side of Hillside Drive. It is recommended that homeowner education emphasize prudent construction practices and controlling on-site drainage systems.

13.9.2 32nd Avenue E.

In the 32nd Avenue E. Stability Improvement Area, as designated on Figure B-28, six landslides are indicated. Both deep-seated and shallow colluvial landslides occurred. The landslides in this area have taken place along the west-facing slope generally between 34th Avenue E. (uphill and east of 32nd Avenue E.) and 32nd Avenue E. near the toe of the slope. Instability in this area was reported as early as 1910, shortly after the grading of 32nd Avenue in 1907. One landslide reportedly was related to piping (soil movement) in trench fill for the water service to a residence in the 300-block of 34th Avenue. The most recent instability took place during the severe winter storm in 1972, which damaged several houses along the west side of 33rd Avenue E. in the 1700 block. This landslide was reported to be related to an excavation downslope of the affected residences.

The subsurface conditions consist of relatively thick colluvium on the moderately-steep slopes overlying glacially overridden native soils. The sand-clay contact extends through this area. The original construction of 32nd Avenue likely included cutting into the toe of the slope between E. Denny Way and E. John Street. After several failures of a wood bulkhead prior to 1941, the City constructed a 4- to 6-foot-high concrete bulkhead/toe wall along much of the east side of 32nd Avenue between Denny Way and John Street.

The factors that contribute to instability in this area are the thick colluvium, high groundwater levels with associated seepage along the sand-clay contact in the vicinity of the 33rd Avenue right-of-way, piping of soil in trench fill, and cutting and filling.

In the long term, there does not appear to be any additional remedial measures that the City could take to prevent the natural occurrence of landsliding in this area other than an evaluation of the existing bulkhead along 32nd Avenue, particularly where the wall appears to be slightly bowed, and homeowner education.

13.9.3 Madrona Drive

The Madrona Drive Stability Improvement Area is the east-facing slope situated as shown on Figure B-28. In this area, six landslides have been reported since 1935 in the 1500-block along the east side of Madrona Drive. Two other landslides occurred in 1951 and 1960 just uphill from Lake Washington Boulevard E. Some of the landslides were related to improper fills along the east side of Madrona Drive in the 1500 block, and others were related to groundwater seepage. Two shallow colluvial landslides during the 1960s (the two youngest slides reported in this area) damaged structures located in a small ravine at the toe of the slope.

The subsurface conditions consist of an estimated 5 to 15 feet of colluvium on the steep slopes overlying glacially overridden outwash sand and lacustrine silt and clay. Although the sand-clay contact (Tubbs, 1974) is mapped at higher elevations and west of this Stability Improvement Area, groundwater seepage, likely from pervious zones within the silt/clay soils, exists along the steep slope east of Madrona Drive. In some areas, fill may be present along the east side of Madrona Drive. Settlement along the east margin of the street was observed during our field reconnaissance in October 1998 where the curb and sidewalk settled to below the street surface in some locations. This settlement permits surface water from Madrona Drive to flow onto downslope properties. In 1936, the WPA performed extensive drainage work in the vicinity of the 1500 block of Madrona Drive.

The factors that contribute to instability in this area are fills along the east side of Madrona Drive, steep topography, and high groundwater levels/seepage. The triggering mechanism is heavy rainfall with surface water runoff and infiltration.

Stability improvements that the City could consider consist of improving storm drainage and educating homeowners. For a distance of about 500 feet, drainage improvements could be considered along the east margin of Madrona Drive (curbs/gutters/catch basins, etc.). It is recommended that homeowner education emphasize the management of surface water using suitable drainage systems (such as discharge to storm drains) to reduce the risk of shallow colluvial and deep-seated landslides that may damage downslope and upslope properties.

13.9.4 Madrona Park

In the Madrona Park Stability Improvement Area, located northwest of Madrona Park, six shallow colluvial landslides are indicated, refer to Figure B-28. The landslides took place along the east-facing slope between 36th Avenue (two blocks west and uphill of Madrona Park) and 38th Avenue/Newport Way, located near Madrona Park. Instability in this area was reported as early as 1914. The two most recent landslides in this area took place in the winter of 1986. A contributing cause of one of these landslides was reported to be surface water runoff from the storm drain at the 38th Avenue dead-end. Infiltration of this surface water into the colluvial soils adjacent to a residence apparently triggered a shallow colluvial landslide that undermined the foundation of a residence. The property owner obtained professional geotechnical advice and constructed a 6- to 10-foot-high concrete retaining wall in the vicinity of the headscarp.

The two landslides that occurred in this area prior to 1960 were reported to have been related to excavations for Newport Way (December 1914) and 38th Avenue (December 1956). The instability that took place following 1960 was generally related to storm-water drainage and over-watering during the summer.

The subsurface conditions in this area consist of silt-clay colluvium located over glacially overridden outwash sand and glaciolacustrine clay. The sand-clay contact is mapped in this area, generally between 36th and 38th Avenues. Abundant groundwater seepage was observed along Newport Way and 38th Avenue during our field reconnaissance in October 1998. The factors contributing to instability are the wet soil conditions on the slope (high groundwater levels/seepage), undercutting of the slope, and surface water runoff, specifically near the stairway along the 38th Avenue right-of-way extending down to Newport Way.

Recommended actions for consideration in this area include storm drainage systems maintenance and/or improvement, installation of springhead and/or finger drains, and homeowner education. The storm drain and surface water catchment area located at the dead end of 38th Avenue could be improved by increasing the capacity of the catch basin to prevent overflow of ponding water onto the downslope stairway area. Springhead and/or finger drains installed along the west (uphill) side of Newport Way and 38th Avenue (located just west of Madrona Park) could be effective in reducing instability associated with groundwater seeps and springs. It is recommended that the gutter area along the west side of Newport Way be cleared of vegetation and other debris to facilitate proper drainage. Homeowner education is recommended to specifically address the need to check and clean vegetation and other debris from public stormdrains in the vicinity of their homes.

13.9.5 Lake Dell

The Lake Dell Stability Improvement Area consists of a south- and east-facing steep slope situated as shown on Figure B-28. In this area, 16 shallow colluvial and deep-seated landslides have been recorded beginning in 1897. Some of the landslides occurred upslope of Lake Dell Avenue E. and others took place on the moderately-steep slope downhill. Lake Dell Avenue, cut prior to 1920, traverses this steep slope. The most recent instability occurred uphill from Lake Dell Avenue in March of 1997, related to a cut made for a private driveway in the 200-block. Eight of the landslides in this area reached Lake Dell Avenue and at least four of these blocked portions of Lake Dell Avenue.

The subsurface conditions consist of colluvium on the steep slopes overlying glacially overridden outwash sand and glaciolacustrine silt and clay. The sand-clay contact (Tubbs, 1974) extends through this area. In some areas, fill may be present, such as for backyards. The original construction of Lake Dell Avenue likely included some cuts along the west and north side and filling along the east and south. Two 1933 landslides that occurred near the 100-200 block of Lake Dell Avenue were attributed to surface water runoff onto downslope areas; however, we observed adequate curbs and gutters along the east and south side of Lake Dell Avenue during our field reconnaissance in October 1998.

The factors that contribute to instability in this area are the steep topography, road and private property cutting and/or filling, high groundwater levels and associated seepage near the sand-clay contact, and private storm-water discharge. Many of the reviewed landslides were reportedly triggered by periods of heavy rainfall that resulted in surface runoff and infiltration into the slope soils.

Recommended action for consideration by the City in this area includes retaining/ catchment wall construction, storm drainage systems maintenance and/or improvement, springhead drains installation, and homeowner education. A retaining/catchment wall along Lake Dell Avenue on the oversteepened, upslope side of the road could be effective for increasing upslope stability and preventing landslide debris from blocking the roadway. Several springhead drains placed at known points of groundwater seepage and springs could be effective in reducing instability along the uphill side of Lake Dell Avenue. It is recommended that homeowner education emphasize management of on-site drainage systems to prevent improperly directed surface water runoff.

13.9.6 Lakeside North

The Lakeside North Stability Improvement Area consists of an east-facing moderately-steep slope generally between 35th Avenue S. (uphill and west of Lakeside Avenue S.) and Lakeside Avenue S. at the toe of the slope, along the shore of Lake Washington, as shown on Figure B-28. In this area, a total of eight landslides are recorded, consisting of three deep-seated and five shallow colluvial landslides, since 1928. The most recent instability occurred in 1991, which was related to an excavation by a private property owner downhill from the 300-block of 35th Avenue. Other instability was reported in the vicinity of the stairway in the S. Jackson Street right-of-way in 1948 and 1958. In the vicinity of S. Leschi Place and Lakeside Avenue S., two landslides were reported, one of which overtopped the concrete bulkhead on the west side of Lakeside Avenue in 1991.

The subsurface conditions in this area consist of a silt-clay colluvium located over stiff to hard clay. The sand-clay contact is located upslope of this Stability Improvement Area. The contributing factors to instability are the soil conditions on this slope (colluvium over stiff to hard clay), road cuts, and groundwater seepage. A 15- to 20-foot-high concrete retaining wall exists along the road-cut on the west side of the 400-block of Lakeside Avenue. Above the wall, west of Lakeside Avenue, an upper road runs parallel to Lakeside Avenue. The cut-slope along the west side of the upper road was wet from high groundwater levels/seepage and appeared to be unstable in the vicinity of the Jackson Street stairway.

Stability improvements that the City could consider consist of wall construction and homeowner education. A catchment/retaining wall along the west side of the upper roadway in the vicinity of the Jackson Street right-of-way could be considered along with improvement of the surface drainage in this area. A catchment/retaining wall along the cut-slope along the 300-block of Lakeside Avenue, north of Leschi Place, could also be considered to prevent landslide debris from blocking Lakeside Avenue. It is recommended that homeowner education emphasize prudent construction practices and management of on-site drainage systems.

13.9.7 Lakeside South

The Lakeside South Stability Improvement Area consists of the east-facing slope as shown on Figure B-28. In this area, deep-seated and shallow colluvial landslide types make up 16 of the 17 total landslides reported since 1925. One landslide was not identified as to its type. The landslides in this area took place along the east-facing slope generally between 32nd Avenue S. (uphill and west of Lake Washington Boulevard S.) and Lakeside Avenue S. (at the toe of the slope, east of Lake Washington Boulevard S.). There are two general areas of instability. The first area, upslope of Lake Washington Boulevard (1100-block), consists of mostly pre-1955, deep-seated type landslides. The second area, located upslope of Lakeside Avenue (1300-block), consists of both recent and older, deep-seated, and shallow colluvial landslides. The most recent instability was a reactivation of a 1983 landslide that took place along the 1300-block of Lakeside Avenue in 1986. Upslope of the 1100-block of Lake Washington Boulevard, several old (pre-1940) deep-seated landslides reportedly affected structures along the shore of Lake Washington. The WPA performed extensive subsurface drainage work in the vicinity of S. Judkins Street and Lake Washington Boulevard S. during the 1930s. With the exception of three records of instability during the 1986 winter storm, the Lakeside South Stability Improvement Area has been relatively stable for the past 15 years.

The subsurface conditions in this area consist of colluvium overlying glacially overridden glaciolacustrine silt and clay. The sand-clay contact (Tubbs, 1974) is mapped uphill from Lake Washington Boulevard. The contributing factors to instability are the high groundwater levels with associated seepage and wet soil conditions, cutting at the toe of the slope and filling near the top, and probable pre-existing, ancient landslide blocks in the vicinity of the 1100-block of Lake Washington Boulevard S.

To improve the stability of the Lakeside South Stability Improvement Area, the City could consider surface drainage maintenance and/or improvement, catchment/retaining wall construction, finger drains or springhead drains, and homeowner education. It is recommended that the storm-water gutter along Lakeside Avenue, at the toe of the slope, be cleaned and maintained. Abundant groundwater seepage exists in this area. Therefore, several finger drains or springhead drains along Lakeside Avenue could be effective in reducing groundwater seepage along the slope. A drained catchment/retaining wall could be constructed at the toe of the steep slope along the west side of the 1300-block of Lakeside Avenue, to prevent shallow colluvial and occasional deep-seated landslides from encroaching onto the southbound lanes of Lakeside Avenue. It is recommended that homeowner education emphasize proper construction methods, and maintenance of on-site private and public storm drainage systems.

Stability Improvement Area [1,2]

West Seattle

Stability Improvement Area [1,2]	Unit [10]	23rd Avenue S.W.	Admiral Way	Fairmount Gulch	Harbor Avenue	Alki Avenue	Boyd/Chilberg Place	Jacobsen Road	Beach Drive/Atlas Place	47th Avenue S.W.	Seola Beach
Number of Landslides											
High Bluff Peeloff											
Groundwater Blowout					5	7		1		1	
Deep-seated		10	9		8	11	4	6	6	1	2
Shallow Colluvial		6	17	10	48	86	3	11	19	19	4
Unidentified		8		1		2					
Total		24	26	11	61	106	7	18	25	21	6
Subsurface Conditions [3]											
Colluvium Over Glacially Overridden Clay		X									
Colluvium Over Glacially Overridden Sand and Gravel											X
Colluvium Over Glacially Overridden Sand-Clay			X	X	X	X	X	X	X		
Colluvium Over Glacially Overridden Till-Sand-Clay										X	
Colluvium Over Glacially Overridden Till-Clay											
Sand-Clay Contact (Tubbs, 1974) Mapped in Area			X	X	X	X	X	X	X	X	
Contributing Causes of Instability											
Steep Topography			X	X	X	X	X	X	X	X	X
Loose Fill or Colluvium on Slope				X		X	X	X	X		X
Colluvium Over Clay		X	X		X						
High Groundwater Levels (Seepage and Springs)		X	X	X	X	X	X	X		X	
Road Cuts and Fills (Public)			X			X	X	X	X	X	
Undercutting and Filling (Private)		X	X		X	X	X	X	X	X	X
Improperly Directed Surface Water			X						X		
Heavy Rainfall with Surface Runoff (Trigger Mechanism)		X	X	X	X	X	X	X	X	X	X
Stability Improvements	**Unit [10]**										
Homeowner Education		X	X	X	X	X	X	X	X	X	X
Strom Drain Maintenance/Improvement (Curbs/Gutters/Catchbasins)		X	X	X	X	X	X	X	X	X	
Trench Subdrains (10 ft deep)	LF							1,100	1,400		
Trench Subdrains (15 ft deep w/ trenchbox)	LF		1,100		1,500	1,600		900			
Finger Drains	EA	10	5								
Springhead Drains	EA			5	5	5			5		
Mechanically Stabilized Earth Wall [4, 8]	SF										
Geotextile Reinforced Soil Slope [5]	SF										
Combined Flattened Slope and Interceptor Trench [6]	LF										
Slope Grading (Excavation)	CY										
Machine Formed Curbs	LF							1,600	3,800	1,200	
Retaining/Catchment Wall (10 ft high)	SF		23,000		20,000	36,000			9,000	8,000	
Fill Stabilization-Excavation and Replacement (20 ft wide, 7 ft deep) [7]	CY				2,100				2,900		
Excavation	CY				2,100				2,900		
Soil Backfill and Compaction	CY				2,100				2,900		
Asphalt Paving	SY				900				1,250		
Machine Formed Concrete Curbs	LF				400				560		
Drainage Improvements [9]											

Magnolia/Queen Anne

Stability Improvement Area [1,2]	Unit [10]	Perkins Lane North	Perkins Lane South	32nd Avenue W.	W. Galer Street	Magnolia Way	Kinnear Park	West Queen Anne	Northwest Queen Anne	East Queen Anne
Number of Landslides										
High Bluff Peeloff		11	7	3	2		1			
Groundwater Blowout		4			1	1	2	4		
Deep-seated		40	6		1		2	7	7	1
Shallow Colluvial		56	4	5	6	9	7	12	7	19
Unidentified										
Total		111	17	8	10	10	12	23	14	20
Subsurface Conditions [3]										
Colluvium Over Glacially Overridden Clay		X								X
Colluvium Over Glacially Overridden Sand and Gravel										
Colluvium Over Glacially Overridden Sand-Clay		X							X	
Colluvium Over Glacially Overridden Till-Sand-Clay		X			X	X	X			
Colluvium Over Glacially Overridden Till-Clay			X	X				X		
Sand-Clay Contact (Tubbs, 1974) Mapped in Area		X	X	X	X	X	X	X	X	
Contributing Causes of Instability										
Steep Topography		X	X	X		X	X	X		X
Loose Fill or Colluvium on Slope		X	X	X			X	X		
Colluvium Over Clay		X	X							X
High Groundwater Levels (Seepage and Springs)		X	X	X	X	X	X	X	X	X
Road Cuts and Fills (Public)					X			X	X	X
Undercutting and Filling (Private)							X	X	X	X
Improperly Directed Surface Water					X	X		X		X
Heavy Rainfall with Surface Runoff (Trigger Mechanism)		X	X	X	X	X	X	X		X
Stability Improvements	**Unit [10]**									
Homeowner Education		X		X	X	X	X	X	X	X
Strom Drain Maintenance/Improvement (Curbs/Gutters/Catchbasins)		X		X	X	X	X	X		X
Trench Subdrains (10 ft deep)	LF				350					
Trench Subdrains (15 ft deep w/ trenchbox)	LF							850		800
Finger Drains	EA	12								
Springhead Drains	EA	6		8-10	7			5		
Mechanically Stabilized Earth Wall [4, 8]	SF						6,250			
Geotextile Reinforced Soil Slope [5]	SF						6,250			
Combined Flattened Slope and Interceptor Trench [6]	LF						250			
Slope Grading (Excavation)	CY									
Machine Formed Curbs	LF					1,300				
Retaining/Catchment Wall (10 ft high)	SF	7,000					7,000			25,000
Fill Stabilization-Excavation and Replacement (20 ft wide, 7 ft deep) [7]	CY									
Excavation	CY									
Soil Backfill and Compaction	CY									
Asphalt Paving	SY									
Machine Formed Concrete Curbs	LF									
Drainage Improvements [9]										

Madrona

Stability Improvement Area [1,2]	Unit [10]	Hillside Drive	32nd Avenue E.	Madrona Drive	Madrona Park	Lake Dell	Lakeside North	Lakeside South
Number of Landslides								
High Bluff Peeloff								
Groundwater Blowout		1						
Deep-seated		2	3	3		2	3	6
Shallow Colluvial		2	3	5	6	14	5	10
Unidentified								1
Total		5	6	8	6	16	8	17
Subsurface Conditions [3]								
Colluvium Over Glacially Overridden Clay				X			X	X
Colluvium Over Glacially Overridden Sand and Gravel								
Colluvium Over Glacially Overridden Sand-Clay		X	X	X	X	X		
Colluvium Over Glacially Overridden Till-Sand-Clay								
Colluvium Over Glacially Overridden Till-Clay								
Sand-Clay Contact (Tubbs, 1974) Mapped in Area			X		X	X		
Contributing Causes of Instability								
Steep Topography				X		X		
Loose Fill or Colluvium on Slope		X	X					
Colluvium Over Clay							X	
High Groundwater Levels (Seepage and Springs)		X	X	X	X	X	X	X
Road Cuts and Fills (Public)			X	X	X	X	X	X
Undercutting and Filling (Private)			X		X	X		
Improperly Directed Surface Water		X			X	X		
Heavy Rainfall with Surface Runoff (Trigger Mechanism)				X	X	X		
Stability Improvements	**Unit [10]**							
Homeowner Education		X	X	X	X	X	X	X
Strom Drain Maintenance/Improvement (Curbs/Gutters/Catchbasins)		X		X	X	X	X	X
Trench Subdrains (10 ft deep)	LF							
Trench Subdrains (15 ft deep w/ trenchbox)	LF							
Finger Drains	EA	5			5			4
Springhead Drains	EA	5			5	10		3
Mechanically Stabilized Earth Wall [4, 8]	SF							
Geotextile Reinforced Soil Slope [5]	SF							
Combined Flattened Slope and Interceptor Trench [6]	LF							
Slope Grading (Excavation)	CY							
Machine Formed Curbs	LF			500				
Retaining/Catchment Wall (10 ft high)	SF		2,000			12,000	3,250	6,500
Fill Stabilization-Excavation and Replacement (20 ft wide, 7 ft deep) [7]	CY	2,600						
Excavation	CY	2,600						
Soil Backfill and Compaction	CY	2,600						
Asphalt Paving	SY	1,100						
Machine Formed Concrete Curbs	LF	500						
Drainage Improvements [9]								

General Note:

The Stability Improvements presented here are general types of measures that could be considered by the City, private property owners, or both, to improve stability. The number, length, square footage, etc., listed are very rough estimates of work on City and/or private properties presented only as a basis to formulate order-of-magnitude budgets.

Notes:

1. This table should be used in conjunction with the text describing each Stability Improvement Area, and with the cost data presented in Table 2-1.
2. The stability improvements listed here are preliminary and are presented to provide the city and private property owners with data for use in prioritizing work and developing order-of-magnitude budgets. Final scopes of work and corresponding cost estimates should be based on additional engineering studies and subsurface explorations.
3. Subsurface conditions may vary within a particular Stability Improvement Area. Many sites contain fill material on a slope or at the top of the slope.
4. Option 1 (Kinnear Park): Estimated cost for Mechanically Stabilized Earth (MSE) Wall (250 feet long, 25 feet high) is $350,000.
5. Option 2 (Kinnear Park): Estimated cost for Geotextile-Reinforced Soil Slope (250 feet long, 25 feet vertical height) is $244,000.
6. Option 3 (Kinnear Park): Estimated cost for Combined Flattened Slope and Interceptor Trench (25 feet deep) is $215,000.
7. Includes excavation of listed volume of material (CY), replacement soil backfill and compaction, installation of drainage improvements (if necessary), asphalt paving, and installation of machine formed concrete curbs. See individual costs for each of these items, as deemed necessary.
8. Standard MSE wall for other than Kinnear Park.
9. If necessary, type and quantity will depend upon site conditions.
10. CY = cubic yard, EA = each, LF = lineal foot, SF = square foot, SY = square yard

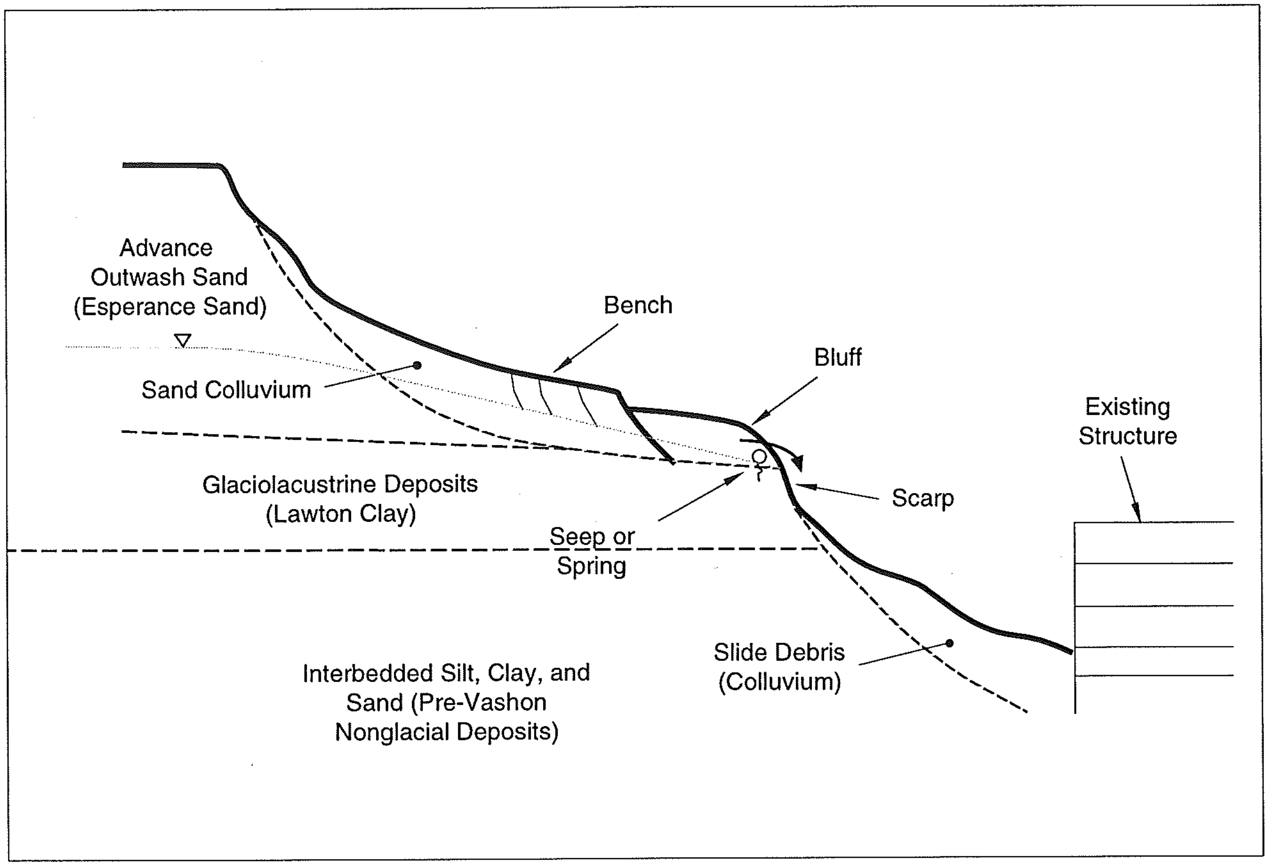
Seattle Landslide Study
Seattle Public Utilities
Seattle, Washington

STABILITY IMPROVEMENT AREAS
WEST SEATTLE,
MAGNOLIA/QUEEN ANNE, MADRONA

January 2000 W-7992-01

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

TABLE 3-1



Not to Scale

LEGEND

Seepage

Groundwater Table

NOTE

The thicknesses and elevations of these geologic units vary from place to place.

Seattle Landslide Study
Seattle Public Utilities
Seattle, Washington

IDEALIZED GEOLOGIC CONDITIONS DUWAMISH HEAD AREA

July 1999 W-7992-01

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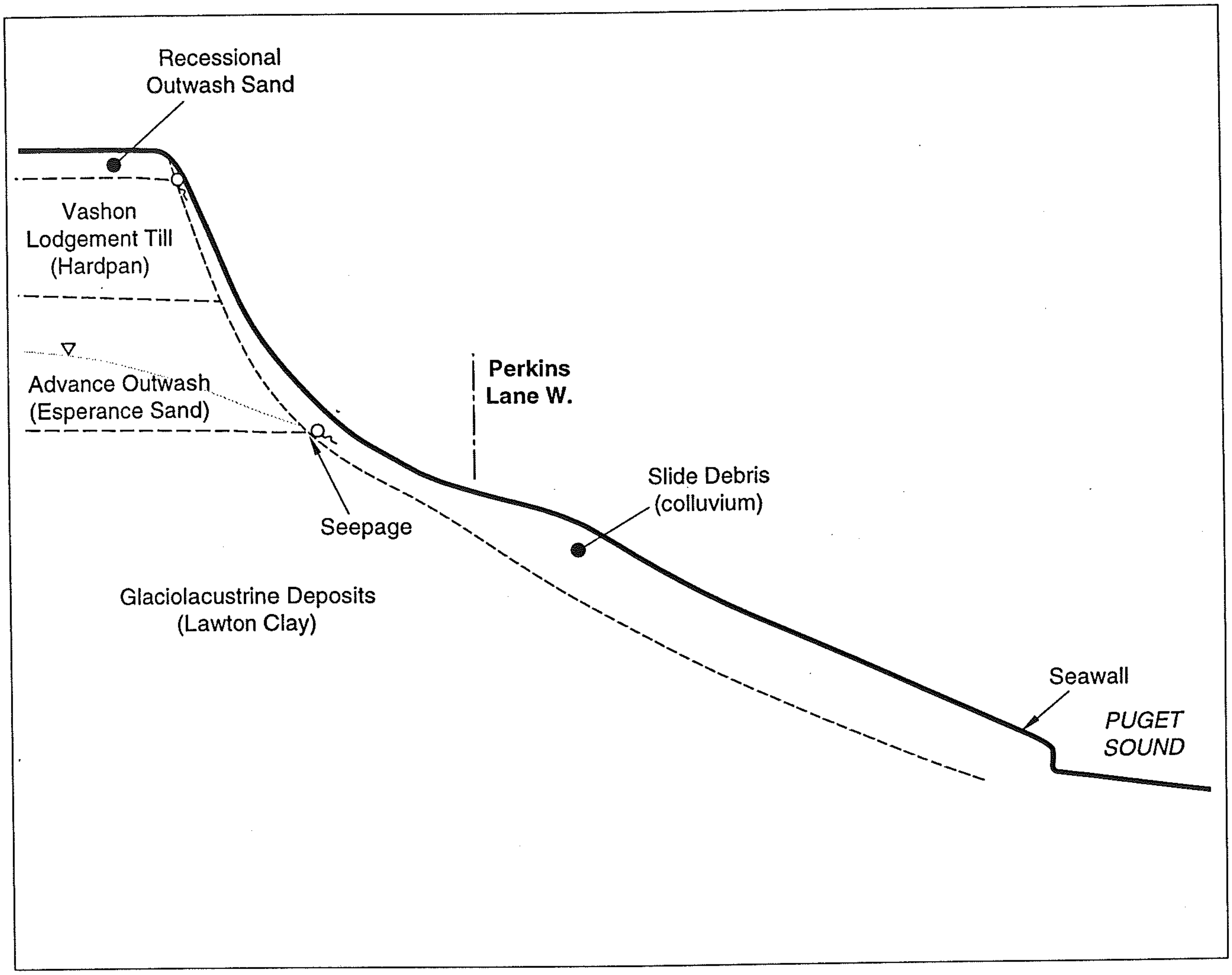
FIG. 3-1

I/Documents/Mac/W Jobs/W-7900s/W-7992-01 Fig. 3-1



Not to Scale

LEGEND

Seepage

Groundwater Table

NOTE

The thicknesses and elevations of these geologic units vary from place to place, and a unit could be missing due to previous erosion.

Seattle Landslide Study
Seattle Public Utilities
Seattle, Washington

IDEALIZED GEOLOGIC CONDITIONS WEST MAGNOLIA BLUFF

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FIG. 3-2

PART 4. LANDSLIDES IN NORTH, CENTRAL, AND SOUTH SEATTLE

14.0 GENERAL

14.1 Purpose and Scope

Part 4 of the report presents a general geologic and geotechnical evaluation of four additional specific study areas in Seattle not addressed in Part 3. (The original three study areas covered in Part 3 were West Seattle, Magnolia/Queen Anne, and Madrona.) The additional areas include Northwest Seattle, Northeast Seattle, Capitol Hill, and South Seattle; refer to Figure C-1, Appendix C, Volume 2 of this report. The emphasis is on evaluating factors that influence soil stability and presenting remedial measures for the types of slope instability found in the aforementioned areas of Seattle. It is to be noted that Parts 3 and 4 essentially cover the entire city.

The purpose for our studies and recommendations regarding stability improvements in Northwest and Northeast Seattle, Capitol Hill, and South Seattle is to provide the City of Seattle (City) with an analysis of 17 additional areas (Stability Improvement Areas) in Seattle where landslide activity has been prevalent, and an updated landslide database with verified locations and attributes. The purpose is also to provide the City with information for prioritizing remedial efforts and to develop order-of-magnitude budgets based on the cost data from Part 2, Section 8.0 of this report. The remedial measures presented are intended to be preliminary, with final scopes of work and corresponding cost estimates based on additional engineering studies and subsurface explorations.

The purpose described above has been accomplished in accordance with the following scope of services:

- We completed field verifying the balance (those not field checked for Part 3) of the reported landslides within the City. During this effort and an additional field visit, we evaluated the alternatives for stability improvements in areas of concentrated historical landslide activity based on the conditions observed (slide type, groundwater and surface water conditions, soil stratigraphy, etc.).

- For each additional study area, we prepared a brief description of the topography, geologic and groundwater conditions, slide types, timing, and slide locations.

- In Part 3 of the report, we identified 26 Stability Improvement Areas for West Seattle, Magnolia/Queen Anne, and Madrona where landslide activity has been prevalent. For

Part 4, we performed the same analysis for the 17 additional Stability Improvement Areas.

- Based on the above, we formulated stability improvements for consideration in the 17 additional Stability Improvement Areas. In this part of the report, we describe the factors contributing to instability in each area and propose remedial measures. The results were tabulated and are presented in Table 4-1. This table provides preliminary estimates of quantities (length, square footage, etc.) and other information (homeowner education, drainage maintenance/improvement, etc.) relative to improvements in the various areas. The types of recommended improvements are described in Part 2, along with costs relative to the various types of improvements.

In general, two site visits were made to each Stability Improvement Area, as indicated above. The first site visit, actually made prior to formulating the improvement areas, was primarily to field check the database locations and make appropriate changes in the database. The second site visit was for the purpose of formulating general types of measures that could be considered by the City and/or private property owners to improve stability and reduce landslide risk. Specific sites were not evaluated. The stability improvements listed on Table 4-1 include homeowner education; existing storm drainage facilities maintenance; storm drainage facilities improvement, as may be indicated by future observations or studies; subdrainage systems; fill stabilization; and retaining wall construction. The number, length, square footage, etc., listed on the table are rough estimates presented only to formulate order-of-magnitude budgets. Upon further studies needed to prioritize improvements, such studies may conclude that the extent or type of recommended improvements may or may not be needed, or that changes and/or additions may be advisable.

It should be mentioned here that some landslides have occurred outside the designated Stability Improvement Areas. These are usually isolated cases and the improvement areas were selected for locations where instability was prevalent. For landslides outside the designated areas, the stability improvement methods described in Part 2 of this report would apply, including homeowner education and drainage control.

As was stated in Part 3 with respect to stability improvements, the stability measures recommended in Part 4 also do not consider the location of property lines and relate to improvements made on City property, private properties, or both. Since landslides and areas of potential instability do not obey property boundaries, improvements are sometimes necessary on both public and private land to suitably improve stability in an area. Therefore, the improvements recommended in Part 4 are those that could be made by the City to protect utilities, drainage features, streets, and other City facilities; and also those measures or actions to be taken by the City and/or adjacent property owners to improve stability of an unstable slope. In the latter case, the City and private property owners should coordinate efforts to improve stability and/or provide protection (such as catchment walls) should instability take place. It is anticipated that some improvements will be made by the City, while other improvements or protection will be the responsibility of private property owners.

It should again be noted that there are always risks of damage to property and structures involving landslides for property located on or adjacent to a slope. Property owners need to accept those risks. Although the recommended improvements and homeowner education can lead to immediate or eventual improved slope stability conditions, private property owners should also obtain professional geotechnical advice to reduce current risks for their properties.

The analyses and recommendations presented in Part 4 of this report must be considered only in conjunction with the Limitations Section 1.5 presented in the Preface of this report.

The information presented in the next three sections 14.2, 14.3, and 14.4 is the same as that presented in Sections 10.2, 10.3, and 10.4 of Part 3. The information is repeated so that Part 4 will be complete without the need to refer back to Part 3.

14.2 Actions by City

In the succeeding sections of Part 4, various improvement measures and other actions are presented that we recommend be considered by the City. These actions include:

- Providing homeowner education materials regarding actions private property owners can take to reduce instability.
- Maintaining and/or improving storm drainage facilities.
- Conducting further detailed engineering studies in areas of prevalent landslides, including subsurface explorations.
- Implementing stability improvements.
- Coordinating stability improvements with private property owners.

Homeowner education is important so that the public is made aware of the factors that cause landslides and the steps homeowners should take to improve stability. Information should be provided to homeowners relative to prudent construction practices and obtaining professional advice for improving stability for existing homes, additions, or new construction. It is particularly important that homeowners learn that filling on a slope (especially at the top of a slope), or cutting into a slope (especially at the toe), can lead to instability and should only be undertaken with proper advice and consultation with competent geotechnical engineers or engineering geologists. Even the placement of yard waste on a slope decreases stability and, therefore, should be properly composted on flat ground or taken off-site. Homeowners should also be required to properly maintain and control their on-site drainage systems and to discharge drainage in accordance with applicable regulations, since improperly channeled water decreases slope stability, particularly when concentrated.

In addition to the above, we recommend that the City continue to conduct neighborhood informational meetings to facilitate two-way discussion regarding stability matters. Valid concerns of homeowners should be taken into account in planning and implementing improvements. We also recommend that the general public be made aware of a telephone "hot line" that can be readily reached to report locations of poor drainage, landslides, or potential instability.

In areas of potential landsliding, it is important that existing storm drainage facilities be maintained. In addition, storm drainage improvements could be considered when indicated by subsequent observations and studies. In this regard, the City has retained a consulting engineering firm (Black & Veatch) to evaluate surface drainage systems throughout the city. The scope of this "Needs Assessment" included visual observation of the roadway runoff where it had potential to impact landslide-prone slopes. Their studies are to be coordinated with the landslide studies presented herein, with the goal of improving stability conditions. In the succeeding sections of this report, recommendations regarding maintaining and/or improving storm drainage facilitates are subject to the evaluations and recommendations to be made by Black & Veatch. Therefore, prioritizing and budgeting relative to surface drainage improvements are beyond this current landslide study.

As stated previously, the stability improvements presented in Part 4 are preliminary and for the purpose of providing the City with information they can use to prioritize remedial efforts and develop "ballpark" budgets. Further detailed studies, including subsurface explorations, should be undertaken by the City to determine final scopes and design of remedial measures, and more accurate cost estimates. Geotechnical and other consultants should be used as appropriate. Implementing stability improvements by the City would consist of preparing plans and specifications using the data presented in Part 2 of this report, and observing actual construction to verify suitable conformance with project requirements.

Since landslides and potential instability cut across property boundaries, a cooperative effort between property owners is advisable in obtaining the greatest benefits of stability improvements. In addition to homeowner education, previously discussed, the City should facilitate the processing of permits submitted by private property owners so remedial work can take place expeditiously to improve stability. Variances to code requirements should be allowed where needed to improve stability for private and/or public properties. Temporary and/or permanent easements on or across City property could be granted, where allowed by ordinance, such as when needed to construct protective structures or to allow gravity flow, in lieu of pumped drainage, for suitably designed drainage facilities on private properties. Coordination between the City and private property owners may also include shared costs, such as by Challenge Grants or Local Improvement Districts (LIDs).

14.3 Actions by Private Property Owners

Improvement of stability involves actions not only by the City, but actions by private property owners. Such actions by private property owners should include accepting existing conditions and the risks of slope instability. Measures should accordingly be implemented on private properties as may be needed to protect and improve stability for existing property, structures, additions, or new construction. Those measures to be taken by private property owners are the same types of improvements presented in Part 2 of this report, and professional advice should be obtained from geotechnical and other appropriate consultants regarding the improvements. Such advice should also be obtained by prospective buyers of property in slide potential areas.

Stability improvements would include proper drainage of surface water, including suitable discharge of roof gutter downspouts. Surface water should not be improperly channeled to or concentrated on slopes and particularly not onto adjacent property. Other remedial measures would consist of properly designed subdrains, site grading, soil retention systems (walls, soil reinforcement, tieback anchors, etc.), drilled drains, or other measures as conditions may dictate.

Of particular concern are structures located above or at the bottom of a potentially unstable slope. Private property owners should seek professional advice regarding such measures as underpinning walls and/or tieback anchors near the top, or catchment/retaining walls at the bottom of a slope.

Private property owners should take advantage of the homeowner education materials prepared by the City or other entities. Cooperation with the City and with adjacent property owners is also important so that remedial measures can be coordinated to achieve the greatest benefits of stability improvement. Private property owners should also notify the City regarding areas observed with poor drainage, landsliding, or potentially unstable ground, so that drainage and stability improvements can be coordinated between City and private property owners as appropriate.

14.4 Additional Considerations

The contributing factors to instability, as described for the Stability Improvements section of this report, include terms such as surface drainage, runoff, storm water runoff, surface water runoff, etc. Such drainage or runoff includes that from pavement areas as well as from soil or vegetated areas. The more pervious the soil, such as sand and/or gravel, the more that rainfall will infiltrate the ground, which reduces the amount of runoff. Conversely, for more impervious soils like silt or clay, runoff will be greater. Runoff also takes place from vegetated slopes, being greater for areas of spare vegetation than for slopes with heavy vegetation.

Cuts at or near the toe of a slope, or fills on or near the top, are also contributing factors to instability. Such factors, particularly where cuts or fills took place years ago, may still have some influence on the stability of an area; however, such a factor may or may not be the predominant cause of recent or future instability. For example, a road cut area may remain stable for years, yet experience instability as the direct result of such things as a leaking or broken pipe, improper drainage from adjacent property, new filling or excavation on a slope, or other unwitting actions by owners or adjacent property owners. Each occurrence of instability requires evaluation to assess the predominant factor or factors leading to slope failure.

In describing some of the Stability Improvement Areas, we noted remedial measures of landslides that had recently been completed or were taking place. However, there are probably other remedial measures being planned, in progress, or completed by the City or private property owners that are not mentioned. Furthermore, we have not mentioned specific locations where surface drainage improvements have recently been undertaken or are being planned in conjunction with the "Needs Assessment" portion of the surface drainage studies by Black & Veatch.

15.0 NORTHWEST SEATTLE

15.1 Site Description

Northwest Seattle is defined in this study as the area north of the Lake Washington Ship Canal and west of Interstate 5 (refer to Figures C-1, C-2, and C-3, Appendix C, Volume 2). From the ship canal, the ground surface rises up to the north gradually as a broad undulating plain, nearly reaching elevation 500 feet near the north city limit. It is broken by depressions such as Green Lake, Haller Lake, and Bitter Lake. It has also been incised by Pipers Creek, a west flowing, steep gradient drainage in the vicinity of Carkeek Park. The Scenic Subdivision of the Burlington Northern Santa Fe Railroad (BNSF RR) extends along the toe of the relatively steep bluff along the western margin of the study area.

The stratigraphy of Northwest Seattle is typically comprised of Vashon glacial sediments overlying a relatively thick sequence of older, pre-Vashon glacial and non-glacial deposits. The contact between the Lawton glaciolacustrine clay and the overlying Esperance outwash sand (both Vashon glacial units) is mapped by Tubbs (1974) in the vicinity of the steep bluffs along Puget Sound and in the Pipers Creek drainage. Abundant groundwater seepage and springs are associated with this contact.

The distribution of recorded historical landslides within the Northwest Seattle study area is generally confined to the steep slopes facing Puget Sound, above the BNSF RR. The type of instability occurring in Northwest Seattle consists of high bluff peeloff-type landslides along the upper portions of the steep bluffs above Shilshole Bay Marina, in the North Beach area, and north of Carkeek Park. Shallow colluvial-type landslides are dispersed all along the west-facing slope bordering Puget Sound and along the steep slopes of the Pipers Creek drainage (Carkeek Park). Groundwater blowout-type landslides are also confined to the bluffs adjacent to Puget Sound. Although few deep-seated landslides are recorded in the Northwest Seattle Study area, one of the largest recorded instances of instability in Seattle is located just east of Golden Gardens Park along View Avenue N.W.

15.2 Stability Improvements

This section, like Part 3 of this study, presents possible stability improvements that could be made by the City to protect utilities, drainage features, streets, and other City facilities. It also presents measures that could be made by the City and adjacent property owners to improve the stability of an unstable slope. We present further comments regarding educating private property owners on steps they may take to improve stability.

The Northwest Seattle area has been divided into six smaller Stability Improvement Areas, where landslide activity has been prevalent. As shown on Figure C-3 (Appendix C, Volume 2), the six areas are as follows:

1) Broadview
2) 25th Avenue N.W.
3) Carkeek Park
4) Blue Ridge
5) Golden Gardens
6) Shilshole

For each area, we will summarize the general subsurface conditions, landslide types and causes, and present actions that could be considered for improving slope stability. Also refer to Table 4-1, located following the text in Part 4 of this report.

15.3 Broadview

The Broadview Stability Improvement Area is located in the northwest corner of the City and north of the Carkeek Park Stability Improvement Area; refer to Figure C-3. In this area, a total of 47 landslides were recorded, interpreted from the records as high bluff peeloffs (3), groundwater blowouts (18), deep-seated landslides (2), and shallow colluvial landslides (22). Two landslides were not identified as to type. In general, most of these landslides occurred on the west-facing bluff and steep slopes located east of the BNSF RR tracks. At some locations, the landslides affected the backyard areas of residential sites located at the top of the bluffs/slopes. Toward the north, instability damaged N.W. Culbertson Drive; this area has been repaired by a crib wall. Toward the central section, some landslides occurred on the side slopes of a north-south-trending gully located uphill and east of the steep slope down to the railroad tracks. Stream-bank erosion may contribute to these instabilities. The recorded landslides in the Broadview improvement area have occurred throughout the years beginning in 1933 and extending into 1997.

The subsurface soils in this area generally consist of colluvium or fill overlying glacially overridden glacial till, outwash sand, and/or lacustrine clay/silt. The sand-clay contact (Tubbs, 1974) is present in this area. The primary contributing factors to instability consist of steep topography, loose fill at the top and/or colluvium on the slope, high groundwater levels with associated seepage particularly near the sand-clay contact, and heavy rainfall (triggering cause).

It is recommended that work by the City include maintaining existing storm drainage facilities and improving them when indicated to be appropriate by future observations. Homeowner education is recommended to include providing information regarding prudent construction and drainage practices, and obtaining professional advice for improving stability for existing property, additions, or new construction.

15.4 25th Avenue N.W.

Eight shallow colluvial landslides have been recorded for this Stability Improvement Area. All but two of these landslides took place during the December 1996/January 1997 storm. They occurred at random locations as shown on Figure C-3. The other two landslides took place in 1970 and 1972. The subsurface conditions in this area consist of fill and/or colluvium overlying glacially overridden sand and/or clay. The sand-clay contact (Tubbs, 1974) extends across this area as shown. The factors contributing to instability are steep topography, loose fill at the top and/or colluvium on the slope, high groundwater levels/seepage particularly near the sand-clay contact, and heavy rainfall (triggering cause).

Recommended action for this area includes homeowner education and storm drainage systems maintenance and/or improvement. It is to be noted that one of the 1996/1997 landslides has been repaired by the City utilizing a mechanically stabilized earth (MSE) wall.

15.5 Carkeek Park

In the Carkeek Park Stability Improvement Area, as designated on Figure C-3, 13 landslides were recorded. Twelve landslides were identified as shallow colluvial and one as a deep-seated landslide. The earliest slide was reported in January 1960. The most recent landsliding reportedly took place on or about January 14, 1998.

The landslides in this area occurred primarily on or adjacent to ravine slopes. At some locations, creek erosion of the slope toe may have contributed to the instability. Construction of N.W. Carkeek Park Road, which lies near the center of this area, likely included some fills along the downhill side and cutting along the uphill side. At some locations, private property owners have placed backyard fills. At one location, a landslide was reportedly related to cutting into the toe of a roadway fill by a homeowner. Several of the landslides reported in this area were caused or exacerbated by private utility pipeline breaks, probably the latter.

The subsurface soils in this area, based on geologic mapping and our experience in this area (no explorations reviewed), consist of colluvium overlying glacially overridden soils. The overridden soils consist of sand over clay, and the sand-clay contact (Tubbs, 1974) is present in the Carkeek Park ravine areas. Groundwater seepage can be expected near the sand-clay contact.

The factors that contribute to instability in this area are steep topography, colluvium on the slope, and cutting or filling on the slope. The landslides were triggered by heavy rainfall that resulted in surface runoff and infiltration into the colluvium.

Recommended actions in this area include storm drainage systems maintenance and/or improvement and homeowner education. Curbs and gutters along N.W. 118th Street could be considered for controlling street drainage. Homeowner education is recommended to inform property owners of the landslide risks involved with backyard fills on or near the top of a slope, and the need to properly control site drainage including downspout discharge. Side sewers on or in the slope should be checked frequently for proper functioning. In addition, the City could consider the installation of a catchment/retaining wall along the uphill side of portions of N.W. Carkeek Park Road in order to prevent landsliding onto the road.

15.6 Blue Ridge

The location of the Blue Ridge Stability Improvement Area is shown on Figure C-3. In this area, 21 landslide records have been reviewed, of which 2 were interpreted as high bluff peeloffs, 11 groundwater blowouts, 3 deep-seated landslides, and 5 shallow colluvial landslides. The earliest recorded landslide was in 1933, and instability has occurred throughout the years. Several landslides were recorded in January 1997.

Most of the landslides in this area took place on the steep northwest-facing slope located between uphill residences and the BNSF RR tracks at the toe of the slope. Most were natural occurrences, although some may have taken place because fill was placed behind residences (contributing factor). Two of the 21 landslides were reportedly caused by plugged catch basins that resulted in runoff onto the slope behind houses. Eleven of the landslides, which were the ones listed as groundwater blowouts, took place between 1933 and 1960, and apparently brought debris down to the railroad tracks.

The subsurface soils in this area consist of colluvium overlying glacially overridden soils. Toward the south in this improvement area, where most of these slides occurred, the overridden soils consist primarily of clay. To the north, the overridden soils consist of sand over clay, and the sand-clay contact (Tubbs, 1974) is present. Seepage near this contact likely contributed to some of the landslides.

The factors contributing to instability are steep topography, loose fill and/or colluvium on the slope, high groundwater levels and associated seepage near the location of the sand-clay contact, and heavy rainfall (triggering cause) that results in surface runoff and also infiltrates and reduces stability for colluvium and loose fill areas.

Recommended action in this area consists of storm drainage systems maintenance and/or improvement and homeowner education.

15.7 Golden Gardens

The Golden Gardens Stability Improvement Area is the area generally east and northeast of Golden Gardens Park, as shown on Figure C-3. On the west- and northwest-facing slopes, a total of 26 landslides have been recorded through the years beginning in 1930. Most of the landslides listed were of the shallow colluvial type (17), while some were listed as high bluff peeloffs (6), groundwater blowouts (1), and deep-seated landslides (2). The most recent instability recorded took place on or about March 19, 1997.

The high bluff peeloff landslides occurred in the northern section of this improvement area, where a steep, northwest-facing bluff rises above the BNSF RR tracks and is present behind and northwest of residential sites fronting on N.W. Esplanade Street. (A note on two of the landslide entries listed in the database indicates that as many as 13 landslides onto railroad property took place in this area from 1949 to 1960). One shallow colluvial and the one recorded groundwater blowout also took place in this area. The rest of the shallow colluvial and the deep-seated landslides took place further south in this improvement area on the steep slopes uphill from N.W. Esplanade and downhill from View Avenue N.W. and Golden Gardens Drive N.W.

The subsurface conditions in this area generally consist of fill and/or colluvium overlying glacially overridden soils. To the north in the area where the high bluff peeloffs occurred, the overridden soils (which slab off or slide) consist of glacial till or outwash sand. Toward the south, the glacial till is generally absent and the fill and/or colluvium overlies lacustrine clay/silt (north of Golden Gardens) or outwash sand (east of Golden Gardens). The sand-clay contact (Tubbs, 1974) is present to the east of Golden Gardens, as indicated on Figure C-3. With respect to groundwater, seepage can be expected in the colluvial layer, at or near the sand-clay contact, and from pervious layers within the lacustrine clay/silt stratum.

In the area uphill of Golden Gardens Park, a large, deep-seated landslide destroyed three houses along the west side of View Avenue. Movement was first detected in early spring of 1974, and movement continued into July of that year, at which time 48 horizontal drains were installed for drainage of soils at or near the sand-clay contact. The drains were installed from a bench on

Park Department property. Movement of the upper portion of the slope near View Avenue was stopped in August 1974 and, to our knowledge, further deep-seated movement affecting the upper slope has not occurred. However, it is our understanding that some movements on the Park Department bench and steep slope down to the railroad tracks have occurred since 1974.

The primary factors that contribute to instability in this area are heavy rainfall (triggering cause), steep topography, fill at the top and/or colluvium on the slopes, high groundwater levels and associated seepage, and pipeline discharge from private properties (storm drainage). In the steep slope area located between View Avenue and Esplanade, a number of erosional gullies or slide debris chutes are present. During periods of heavy or prolonged rainfall, mud and debris flows have taken place in these chutes. At some locations, pipeline discharge has contributed to the debris flows.

Recommended actions for consideration by the City in this stability improvement area include construction of catchment/retaining walls, maintenance and/or improvement of storm drainage systems, and education of homeowners. Catchment/retaining walls are recommended along the uphill side of N.W. Esplanade to protect against landslides onto the roadway. In addition, an MSE wall is recommended along the west (downhill) edge of Golden Gardens Drive N.W. to provide support for the edge of the road where signs of instability are present.

Homeowner education should emphasize the suitable discharge of site drainage including downspout discharge. Homeowners located at the toe of slopes should be advised to consider constructing catchment walls to protect against debris slides from uphill land. One such catchment wall was constructed in 1998 for protection of one house along N.W. Esplanade.

With respect to the area where the deep-seated landslide took place in 1974, it is recommended that a comprehensive study be made to evaluate the current stability of this area. This could include evaluating the horizontal drain system that may or may not still be operating suitably. We suspect that many of these drains may have been severed by slide movement shortly after installation in 1974. The City and/or private property owners could consider cleaning or replacing these drains as indicated. Additional stability improvement measures could also be indicated in order to improve stability. For this current study, we recommend that potential costs relative to this site be determined by assuming the cleaning of existing drains and the installation of 30 additional horizontal drains to replace non-functioning drains. In addition, deep trench subdrains on the lower bench could be assumed for cost estimating purposes; refer to Table 4-1.

15.8 Shilshole

In the Shilshole Stability Improvement Area (refer to Figure C-3), a total of 19 landslides have been recorded. Four were interpreted to be high bluff peeloffs, six groundwater blowouts, and nine shallow colluvial landslides. The earliest recorded landslide was 1933, and landslides have occurred throughout the years. The most recent slide occurred in February 1999. Most of these landslides (14 of 19) occurred on the west-facing bluff located uphill from the BNSF railroad tracks. Debris from these landslides sometimes reached the railroad tracks, and at least two debris flows came down onto Seaview Avenue N.W. and/or the parking area for the Shilshole Bay Marina. The other five landslides occurred on residential sites located in the southern portion of this improvement area.

The subsurface conditions in this area generally consist of fill and/or colluvium overlying glacially overridden soils. The overridden soils include glacial till and sand over clay. The sand-clay contact (Tubbs, 1974) is mapped only in about the northern third of this improvement area.

The factors that contribute to instability in this area are steep topography, high groundwater levels/seepage, and improper fills at the top or on slopes. The triggering mechanism is generally heavy rainfall. Where existing residences are located at the top of the slope, surface runoff from the top and/or storm water discharge (downspouts) into slope soils could contribute to instability unless suitably controlled.

Recommended action consists primarily of homeowner education. Maintaining and/or improving storm drainage would also be appropriate.

16.0 NORTHEAST SEATTLE

16.1 Site Description

Northeast Seattle is defined in this study as the area north of the Lake Washington Ship Canal and east of Interstate 5 (refer to Figures C-4 and C-5, Appendix C, Volume 2). From the ship canal, the ground surface rises up to the north relatively gradually as a broad undulating plain, nearly reaching elevation 450 feet near the Maple Leaf area. The northern two-thirds of the study area is incised by Thornton Creek and its tributaries and the southern third by Ravenna Creek. Steep slopes predominate along the eastern portion of this study area adjacent to Lake Washington. The Burke Gilman Trail (formerly railroad tracks) is located at the toe of the steep bluff along the shore of Lake Washington and extends south from the northern city limit to the ship canal.

The stratigraphy of Northeast Seattle is comprised of Vashon Glacial sediments overlying a relatively thick sequence of older, pre-Vashon glacial and non-glacial deposits. The contact between the Esperance outwash sand and the underlying Lawton glaciolacustrine clay (both Vashon glacial units) is mapped by Tubbs (1974) along the steep slopes above the Burke Gilman Trail and within the Thornton Creek drainage basin.

The locations of recorded landslides within the northeast Seattle study area are generally confined to the steep slopes facing Lake Washington with the exception of the instability recorded along Thornton Creek and its tributaries. The type of instability occurring in this study area primarily consists of shallow colluvial-type failures. Several deep-seated failures are recorded in the vicinity of the Inverness area. Groundwater blow-out-type landslides are documented along the shore of Lake Washington in the northeast portion of the study area where the sand-clay contact extends along the steep slope just west of the Burke Gilman Trail.

The instability recorded in Northeast Seattle has primarily occurred after 1940 and the majority of the older recorded events are confined to the Laurelhurst neighborhood. Several landslides are recorded in the Inverness neighborhood between 1950 and 1970; these are primarily related to grading and excavations during development.

16.2 Stability Improvements

This section presents possible stability improvements that could be made by the City to protect utilities, drainage features, streets, and other City facilities in the Northeast Seattle area. Furthermore, this section includes measures that could be made by the City and adjacent property owners to improve the stability of an entire landslide or unstable slope. We further present comments regarding educating private property owners on steps they may take to improve stability.

The Northeast Seattle area has been divided into three smaller Stability Improvement Areas where landslide activity has been prevalent, in order to describe various improvements and homeowner education suggestions. As shown on Figure C-5 (Appendix C, Volume 2), the three areas are as follows:

1) Burke Gilman
2) Inverness
3) Laurelhurst

For each area, we will summarize the general subsurface conditions, landslide types and causes, and present actions that could be considered for improving stability. Also refer to Table 4-1.

16.3 Burke Gilman

The Burke Gilman Stability Improvement Area is located in the northeast corner of the City, as indicated on Figure C-5. In this area, a total of 39 landslides were recorded. Most of them (35) were shallow colluvial landslides. Two were high bluff peeloffs, one groundwater blowout, and one deep-seated landslide. The landslides generally occurred on the east-facing slope that is present in this area. The earliest recorded landslide took place in 1955, and instability was reported throughout the years including January 1999.

Fourteen of the landslides were recorded for the bluff area located between the Burke Gilman trail on the east and private properties uphill to the west. Many other non-recorded landslides have occurred here as well. Many of these brought debris down onto the trail. Ten of the recorded 14 landslides took place in early January 1997, one in March 1972, one in May 1983 (due to sprinkler left running), and two in February 1996. The landslide database indicates that at least one of these landslides resulted in some damage to a residence at the top of the slope. The other landslides in this improvement area occurred on the uphill and downhill sides of various streets (38th to 42nd Avenues N.E.), and on the east-facing slope located behind and to the east of a number of residences on these streets.

The subsurface soils in this area generally consist of colluvium overlying glacially overridden sand over clay. The sand-clay contact (Tubbs, 1974) is generally located just east (upslope) of the Burke Gilman trail. At some locations, fill located behind (east of) residences was involved in instability. Groundwater seepage can be expected at the sand-clay contact.

The factors that contribute to instability in this area are steep topography, colluvium on the slope, high groundwater levels and associated seepage at the sand-clay contact, and heavy rainfall (triggering cause). At some locations, fill at the top or on a slope contributed to landslide potential. As indicated previously, one reported landslide was due to a sprinkler that was left running.

Recommended actions in this area include storm drainage systems maintenance and/or improvement and homeowner education. Surface drainage along 40th Avenue N.E. (13700-block) could be evaluated and improved as needed. Homeowner education is recommended to inform property owners of the landslide risk involved with a steep slope when located behind a residence, particularly with backyard fills on or near the top of a slope. Site drainage including downspout discharge also needs to be properly controlled. In addition, the City could consider the installation of a continuous catchment/retaining wall along the uphill side of 40th/41st Avenue N.E. between N.E. 142nd and N.E. 144th Streets. This is to prevent landslide debris from coming onto the road. To improve stability for the downhill edge of this road, an MSE wall could be constructed.

16.4 Inverness

The Inverness Stability Improvement Area is located uphill of Sand Point Way and is shown on Figure C-5. In this area, 22 landslides are recorded, consisting of 5 deep-seated and 17 shallow colluvial landslides. The earliest recorded landslide occurred in February 1955. Instability has taken place throughout the years. The most recent instability was noted in January 1999. In general, instability in this area has occurred on ravine slopes often where filling has occurred in conjunction with residential development.

The subsurface soils in this area generally consist of fill and/or colluvium overlying glacially overridden sand over clay. Most of the landslides occurred near the sand-clay contact (Tubbs, 1974) mapped for this area. Seepage at the contact is likely. The primary contributing factors to instability consist of steep topography, loose fill at the top and/or colluvium on the slope, high groundwater levels and associated seepage near the sand-clay contact, and heavy rainfall (triggering cause).

Recommended actions in this area consist of storm drainage systems maintenance and/or improvement and homeowner education. Homeowner education is appropriate, particularly involving the instability risks regarding fills on or near the top of slopes. It is recommended that homeowners also be provided with information regarding prudent drainage practices including downspout water discharge.

16.5 Laurelhurst

Twenty landslides have been recorded for the Laurelhurst Stability Improvement Area; refer to Figure C-5 for location. All 20 recorded landslides have been listed as shallow colluvial events. Beginning in December 1933, instability has reoccurred in this area at about 10-year intervals. The last recorded event was January 1997. Most of the landslides in this improvement area consist of instability on the southeast-facing slope west of Lake Washington and rising above

N.E. Laurelcrest Lane. Two recorded landslides occurred further north and on the downhill side of residences located on 55th Avenue N.E.

The subsurface soils in this area consist of colluvium overlying glacially overridden soils. The sand-clay contact (Tubbs, 1974) is not shown in this area. Based on geologic mapping and our experience in this area (no explorations reviewed), the overridden soils consist of till, sand, and/or clay. The factors that contribute to instability consist of steep topography, colluvium on the slope, and heavy rainfall (triggering cause).

Recommended actions in this area include storm drainage systems maintenance and/or improvement and homeowner education. It is recommended that homeowner education include informing uphill property owners of the risks involved with fills on or near the top of a slope, and the need to properly control and maintain site drainage including downspout discharge. Homeowner education could also include information regarding construction of catchment walls at the toe of slopes to retain landslide debris and protect residences, garages, and the private drive that extends south of N.E. Laurelcrest Lane. The City could also consider the installation of a catchment/retaining wall along the uphill side of portions of N.E. Laurelcrest Lane in order to prevent landsliding onto the road.

17.0 CAPITOL HILL

17.1 Site Description

The Capitol Hill area is defined in this study as the peninsular area east of Lake Union, south of Portage Bay and the Montlake Cut, west of the Arboretum (see Figure C-1), and north of East Roy Street (refer to Figures C-6 and C-7, Appendix C, Volume 2). Capitol Hill proper, is a north-south-trending ridge that rises gently to the south with relatively steep west-facing slopes along Interstate 5 on the west side and a highly incised drainage (Interlaken Park) to the east. Interstate 5 extends north-south just west of the longitudinal axis of Capitol Hill and State Route 520 extends east-west, south of the Montlake Cut, to its western terminus at Interstate 5.

The stratigraphy of Capitol Hill is comprised of a thin veneer of Vashon Glacial sediments overlying a relatively thick sequence of pre-Vashon glacial and non-glacial deposits. The sand-clay contact, representing the interface between the Lawton glaciolacustrine clay and the overlying Esperance outwash sand (both Vashon glacial units) is mapped by Tubbs (1974) in the vicinity of the steep slopes east of Interstate 5 and west of the southern margin of the University of Washington Arboretum. The contact is conspicuously absent around the northern margin of

Capitol Hill. Based on exploratory borings for the Sound Transit project, it is believed that a pre-Vashon, east-west trending fluvial (outwash) channel extends through Capitol Hill just south of the I-5/SR 520 interchange. Our opinion that these natural cut-and-fill deposits extend in an east-west-direction is corroborated by the City of Seattle shaded relief map of the area (refer to Figure A-1, Appendix A, Volume 2). This map shows erosional ravines and landslide bowls on the east (Interlaken) and west (Lakeview Drive) hillsides of Capitol Hill. In our opinion, these features are indicative of cohesionless soils, such as outwash sand, and thus explain the absence of glaciolacustrine clay in portions of the Interlaken area.

The distribution of recorded landslides within the Capitol Hill study area is generally confined to three areas: the steep slope just east of Interstate 5 (I-5) along Lakeview Boulevard East, upslope of Portage Bay Place East on the east side of Capitol Hill, and in the highly incised area of Interlaken Park. Shallow colluvial and deep-seated-type landslides predominate in the Capitol Hill area. The absence of groundwater blowouts, in our opinion, may be a result of the lack of detailed information in the City files as well as the absence of glaciolacustrine clay, especially in the Interlaken area. Because the Capitol Hill area is among one of the older neighborhoods in Seattle, the record of landsliding dates back to the early 1900s.

17.2 Stability Improvements

This section presents possible stability improvements that could be made by the City to protect utilities, drainage features, streets, and other City facilities. Measures are also presented that could be made by the City and adjacent property owners to improve stability of an unstable slope. We present further comments regarding educating private property owners on steps they may take to improve stability.

The Capitol Hill area has been divided into three smaller Stability Improvement Areas, where landslide activity has been prevalent. As shown on Figure C-7 (Appendix C, Volume 2), the three areas are as follows:

1) North Capitol Hill
2) Interlaken
3) West Capitol Hill

For each area, we will summarize the general subsurface conditions, landslide types and causes, and present actions that could be considered for improving slope stability. Also refer to Table 4-1, located following the text in Part 4 of this report.

17.3 North Capitol Hill

Fourteen landslides have been recorded for the North Capitol Hill Stability Improvement Area; refer to Figure C-7 for location. One landslide was noted as deep-seated, 8 as shallow colluvial, and 5 were unidentified as to landslide type. The earliest slide was reported in 1923, and instability has occurred through the years. Five landslides were listed for January and March 1997.

Seven of the landslides in this area took place on the steep slope located behind (east of) buildings on Fuhrman Avenue E., extending downhill to Portage Bay Place N.E. Four of these landslides occurred in 1997, which apparently resulted in the construction of a soldier pile and wood-lagged retaining wall along a portion of the slope toe on the west side of the lower road. Another landslide took place behind a building on Fuhrman Avenue in August 1986, reportedly caused by excessive landscaping watering. The instability uphill of Portage Bay Place can and has caused debris to block the roadway.

The other seven landslides in this improvement area took place at the locations shown on Figure C-7. Four of them involved fill placed by private property owners.

The subsurface soils in this North Capitol Hill area generally consist of fill and/or colluvium overlying glacially overridden glacial till, outwash sand, and/or lacustrine clay/silt. The sand-clay contact (Tubbs, 1974) is not mapped in this area. The primary contributing factors to landsliding consist of improper fills, steep topography, colluvium on the slope, improper irrigation (one instance), and heavy rainfall (triggering cause).

It is recommended that actions by the City include maintaining existing storm drainage facilities and improving them when indicated by future observations. Homeowner education is also recommended to stress the risks involved with improper filling. Information regarding prudent construction and drainage practices should also be made available to private property owners. The City could also consider the installation of a catchment/retaining wall along currently unsupported portions of the toe of slope west of Portage Bay Place, including adding more debris catchment height to the recently constructed retaining wall.

17.4 Interlaken

The Interlaken Stability Improvement Area is generally a northeast-facing slope located as shown on Figure C-7. Numerous landslides have taken place through the years in this area. Forty-two landslides have been recorded, interpreted as one groundwater blowout, 10 deep-seated events, 22 shallow colluvial landslides, and 9 unidentified as to type. The earliest recorded landslide was in February 1927. A number of landslides occurred in 1997, 1998, and 1999.

Most of the landslides in this improvement area occurred upslope of E. Interlaken Boulevard and Interlaken Drive E. A number of landslides also took place downslope of E. Boston Terrace, and others occurred upslope of Delmar Drive E. and 14th Avenue E.; refer to Figure C-7. Sixteen of the 42 landslides in this area reportedly involved fill material of which ten were fills at residential sites. A number of the landslides in this area have received remedial measures consisting of retaining structures, grading, and/or subsurface drainage.

The subsurface soils in this Interlaken area generally consist of fill and/or colluvium overlying glacially overridden sand, silt, and clay. The colluvium consists of intermixed sand, silt, and clay. High groundwater levels occur in the colluvium during the wet-weather times of the year. The sand-clay contact (Tubbs, 1974) is not mapped in this area. The primary contributing factors to landsliding consist of improper fills, steep topography, high groundwater levels/seepage in colluvium on the slope, and heavy rainfall (triggering cause).

It is recommended that actions by the City include maintaining and/or improving storm drainage systems in this area. Homeowner education is also recommended to stress the risks involved with improper filling and storm drainage practices. The City could also consider construction of the improvements described in the following paragraphs.

A trench subdrain may be appropriate to improve stability for a portion of the slope uphill from Interlaken Drive. Such a subdrain would be parallel to slope contours in the area where previous landslides have occurred, and should be extended through the colluvium and into the glacially overridden soils. An MSE wall could be considered at two locations: 1) along the downhill shoulder of Interlaken Drive (near the south end of the improvement area), and 2) along the downhill side of 20th Avenue E. near its transition to Interlaken Place E. The purpose for the MSE walls is to strengthen the roadways.

The construction of retaining/catchment walls could be considered for support and/or debris catchment along the uphill sides of Interlaken Drive (where landslides have previously occurred), Delmar Drive E. (north of 14th Avenue E.), and 14th Avenue E. (between Delmar Drive and Boyer Avenue E.) to protect the street. We also recommend consideration of fill stabilization (roadway replacement) for portions of Interlaken Drive and 20th Avenue.

17.5 West Capitol Hill

Sixteen landslides have been reported for the West Capitol Hill Improvement Area, located as shown on Figure C-7. Of these 16 landslides, 2 were noted as deep-seated, 11 as shallow colluvial, and 3 not identified as to landslide type. The earliest recorded landslide date was 1916. Four landslides occurred in the 1930s, one in 1961, one in 1974, two in 1986, and seven in 1997.

In general, the landslides in this area have taken place on the steep, west-facing slope located between 10th Avenue E. and Lakeview Boulevard E. A number of the landslides resulted in debris blocking Lakeview Boulevard. One landslide in 1997 occurred on the downhill side of Lakeview Boulevard. A few of the landslides involved fill material.

The subsurface soils in this area generally consist of fill and/or colluvium overlying glacially overridden till, sand, or clay. The sand-clay contact (Tubbs, 1974) is mapped downslope to the west of this improvement area. The primary contributing factors to instability consist of steep topography, fill and/or colluvium on the slopes, and heavy rainfall (triggering cause). High groundwater levels/seepage is also a contributing factor, particularly in the vicinity of Lakeview Boulevard.

Recommended actions in this area consist of storm drainage systems maintenance and/or improvement and homeowner education, particularly involving prudent drainage and construction practices. The risks of instability involving fills on or near the top of a slope could also be emphasized. The City could also consider the installation of retaining/catchment walls along the east side of Lakeview Boulevard, as protection against landslide debris blocking the roadway.

18.0 SOUTH SEATTLE

18.1 Site Description

South Seattle is defined in this study as the area south of Interstate 90, north of the City limit, west of Lake Washington and east of 21st Avenue S.W. (refer to Figures C-8 and C-9, Appendix C, Volume 2) The area is characterized by the broad, north-south-trending, floodplain of the Duwamish River that is bounded on the west by the steep, east-facing slope of Puget Ridge and on the east by Beacon Hill. Other significant slopes in this study area are located along the west shore of Lake Washington near the Rainier Beach and Mount Baker neighborhoods. Interstate 5 extends in a north-south direction along the toe of the west-facing slope of Beacon Hill.

The stratigraphy of South Seattle is comprised of Vashon glacial sediments overlying a sequence of pre-Vashon glacial and non-glacial deposits. Older, Tertiary bedrock crops out sporadically throughout the South Seattle study area, with notable exposures along the east side of Interstate 5. The contact between the Lawton glaciolacustrine clay and the overlying Esperance outwash sand (both Vashon glacial units) is mapped upslope of West Marginal Way S.W., and around the northern tip and along the west side of Beacon Hill. Abundant groundwater seepage and springs are associated with this contact as well as other contacts between relatively permeable glacial units (older glacial outwash deposits) and relatively impermeable soils of older, pre-Vashon glacial deposits and Tertiary bedrock.

The distribution of recorded historical landslides within the South Seattle study area is generally confined to the steep slopes on both sides of Beacon Hill, the east-facing slope along West Marginal Way S.W., and the steep, northeast-facing slope in the Rainier Beach neighborhood. Deep-seated and shallow colluvial-type landslides predominate along the margins of the Duwamish floodplain, while nearly all of the documented landslides in the Rainier Beach area are classified as shallow colluvial. Groundwater blowout-type landslides are documented along the west-facing slope adjacent to Interstate 5 and south of Highland Parkway S.W. in the vicinity of the contact between the underlying Lawton glaciolacustrine silt and clay and the Esperance glacial outwash sand and gravel. There are no high bluff peeloff-type landslides documented within the South Seattle study area.

The timing of landslides within the South Seattle study area is strongly influenced by the construction of public works projects. For example, the majority of the instability along the west side of Beacon Hill was recorded prior to the 1960s before the construction of I-5. The construction of Interstate 5 effectively increased stability, with cylinder piles and retaining walls, for large portions of the chronic landslide areas along the west-facing slope. Conversely, the grading of Rainier Avenue S., in the Rainier Beach neighborhood, along the toe of the steep, northeast-facing slope may have oversteepened the slope and exacerbated instability of the colluvium-covered till slope.

18.2 Stability Improvements

This section presents possible stability improvements that could be made by the City to protect utilities, drainage features, streets, and other City facilities. Measures are also presented that could be made by the City and adjacent property owners to improve stability of an unstable slope. We present further comments regarding educating private property owners on steps they may take to improve stability.

The South Seattle study area has been divided into five smaller Stability Improvement Areas, where landslide activity has been prevalent. As shown on Figure C-9 (Appendix C, Volume 2), the five areas are as follows:

1) Mount Baker
2) 25th Avenue S.
3) West Beacon Hill
4) Duwamish
5) Rainier Beach

For each area, we will summarize the general subsurface conditions, landslide types and causes, and present actions that could be considered for improving slope stability. Also refer to Table 4-1, located following the text in Part 4 of this report.

18.3 Mount Baker

Fourteen landslides were listed in the database for the Mount Baker Stability Improvement Area, categorized as three deep-seated events, eight shallow colluvial landslides, and three unidentified as to landslide type. The earliest landslide was recorded for 1922. Thereafter, two took place in the 1930s, five in the 1960s, and three each in 1986 and 1997.

The landslides in this area generally have occurred on the east-facing slope that extends downhill to Lake Washington Boulevard S. The seven southernmost landslides involved Park Department property located between Lake Washington Boulevard and private properties located on the next street (Lakewood and Cascadia Avenues S.) uphill to the west. In three or four of these landslides, debris from private properties came down onto Park Department land, reaching Lake Washington Boulevard in two reported events. In the other landslides, instability apparently occurred on Park Department property to the east of private property. One residence was reported to be threatened by a 1986 landslide. In connection with two of the seven landslides, property owners claimed that sewer backup, leakage, and/or surface drainage led to the instability.

Three shallow colluvial landslides occurred immediately uphill from Mount Claire Drive S., in 1961, 1963, and 1997. These apparently involved Park Department property (Mount Claire Park). In the 1997 event, landslide debris from Park Department property came down and crossed Mount Claire Drive. Further to the west, an unidentified type of landslide occurred on private property, involving failure of a rubble wall and fill material. To the north, three reported deep-seated landslides took place in 1922, 1933, and 1936. Stability in this latter area was improved by drainage facilities installed as a Works Progress Administration (WPA) project in 1935 and 1936.

The subsurface soils in this area generally consist of fill or colluvium overlying glacially overridden till, sand, and/or clay. The sand-clay contact (Tubbs, 1974) is not mapped in this area. The factors contributing to instability in this area are relatively steep topography, fill and/or colluvium on the slope, and heavy rainfall (triggering cause). In several instances, as previously mentioned, sewer backup, leakage, and/or surface drainage contributed to the instability.

Recommended actions in this area include storm drainage systems maintenance and/or improvement and homeowner education. In addition, the City could consider installing a retaining/catchment wall along portions (two sections) of the west side of Mount Claire Drive, to protect the roadway and private property from potential landslide debris.

18.4 25th Avenue S.

In the 25th Avenue S. Stability Improvement Area (refer to Figure C-9), a total of ten landslides have been recorded. They are listed as four deep-seated events and six shallow colluvial landslides. With the exception of one recorded landslide in March 1997, the others occurred in 1974 and before. The earliest listed events (two landslides) took place in December 1933. The landslides here have occurred at approximately 20-year intervals.

In general, this area straddles 25th Avenue S. and slopes downward toward Rainier Avenue S. Eight of ten landslides reportedly involved fill material, presumably placed in conjunction with street grading or residential construction. Two of the deep-seated landslides took place on the downslope side of 25th Avenue S. just south of S. McClellan Street. One of these was initiated by an excavation made for a building that was constructed on the west side of Rainier Avenue S.

The subsurface conditions in this area consist of fill and/or colluvium overlying glacially overridden clay. The sand-clay contact (Tubbs, 1974) is not mapped in this area. The predominant factors contributing to instability are the soil conditions on this sloping area (fill and/or colluvium overlying glacially overridden clay), undercutting or filling on the slope, and heavy rainfall (triggering cause). Other possible contributing factors of instability are steep topography at some locations and high groundwater levels/seepage in the colluvium.

Recommended actions in this area include homeowner education and storm drainage systems maintenance and/or improvement. To improve subsurface drainage, a curb could be installed along the east side of Cheasty Boulevard S. between S. Hinds Street and S. Winthrop Street with the curb extending along Winthrop to 27th Avenue S. The City could also consider construction of an MSE wall along a portion of Cheasty Boulevard (east side) to increase support for the downhill side of the roadway.

18.5 West Beacon Hill

In the West Beacon Hill Stability Improvement Area, as designated on Figure C-9, 38 landslides are listed in the database. These consisted of groundwater blowouts (6), deep-seated landslides (13), shallow colluvial landslides (16), and 3 not identified as to landslide type. The earliest landslide was recorded in 1921, and 21 of the 38 instabilities occurred prior to 1960. Others occurred in the 1960s (7), one in 1972, two in 1986, one each in 1987 and 1990, three in 1997, and two were noted to have taken place in 1999.

The landslides in this improvement area took place on the west-facing slope downhill from 15th Avenue S., and uphill from the Interstate 5 (I-5) alignment (I-5 constructed in this area in the 1960s). As previously noted (Section 18.1), construction of I-5 effectively increased stability in this location. A number of landslides occurred in areas of residential fills.

The subsurface soils in this area consist of fill and/or colluvium overlying glacially overridden till, sand, or clay. The sand-clay contact (Tubbs, 1974) runs in a north-south direction through this area, where groundwater seepage can be expected into near-surface soils. The factors contributing to instability in this area consist of steep topography, fill and/or colluvium on the slope, high groundwater levels with associated seepage near the sand-clay contact, and heavy rainfall (triggering cause).

Recommended actions consist of storm drainage systems maintenance and/or improvement and homeowner education. Homeowner education could stress prudent drainage and construction practices, and filling should not take place unless suitably supported using competent geotechnical advice. There are a number of locations where steep slopes exist adjacent to residential properties or roadways. Yard waste and filling at the top or over the slope should not take place. The City could also consider the construction of a new retaining/catchment wall on the east of 13th Avenue S. between Bayview and S. Lander Streets to protect the roadway.

18.6 Duwamish

Figure C-9 shows the location of the Duwamish Stability Improvement Area, located west of the Duwamish Waterway. Twenty-four landslides occurred in this area throughout the years beginning with a 1922 event. Three occurred in 1997 and two were observed to have occurred in early 1999. Most of these landslides (19) were recorded as shallow colluvial events, while the others were groundwater blowout (1), deep-seated (3), and unidentified as to type (1).

Many of the landslides in this area brought debris down onto W. Marginal Way S.W. Two early landslides (one dated 1922; the other 1923) were reportedly related to the grading of W. Marginal Way. Another landslide, dated 1926, was reportedly related to the grading for 9th Avenue S.W. (located near Highland Park Way S.W.). Along the west side of W. Marginal Way and some distance to the north of Highland Park Way, an ivy-covered toe wall (appears to be wood), approximately 500 feet in length, is present.

This improvement area is generally an east-facing slope, except near the north and south portions of this improvement area. At the north end, a portion of this area slopes down to the north. In the south where Highland Park Way follows a ravine uphill to the west and south, landslides have occurred on slopes facing east, north, and west.

Colluvium overlying glacially overridden clay is generally present in this area. At two locations, the glacially overridden soils were listed as sand or glacial till, and at two other locations, fill material was involved in the instability. The sand-clay contact (Tubbs, 1974) runs in a north-south direction through this area as shown on Figure C-9. Groundwater seepage can be expected near the sand-clay contact.

The factors that contribute to instability in this area are steep topography, colluvium on the slope (mostly overlying glacially overridden clay), cutting and filling on the slope, and high groundwater levels/seepage. The landslides were triggered by heavy rainfall that results in surface runoff and infiltration into slope soils.

It is recommended that actions by the City to improve stability include maintaining existing storm drainage facilities and improving them when indicated by future observations in this area. Homeowner education could stress prudent construction practices. The City could also consider construction of a retaining/catchment along a portion of W. Marginal Way on the west side of the roadway to protect the roadway; its location would be further north than the existing toe wall mentioned previously.

18.7 Rainier Beach

The Rainier Beach Stability Improvement Area is located in the southeast corner of the City as indicated on Figure C-9. In this area, 27 landslides are shown. The earliest landslides were recorded in 1914 (two events), 1918, and 1924. No further instability was reported until 1951, and then landsliding occurred in the 1950s (3), 1960s (4), 1970s (3), 1980s (8), and 1990s (5). The most recent instability was observed in 1999. Most of the landslides in this area were shallow colluvial instability (21). The others were a groundwater blowout (1), deep-seated landslides (4), and one that was not identified as to type.

The landslides in this area generally occurred on the northeast-facing hillside that slopes down to Lake Washington. A number of the slides took place immediately upslope from Rainier Avenue S., bringing landslide debris down onto the sidewalk or roadway. At present, portions of the sidewalk along the south side of Rainier Avenue are permanently closed. One landslide that occurred in 1914 was reportedly related to the grading work (cutting at slope toe) for Rainier Avenue. Uphill from Rainier Avenue, a number of landslides occurred in fill material placed on private properties.

The subsurface conditions in this area consist of fill and/or colluvium overlying glacially overridden soils. For the most part, based on the database information, the overridden soils consist of clay. (Geologic maps indicate that glacial till may also be present in this area.) The sand-clay contact (Tubbs, 1974) is not mapped in this area. Recent (1999) field visits have noted that groundwater seepage is present in this area, particularly in the bowl-shaped areas extending uphill (west) from Rainier Avenue.

The factors that contribute to instability in this area are steep topography, fill and/or colluvium on the slope, high groundwater levels/seepage, cutting or filling, and heavy rainfall (triggering cause).

Recommended actions include storm drainage systems maintenance and/or improvement and homeowner education. In addition, the City could consider the construction of a retaining/catchment wall along the west side of Rainier Avenue to prevent debris from accumulating on the sidewalk or roadway.

Stability Improvement Area[1,2]	Unit[7]	NORTHWEST SEATTLE: Broadview	NORTHWEST SEATTLE: 25th Ave. N.W.	NORTHWEST SEATTLE: Carkeek	NORTHWEST SEATTLE: Blue Ridge	NORTHWEST SEATTLE: Golden Gardens	NORTHWEST SEATTLE: Shilshole	NORTHEAST SEATTLE: Burke Gilman	NORTHEAST SEATTLE: Inverness	NORTHEAST SEATTLE: Laurelhurst	CAPITOL HILL: North Capitol Hill	CAPITOL HILL: Interlaken	CAPITOL HILL: West Capitol Hill	SOUTH SEATTLE: Mount Baker	SOUTH SEATTLE: 25th Avenue South	SOUTH SEATTLE: West Beacon Hill	SOUTH SEATTLE: Duwamish	SOUTH SEATTLE: Rainier Beach
Number of Landslides																		
High Bluff Peeloff		3			2	6	4	2										
Groundwater Blowout		18			11	1	6	1				1				6	1	1
Deep-seated		2		1	3	2		1	5		1	10	2	3	4	13	3	4
Shallow Colluvial		22	8	12	5	17	9	35	17	20	8	22	11	8	6	16	19	21
Unidentified		2									5	9	3	3		3	1	1
Total		47	8	13	21	26	19	39	22	20	14	42	16	14	10	38	24	27
Subsurface Conditions[3]																		
Colluvium Over Glacially Overridden Clay					X	X						X		X	X		X	X
Colluvium Over Glacially Overridden Sand and Gravel																		
Colluvium Over Glacially Overridden Sand-Clay			X	X	X	X		X	X					X				
Colluvium Over Glacially Overridden Till-Sand-Clay		X				X	X			X	X		X	X		X		
Colluvium Over Glacially Overridden Till-Clay														X				X
Sand-Clay Contact (Tubbs, 1974) Mapped in Area		X	X	X	X	X	X	X	X							X	X	
Contributing Causes of Instability																		
Steep Topography		X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X
Loose Fill or Colluvium on Slope		X	X	X	X	X	X	X		X	X	X	X	X	X	X	X	X
Colluvium Over Clay						X						X		X	X		X	X
High Groundwater Levels (Seepage and Springs)		X	X		X	X	X	X	X			X	X		X	X	X	X
Road Cuts and Fills (Public)				X											X		X	X
Undercutting and Filling (Private)				X		X	X	X			X	X		X	X		X	X
Improperly Directed Surface Water					X	X								X				
Heavy Rainfall with Surface Runoff (Trigger Mechanism)		X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X
Stability Improvements																		
Homeowner Education		X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X
Strom Drain Maintenance/Improvement (Curbs/Gutters/Catchbasins)		X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X
Trench Subdrains (10 ft deep)	LF																	
Trench Subdrains (15 ft deep w/ trenchbox)	LF					1,600						200						
Finger Drains	EA																	
Springhead Drains	EA																	
Horizontal Drains-Cleaning						8,450												
New Horizontal Drains						7,500												
Mechanically Stabilized Earth Wall[5]	SF					5,000		5,000				2,200			1,200			
Geotextile Reinforced Soil Slope	SF																	
Slope Grading (Excavation)	CY																	
Machine Formed Curbs	LF			400											1,300			
Retaining/Catchment Wall (10 ft high)	SF			17,000		4,000		5,000		500	5,000	9,400	7,000	5,500		2,250	3,000	19,500
Fill Stabilization-Excavation and Replacement (20 ft wide, 7 ft deep)[4]	CY											2,000						
Excavation	CY											2,000						
Soil Backfill and Compaction	CY											2,000						
Asphalt Paving (4-inch thick including base)	SY											860						
Machine Formed Concrete Curbs	LF											385						
Drainage Improvements[6]																		

General Note:

The Stability Improvements presented here are general types of measures that could be considered by the City, private property owners, or both, to improve stability. The number, length, square footage, etc., listed are very rough estimates of work on City and/or private properties presented only as a basis to formulate order-of-magnitude budgets.

Notes:

1. This table should be used in conjunction with the text describing each Stability Improvement Area, and with the cost data presented in Table 2-1.
2. The stability improvements listed here are preliminary and are presented to provide the city and private property owners with data for use in prioritizing work and developing order-of-magnitude budgets. Final scopes of work and corresponding cost estimates should be based on additional engineering studies and subsurface explorations.
3. Subsurface conditions may vary within a particular Stability Improvement Area. Many sites contain fill material on a slope or at the top of the slope.
4. Includes excavation of listed volume of material (CY), replacement soil backfill and compaction, installation of drainage improvements (if necessary), asphalt paving, and installation of machine formed concrete curbs. See individual costs for each of these items, as deemed necessary.
5. Standard MSE wall.
6 If necessary, type and quantity will depend upon site conditions.
7. CY = cubic yard, EA = each, LF = lineal foot, SF = square foot, SY = square yard

Seattle Landslide Study
Seattle Public Utilities
Seattle, Washington

STABILITY IMPROVEMENT AREAS
NORTHWEST, NORTHEAST,
CAPITOL HILL, SOUTH SEATTLE

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SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

TABLE 4-1

Table.4-1.2nd

PART 5. POTENTIAL SLIDE AREAS

19.0 PURPOSE AND SCOPE

This part of the report presents an update of the Potential Slide Areas (PSAs), a designation used primarily by the Department of Design Construction and Land Use (DCLU) in the regulation of residential and commercial construction in the City of Seattle (City). PSAs have been used in conjunction with Steep Slope Areas and Known Slide Areas to identify those areas of the City where additional reconnaissance and engineering work needs to be accomplished when new construction or major remodel work takes place. However, it became apparent in the course of this Seattle Landslide Study that well-known landslide-prone areas were not represented on the PSA map. Therefore, this analysis of the PSAs was undertaken to discover and document the inconsistencies between the maps and the features on the ground.

All areas of the City were evaluated in this part of the study. Areas with high concentrations of landslides were printed on 26 maps at a scale of 1:4800 using Arc View Geographic Information System (GIS) and City of Seattle ArcInfo coverages. Base map coverage information included topography, property lines, streets, historical landslide initiation locations, trace of the sand/clay contact, and existing PSA boundaries. Revisions to the PSA boundaries were plotted on the maps in the office, based on landslide concentrations, topography, and general knowledge of geologic conditions. Based on previous knowledge of certain landslide-prone areas and the historical record, about one-quarter of the sites were delineated with high confidence.

After the preliminary re-delineation of the PSAs in the office, we performed a field review of the proposed revisions to the PSA maps. The remaining three-fourths of the areas were visited to evaluate the field conditions and to accurately draw the boundaries. This also included the delineation of runout zones at the toes of hillsides and setback zones at the tops of very steep slopes or bluffs.

Meetings were held with representatives of DCLU and Seattle Public Utilities (SPU) before the revision process to discuss the criteria to be used and after the field verification of the revised boundaries to discuss the results.

20.0 BACKGROUND

The existing PSA maps used by DCLU, primarily in the building permit process, are based on conclusions in the Ph.D. dissertation of Mr. Donald Tubbs, *Causes, Mechanisms and Prediction of Landsliding in Seattle*, 1975. Mr. Lloyd Finney of the Seattle Engineering Department started methodical classification of landslide information in the 1960s, but zonation of the landslide-prone areas of the City were not codified until the Environmentally Critical Areas (ECA) were published in a map folio in 1992. The criteria used for the PSAs were taken directly from the Tubbs dissertation and they included the following factors:

(1) All Class 4 zones, a strip of approximately 200 feet wide along the trace of the Esperance Sand/Lawton Clay or pre-Vashon sediments contact.

(2) All Class 3 zones, areas steeper than 15 percent slope gradient and underlain by the Lawton Clay or pre-Vashon sediments.

(3) Areas with springs or groundwater seepage; however, this criterion was not mapped.

During analysis of the landslide study information, it became apparent that there were many inconsistencies among the actual landslide pattern, the electronic layer of the PSAs on the City's GIS system, and the ECA map folio. The reasons appear to be:

(1) Differences between the 1958 topographic base map used by Tubbs to construct the original hand-drawn maps and the topographic base map used in the ECA folio.

(2) Large areas of PSA not properly transferred from the original hand-drawn maps to the GIS layer.

(3) Geologic factors not previously considered, such as setbacks from steep bluff, runout zones at the toes of hillsides, instability not related to the Lawton Clay or pre-Vashon sediments, and geologic conditions unknown at the time of Tubbs' work.

21.0 RESULTS

Several criteria were used to revise the boundaries of the PSAs. They include the following factors:

(1) Areas with historical record of landsliding.

(2) Signs of past landsliding observed in the field, such as landslide scars and deposits.

(3) Signs of potential landsliding observed in the field, such as springs, groundwater seepage, and bowed or backtilted trees.

(4) Topographic expression of runout zones, such as fans and colluvial deposition at the toes of hillsides.

(5) Setbacks from very steep slopes or bluffs.

(6) Extrapolation of the above factors to areas of similar and contiguous topography and geology.

The most important criterion used in this evaluation was the historical record. The other criteria were used to supplement the analysis.

Twenty-six work maps were prepared with field notes at a scale of 1:4800 showing the original PSA boundaries and the revisions to them. Two larger scale maps with the same information show the entire City on one map. Figure D-1 indicates the existing (1992) PSAs based on the current data layer from the City of Seattle GIS department, and Figure D-2 shows the proposed revised PSAs. In some cases, entirely new PSAs were created; however, in most cases, the boundaries were shifted slightly. In some parts of the City, areas previously mapped as PSAs were eliminated or reduced because current geologic or topography information could not justify the original boundary. Absent any new information to change a boundary, no revisions were made. At the request of DCLU, a map (Figure D-3) was prepared that indicates areas where regulated land, which includes steep slopes (steeper than 40 percent) and PSAs, would be lost or gained due to the proposed revisions to the PSAs. The following is a brief summary of the significant revisions recommended for the PSA maps.

Northeast Seattle – The original PSA was a strip along the lower portion of the slope overlooking Lake Washington. The boundary was extended uphill to encompass the 23 landslides that were recorded on the higher elevation slopes.

Inverness/Sand Point – The PSA was extended southward about 4,000 feet because of an apparent error in the transfer of information from the ECA map and the GIS layer.

Windermere – A new PSA was delineated based on the presence of three landslides on very steep slopes that had several signs of past landsliding.

Laurelhurst – A new PSA was created because of 18 reported landslides, very steep topography, and the presence of springs and landslide scars.

Interlaken – The largest new PSA in the City was created in this very steep ravine system, based on the presence of widespread landslide features and 37 reported landslides.

North Capitol Hill – The slope facing Lake Union, along Eastlake, was eliminated as a PSA owing to minimal occurrence of landsliding in the area, and because the steeper portions of the areas would be covered by the steep slope criteria. The steep slope between Furman Avenue E. and Portage bay Place E. was placed in a PSA because of the high concentration of reported landslides, seepage, and widespread evidence of other landsliding.

St. Mark's Greenbelt – The PSA boundaries were adjusted by eliminating an apparently landslide-benign slope at the north end of the area, and by extending the boundary uphill to the top of the steep slope to encompass 10 reported landslides on the upper portion of the hillside.

Colman Park – The PSA boundaries were extended southward to cover Colman Park, a large amphitheater-like ravine, which contained seepage and widespread evidence of past landsliding. Two other new PSAs were created to the south of this area where reported landslides, old landslide features, and very steep topography were contiguous.

Rainier Beach/Taylor Creek – A new PSA was created on the steep slopes south and west of Rainier Avenue s. due to the presence of 25 reported landslides on steep ground that included many areas of groundwater seepage, and extended up the Taylor Creek drainage, based on the widespread occurrence of springs on hillsides, old landslide scars, and very steep topography.

Myers way S. – A new PSA was created owing to the presence of widespread springs, signs of landslide topography, and a history of slope stabilization measures along this hillside for SR-509.

Seola Beach Drive S.W. – A new PSA was created to encompass a concentration of seven landslides and very steep slopes in the upper portion of the ravine along Seola Beach Drive.

S.W. 47th Street – The PSA was extended southward about ¾ miles because of an apparent error in the transfer of information from the ECA map to the GIS layer.

Fauntleroy Creek – An existing PSA was extended up this ravine to encompass the zones of heavy seepage and hummocky ground surface, both indicative of unstable slopes.

Schmitz Park – An existing PSA was extended up this ravine to encompass the zones of heavy seepage and hummocky ground surface, both indicative of unstable slopes.

Alki Avenue/Sunset Avenue – The existing PSA was extended uphill to encompass 29 reported landslides along the upper portion of this steep slope, and downhill to encompass the potential or historical runout zones of landslides or debris flows.

Harbor Avenue/Admiral Way – The existing PSA was extended uphill to encompass 26 reported landslides along the upper portion of this steep slope and widespread signs of past landsliding activity.

East Queen Anne – The original PSA was a strip along the lower portion of the slope west of Lake Union. The boundary was extended to the very steep slope west of Aurora Avenue to encompass ten landslides that were recorded on the higher elevation slopes.

North Magnolia – The area around discovery Park was encompassed because of very steep slopes that showed signs of past instability. The area along the entrance to the Government Locks was created because of a concentration of reported and observed landslides on this steep slope.

Northwest Seattle – Several areas of existing PSA were extended uphill to encompass very steep slopes that contained reported landslides, and some were revised due to differences in topographic contours since the 1958 survey.

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