OREGON DEPARTMENT OF TRANSPORTATION

Technical Services Branch Geo-Environmental Section

Geotechnical Design Manual

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Geotechnical Design Manual

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1 Introduction

1.1 General

The *ODOT Geotechnical Design Manual (GDM)* establishes standard policies and procedures regarding geotechnical work performed for ODOT. The manual covers geotechnical investigations, analysis, design and reporting for earthwork and structures for highways. The purpose of the geotechnical investigation and design recommendations is to furnish information for an optimum design which will minimize over-conservatism as well as to minimize under-design and the resulting failures commonly and mistakenly attributed to unforeseen conditions.

It is to be understood that any geotechnical investigation and design will leave certain areas unexplored. Further, it must also be understood that it would be impractical to provide a rigid set of specifications for all possible cases. Therefore, this manual will not address all subsurface problems and leaves many areas where individual engineering judgment must be used. It is intended that the procedures discussed in this manual will establish a reasonable and uniform set of policies and procedures while maintaining sufficient flexibility to permit the application of engineering analysis to the solution of geotechnical problems.

This manual references publications which present specific engineering design and construction procedures or laboratory testing procedures. Each chapter contains a listing of associated references for the subject area of the chapter. Among the commonly referenced materials are the publications of the American Association of State Highway Transportation Officials (**AASHTO**), the Federal Highway Administration (**FHWA**), and the American Society for Testing and Materials (**ASTM**). Relative to testing and design procedures, the methods presented by AASHTO and FHWA are often followed.

Figures presented in the manual have been redrafted from the original published figures. The figures in this GDM are only to be used for illustrative purposes and should not be used for design.

1.1.1 Acknowledgments

This *ODOT Geotechnical Design Manual* is a completely new manual and is the product of the combined efforts of the personnel in the HQ Engineering and Asset Management Unit. Thanks for their work and appreciation for their contributions are extended here. Continued work is required to edit and update the manual and their help will be appreciated in the future. An additional thanks and acknowledgement is given to Tony Allen of WSDOT for permission to use, en masse, whole sections, paragraphs, and even an entire chapter or two in the development of this manual.

The completion of the *WSDOT Geotechnical Design Manual* in September 2005 provided the spark and impetus for ODOT to finally, after many years of wishing it, produce a Geotechnical Design Manual worthy of the importance of geotechnical design on highway projects.

1.2 Manual Review and Comment Process

The ODOT Engineering and Asset Management Unit of the Geo-Environmental Section is responsible for the publication and modification of this manual. Any comments or questions about the *ODOT Geotechnical Design Manual* should be directed to:

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1.2.1 Manual Revision Procedure

It is intended that the GDM will be continually updated as required to clarify geotechnical practice in ODOT and include new information. Submittals for revisions are encouraged from all users. The following submittal procedure should be used:

1. **Define the problem**

   Discuss the suggestion or revision of the GDM with others that have a stake in the outcome. If it is agreed that the item should be proposed, develop a written proposal. Changes to design policy, design practice or procedure can have wide ranging effects and will affect some or all of those involved in the preparation of contract documents for ODOT.

2. **Put it in writing**

   Research and develop a written proposal using the three general subject headings:

   - Problem Statement,
   - Analysis/ Research Data; and
   - Proposal.

Check the finished product by reviewing the following guiding comments:

- Is the existing problem clearly stated?
- Is the research and analysis of the problem and potential solution thorough and understandable?
- Is the proposed solution well thought out, is supported by facts and does it solve the problem?
- Have the impacts on other areas been considered?
- Have the details been coordinated with other units or organizations that may be affected?
- Do any questions remain that need to be answered before implementation?

3. Review and Approval

After reviewing the written proposal for completeness, the Engineering and Asset Management Unit will either:

- Accept, without further review, manual corrections for inclusion in the GDM; or
- Distribute a copy with the due date and a Geotechnical Design Practice Approval Form for review and comments.

After reviewing the returned Geotechnical Design Practice Approval Forms, the Engineering and Asset Management Unit will do one of the following:

- Proposals approved for revision of the GDM will be implemented in a technical bulletin and will be placed into the next upcoming version of the GDM;
- Proposals needing more research or clarification will be returned to the originator for revision and resubmittal.

Regardless of whether or not a proposal is accepted, the Engineering and Asset Management Unit will reply in writing to the person making the submittal.

4. Implementation of Approved Revision

After a proposal has final approval, a revised GDM page will be prepared for inclusion into the manual. All revisions will be highlighted in yellow to indicate that the text has been revised or added and the month and year in the right hand bottom corner of each page will also be highlighted to indicate when the revisions or changes were made.

The revised GDM will be published electronically on the ODOT Geo-Environmental web page as soon as practical.

1.3 ODOT Geotechnical Organization

The functions of geotechnical design in ODOT are generally managed and performed within the 5 region offices. Tech Centers within each region are staffed with Geotechnical Engineers, Engineering Geologists, Hydraulics Engineers, and HazMat specialists. The geotechnical design, construction and maintenance support may be performed in-house or contracted out to specialty consultants. The ODOT Technical Leadership Center (TLC) Engineering and Asset Management Unit provides on-call geotechnical design assistance and review, training and software, coordination of section initiatives, and other functions involving development of standards and practices for geotechnical work. Material source and aggregate material program needs are also a function of the headquarters unit.

1.4 Location of Existing Project Information

In general, the regional offices keep file information on past projects. The first inquiry into project geotechnical information should be to the appropriate region Geotechnical office. In addition, project information for past projects involving geotechnical analysis and design has been archived and stored in the ODOT Salem Airport Road complex. A database listing of the projects archived is located in the HQ Salem Engineering and Asset Management office in Salem. The Salem Bridge Engineering Section keeps pile record books for past projects where pile driving was performed. In addition, bridge archives are available that include Foundation Reports, boring logs, as-constructed bridge plans and foundation data sheets. Inquiries regarding bridge foundation records and archives should be directed to the HQ Bridge Section office.

1.5 Consultant Contracting for Geotechnical Work

ODOT has a set of specialty consultants retained to perform geotechnical work as needed. The current list of geotechnical consultants can be obtained from the ODOT Procurement Office (OPO). A Scope of Work Template has been developed for use by those needing to have a consultant perform geotechnical work and is located on the **ODOT Geo-Environmental** website.

1.6 ODOT Professional of Record Policy

Designers of ODOT projects are required to follow the ODOT Technical Services Directive TSB11-02(D) and the ODOT Design Policy DES 05-02. The documents can be found at the following link:

Technical Services Professional of Record Guidance

Chapter

2

2 Project Geotechnical Planning

2.1 General

General geotechnical planning for projects with significant grading, earthworks, and structure foundations, from the earliest project concept plan through final project design are addressed in this chapter. Detailed geotechnical exploration and testing requirements for individual design are covered in detail in **Chapter 3, Chapter 4** and **Chapter 5**. This chapter also provides direction for geotechnical project definition and creation of the subsurface exploration plan for the project design phases. General guidelines for subsurface investigations are provided in **Chapter 3** in addition to specific guidelines regarding the number and types of explorations for project design of specific geotechnical features.

The success of a project is directly related to the early involvement of the geotechnical designers in the design process. For larger projects that involve an Environmental Impact Statement (EIS), the geotechnical designer needs to be involved with the assessment of various options or corridor selections. Ideally, for all projects, the geotechnical designer will be involved during the first scoping efforts. At this point, a study of the project concept is begun by gathering all existing site data and determining the critical features of the project. This information can then be presented at the project kick-off meeting and/or scoping trip. The project scoping trip is a valuable opportunity to introduce the roadway and structural designers, and project leaders to the geologic/geotechnical issues that are expected to impact the project. Continued good communication between the geotechnical designer and the project leader and project team is vital.

2.1.1 Geotechnical Project Elements

All proposed project scopes should be reviewed by an engineering geologist and/or geotechnical engineer for a determination of the project elements (if any) that require a geologic investigation and geotechnical design. This allows the geotechnical designers to begin formulating a prospective scope of work and budget estimate. There are common project elements that are always the subject of a geotechnical investigation and design such as bridge foundations and landslide mitigations, and there are project elements that, depending on the site history and underlying geology, may or may not need investigation and design, or may require different levels of effort. The geotechnical designers will be able to determine the level of effort based on their own or other's knowledge and experience of the site to make these judgments. Because of the underlying site conditions, elements that generally don't warrant geotechnical design for most sites may require it at others. Conversely, investigation and design efforts may be scaled back or eliminated at other sites due to known favorable conditions, and the significance of the project feature. It is the responsibility of the geotechnical designers to make these decisions.

The common project elements on transportation projects that are the subject of engineering geologic investigation and geotechnical design for construction are:

- Structure Foundations (bridges, viaducts, pumping stations, sound walls, buildings, etc.)

- Retaining walls over 4’ in height as measured from the base of the wall footing to the top of the wall and any wall with a foreslope or backslope
- Cuts and embankments over 4’ in height
- Tunnels and underground structures
- Poles, masts and towers
- Culverts, pipes and conduits

This last group of elements, culverts, pipes and conduits, exemplify the broad range of design and investigation that may occur on any project. A 24” culvert replacement at a depth of 3 feet below a proposed roadway alignment would normally require the hand-collection of soil samples from the pipe location, submittal of those samples to the laboratory for chemical properties testing, and forwarding the results to the project designer for selection of the appropriate pipe materials for that location. If however, that same culvert was to be installed under a large, existing embankment while under traffic using trenchless methods, then the required investigation and design effort would be close to what is required for a tunnel or underground structure.

2.1.2 Geotechnical Project Tasks and Workflow

The expected milestones for geotechnical input on projects and the review of geotechnical work is outlined in **APPENDIX 2-A – Geology/Geotechnical QC MATRIX**, and the Project Flowchart.

Certain project checkpoints and tasks may be added or eliminated based on the project scope and/or requirements. Each individual project prospectus should be consulted to determine which tasks and QC checkpoints will apply.

2.2 Preliminary Project Planning

2.2.1 Overall

The creation of an efficient geologic/geotechnical investigation and identification of fatal flaws or critical issues that could impact design and construction as early in project development as possible is essential. Use the maximum amount of effort to obtain the greatest amount of information as early in each phase of investigation as possible so that each successive phase can capitalize on the information previously gathered. The result is a more thorough and cost-effective geologic and geotechnical investigation program.

Projects with a small number of defined structure locations or limited earthwork typically do not require numerous phases of investigation. Such projects normally proceed through an initial background study, site reconnaissance and ensuing subsurface exploration at the TS&L phase. Larger projects in contrast, will usually benefit from a phased sequence of field exploration. The geologic/geotechnical investigation will occur as a reconnaissance-level examination and preliminary subsurface exploration during the Field Survey phase of the project. More detailed, site-specific exploration is accomplished later as the project develops through the TS&L and Approved Design phases.

Phased subsurface exploration is beneficial because:

- Phased subsurface exploration allows information to be obtained in the early stage of the project that can be used to focus the exploration plan for the more detailed design stages. This is where previously gained information can be used to maximize the efficiency of the final exploration, and to assure that previously identified geotechnical problems and/or geologic hazards are thoroughly investigated and characterized.

- Additionally, the Exploration Plan can be more clearly defined and easier to manage. In this regard, the number of borings, their depths, and laboratory testing programs can be determined in advance of actual mobilization of equipment to a project area.

  For most projects, mobilization costs for exploration equipment are high, so efforts should be made to reduce the number of subsurface investigation phases whenever practical. However, the site location, project objectives, and other factors will necessarily influence the investigation phases and mobilizations. Some of the additional factors to consider are site access, availability of specialized equipment, environmental restrictions, safety issues, and traffic control.

  To economize field investigations and provide contingencies for ongoing project changes, consider the following:

- A substantial amount of background study should take place prior to mobilization to a project site. The information derived from this research provides a basis for the design of the Exploration Plan and help focus the on-site investigation.
- In addition, all resources used in the development of the background study should be organized and documented in such a manner that another geotechnical designer would be able to continue the project without going back to the beginning to get the same information. Keep a list of all documents used in the background study, such as field notes and sketches from initial site reconnaissance, reports or investigations from previous or nearby site investigations, and other published literature.
- Any critical issues such as geologic hazards, problem materials or conditions, or contamination identified during the initial study should be clearly documented and highlighted throughout the project to avoid any surprises later on in the design or construction phases.

2.2.1.1 Project Scale and Assignment of Resources

Geotechnical designers should use their professional judgment with respect to the scope, scale, and amount of resources to utilize during preliminary project studies. Larger projects obviously necessitate a greater effort in the early examination of background materials such as previous reports for an area, maps, published literature, aerial photographs and other remote sensing.

Even the smallest bridge replacement or grading project, background study is just as important, and although of a smaller scale, should be carried out with the same diligence as a similar study for a major realignment. A thorough and expedient background study is essential for these smaller projects since unforeseen conditions and additional unplanned field investigations are much more difficult to absorb in a smaller project budget. It follows that for a larger project; a more thorough background investigation is warranted since unforeseen conditions can have a compounding effect during design and construction that may impact even the most generously funded projects.

The amount of background research needed for a project is usually unknown until the study begins and the potential site conditions are assessed to some degree. It is up to the geotechnical designer to determine the amount of background study needed and the cost-benefit of such studies with respect to the project design.

Using Remote Sensing and Existing Information

Ordering new remote sensing studies to assess surrounding landforms is probably not necessary for in-kind bridge replacement projects unless some special conditions are observed during the field or office study. However, failure to procure and study a set of aerial photographs along a proposed realignment would probably be somewhat negligent. Project background studies for major realignment projects and landslide mitigations typically make more use of remote sensing and published literature while replacement and modernization projects will rely more heavily on previous site studies and reports.

2.2.2 Office Study

The foremost objectives of initial office study are 1) early identification of critical issues that will affect the project's scope, schedule, or budget, and 2) efficiently plan detailed site studies and formulate a subsurface investigation program.

2.2.3 Project Stage 1

The first stage of any project should begin with a review of the published and available unpublished literature to gain a thorough understanding of the existing site conditions and composition. Such an understanding includes knowledge of the geologic processes that have been the genesis of, or have in some way affected the project site. The site geomorphology should receive the most scrutiny from the geotechnical designer since characteristic landforms are created by specific geologic processes, and composed of particular materials. The site geomorphology, coupled with the literature and results of previous studies, will aid the geotechnical designer in predicting what materials will be encountered, and how they will be distributed across the site.

2.2.4 Project Stage 2

The second stage of a project involves the detailed examination of the proposed project components and in particular, the geotechnical elements. This includes an appraisal of the project prospectus as well as any conceptual or preliminary plans available from the roadway designer or project leader. The project geotechnical features such as bridge foundations; earth retaining structures, cuts, embankments and any other earthworks should be identified and located. Once the project geotechnical features are recognized, they can then be analyzed with respect to the background information previously collected.

2.2.4.1 Existing Information and Previous Site Investigation Data

Current transportation projects take place almost exclusively on or near existing routes, for which a considerable amount of subsurface information already exists, in most cases. Since many transportation projects take place in urban areas, additional information may also be available from other nearby public works projects and private developments involving structures and earthworks. Local agencies may possess subsurface information for their projects as well as data provided by consultants.

Subsurface information collected for ODOT projects resides in the region geology office in which the data was collected. Subsurface information is collected for bridge foundations, retaining walls, cut slopes, embankments, and landslides. Additional subsurface data has also been collected for incidental structures such as sound walls, sign bridges, poles, masts and towers, and facilities such as water tanks and maintenance buildings.

The **Oregon Water Resources Department** maintains a database of boring logs on its website. By law, reports must be filed with this agency for all geotechnical holes and water, thermal, and monitoring wells. Thus, the database is fully populated, and may be queried in many ways geographically or by owner, number, constructor, or purpose. These logs are beneficial in rural or remote areas with a dearth of subsurface information.

Note:
A wealth of information can be contained on the logs especially regarding groundwater and depth to bedrock information. There is an entry for soil and rock descriptions on the reporting forms. However

this information should be used with caution since there are no standard reporting formats and thus, the soil and rock descriptions on the Water Resources forms vary in content and accuracy.

The Oregon Department of Water Resources Database (ORWD) can be accessed at the following location:

http://apps.wrd.state.or.us/apps/gw/well_log/

In addition to the information provided on the OWRD forms, it is important to simply note the presence of wells in the area that may be affected by the project construction. Projects involving large cut slopes or dewatering efforts can affect the yield of nearby wells. Where this occurs, ODOT typically includes replacement or deepening of the well as part of the Right-of-Way acquisition.

2.2.4.2 Construction Records

Since most current ODOT projects are modernization, replacement, or rehabilitations of existing transportation facilities, construction records are commonly available from various sources throughout the agency. Such records may be in the form of as-built plans, construction reports, pile-driving records, and other technical memoranda addressing specific issues and recommendations during project construction. Locate information using:

- **As-built plans**: As-built plans are normally located in the region office where the project was constructed. The Geometronics Unit maintains the engineering documents in Room 29 of the Transportation Building in Salem where mylars of project plans reside in addition to some of the as-built plans.
- **Pile records**: Pile record books are maintained by the headquarters office of the Bridge Section.

Region project engineers and construction project managers that have completed previous projects in the area should be consulted with respect to the geologic/geotechnical conditions as well as the construction issues related to those conditions. In addition, section maintenance personnel with a long history in an area will posses a wealth of information regarding the performance of existing facilities, problems encountered, and repair activities that have taken place at a particular site.

2.2.4.3 Site History

Past use of a site can greatly affect the design and construction of a project and can also make a significant impact to its timeline and budget. Typically, much of a site's background and past use will be researched and described for a Phase I or II Environmental Site Assessment produced by the environmental specialists or their consultants in the region geology offices. Information concerning the development of Environmental Site Assessments and other site use resources can be found in the HazMat Manual. Environmental Impact Statements (EIS) for previous projects in the area are also an important and concise source of previous and current site use information. Some of the remote sensing methods previously discussed may also help determine previous site use in the absence of historic records.

Hazardous Materials

The presence of hazardous materials in the subsurface not only affects the geotechnical design, and the construction approach to a project, but it also greatly affects how the subsurface investigation program is carried out. For this reason itself, it becomes important for the geotechnical designer to determine if previous use of the site, or surrounding locations could have potentially resulted in subsurface contamination. Such uses include any facility or enterprise engaged in the production, distribution, storage, or use of hazardous substances. Hazardous substances are defined by the **Environmental Protection Agency (EPA)** in 40CFR§261.31 through 261.33. In addition, the EPA further includes as hazardous wastes, such substances with characteristics of Ignitability, Corrosively, Reactivity, and Toxicity according to 40CFR§261.21 through 261.24. For transportation projects, the most commonly contaminated sites are those that are presently, or have previously been occupied by service stations. However, larger manufacturing and processing sites with substantial amounts of contamination are encountered. When geotechnical investigation must be conducted under such conditions, significant preplanning is required not only to protect the field crew, but also to comply with the numerous environmental regulations that govern everything from required PPE to disposal of contaminated drill cuttings.

Previous Site Use

In addition to contaminated materials, previous site uses have the possibility of leaving behind materials and/or conditions that can be detrimental to the construction or performance of a facility if not properly mitigated. In this regard, deleterious fill materials such as wood waste and ash are commonly associated with timber processing and other operations throughout the state while reclaimed quarries may be filled with deep, unconsolidated debris and spoils. Underground mines and tunnels are present in various locations throughout Oregon. Although uncommon, some instances of such features unexpectedly encountered during construction have occurred. In addition to their obvious geotechnical impacts, such features may be historic locations and thus, be protected by Federal law.

Previous Site Occupation

In addition to previous site use, the geotechnical designer must also consider previous site occupation. A site previously occupied by Native Americans can contain artifacts, or be of significance to contemporaries. Such occupation may require archaeological investigation or preservation activities by qualified personnel. It is also possible that the exploration plan, or even significant project design changes prior to on-site geotechnical investigation will be required. Historic sites, structures, and even trees will also be protected in some instances that will necessitate adjustments to the proposed investigation. Clearly, much of the archaeological and historical issues in connection with a site are outside the purview of engineering geology and geotechnical engineering. However, the geotechnical designer must be aware of these issues to assure that field investigation activities are compliant with the laws and regulations that protect these resources.

2.2.4.4 Office Research for Bridge Foundations

In addition to the sources of information listed above, office research for bridge foundation work generally consists of a review of foundations for the existing structure and any other pertinent foundation information on other nearby structures. The structure owner may have subsurface information such as soil boring logs or “as-constructed” foundation information such as spread footing elevations, pile tip elevations, or pile driving records. The HQ Bridge Section archives contain Foundation Reports and boring logs for many bridges constructed between the mid 1960s to about 2001. Subsurface information on some earlier ODOT bridges may also be available in the Bridge Section construction records. Between about 1999 and 2004, bridge foundation files, reports and records for most bridges were stored in the Salem Geo/Hydro Section archives (now the Geo-Environmental Section archives). Copies of these reports should also exist in the region offices.

Maintenance and construction records for existing bridge(s) should also be reviewed for information relevant to the design and construction of the proposed structure. These records are available in the HQ Bridge Engineering Library or from the Bridge Section Archives. As-Constructed bridge drawings are available online, internally to ODOT through the ODOT Bridge Data System (BDS). Pile driving record books are also available from the HQ Bridge Section.

Office research work for structure foundations typically includes (but is not limited to) gathering the following information for the existing structure(s):

- Location and structure dimensions, number of spans, year constructed
- Superstructure type (e.g. RCDG, composite, steel beam)

- Subsurface data (e.g. foundation reports, boring logs, data sheets, groundwater conditions, etc.)
- Type of Foundation (e.g. spread footings, piles, shafts)

Applicable "as-constructed" foundation information such as:

- Spread footing elevation, dimensions, and design or applied load
- Pile type and size, pile tip elevations or lengths, design or actual driven pile capacity and the method used to determine capacity (resistance) (dynamic formula (ENR, Gates), wave equation, PDA/CAPWAP)
- Drilled shaft diameter, tip elevations
- Construction problems (e.g., groundwater problems, boulders or other obstructions, caving, difficult shoring/cofferdam construction).
- Foundation–related maintenance problems (e.g., approach fill or bridge settlement, scour problems, rip rap placement, corrosion, slope stability or drainage problems)

A review of old roadway design plans, air photos, and soil and geology maps and well logs may also be useful. Particular attention should be given to locating any existing or abandoned foundations or underground utilities in the proposed structure location. Any obstructions or other existing conditions that may influence the bridge design, bent layout or construction should be communicated directly to the structural designer as soon as possible so these conditions can be taken into account in the design of the structure.

This information should be summarized and provided in the Geotechnical Report. All applicable "as-constructed" drawings or boring logs for the existing structure should be included in the Geotechnical Report Appendices.

2.2.4.5 Site Geology

The underlying geology of a project site provides important information concerning the conditions that may be encountered during the investigation and construction phases of a project. Of equal importance is the indication of conditions that either may not be encountered, or will require specific procedures to determine if they do exist. Some particularly deep bedrock horizons, groundwater surfaces, and boulders or other obstructions are examples. Certain conditions can be expected due to the nature of the project site geology.

Oregon has specific geologic terrains, formations and units with distinct constituents, properties, and characteristics that greatly affect the design and investigation of a transportation project. For example:

- Many of the volcanic rocks that compose the Coast range, Willamette Valley, and Cascades can exhibit deeply weathered soil horizons with isolated zones of less weathered materials, interbeds of weak tuff and other unconsolidated tephra.
- Many of the coastal and inland valleys contain deep, soft sedimentary deposits formed by a rising sea level at the end of the Pleistocene.
- The Klamath Terrain in the southwestern portion of the State is a complex mixture of materials that present difficult conditions for the exploration as well as construction.

Numerous published and unpublished documents are available that provide enough information upon which to base a background study. Naturally, many portions of the State have more information than others depending on population densities and previous site uses. However, some basic information is available throughout the state that can be used for most projects. The following sections provide a discussion of the most common publications and how they contribute to a background project study.

Procedures and techniques for the interpretation of maps, aerial photographs, and other remote sensing products can be found in a wide variety of texts and other publications. Several engineering geology textbooks provide a good background in geologic interpretation for engineering projects. However, landform recognition methods are also very well presented in numerous geography texts and other related books devoted entirely to remote sensing and/or GIS. Geologic interpretation with specific emphasis on landslides is treated in Chapter 8 of the 1996 *TRB Landslides* publication.

Topographic Maps

The **U.S. Geological Survey (USGS)** prepares and publishes 7.5-minute topographic maps at a scale of 1:24,000 for the entire State, and for most of the rest of the U.S. Topographic maps can be used to extract both physical and cultural information about the landscape and their consultation should be the first step in any site investigation. Contour lines provide information about slopes as well as indications of the underlying geology and geomorphology. The drainage patterns that develop in the contour lines also suggest geologic and human factors that may have influenced site conditions. Transportation and development patterns portrayed on USGS quad sheets are an often-overlooked source of information. Many roads are aligned to avoid existing geologic hazards or areas where construction difficulties are expected such as wetlands, steep slopes, or hard, resistant rock cuts. Quarry and mine site locations are also an important clue with respect to the location and distribution of bedrock materials.

15-minute topographic maps, also produced by the USGS at a scale of 1:62,500 are also commonly available, but since they have been discontinued in favor of the 7.5' quad sheets, are becoming increasingly rare. The advantage of the 15-minute maps is that they can be very old and may show how land-use has changed in an area since their original survey. Previously existing wetlands that have since been filled or drained, waste areas, quarries, abandoned mines and other problematic areas with respect to transportation projects may be identified. Topographic maps should always be used to identify the arcuate headscarps and hummocky terrain indicative of landslides, wetlands, and general site accessibility with respect to investigation as well as construction.

Sources of Aerial Photos

Aerial photography is the most common, reliable, easy to use, and usually the cheapest source of remote sensing available. Aerial photos are very useful in planning subsurface investigation programs from gaining general knowledge regarding the geology, the extent and distribution of materials, the location of geologic hazards, potential for encountering contaminants, and determining access for exploration equipment.

Aerial photographs are widely available through a variety of sources. The ODOT Geometronics Unit would be the first source for aerial photos as their archives date back to the early 1950s and primarily cover the areas around the State's highways and the Oregon coastline.

Instructions and forms for ordering aerial photographs from the ODOT Geometronics Unit will be found on the Agency's website at:

http://egov.oregon.gov/ODOT/HWY/GEOMETRONICS/AerialPhoto.shtml.

Additional sources of aerial photography are:

ODOT Geometronics Unit –

http://egov.oregon.gov/ODOT/HWY/GEOMETRONICS/AerialPhoto.shtml

The US Geological Survey

http://www.usgs.gov/science/science.php?term=700

USGS EROS Data Center

http://edcwww.cr.usgs.gov/

The USDA Aerial Photo Archives

http://www.apfo.usda.gov/

Bureau of Land Management

http://www.blm.gov/nstc/library/pdf/TN428.pdf

University of Oregon's Aerial Photography Library

http://libweb.uoregon.edu/map/orephoto/mapresearch.html

WAC Corporation

http://www.waccorp.com/

Spencer B. Gross, Photogrammetric Engineering

http://www.sbgreno.com/index.html

Intermountain Aerial Survey

http://www.ias-map.com/

Bergman Photographic Services

http://www.bergmanphotographic.com/

Many County Surveyor and/or Assessors offices throughout the State are an additional source of aerial photography. There are also a number of internet resources for low-resolution images for site location or other less-detailed applications.

General Use of Arial Photography

Aerial photographs may be taken on either black and white or color film. Each of them have characteristics that make them superior to one another for different applications although color photographs are generally considered better since many objects are easier to identify when shown in their natural colors. Things to consider include:

- Color photos also allow for the application of color contrasts and tonal variations to interpretations. In some circumstances, black and white photographs allow the geologist or engineer to resolve changes in slope or elevation that may otherwise be lost in the subtle color changes when using natural color aerial photos.
- Another, less commonly available type of aerial photograph are those taken in false color or infrared (IR). Color IR photography responds to a different electromagnetic spectrum than natural photography. Differences in soil moisture, vegetation type and soil and rock exposure are more readily identified on color IR film.
- Ideally, both black and white as well as color photos of a site should be analyzed for a complete analysis of all features unless color IR photos are available in which case it is generally agreed that for engineering geologic interpretation, natural color and color IR transparencies provide the best information.

With a general understanding of the site geology, the lateral extent of certain geologic features and deposits can be estimated from aerial photography. With a stereo-pair of photographs, the vertical extent can also be estimated in some circumstances. The use of stereo-pairs significantly increases the ease and accuracy of geomorphic interpretation. Subtle landforms may be discerned that may otherwise be hidden from view either on-site or on a two-dimensional image.

Geomorphic Identification from Aerial Photography

Landform identification regularly allows the general subsurface conditions to be determined within the boundaries of that particular feature and thus, an opening impression of the materials to be encountered. Recognized landforms result from particular geologic mechanisms that allow such determinations to be made. These landforms are formed by distinct processes such as fluvial, glacial, or Aeolian and so they are composed of particular materials and compositions. Drainage patterns that develop within or as a result of certain landforms and geologic structures can be used as a diagnostic feature when studying aerial photographs. One of the more important landforms to distinguish during a preliminary study of aerial photographs is landslides. Landslides are readily identified by their characteristic arcuate headscarps, patterns of disturbed soil and vegetation, standing water on slopes with no apparent source or discharge (sag ponds), abrupt changes in slope, disrupted or truncated drainage patterns, and upslope terraces.

Other Applications of Aerial Photography

Vegetation is another important feature to evaluate on aerial photographs since it frequently reveals certain subsurface conditions. Vegetative cover is related to numerous factors including soil development on certain bedrock units, depth of the soil profile, drainage and natural moisture content, climate, and slope angle. The relationship between clear-cutting of forests and debris flows or ad jacent land instability is becoming increasingly important. Consequently, identification of such conditions within or near a project site is essential. In addition to the geologic characteristics, the condition or absence of vegetation may be a sign of soil contamination. Zones of dead or discolored vegetation can indicate the presence of a spill or chemical dump site that field exploration crews may not be prepared to encounter.

It is also important to review a sequence of aerial photographs from different years to determine the history of site use and the natural or human-caused changes that have occurred. Significant changes in the ground contours and shapes can indicate changes due to geologic processes such as landslides, erosion, and subsidence or changes due to construction on the site such as filling and excavation. Other aspects of the site's history that can be determined are the activities that occurred on site such as chemical processing, fuel storage, waste treatment, or similar activities which may leave contaminated or other deleterious materials behind.

Geologic Maps

The **Oregon Department of Geology and Mineral Industries** (DOGAMI), USGS, **US Department of Energy**, and other agencies publish geologic maps of most of the state at various scales. The USGS has published a map of the entire state at a 1:500,000 scale. These geologic maps generally use the USGS topographic maps as a base layer. Geologic maps portray the distribution of geologic units and provide a general description of each that includes the rock or sediment type, geologic age, origin, and brief summary of its properties and physical characteristics. Additional information concerning geologic hazards, groundwater, and economic geology is typically included.

DOGAMI also publishes special studies on geologic hazards in certain heavily populated or problematic areas of Oregon. Geologic Hazard maps are generally produced to portray specific themes such as slope stability, liquefaction potential, amplification of peak rock accelerations, and potential tsunami inundation zones. Such maps provide a general indication of the extent and magnitude of the hazards they were produced to portray.

Geologic maps for the state are available from DOGAMI and at most of the State Universities libraries. Publications are also available for purchase on line from DOGAMI at **http://www.naturenw.org/**. In addition, many local agencies and municipalities have contracted for hazard mapping and planning. These publications may be available from the local agency offices. DOGAMI is now in the process of a digital map compilation for the state. This compilation allows for the electronic querying of geologic information published in a selected area. The geologic information contains pertinent engineering characteristics in many areas. Currently, the compilation map for the NE sextant of the state is available on CD.

Soil Surveys

The US Department of Agriculture, Soil Conservation Service has published soil surveys for all of the counties in Oregon. Although these reports are intended for agricultural use, they provide valuable information on the surficial soils in and around a project area. These bound volumes include maps and aerial photographs showing the lateral extent of soil units and a description of the overall physical geography including local relief, drainage, climate, vegetation, and description of each soil unit together with its genesis. Commonly, the soil units are overlain on a topographic and aerial photographic base. The reports contain engineering classifications of the surficial soil units, a discussion of their characteristics such as drainage and susceptibility to erosion suitability for use in some construction applications.

Remote Sensing and Satellite Imagery

Remote sensing, by the largest definition, involves the collection of data about an area without actual contact. By this definition, the previously discussed methods of air photo and map interpretation would be classified as remote sensing. However; for this section, remote sensing is restricted to imagery obtained by systems other than cameras, or images that are enhanced to distinguish different characteristics of the earth's surface.

Remote sensing as discussed in this section generally utilizes sensors that detect particular electromagnetic energy spectra that is mostly generated from the sun and subsequently reflected or emitted from earth. In addition, active systems that transmit and detect energy from the same platform such as an airplane or satellite are also used to collect imagery. The primary purpose of this distinction is that aerial photographs allow examination of images in the electromagnetic spectrum visible to the human eye. Other imagery allows examination of features with reflectance or energy emission properties that are either outside the spectrum visible to humans or occur with other features with overlapping spectral reflectance that obscures them to the human eye. Examples of these other remote sensing systems are; Multispectral Scanning Imagery (MSS), Thermal Infrared Imagery (Thermal IR), Microwave Imagery (Radar), and Light Detection and Ranging (LiDAR). Despite their advantages, these remote sensing systems are not a substitute for stereo photographs and their higher detail, interpretive returns, and overall economy. They are merely a tool to allow additional interpretation capability for engineering geologic studies.

Thermal Infrared Imagery

These systems obtain images from the thermal wavelength range, generally from 8µm to 14µm, and contain the energy emitted from the earth that was previously stored as solar energy. The thermal properties such as conductivity, specific heat and density of various materials produce different responses to temperature changes. Such responses can be measured to allow differentiation of various surface materials. In a sense, thermal IR imagery can be described as a photograph of the earth's albedo.

Obviously, the longer wavelength of thermal IR images will result in a much lower resolution than a corresponding photographic image. For this reason, thermal data is used to enhance images of areas with certain surface conditions that are not generally detected by aerial photography. In this regard, areas composed of materials with similar or overlapping reflectance properties may not show up on an aerial photograph, but their different thermal properties will make them stand out on a thermal IR image.

The primary uses of thermal IR imagery are for mapping changes in soil and rock compositions and anomalous groundwater flow characteristics on an aerial photograph base. Typical engineering geology applications of thermal IR imagery are:

- Fault delineation
- Locating seepage at soil and rock contacts
- Mapping variations in weathered rock profiles
- Mapping near-surface drainage
- Multispectral Scanning Imagery (MSS)

MSS systems produce imagery from several distinct ranges, throughout the photographic and thermal spectrum. These distinct spectra are typically referred to as a band. Each spectral is concurrently recorded by the scanning instruments along the aircraft or satellite flight line. Much of the data available came from the Landsat satellite program during the 1970s and 1980s. The early Landsat satellites used only four spectral bands and achieved a resolution of about 80 meters. Later satellites used 7-band sensor array with a 30-meter resolution from 6 of those bands. The seventh was a thermal IR sensor. Special aircraft flights with 24-band sensors can also be obtained.

Images from MSS data can be used to examine the spectral signatures and reflectance of surficial materials and objects. Different soil and rock materials, as well as the extent of rock weathering, can be identified by comparing color variations from the different spectral bands. MSS image analysis for engineering geology is typically used to identify major landforms and tectonic features. Also, the length of time over which the images were collected allows observation of changes in vegetation, land use, and the locations of catastrophic events such as fault rupture, flooding, and landslides. As with thermal IR imaging, MSS is generally used as an enhancement of aerial photography rather than a substitute for it.

Microwave Imagery (Radar)

Radar utilizes electromagnetic energy from the microwave spectrum, typically with wavelengths from 1mm to 1m. Radar imaging may come from either an active or a passive system. In this regard, passive systems are a form of thermal IR imaging using the wavelengths that increase to the range of microwaves whereas active systems emit pulses of energy that are transmitted to the earth's surface where they are reflected back to a receiver.

The most common technique for this type of imagery is Side-Looking-Airborne-Radar (SLAR). For this technique, the radar scans a portion of the earth's surface laterally from an aircraft in a direction perpendicular to the flight line and at a depression angle measured downward from the horizontal. Overlapping images created from this method allow stereo viewing of surface features and objects. Objects that are more perpendicular to the pulse provide a strong energy return to the receiver while smooth or horizontal surfaces reflect the energy away from the receiver resulting in a dark image. It then follows that reflection angles and surface roughness as well as vegetation and moisture content influence the energy returned to the receiver. Objects and features extending above the surface project radar shadows that are related to the angle of incidence of the energy transmitted and received. These shadows accentuate the surface topography and thus, structural trends.

SLAR images are typically used in an engineering geology application to identify the surficial expression of geologic structures, drainage features, structural patterns, and trends. SLAR imagery is complimentary to aerial photography and should not be a substitute for it. However, SLAR images have many advantages that provide additional information that is difficult to extract from an aerial photograph. Their primary advantage is the enhancement of major features that are obscured by the greater detail of an aerial photograph. Another advantage of SLAR is the ability to obtain clear images at night and in heavy cloud cover.

Light Detection and Ranging (LiDAR)

This relatively new technology utilizes an active system that is similar to radar in the manner by which it creates an image. In this regard, energy is emitted from a source and reflected from the earth's surface back to a receiver. However in this case, a laser is used to measure the distance to specific points and generates a digital elevation model of the earth's surface similar to standard photogrammetric methods. LiDAR equipment is typically mounted in an aircraft although numerous ground-based applications have been developed that are beneficial to highway engineering geology, and in particular, rock slope design.

The primary advantage of LiDAR is during post-processing of the data that allows vegetation to be stripped from the data to provide a bare-earth terrain model. This is a particularly useful technology in much of Oregon where heavy vegetation obscures much of the ground surface. Landforms that would typically be obscured stand out in sharp resolution on a LiDAR image where the vegetation has been removed. In addition to vegetation, structures and dwellings can also be removed. This is also advantageous where development has occurred over large, ancient structures to the extent where they completely obscure its features. Disturbed areas and earthworks are also plainly visible on bare-earth LiDAR images. This allows clear distinctions to be drawn between fills and embankments, and natural ground surfaces. Bare-earth models also provide a clear resolution of existing stream courses and channels. Other imagery and photogrammetry-derived mapping often contain erroneously-located stream segments due to forest cover and/or ongoing lateral migration. LiDAR images not only provide an unmistakable location of the stream course, but also a clear rendition of the stream banks and terraces.

ODOT currently stores LiDAR bare-earth and reflective imagery files on the GIS server as hillshade images and Digital Elevation Models (DEM) files. This server is accessible on the ODOT system and located at:

\\Sn-salemmill-1\GIS\IMAGES\LIDAR.

Raw ASCII and .LAS-format files are available from ODOT's GIS unit as requested. In order to load the raw or binary datasets, an external hard drive of at least 500 GB capacity must be provided as these files are extremely large. LiDAR imagery and DEMs are normally viewed, manipulated, and analyzed with GIS software and specific GIS software extensions. Specialized software is also available for LiDAR data and imagery analysis. ASCII and .LAS files can be used to produce a .dtm file compatible with later versions of Bentley InRoads.

Numerous contractors are available that can provide LiDAR data products; however, ODOT participates in the Oregon LiDAR Consortium (OLC) for new acquisitions. The Oregon Department of Geology and Mineral Industries (DOGAMI) was given a legislative mandate to extend LiDAR coverage throughout the state. The consortium model was approved for funding, collection, and sharing new LiDAR datasets. DOGAMI, as head of the consortium retains the LiDAR contractor and develops cooperative agreements between consortium members. The consortium benefits all members by provided additional coverage for lower cost. As the aerial extent of each acquisition order increases, the cost per square mile decreases. In addition to lowering the unit cost, more contiguous areas of LiDAR data are acquired providing greater benefit to all members. Members of the OLC include Federal, State, and Local agencies, Tribal governments, private entities, and not-for-profit organizations.

2.2.5 Site Reconnaissance

2.2.5.1 General

The purpose of site reconnaissance in geotechnical project planning is to verify the results of the office study completed in **Section 2.2.1** and **Section 2.2.2**, and to begin formulation of a site-specific exploration program that will address the issues identified, and determine some of the logistics required to complete the next phase of investigation. At this stage, the geotechnical designer should know what to look for at the site, and, with preliminary or conceptual plans in hand, should observe the anticipated conditions with respect to the proposed project features. Surficial expression of features and landforms should be checked on the project plans as well as delineating additional features noted during the site reconnaissance. It is also important to assure that the project maps are accurate with respect to the actual site conditions, and that significant features were not overlooked or misrepresented on the preliminary or conceptual design phase maps. The scope of the site reconnaissance depends greatly on the site conditions, accessibility, and project complexity. The value of the site reconnaissance is realized later on in the project through a more efficient and thorough site exploration and geotechnical design. Therefore; site reconnaissance should be complete and systematic to achieve the final objectives of the office investigation, and may involve a significant level of effort in the field depending on the project site itself.

2.2.5.2 Verification of Office Study and Site Observations

The topography and geomorphology of a site should be reconciled in the field with what was anticipated in the office study and shown on any maps or aerial photographs. Review and assess the following:

- Outcroppings, road cuts, stream beds, and any other subsurface exposures should be noted to verify the anticipated conditions based on the published geologic maps and literature. The presence of artificial fills should be noted and described with respect to its composition, lateral extent, and estimated volume.
- Surface waters, springs, wetlands and other potentially sensitive areas that may impact the project work should also be noted. In addition, an effort should be made to identify the 2-year flood zone for future reference.
- Boulders, blocks, and oversized materials in stream beds, or projecting from embankments should be noted as they may be indicative of obstructions in the subsurface. Such obstructions are one of the most common sources of changing site conditions claims on projects that involve pile driving, shaft/tieback/soil nail drilling, and

excavations. Oversized materials observed on the surface may not be encountered during exploratory drilling and thus, the field reconnaissance may be the only record of their occurrence. In addition to boulders and blocks, existing, abandoned structures such as foundations and utility vaults can also be an obstruction to foundation installation and excavation.

- Any landslide features observed in the office study should be examined in addition to any new features discovered during the site reconnaissance. All indicators of unstable slopes such as springs sag ponds, bent tree trunks, disturbed plant communities, abrupt vegetation changes, and hummocky terrain should also be noted. Measurement and delineation of all features and indications of slope stability should be completed during the reconnaissance. Complete investigation of slope stability affecting a project area necessarily involves areas that may extend a substantial distance away from the proposed alignment.

- The performance of existing and nearby structures should be evaluated during the site reconnaissance. Evidence of settlement, deformation, tilting, or lateral movement can indicate site conditions that possibly will impact the project design and further exacerbate the performance issues during construction.

- At bridge sites, the existing footings should be evaluated with respect to stream scour. Exposed pile caps or footings as well as riprap protection generally indicate that scour has been a concern at the site previously.

2.2.5.3 Preparation for Site Exploration

Potential boring locations should be identified with respect to the preliminary or conceptual plans available at the time of the site reconnaissance. Once the locations are determined, an assessment can be made in connection with how they will be accessed by exploration equipment and personnel. Many projects can be investigated by routine methods with common equipment. However, for some projects, site access can cost almost as much if not more than the actual subsurface exploration itself in many circumstances. Physical site access, traffic control, environmental protection, and many other issues can arise that increase the complexity, and subsequently, the cost of the exploration program. Every site is different, so each must be assessed individually to determine what methods, procedures, equipment, and subcontractors will be needed. Some of the most common issues that need to be addressed are:

- **Traffic Control** – Flagging, lane restrictions, and pilot cars are required when working in or near the travel lanes. In such instances, traffic will need to be controlled for the entire time the exploration crew is on site. In other areas, traffic control may be needed while loading or unloading equipment and supplies. In many areas, lane restrictions are only allowed for nighttime operations. In every case, all efforts will be made to minimize the impact to the traveling public.

- **Equipment Required** – Determining whether the site can be accessed using a standard truck-mounted drill rig or whether a track-mounted drill will be needed. It may also be necessary to consider difficult-access equipment that must be transported by crane, helicopter, or hand-carried.

- **Physical Access** – Considering additional equipment to access a site and analyzing the cost-benefits of their use vs. other drilling equipment and investigative methods. For some sites, bulldozers and excavators may be needed to construct an access road for drilling equipment, barges may be needed for in-water work, and special low-clearance equipment may be needed for work in and around utilities. Where access roads are problematic due to environmentally sensitive areas that need to be avoided, overall impact, cost, and reclamation requirements; alternative equipment or methods should be looked upon as a potential cost or problem-saving measure where the integrity of the exploration information is not compromised.

For in-stream work, project scheduling becomes a significant issue since restrictions will be imposed on the times of the year when such activities will be allowed. Furthermore, the logistics of carrying out in-water work bring additional requirements such as determining the draft of the barge needed for the depth of the water, how the barge will be anchored, where the barge will be launched from, how the crew will access the barge during a shift change, and determining the effects of tidal or current changes on the drilling operations. A marine surveyor should be engaged for particularly complex over-water operations, and on some waterways, their review of operations is required.

Where bridges are replaced at their present location, and conditions allow, drilling may be conducted through the existing bridge deck although efforts must be made to assure that only the deck and not the superstructure are penetrated.

- **Drilling Conditions** – Where high groundwater levels, deep water, and loose or heaving sands and gravels, and obstructions are anticipated, the appropriate drilling methods and materials should be specified.
- **Materials and Support** – Remote locations may require special considerations for supporting the field crew and the equipment. In this regard, additional logistics may be needed for delivering drilling supplies, fuel, lubricants, etc., and for the timely delivery of samples back to the laboratory and office. All-terrain vehicles may be needed to support the drill crews in such situations, or else preplanning needs to be carried out to schedule or arrange for extra site provision. Locations for drill water should be identified ahead of time, and where an ODOT facility is not available, permits will need to be obtained ahead of time for fire hydrants, private sources, or extraction from streams and lakes.
- **Right-of-Way** –The methods by which permits of entry for exploration on private property are obtained vary from region to region, and frequently, within a region. For all cases, the region Right-of-Way section in which the project is taking place should be consulted prior to exploration, and then notified in advance, when and which private properties will be accessed. The Right-of-Way section manager or their subordinate will recommend either a standard permit of entry form, or they will obtain the permit of entry internally.

  In many instances, private property owners will refuse to grant entry. For these, the right-of-way section will be required to handle the negotiations for site access, and determine the terms and conditions.
- **Utility Conflicts** – During the site visit, the location and type of utilities should be noted. The names and contact information located on the utility risers, stakes, and poles should be recorded. In all cases, the Utility Notification ("One-Call") Center must be contacted at least 2 working days prior to commencement of site operations at 1-800-332-2344. The One-Call Center will recount the utility services that they will notify based on their records. The geotechnical designer or drilling supervisor will be responsible for notifying any other utilities operating in the area based on their observations of facilities during the site reconnaissance. Responsibility for maintaining the utility location markings during site operations belongs to the field exploration crew.

2.2.5.4 Reconnaissance Documentation

During the field reconnaissance, photographs should be taken of all the predominant features previously discussed. Each photograph should be appropriately labeled with the object of the photo, the direction it was taken, where it was taken from, the date, and ideally, the latitude and longitude of the photograph's origin obtained with GPS equipment.

The observations taken during the site visit should be documented in a memorandum or short reconnaissance report depending on the scope and complexity of the project. The report should provide a list and a description of all the observations made, and the prominent features encountered during the office study and site reconnaissance. Each feature should be located with reference to the project stationing or reference grid. Once again, there is considerable benefit to locating features with GPS equipment for long-term record keeping. Project stationing can change, projects can be postponed for long periods of time, and future projects will occur that will utilize this document see **Section 2.2.1.1**. Preplanning for geotechnical design is correlative to any other investment; the earlier in the process the work takes place, the longer the benefits can be reaped.

2.3 References

American Association of State Highway and Transportation Officials, Inc., 1988, *Manual on Subsurface Investigations*.

C.H. Dowding, Ed., *Site Characterization & Exploration*, ASCE Specialty Workshop Proceedings, Northwestern University, 1978.

U.S. Department of Transportation, *Federal Highway Administration Evaluation of Soil and Rock Properties, Geotechnical Engineering Circular No. 5, FHWA-IF-02-034*, April, 2002.

Turner, Keith A., and Schuster, Robert L., Eds., *LANDSLIDES Investigation and Mitigation, Transportation Research Board Special Report 247*, 1996, Pages 140-163.

Geotechnical Investigations, U.S. Army Corps of Engineers Engineering and Design Manual, EM 1110-1-1804, January 2001.

APPENDIX 2-A – Geology/Geotechnical QC MATRIX

Table 2-1. Geology / Geotechnical Matrix Checklist QC Check #1 – Scoping

	Geology			Geotech			Rock Slopes		
	YES	**NO**	**N/A**	**YES**	**NO**	**N/A**	**YES**	**NO**	**N/A**
Scope									
Project Name and Key Number									
Existing structures, earthworks and known hazards									
Proposed structures and earthworks									
Design Narrative, defined project area									
Project Geography									
Bodies of water									
Terrain Features									
Climate									
Region									
Project Geology									
Province									
Bedrock and Quaternary Geology									
Structural Geology									
Geologic Hazards									
Geomorphology									
Geologic Impacts/Performance of existing structures									
Performance of existing structures									
Previous design efforts in the project area									
Cost Estimates for Proposed Work (Design and Construction)									
Monitoring period									
Summary of findings and project implications									

Table 2-2. Geology / Geotechnical Matrix Checklist QC Check #2 – Scope of Work

	Geology			Geotech			Rock Slopes		
	YES	NO	N/A	YES	NO	N/A	YES	NO	N/A
Project Scope									
Schedule of work									
Geology Scope of Work									
Geotechnical Scope of Work									
Rock Slope Scope of Work									
Exploration Scope of Work									
Geology project budget									
Geotechnical project budget									
Rock slopes project budget									
Monitoring period schedule and budget									

Table 2-3. Geology / Geotechnical Matrix Checklist QC Check # 3 – EIS

	Geology			Geotech			Rock Slopes		
	YES	NO	N/A	YES	NO	N/A	YES	NO	N/A
Survey of proposed alignments and alternatives									
Bedrock units to be encountered									
Surficial units to be encountered									
Physical geography – effects on proposed alignments and/or slope geometries									
Location									
Extent									
Climate									
Topography									
Geologic Province									
Character of expected geologic units and their performance history									
Geologic hazard potential									
Summary of known geologic hazards									
Summary of known geologic impacts to existing features									
Performance of structures and earthworks along proposed corridors or alignments									
Known geotechnical-related problems in existing structures and earthworks in the proposed project area									
Mitigation methods and costs for potential geotechnical issues									
Geotechnical characterization/estimated properties of geologic units									
Discussion of the performance of project area materials and geologic units									
Correlation of properties of expected materials with similar studies									
Cost-benefit analysis of proposed alignments and/or locations									

Table 2-4. Geology / Geotechnical Matrix Checklist QC Check # 4 – Concept

	Geology			Geotech			Rock Slopes		
	YES	NO	N/A	YES	NO	N/A	YES	NO	N/A
Concept Plan Review									
Reconnaissance Report (File Summary Survey)									
Consultation of published literature									
Consultation of unpublished literature									
Aerial photographs and other remote sensing									
Aerial photographs from different years to review varying conditions through time and site history									
As-built plans									
Maintenance records									
Region file survey									
Consultant reports									
RHRS/Unstable slope inventory									
Review of maintenance activities that have affected the site (e.g. rockfall containment, slope stability, drainage)									
Review of geographic and geologic conditions affecting slope stability with respect to conceptual evaluation of landslide/rockfall remediation schemes									
Determine the potential affect of outside stakeholders on the remediation options (USFS, Gorge Commission, Tribal Governments, etc.)									

Table **2-5**. Geology / Geotechnical Matrix Checklist QC Check #5 – Exploration Plan (10% TS&L)

	Geology			**Geotech**			**Rock Slopes**		
	YES	**NO**	**N/A**	**YES**	**NO**	**N/A**	**YES**	**NO**	**N/A**
Exploration Plan									
Exploration Plan Summary									
Survey Requirements									
Work Products									
Scope, Schedule, Budget									
Project Features requiring subsurface investigation									
AASHTO compliance for project features									
Boring/Exploration spacing									
Boring/Exploration depth									
Sampling frequency									
FHWA recommended standard practices for rock slopes									
Evaluation/inclusion of alternative or supplementary exploration methods									
Consideration of alternative tests and/or techniques that would provide better quality and economy									
Appropriate rock slope mapping and drilling programs for the proposed mitigation measure									
Evaluation of the expected site conditions and compatibility with standard exploration procedures									
Minimum explorations for trenchless pipe installation and associated features									
Exploration Plan Review									
Structures and earthworks for exploration									
Proposed exploration at each structure location									

Table 2-6. Geology / Geotechnical Matrix Checklist QC Check #6 – 2/3 TS&L)

	Geology			Geotech			Rock Slopes		
	YES	NO	N/A	YES	NO	N/A	YES	NO	N/A
Field Exploration Review									
Site-specific field explorations									
Borings									
Test Pits									
Hand-auger holes									
Geophysics									
In-Situ testing									
Site and vicinity reconnaissance									
Project-level geologic mapping									
ASTM conformance									
Drilling methods									
Sampling and testing									
Deviations from standards noted and described									
Review of alternative tests or techniques									
Quantity of samples for laboratory testing (collection and recovery)									
Adequate samples and laboratory testing to characterize and determine the extent of subsurface materials									
Undisturbed samples in cohesive and/or compressible materials									
Core drilling procedures									
ODOT standard core box placement and labeling									
HQ or larger-sized core diameter									
Triple-tube recovery system									
Recovery appropriate for the materials encountered (never less than 80% unless special conditions exist)									
Core specimens labeled and photographed while wetted									
Legible and appropriate core photography									
Specimens removed for laboratory testing replaced in the core box with the appropriate marker									
Drilling techniques correspond to the materials encountered									
Augers used while investigating for the piezometric surface in soil									
Indication where natural moisture content was altered by introduced fluids									

QC Check #6 – (⅔ TS&L) (continued)

	Geology			Geotech			Rock Slopes		
Methods Used to Determine Piezometric Surface in Rock	**YES**	**NO**	**N/A**	**YES**	**NO**	**N/A**	**YES**	**NO**	**N/A**
Fluids used to stabilize boreholes in sandy material or other heaving conditions									
Measures to avoid affecting SPT and other testing values and intervals in heaving conditions									
Drilling activities recorded on standard boring log forms									
Fluid return and color changes									
Drill action and rate									
Shift/personnel changes									
Bit wear									
Drilling techniques									
All information used for interpretation of subsurface conditions									
Locations where groundwater was encountered									
Open hole water levels recorded at the beginning of each drilling shift									
Dry holes specifically noted									
Types, quantities, ad depths of backfill and sealing materials									
Soil and rock materials identified, classified, and described according to the current version of the ODOT Soil and Rock Classification Manual									
Complete soil and rock descriptions									
Additional physical properties, diagnostic, or distinguishing features recorded on the logs									
Boring locations surveyed with respect to State Plane Coordinates and true elevations									
Conversion to SPC/true elevation where assumed values are used									
Borings referenced by project stationing									
Borings referenced by bearing and distance to permanent features or reference points in the absence of an existing base map or survey									
Preliminary subsurface drawings and/or model for adjusting exploration according to current findings									

QC Check #6 – (⅔ TS&L) (continued)

	Geology			Geotech			Rock Slopes		
	YES	NO	N/A	YES	NO	N/A	YES	NO	N/A
Boreholes abandoned according to Water Resources standards									
Instruments installed according to their purpose (e.g. inclinometers installed below the slide plane, piezometer sensing zones in the water-bearing strata, etc.)									
Records of piezometer casing type/size, slotted zones, slot size/frequency									
Records of sealing and filter pack placement, sizes and grades of the materials									
VWP Installations									
Manufacturers calibration sheets									
Field calibration results									
Initial reading consistent with manual observation									
Inclinometers									
Appropriate slurry mixture									
Slurry quantity recorded									
Distinct zones of grout-take noted									
A0 direction noted, proper A0 inclinometer alignment									
Tube stick-up recorded									
Water Resources Hole Reports completed correctly and filed within the 30-day requirement									
Appropriate rock mass classification system used to evaluate rock slope excavation performance									
Rock slope surface mapping									
Overburden thickness and type									
Discontinuity thickness, type, surface roughness, spacing, orientation, and shape									
Zones of differential weathering on the slope									
Location and volume of seeps and springs									

QC Check #6 – (⅔ TS&L) (continued)

	Geology			Geotech			Rock Slopes		
	YES	NO	N/A	YES	NO	N/A	YES	NO	N/A
Preliminary Geotechnical Recommendations									
TS&L Foundation Design Memo									
Description of proposed project									
Anticipated subsurface conditions									
Preliminary foundation design recommendations									
Foundation types									
Preliminary capacities									
Rational for selecting the recommended foundation type and capacity									
Discussion of liquefaction potential and associated effects									
Suggested retaining wall types									
Preliminary slope recommendations									
Site Model Review									
All exploration locations located on plan view maps referenced to the project									
Plan view maps developed to the appropriate scale to show the necessary features with respect to the overall project									
Appropriate plan map contour interval and labeling									
Borehole collar elevations consistent with nearest contours									
Standard map elements									
Cross-sections, fence diagrams, profiles and/or block diagrams used to display the 3-dimensional distribution of geologic units, features, structures, and engineering properties									
Geologic model consistent with engineering properties of defined units									
Material properties/laboratory testing results recorded on the drill logs									
Laboratory testing used to develop engineering geologic units									
Laboratory testing results displayed graphically to support the engineering geologic model (e.g. graphs or charts plotting engineering properties with depth or along a graphic lithology column)									
Laboratory testing program included samples from each boring or test pit to confirm the field and visual classification									
Laboratory results incorporated into the final drill logs and subsurface model									
Laboratory testing to verify or confirm interpretations or further characterize a unit									

QC Check #6 – (⅔ TS&L) (continued)

	Geology			Geotech			Rock Slopes		
	YES	NO	N/A	YES	NO	N/A	YES	NO	N/A
Final drill logs match the interpretive drawings and preliminary drawings for the Geotechnical or Foundation Datasheets									
Clear distinction between observed and inferred features and relationships in the geologic model									
Review laboratory test results to determine if modifications are required in specific geologic units at different locations in the subsurface model									
Process developed to incorporate laboratory testing to assure correct and consistent material classification and description between borings and to develop engineering geologic stratigraphy from the test results									
Review physical properties testing to determine if initially misidentified materials occur elsewhere in the project subsurface									
Related soil classifications modified as a result of physical properties test results									
Results of instrumentation programs match the engineering geologic model									
Geologic model encompasses the project design details to show the effect of the geology on the facility									
Proposed cut lines, excavations, tunnel/pipe alignment, and foundations all plotted in the subsurface model									
Geologic features affecting the design such as seeps, springs, piezometric surfaces, and daylighted adverse structures clearly shown and identified in the model									
Blocky or rubble-zones that could produce overbreak in rock cuts or excavations									
Boulders or other obstructions in proposed excavations or pile and shaft foundations									
Groundwater surfaces									
Delineation of collapsible or expansive soils									
Cuts or fills on known or potential slide areas									

QC Check #6 – (⅔ TS&L) (continued)

	Geology			Geotech			Rock Slopes		
	YES	NO	N/A	YES	NO	N/A	YES	NO	N/A
Foundations in or near bog/marsh areas									
Excavations below the groundwater surface, determination of the amount of water that will be encountered and the effect of piezometric drawdown on groundwater resources									
Delineation of potentially soft subgrade on the project plan map									
Geologic interpretation of materials and stratigraphy incorporates the engineering properties of the strata encountered (e.g. geologic units are subdivided down to the level of distinct engineering properties)									
Cross-cutting relationships established									
Quaternary-aged features and discontinuities identified									
Determine if weak or weathered rock sources identified for use on the project are likely to be friable or nondurable									
Slake Durability testing of exposed rock face material									
Thorough representation of materials tested for strength and compressibility rather than reliance on empirical correlations, especially those based upon Standard Penetration Tests									
Appropriate strength tests conducted to distinguish between drained and undrained conditions where needed									
Determine if the total stress envelope of the CIU test with pore pressure measurements has been used improperly to define the relationship of undrained shear strength with depth									
Determine if the existing and proposed state of stress has been accounted for during strength testing									
Evaluation of consolidation tests: reconciliation of the test-derived preconsolidation pressure with the actual stress history of the sample									

Table 2-7. Geology / Geotechnical Matrix Checklist QC Check #7 – Preliminary Plans

	Geology			Geotech			Rock Slopes		
	YES	NO	N/A	YES	NO	N/A	YES	NO	N/A
Engineering Geology Report									
Geotechnical Report									
Rock Slope Report									
Preliminary Geotechnical Datasheets									
Datasheets completed for all required structures or features									
Profiles drawn along project alignment centerlines or specific offsets									
Cross-sections, additional profiles completed to show structure-specific information, or to provide additional information in areas of complex geology									
Sample and property data									
Subsurface model used to develop the Geotechnical Datasheets									
Subsurface information shown on the datasheets matches the final logs									
Drawings made at appropriate scales to show the needed level of detail									
Interpretation shown on the datasheets									
Geotechnical Datasheets completed according to Subsurface Information Policy									
Detail Drawings and Plans									
Review geotechnical items in the bid schedule									
Assure specification writer's review of geotechnical items in the special provisions									
Review specification writer's modifications of geotechnical items in the special provisions									
Correct length and locations for buttresses, surface and subsurface water collection and discharge features shown on the plans									
Correct materials called out on the plans									
Sequence of construction for buttresses									

QC Check #7 – Preliminary Plans (continued)

	Geology			Geotech			Rock Slopes		
	YES	NO	N/A	YES	NO	N/A	YES	NO	N/A
Staged construction sequence for surcharging, wick drains, and ground improvement									
Appropriate drainage discharge locations									
Recontouring of slide areas clearly shown									
Surface water drainage in slide areas addressed in the plans or detail drawings									
Buttress, drainage, or other features shown with the correct elevations and dimensions									
Slope protection mat and rockfall protection fences									
Mesh type									
Anchor spacing									
Quantities									
Special provisions, including those for high-impact fences									
Standard Drawings included in the plans									
Special access issues and requirements									
Standard drawings and special provisions for PVC-coated mesh									
Rock Bolts and Dowels									
Design Loads									
Design Lengths									
Locations									
Quantities									
Corrosion protection									
Performance and proof-testing requirements									
Reference to the Qualified Products List									
Rockfall Retaining Structures									
Type, Size, and Location									
Quantities									
Slopes (Rockfall Protection Berms)									
Backfill type specifications									
Special Provisions									
Rock Slope Drainage									
Location									
Drain lengths									
Drain angles and orientations									
Quantities									
Water collection and disposal									

QC Check #7 – Preliminary Plans (continued)

	Geology			Geotech			Rock Slopes		
	YES	NO	N/A	YES	NO	N/A	YES	NO	N/A
Shotcrete									
Locations									
Areas of coverage									
Quantities									
Anchorage									
Reinforcement									
Standard drawings and details									
Drainage									
Performance requirements									
Installation details									
Temporary Rockfall Protection									
Review type for suitability									
Locations									
Length									
Height									
Required materials and quantity									
Details									
Rock Blasting and Rock Excavation									
Quantity of Controlled Blast Holes									
Overburden slopes and slope breaks shown on the plans									
Special Provisions									
Blast Consultants									
Noise/vibration monitoring									
Preblast survey									
Blasting plan review									

Table 2-8. Geology / Geotechnical Matrix Checklist QC Check #8 – Advanced Plans

	Geology			Geotech			Rock Slopes		
	YES	**NO**	**N/A**	**YES**	**NO**	**N/A**	**YES**	**NO**	**N/A**
Preliminary Wall Drawings									
Review subsurface information on Geotechnical Datasheets for retaining structures									
Retaining Wall Drawing Review									
Type, Size, Location, Height, Backslope									
Quantities									
Backfill types									
Wall drainage									
Special Provisions									
Design Changes and Addenda									
Design calculations for added structures and features									
Design calculations for structures and features that have moved									
Review design assumptions									
Changed Criteria									
Changed Type, Size, Location									
Changed Quantities									
Additional exploration requirements for added structures or features									
Appropriate exploration carried out for added structures or features									
New data incorporated into the overall geologic interpretation									
Further characterization of geologic units with additional data									
Resolution or confirmation of previous inferences and interpretation									
Additional risk assessment									

Table 2-9. Geology / Geotechnical Matrix Checklist QC Check #9 – Final Plans

	Geology			Geotech			Rock Slopes		
	YES	NO	N/A	YES	NO	N/A	YES	NO	N/A
Final Plan Review									
Geotechnical or Foundation Datasheets completed for all structures, facilities, ad features for which they are required									
Geotechnical Datasheets completed according to Subsurface Information Policy									
Engineer or Geologist has stamped all sheets that they are responsible for									
Information provided on the datasheets exactly matches what is presented on the final logs and in the Engineering Geology report									
Final review of detail and plan sheets									
Final review of bid item quantities									
Final review of Special Provisions									

Chapter

3

3 Field Investigation

3.1 Introduction

For any transportation project that has components supported on or in the earth, there is a need for subsurface information and geotechnical data during its planning, design, and construction phases. Any geologic feature that affects the design and construction phase of a project, or has a bearing on site or corridor selection in terms of hazards and/or economics must be investigated and analyzed. Of equal importance is the clear and accurate portrayal of these conditions in a format that is accessible and understandable by all users.

Consider the following during field investigation:

- **Subsurface investigation:** The objectives of a subsurface investigation are the provision of general information on the subsurface conditions of soil, rock and water, and specific information concerning the soil and rock properties that are necessary for the project geotechnical design and construction.
- **Scale of investigation:** For transportation projects in Oregon, the appropriate scale of investigation must be carefully considered. Because of Oregon's geology and geography, subsurface conditions are complex and may vary widely over short distances. A more thorough investigation will provide additional information that will generally decrease the probability of encountering unforeseen conditions during construction, and increase the quality and economy of the geotechnical design of a project.
- **Balance of investigation:** Time and fiscal considerations will constrain the scale and resolution of the field investigation. Therefore; the geotechnical designer must balance the exploration costs with the information required and the acceptable risks.

The technical decisions and details required for site investigations require the input of trained and experienced professionals. Every site has its own particular circumstances, and diverse geologic conditions, professional experience, available equipment, and the previously described time and budgetary restraints all contribute to the most cost-effective site investigations. The implications of site-specific geologic conditions for the type of proposed facility must be investigated for each project. The remainder of this chapter describes established ODOT criteria to be used in field investigations as well as information on any areas where ODOT's criteria differs from the FHWA and AASHTO guidelines. More information can also be found in the Federal Highway Administration *Subsurface Investigations - Geotechnical Site Characterization Reference Manual* (*FHWA NHI-01-031*).

Established Investigation Criteria

Professional experience and judgment are the basis of any field investigation program. This chapter is not intended to provide a prescriptive approach to field investigation, however; there are some established base levels of investigation for transportation facilities that must be mandated to assure consistency and quality throughout the agency, and to address a common level of risk acceptance.

- These baselines were based on Federal guidance and the *AASHTO Manual on Subsurface Investigations*, 1988. ODOT has adopted the baseline requirements for subsurface investigations from the AASHTO Manual.
- However, due to the more variable conditions found in Oregon, ODOT's practice is slightly more rigorous with respect to exploration spacing and sampling. ODOT variance from AASHTO guidelines is outlined in **Section 3.5** and **Section 3.6**. *LRFD Bridge Design Specifications, Section 10* provides an additional resource for subsurface investigations, supplementary to the AASHTO guidelines.

The most important component of subsurface investigation is the personnel that direct the field activities, interpret the information, and present the results in a clear manner to those responsible for the final geotechnical design and construction of the project. The quality of information produced from a subsurface investigation can vary substantially depending on the experience and competence of the personnel charged with its conduct. Radically different interpretations and conclusions can result from substandard investigation programs. Subsurface investigation is an investment in the success of a project with returns that range from 10 to 15 times the cost of the investigation later realized during final design and construction.

3.2 General Subsurface Investigation

For most projects, the main purpose of a subsurface investigation program is to obtain the engineering properties of the soil and rock units and define their vertical and lateral extent with respect to thickness, position in the stratigraphic column – their depth, and aerial extent where they could affect the design and performance of a structural or earthwork feature.

The properties normally evaluated include Index Properties such as:

- Natural moisture content, and
- Atterberg Limits.

Additional physical properties may be evaluated, such as

- Shear strength,
- Density,
- Compressibility, and
- In some cases, permeability.

The location and nature of groundwater is evaluated in every subsurface investigation. In addition to material properties, subsurface investigations are carried out to explore and monitor geologic hazards that were identified in the office studies previously conducted.

For this later purpose, landslides are the most common hazard although caverns, compressible materials, high groundwater, faults, and obstructions may also form the basis or extension of a subsurface investigation program.

3.2.1 Subsurface Investigations – Phases

Subsurface investigations may be carried out with varying levels of intensity depending on the phase of the project for which they are conducted. The typical phases are described in the following sections.

3.2.1.1 Phase 1

For the Field Survey and/or Alternative Design phases (Usually described as “Phase 1”) of a project, the information gathered from the office study is usually sufficient for preliminary geologic/geotechnical input to the project team and for completion of the Soils and Geology chapter of the Environmental Impact Statement (EIS). In the case of a large and/or complex project, or if geologic conditions will have a major impact on the design and construction of a project, then some amount of subsurface investigation will be warranted to determine the exact location and extent of the problems and to devise some preliminary cost estimates and alternatives. Ideally, when performing a subsurface investigation during Phase 1, the exploration would be situated at the location of a major project feature that would be investigated later during project design. However, as this occurs early in the project, or certain other alternatives are under consideration, the precise locations of bridge bents and final alignments may not be known.

3.2.1.2 Phase 2

The project design phase (Field Survey up to Preliminary Plans, usually referred to as “Phase 2”) is where the most intense and focused subsurface investigation occurs for specific project features. Wherever possible, the project design or Phase 2 investigation should capitalize on any previous explorations in the project area. Personnel responsible for the field investigation and geotechnical design should determine the utility of this information.

The project design phase subsurface exploration and testing program provides the geotechnical data specifically required by the project’s geotechnical design team. The investigation provides the aforementioned informational needs for the foundation and earthworks design as well as:

- Additional information applicable to other related project elements such as the chemical properties of soil with respect to corrosion of structural elements, and issues associated with environmental protection and erosion control.
- The project geotechnical design analyses, decisions, and recommendations for construction will be based on the information gathered during the Phase 2 investigations.
  For these reasons, the information gathered during this phase of investigation should achieve a degree of accuracy, thoroughness of coverage, and relevancy to support the project design decisions and to allow for realistically accurate estimates of geotechnical bid items.

3.2.1.3 Other Phases

There will be some instances where additional subsurface investigation is necessary during Advanced Plans, Final Plans, or even during the construction phase of a project. This is not necessarily due to an incomplete investigation during the project design phase, but rather the result of unforeseeable problems that arise during construction, or late design changes following the main investigational effort and/or geotechnical design. Subsurface investigation is conducted to provide design information and is usually adequate, in most cases, for contractor's estimates for construction and bidding. Explorations conducted during construction are uncommon, and are usually carried out to resolve problems or answer questions that arise while the project is being built.

Occasionally, explorations will occur as part of the construction activity to install and monitor needed instrumentation. When design changes occur late in a project, additional subsurface investigation can be necessary to confirm the geotechnical design assumptions or to develop additional information.

3.3 Exploration Plan Development

The Exploration Plan is a document that describes the subsurface investigation activities that will take place to obtain the engineering properties required for geotechnical design. The objective of the Exploration Plan is to:

- Assure that the sampling and testing carried out for the subsurface investigation thoroughly covers each of the geologic units applicable to the geotechnical design
- Verify that the maximum amount of information can be obtained from the fewest number of borings or other higher-cost methods

In order to achieve this, the plan must be updated and modified as exploration proceeds to make sure that the number of samples taken, and tests performed in each unit provides enough numeric measurements of each critical engineering property distributed throughout the geologic unit to provide enough confidence in the property to base the geotechnical design upon. In this regard, the properties of a material at one end of a long alignment may not hold true for the other end, and a geotechnical designer will not want to base all design parameters for that material on only one or a few samples.

Subsurface investigation conducted during the project design phase must fully define the subsurface conditions at a project site to meet the requirements of geotechnical design and construction. The proper execution of the Exploration Plan will assure that samples and tests are numerically adequate and distributed vertically and laterally throughout each geologic unit, and that every important geologic unit at the site is discovered and investigated to the maximum feasible extent. The Exploration Plan will also assure that the site investigation is conducted in accordance with the standards of practice outlined in the 1988 AASHTO *Manual on Subsurface Investigations* and augmented in this manual. These standards are further subject to modification due to the variability of the site geology, sensitivity to potential changes, and risk or potential impact.

Note:
Exploration Plans should be created, reviewed, and executed by an experienced engineering geologist or geotechnical engineer.

The geotechnical designer should comprehensively evaluate the various methods and procedures for subsurface exploration that are currently available to maximize the amount of information gathered while reducing costs to the extent possible. The most common method for achieving this is to gain the most information from the fewest number of borings.

Alternatively, various types of exploration methods may be used where practical in lieu of the more expensive borings to realize those cost savings without compromising the necessary acquisition of information.

3.3.1 Exploration Plan Considerations

One of the leading issues addressed when developing the Exploration Plan is the overall scale or intensity and level of effort for the subsurface investigation. To answer these questions, the expected complexity of the project site's geology must be considered with respect to nature of the proposed project, and the project's requirements from the subsurface investigation.

In effect, there are some primary factors that will necessitate increasing the Exploration Plan for a larger-scale subsurface investigation including:

- Complex site geology
- Complex site conditions
- Scale of the project
- Sensitivity of the facility to variations in site conditions

The subsurface investigation program should be scoped according to these issues rather than from some baseline requirement. Each exploration should be justifiable in terms of the information needed from it. Such informational requirements form the basis of the following criteria:

- The type of boring
- Location
- Depth
- Types of sampling
- Sampling interval

These questions can only be answered by the experience, knowledge, and application of engineering geologic principles by the geotechnical designer. Through careful examination of the results previously obtained by the office study, and their experiences working in the area, they are the essential resource for determining the requirements of the subsurface exploration program.

3.3.1.1 Minimum Requirements for Subsurface Investigations

This does not however, preclude the necessity of established minimum requirements for subsurface investigations. The base level of investigation has value as an initial approach to a subsurface investigation and for preliminary cost estimation of exploration activities as well as assuring that some uniform amount of exploration is accomplished for all geotechnical design. The minimum standards for subsurface investigations are well-defined in the 1988 *AASHTO Manual on Subsurface Investigations* and are broadly accepted in the practice.

Where ODOT Differs from the AASHTO Manual

Where ODOT practice differs from the AASHTO Manual is in the divergence from the minimum amount of investigation. AASHTO allows for a reduction from the minimal amount of exploration in areas of predictable geologic conditions and the absence of any geologic hazards. Such conditions generally do not exist in Oregon and as a rule, prohibit any reduction of the exploration program. Rather, explorations are added to the program due to the unpredictable nature of the state's geology. Much of the work performed during the preliminary office studies will assist in determining the overall scale of the subsurface investigation program.

Such added expenditures are always justifiable when additional exploration, testing, and analyses result in correlative savings on the construction cost and in an overall better geotechnical design.

3.3.1.2 Risk Tolerance

Further consideration in the development of the Exploration Plan should be given to developing an assessment of the risk tolerance of the project to unforeseen subsurface conditions. In this regard, an assessment of the risks assumed by the constructability and function of the design feature without the benefit of site-specific subsurface information should be conducted with respect to the potential for cost overruns during construction and to potential for long term maintenance or increased lifecycle costs. The cost of an over conservative design resulting from a hedge against unknown subsurface conditions is another aspect of risk that should also be evaluated. This is where a design is forced to be based on the worst possible condition known to be present or perceived at a site in order to prevent failure because the lack of information precludes the assessment of other alternatives. Generally, an evaluation of the potential risks at a project site occurs as exploration progresses and the variability of the subsurface is discovered.

3.3.1.3 Structure Sensitivity

The sensitivity of a structure or other facility in terms of performance to subsurface variability also influences the scale of the subsurface investigation. Consider the following in relation to structure sensitivity:

- Where settlement is concerned, structures are much more sensitive whereas embankments overall are able to tolerate more post-construction deflection not withstanding those sections adjacent to bridges.
- Existing structures adjacent to transportation projects also increase the sensitivity of projects in the built-up or urban environment. Where construction is to occur adjacent to existing structures or private buildings, the tolerance for settlement or deflection and even vibration is essentially eliminated, and correspondingly, the need for subsurface information increases.
- Such sensitivity can also extend to environmental, cultural, and archaeological sites where great efforts will be made to mitigate impacts during construction. For these circumstances, significant efforts in pre through post-construction monitoring are often required with instrumentation installed far in advance of contract letting.
- Certain types of construction may also be more sensitive to unanticipated subsurface conditions such as drilled shaft installation where relatively small changes can result in a sizeable cost increase.

Despite the best efforts and most detailed subsurface investigations, every significant subsurface condition may not be discovered or fully examined. The objective here is to reduce the risks accepted to the barest minimum, and to have some understanding of the risks that will remain.

3.3.1.4 Subsurface Investigation Strategy

An important strategy when conducting the subsurface investigation is to complete the most important explorations first with the idea that the project schedule may change, funding may be terminated, or some other decisions made that preclude the completion of all the planned borings. From this standpoint, the important borings are those that:

- Provide information about geologic hazards affecting the project or that require monitoring for mitigation design,
- Provide the information that the engineer needs to design the most critical structures, and
- Again, those locations that provide the most amount of information for the lowest expenditure.

This approach to the subsurface investigation allows design to proceed in the event of the inevitable project schedule or other priority shifts that may have a more urgent need for geologic or geotechnical resources. It is quite common for a planned exploration to be interrupted by the needs of emergency repair work or other critical-path project, and having these explorations complete first allows engineers to continue work on a project rather than having to wait for the emergency to pass before getting the information they need to continue so that the interrupted project doesn't become an emergency itself.

Note:
We recommend referring to Section 7.4.1 AASHTO which provides additional items to consider in determining the layout of a project subsurface investigation in addition to prioritization of the explorations. This bulleted list describes key issues in determining importance and priority of explorations from locations to structures that they are intended for as well as the use of less or even more expensive methods for investigation that may be required.

3.3.1.5 Schedule of Subsurface Investigations

Subsurface investigations are ideally completed as early in the project as possible to allow sufficient time for geotechnical design, quantity estimation, and consideration of alternatives. Clearly, many of the project features must already be known to some degree before the Exploration Plan can be formulated. Right-of-way needs must be established to determine cut and fill slope angles and heights or the need for retaining structures. Even more detailed plans are needed to begin bridge foundation investigations. Typically, the bridge type, size, and location (commonly referred to as "TS&L) must be known in order to obtain ground-truth information at the precise bent locations.

Completion of Exploration Plan

Because of these informational prerequisites, the Exploration Plan is usually completed soon after initiation of the structure TS&L phase with a goal for completion set at the 10% of TS&L completion with respect to its timeline. The target for completion of preliminary geotechnical recommendations is set at 2/3 TS&L.

In order to meet this date, there will be less than 50% of the TS&L timeline to complete the subsurface investigation and provide the needed information to the geotechnical designer charged with making the preliminary recommendations.

Subsurface investigation performed during preliminary phases may be called for at any time prior to Phase 2, particularly during the EIS phase depending on the size of the project or any other special requirements. These investigations are intended to develop project geotechnical constraints and/or to provide general information to assist in alternative route selection, and to address particular requirements of the EIS rather than to gain site-specific geotechnical design parameters. Preliminary subsurface investigation typically takes place on an existing state right-of-way readily accessible areas so there should not be additional time and money spent in acquiring permits of entry, building access roads and reclaiming sites.

Instrument Monitoring Periods

An additional aspect of the subsurface investigation schedule that also needs to be determined is the requirement for instrument monitoring periods. These are particularly important as they commonly extend before and beyond typical project timelines.

- **Landslides:** Projects that involve landslide repair or evaluation are the usual reasons for broadening timelines as it is critical to monitor landslide movements over periods of time that include at least one wet season (usually November through April) to assess the nature of the slide evaluate the relationships between precipitation, groundwater, and slide movement, and determine the correct slide geometry for stability analysis.
- **Groundwater:** It is also important to monitor groundwater for other construction applications throughout seasonal fluctuations to help determine actual construction-time conditions. Grading operations or excavations that would be made "in-the-dry" during certain times of the year may occur below the groundwater surface during other months. Every effort must be made to collect this information regardless of the time of year that exploration is conducted.
- **Post-construction monitoring:** Where post-construction monitoring is necessary, it should also be identified as early in the Exploration Plan development as possible. Critical structures in addition to landslides may require such instrumentation for quality assurance in addition to providing an assessment of long-term performance.

3.3.1.6 Exploration Sites

One of the primary factors affecting the schedule of the subsurface investigation program is providing access to drill sites. This includes acquiring the necessary permits as well as the actual physical occupation of the drill site.

Note:
Preliminary borehole location should have taken place during the initial site reconnaissance and major requirements with respect to accessibility should have been identified at that time. Since access to certain drill sites requires a significant investment of time, it is necessary to start acquiring permits of entry, environmental clearances, and engaging contractors to build access roads or bring additional resources to move the drilling equipment.

The geotechnical designer should clearly indicate the necessary borehole location tolerances to the field crews to assist in determining site access. When situating a borehole, consider the following:

- For some sites, a few extra feet of tolerance available will allow a borehole to be accessed with standard equipment or with minimal disturbance while at others, considerably greater efforts will be necessary to place the borehole at the precise location.

- Where the location of the exploration is crucial, it may be reasonable to mobilize specialty drilling equipment.
- Several factors contribute to the amount of tolerance allowed for an exploration. Among these are the phase of the investigation for which the explorations are performed, in this case, the final design explorations would require the more precise location.
- The types of structure, expected subsurface conditions, and surrounding facilities also have more exacting standards for borehole placement.
- A spread footing on rock, or a tieback wall adjacent to and supporting an existing structure are examples of cases where relatively minor changes in the subsurface conditions have very serious consequences during construction and would therefore warrant the extra expenditure to precisely locate the explorations. In this case, the expenditure for mobilizing special equipment would be far exceeded by orders of magnitude from ensuing claims or even, litigation.

3.3.1.7 Right-of-Way and Permits of Entry

Determining the exact boundaries of the State's right-of-way during exploration planning is essential since this demarcation is very commonly not correlative to the highway centerline nor does it fall at a constant length perpendicular to it. Current right-of-way maps should be consulted to assure the correct property ownership at the exploration site or for any land that must be traversed by exploration equipment and personnel.

Permits of entry (also known as "Right-of-Entry Permits") are required for any site exploration outside of the highway right-of-way whether the site is on private property or on public lands outside the jurisdiction of ODOT. For simple cases, these permits can be obtained by the geotechnical designer in charge of the exploration or other staff. For most circumstances however; these permits should be obtained by the Region's Right-of-Way section. In either case, the region Right-of-Way section should be consulted prior to any entry onto private property. A sample Permit of Entry Form is included in **Appendix 3-A**.

Each permit of entry form should be accompanied by a site map showing the precise location of the exploration with respect to property lines and any structures or features on the private property.

Considerable delay in the exploration timeline can stem from the permit of entry process. In many cases, property owners are unaware of upcoming transportation projects until a geologist or geotechnical engineer asks them for a permit-of-entry for exploration. Even if unopposed or unaffected by the project, the owner may be reluctant to sign a permit of entry for a variety of reasons.

Often, further explanation of the activity and its purpose will be all that is necessary, or just allowing extra time for consideration is all that is required, but will affect the exploration schedule nevertheless.

How to Handle Problems Obtaining Access to Property for Field Investigation

In some cases, landowners are particularly slow in granting access to their property for whatever reason and may even respond to a request for a permit of entry with a letter from their legal council. In these instances, **the Region right-of-way office should be contacted immediately** to take a lead role in negotiations to resolve the issue. Although the Agency has the statutory authority to access any real property for the purpose of survey or exploration, it is an exceedingly rare case for ODOT to exercise this authority for subsurface investigation. The cause for performing a subsurface investigation on such a property must be well-founded and without feasible alternatives.

Note:
When a property owner refuses permission to enter their property, then all further communication and resolution becomes the responsibility of the Right-of-Way Section and the project management. Under no circumstances should field personnel mention or discuss the State's statutory authority to enter upon their property to complete the work, nor should they engage in any bargaining or make agreements other than those stated on the permit of entry form in exchange for access to their properties.

Obtaining Right-of-Way from other Real Property-owing Entities

Other real property-owning entities will take more time in granting a permit of entry. Corporations, governmental agencies, mutually-owned properties, and railroads all have different procedures and requirements for granting access. Corporations may sign permits of entry only from their main offices, governmental agencies may have lengthy policies and procedures for granting permissions, and mutually-owned properties may have numerous non-resident owners that must all be contacted for their consent.

Railway Right-of-Way

Getting permission to access railroad right-of-way is a special case and can be a particularly time-consuming undertaking. For local operators and short lines, getting access may be relatively straightforward. Some larger carriers have a lengthy and rather Byzantine process for handling permit of entry requests that can severely affect a project timeline. If exploration or access is needed on railroad right-of-way, the project timeline should be adjusted accordingly and alternatives sought wherever possible. Permit of entry requests for railroad right-of-way should be forwarded through the headquarters Right-of-Way section.

In the event that the state-owned railroad right-of-way must be accessed, contact **ODOT's Rail Section** to obtain that permit.

Limiting Site Impact

When performing subsurface investigation on private property, all care must be taken to avoid and mitigate the site impact. Access to such sites should be planned with the smallest possible impact. Although some exploration sites will be completely removed during construction, there may be considerable time between then and the time of exploration. The responsibility for complete restoration of exploration sites is placed on ODOT by the same statute that provides legal access to those sites.

3.3.1.8 Utility Location/Notification

Underground and overhead utilities in the project area must be identified and approximately located early in the Exploration Plan development. The presence of utilities may dictate the location of, or access to exploration points.

Warning:
Encountering underground utilities during site investigations can be detrimental to the exploration schedule and budget. Digging or drilling into underground utilities or contacting overhead power lines with drill rig masts or backhoe arms can be lethal. For these reasons, the exact location of all utilities must be determined before any equipment is mobilized to the project site.

Utility Notification Center

In Oregon, the law requires that the **Utility Notification Center** is contacted no less than 48 business hours prior to any ground disturbing operations. This includes all test pit excavation, drilling, and even hand-augering or digging.

Note:
The Utility Notification Center (or "One-Call" Center) can be reached at 1-800-332-2344.

The Utility Notification Center contacts all of the utility services with facilities in the location(s) provided to them based on their records. The individual utilities then dispatch their personnel or contractors to the site to locate and mark the positions of their facilities according to the instructions provided. The following occurs in relation to utility marking:

- The utilities are also required by law to locate their facilities within 48 business hours. If the utility operator does not have facilities near the proposed location site, he or she will mark it as such to indicate that it is safe to proceed. Otherwise, they will mark the approximate location of their facility in the requested vicinity.
- If the utility is close to the proposed exploration, prudence would dictate that the exploration be moved slightly to allow for errors in the utility location, and to further prevent the accidental contact with the utility.
- If the utility has not marked the requested area in the required time frame, they should be contacted prior to commencement of exploration to confirm that the utilities have been contacted, and that they do not have facilities in that area.

The utility operators are often hard-pressed to comply with the 48-hour requirement due to the sheer volume of utility locations – particularly during the summer months when numerous contractors are requesting them. Additional time may be required, so utility location with respect to projected exploration starting times should be planned accordingly. It is also important to look for any other utilities that might be operating in the area in case they are not in the records of the Utility Notification Center. Indications of other utilities are marked riser boxes, manholes, valves, and obvious illuminated structures such as street lighting and advertising. It is the responsibility of the project geologist to notify any other utilities operating in the project area.

Procedures to Perform Prior to calling the One-Call Center

The procedures for utility notification and location are relatively simple, but minor mistakes or overlooked information can result in unnecessary delay and risk to the utilities and the exploration personnel. The following steps should be completed and information gathered prior to calling the One-Call Center:

All proposed exploration sites must be located and clearly marked in the field with a survey lath, painted target on the ground surface, or both. By convention, the survey lath and target should be painted white. Efforts should also be made to make the location as visible as possible for the utility locators such as using additional directional markers and survey flagging.

- Each exploration site should be numbered and labeled as either "proposed test boring" or "proposed test pit".
- The nearest physical address or milepost, and the closest cross-street should be recorded.
- The Township, Range, and quarter Section should also be determined.

When contacting the One-Call Center, the following information will be asked by their operator:

- The caller's identification number (one will be assigned if not already registered)
- For whom the work is being performed
- Who will be doing the work

- Type of work
- Alternate contact
- Location of site (number of exploration points, county, nearest city, address, cross street, township range, section)
- Marking instructions (typically a 25' to 50' radius from each stake or target)
- Presence of any overhead utilities

The operator determines which utilities are known to have facilities in that area and provide the list verbally along with the ticket number which will be used to identify that particular work order. The operator provides the date and time at which the work should be able to proceed. Once this call is complete, the operator will then notify those utilities that will then dispatch their locators. ODOT geotechnical designers use **Utility Notification Worksheet, Appendix 3-B,** to document utility location for future reference while on site.

3.3.1.9 Methods for Site Access

Exploration equipment selected for the subsurface investigation should be matched to the site conditions. Truck-mounted drills are the most commonly available and are capable of accessing most sites with or without additional work and equipment. However, for many sites, access to boring locations can be difficult and even very complex in some cases. Often, the cost for mobilizing special equipment to a project site is more than compensated for in reduced site impact, reclamation effort, time and materials costs, and the additional personnel and equipment that might be needed. Frequently, the method of site access is selected based on one or a combination of desired outcomes whether time and cost, minimizing impact, equipment availability, or equipment capability.

Truck-Mounted Drill Rigs

Truck-mounted drills that are road-legal generally have limited off-road capability even when equipped with 4-wheel or all-wheel drive due to their size and weight. These types of equipment are best suited to work on paved or surfaced areas although they are capable of reaching many off-road locations "in the dry". Because of their axle loading, they can rapidly become mired in wet or soft soils.

In order to use a truck-mounted drill in difficult conditions, access roads may need to be built using one or more additional pieces of equipment. In steep terrain, access roads may require substantial cuts and fills, and where soft ground is encountered, sizeable amounts of rock and geotextile will be needed to surface the road. Special mats or even plywood may be used to distribute the trucks weight over soft ground when accessing a boring location. In any case, such work can be expensive, time-consuming, laborious, and high-impact requiring significant reclamation work after exploration.

Truck-Mounted drills that are off-road capable may require lower-standard access roads, but still need these roads. If a significant amount of winching or vehicle towing is necessary, an alternative method of site access should be strongly considered, if only for safety reasons. The advantage of truck-mounted drill rigs is that they are usually the best-equipped and highest-powered pieces of equipment available, so if a particular type of drilling or deep hole is required, these may be the only option. For accessible sites, truck-mounted drills are usually the cheapest and fastest way to accomplish explorations since they can drive over a site, set up, complete the boring, and move on to the next location with relative ease and with fewer support vehicles.

Track or ATV-Mounted Drill Rigs

Many exploration drill manufacturer's product lines now include drill rigs mounted on a variety of track and rubber-tire ATV platforms with some of the same features and capabilities as their truck-mounted counterparts. In some cases, the drilling equipment is the same, and only the platform varies:

- **Track-mounted drill rigs:** Track-mounted drill rigs offer a much greater off-road capability and ability to access sites in rough terrain and soft ground. Although the track-mounted drill can reach difficult locations, some road-building or at least clearing of trees and vegetation may be required, although to a much lesser degree, than their truck-mounted counterparts. A level pad upon which to set the drill may also need to be constructed. One of the drawbacks of track-mounted drills is that they require slightly more time for set up and moving between longer distances since they must be hauled to project sites on a flatbed truck or trailer. The presence of the trailer or large truck for hauling the drill may also prove to be another encumbrance when working in tight locations or those sites with limited parking or space for maneuvering a long truck and trailer combination. The types of tracks must also be appropriate for the site.

Note:

Older-style steel caterpillar tracks are ideal for traversing steep slopes with a soil cover, but will be harmful to pavements or landscaped areas. Newer developments with rubber tracks offer better traction on bare rock surfaces, and are less harmful to pavements and landscaping but should still be used with caution as their treads can still damage or scar most surfaces.

- **ATV Mounts:** Typical ATV-mounts consist of "balloon" or other oversized rubber tires for use in soft ground or swampy areas. The advantage that such vehicles have over tracks is the lighter load per unit area and correspondingly reduced impact to sensitive areas such as wetlands, landscaping, private properties, etc. Because of their distributed load, these vehicles are more suited to soft or uneven ground applications rather than for sites where traction on steep slopes is most needed. Several manufacturers now produce ATV platforms with tractor-style tires that offer many of the advantages of tracked and "balloon" tires with respect to traction, impact, and load distribution.

Difficult Site Access

A variety of site conditions and subsurface information requirements create substantial difficulties in reaching exploration sites whether in remote, environmentally sensitive areas, or restricted space in the built-up environment. Such obstacles can range from high-angle slopes and physical barriers to restricted work areas such as confined spaces (as defined by OSHA), limited work space due to objects or environmentally sensitive areas, and over-water work. Diverse methods are available to assist with difficult site access as well as drilling contractors that specialize in this type of work.

Methods and equipment for difficult site access are as varied as the sites themselves. The common factor that limits what methods can be used for certain applications is the weight of the equipment with the volume of the machinery also being a limitation.

- **Winching or dragging:** Much of this work in the past has been performed by skid or trailer-mounted equipment with some man-portable also employed in some areas. This equipment has been winched, crane-lifted, or dragged into place by other tractors.

With the advent of track and ATV-mounted drills, winching and skidding drilling equipment into place is no longer necessary or recommended due to the amount of ground disturbance involved.

- **Cranes:** Cranes are often employed to lift equipment into tight work areas although the weight of many of these drill rigs necessitated very large pieces of equipment to move them and had their own space issues.
- **Specialized equipment:** Until recently, most of the skid or trailer-mounted and man-portable drill rigs had restricted power and capabilities. However, drilling technology has advanced to the point where smaller and lighter equipment is capable of performing heavier drilling tasks. Specialized difficult-access drilling contractors generally use their own customized equipment that comes with a specific platform, or breaks down into lighter compartmentalized sections that are reassembled at the boring location. Much of this specialized equipment is light enough to be transported while slung beneath a helicopter.

Most modern drilling equipment not mounted on a truck chassis, with the exception of some man-portable equipment, is capable of completing almost all geotechnical exploration tasks in the same amount of time as their road-legal counterparts. However, these drills will always be restricted by allowable axle loads during transport, and so they will always have a disadvantage with respect to their overall horsepower versus a truck-mounted rig that does not require a truck and trailer combination for roadway transport. This disadvantage is typically only manifest in very deep and/or large-diameter boreholes.

Barge/Over-Water Drilling

Foundation investigation for bridges commonly requires in-stream access to drill sites. To achieve this, barges or other platforms must be used to set the equipment over the foundation location. Over-water work will add extra details to a site investigation, and depending on the location, this can add extensive logistical complexity to a project.

- **Permitting:** Additional permits will be needed to conduct the over-water work from the **US Army Corp of Engineers** and/or the **U.S. Coast Guard**, and from the port authority or harbor master with jurisdiction over the waters in which the investigation is being conducted. An additional staging and launch areas must be identified where equipment can be loaded onto the barge, and where the crew can access the work site for daily operations. The appropriate equipment must also be selected for the site with respect to the currents, depths, river traffic, obstructions, and other details.
- **Launch site:** The site for initially loading and launching the drill barge must be of sufficient size for the type of equipment being used. The launching ramp should have enough grade to provide enough draft for the barge. The facility will also need enough room to either drive or lift the drilling equipment onto the barge and to safely load and unload all other ancillary equipment and supplies. Scheduling the facility for loading and unloading may also be important at different times of the year. Some ports may only be available at certain times due to their ongoing cargo loading operations and public or commercial fishing ramps may be crowded during those seasons. A more proximate and smaller location may be available for launching a skiff or other small craft to support the daily drilling operations and permit crew changes between shifts.
- **Drilling barge:** The barge and any other vessels used for the over-water drilling operations must also be selected and rigged for the conditions.
    - o The drilling barge itself must be of sufficient size not only to support the weight of the drill and other equipment, but must also have enough deck space for whatever sampling and testing operations that will also be carried out.

- The vessel used to transport the drilling barge should also be capable of moving the barge in all conditions of weather and current.
- For work in very slow currents or standing bodies of water, the drill barge may be fixed in place by spud anchors or by lashing to a fixed object such as a driven pile or pier. Where stronger currents occur, whether stream or tidal, a larger vessel may be required to transport and anchor the drill barge during operations. Additional anchoring will be needed in such conditions.
- Where water levels will fluctuate quickly during the conduct of drilling such as in tidal zones and downstream of large dams subject to rapid discharge, allowances must be made for the drill barge to move accordingly with respect to elevation. These operations will usually require the drill barge to use free-moving spud anchors that are also fixed to a more securely anchored vessel.
- The access vessel or skiff must also be capable of operations in all conditions at the site.
- Provision must be made for keeping track of elevation changes during tidal or current changes as this will profoundly affect the drilling operations.

Note:
*As a condition of the **Corps of Engineers** and/or the **Coast Guard** permit, a licensed Marine Surveyor must be engaged to examine the equipment and the site conditions. This professional will then make recommendations concerning the equipment, personnel, and safe conduct of operations. Whether or not a Marine Surveyor is required, their inclusion for over-water work planning is highly recommended for the particular skills and efficiencies that they bring to this rather hazardous aspect of subsurface investigation.*

3.4 Exploration Management and Oversight

The daily field exploration activities on a project should be based primarily on the execution of the Exploration Plan. The Exploration Plan provides a framework for scheduling and adjusting field operations as needed. It will necessarily allow for enough flexibility to modify the subsurface investigation program as information comes in from the field.

- The Project Geologist should maintain a base-level subsurface model from the subsurface information as it is received in order to make the needed modifications.
- The Field Geologist/Drill Inspector will need to provide regular updates on the field activities and information gathered so that changes to the schedule and routine can be made expediently. With the advent of cellular telephones and increasing areas of coverage, field crews should only be a few minutes away from contact with the senior geotechnical designers to inform them of unanticipated field conditions and in turn, receive direction on how to proceed with the modifications.

Because of the costs of subsurface exploration and the rapid use of the data, it is imperative that the subsurface investigation is directly supervised by qualified and experienced personnel. All on-site personnel including drillers, field geologists/engineers, and testing specialists should be instructed and familiarized with the project objectives and their role in achieving those objectives. Special geotechnical or other problems that may be anticipated during exploration including contingencies for addressing them should also be conveyed. All field personnel should be instructed in their role concerning project requirements for schedules, environmental protection, and especially, site safety and health procedures. Field personnel should communicate frequently with project supervisors or geotechnical designers.

Regular transmission of field data such as boring logs, test data, field conditions, and daily driller's reports will streamline and economize the site exploration.

Note:
Any unforeseen site changes, complications, and geologic or geotechnical problems revealed during the investigation that will affect the project scope, schedule or budget should be communicated to the Project Leader without delay. The geotechnical designer charged with the exploration program is responsible for immediately and succinctly informing the Project Leader of the nature of the problem, the expected remediation, and the anticipated impact to the project. The geotechnical designer should then be prepared to offer alternatives and their respective outcomes for the resolution of the problem.

3.5 Subsurface Exploration Requirements

3.5.1 General

The 1988 *AASHTO Manual on Subsurface Investigations* is the basis for subsurface investigations conducted by ODOT. This manual provides guidance on the minimum amount of investigation for the various structures and geotechnical features constructed for transportation projects. The manual states however, in numerous places, that there can never be a set of specifications and guidelines that will determine the amount of exploration that must take place for every project.

Note:
The number of borings, their distribution, sampling interval, and depths of penetration will always be determined by the underlying geology and the size and complexity of the project.

Planning for the subsurface exploration will be based on past knowledge of the site and on the published and unpublished literature that was consulted during the project reconnaissance phase. However, even the most thoroughly studied sites will still reveal previously unknown conditions, and each exploration provides new information about it. In a sense, the site conditions are truly unknown until the exploration begins, and knowledge of it increases as the investigation proceeds so adjustments must be made in the field to economize the investigation while assuring a full investigation of the important geotechnical design elements.

3.5.2 Exploration Spacing and Layout

The layout of explorations on a project is determined by many variables. As previously discussed, the assumed complexity of the underlying geology and the type of facility typically dictate the exploration spacing. Consider the following:

- Where conditions are uniform and a considerable amount of previous, reliable work has been accomplished in a project area, exploration spacing may be increased.

- If the geologic conditions are complex and change significantly over short distances, then explorations will necessarily be conducted on a shorter interval.
- Facilities that will impart a heavy load or are more sensitive to settlement or other movements will also require a more detailed exploration.

The 1988 *AASHTO Manual on Subsurface Investigations* provides a range of exploration spacing for the various structures and features that are typically the subject of subsurface exploration.

These guidelines are modified for use within the State of Oregon where subsurface conditions at the vast majority of sites warrant much tighter exploration spacing due to the highly changeable nature of the state's geology.

3.5.2.1 Spacing and Layout Strategies

Because transportation projects are typically linear, explorations tend to be channeled into a relatively straight and narrow corridor, and are often laid out only along the centerline of many features. This should be avoided as it most often results in poor development of the subsurface model. To avoid this, boreholes should be spread out to either side of the centerline to help determine the strike and dip of the underlying strata, the nature of the contacts (i.e. conformal or non-conformal), and other changes or irregularities across the subsurface profile. Exploration to reveal or characterize geologic hazards such as faults and landslides that affect the proposed project may necessarily be conducted outside of the proposed alignment(s). Material source or disposal site investigations normally take place far away from the project alignment and will have different exploration spacing criteria.

Take special care when conducting explorations in particular alignments and foundation locations. Certain geologic conditions, such as openwork cobbles and boulders, heaving sands, or highly fractured rock may bind exploration tools severely enough that the drill crew is unable to retrieve them from the hole where they subsequently form an obstruction during drilled shafts construction. In areas that experience high artesian pressures, improperly sealed boreholes may form an undesirable conduit for groundwater to enter footing excavations, cut slopes, or cofferdams.

Note:
All borings should be abandoned in accordance to Oregon Water Resources Department Regulations to prevent vertical water migration. Provision should also be made to extract bound drilling tools from the boring with special equipment.

The boring layout guidelines presented here are of a general nature and are intended for use in the preliminary location of site exploration points. The final exploration locations should be developed as the site investigation proceeds. Information must be incorporated into the Exploration Plan as it becomes available to assure the most complete, cost-effective outcome.

3.5.2.2 Embankment and Cut Slope Explorations

The maximum exploration spacing for embankment fills over 10 feet (3.05m) in height is 200 feet (61m). Where changeable conditions or problem areas such as those with soft and/or compressible materials are present, then the exploration spacing should be decreased to 100 feet (30m). In many cases it will be necessary to conduct additional exploration using cone penetrometers, hand augers, or backhoe test pits to further define the properties and boundaries of problem foundation conditions. At least one boring should be located at the point of maximum fill height.

For cut slopes 10 feet (3m) and higher, the maximum boring spacing is 100 feet (30m). Borings should be staggered to each side of the cut line to help determine the strike and dip of the units in the cut slope, and one of the borings should be placed at the maximum depth of the cut. For "through-cuts" where a cut slope will be located on each side of the roadway, boring spacing may be increased to 200 feet (61m) for each cut slope, but the borings must be staggered so that the total 100 foot (30) spacing continues along the length of the cut.

Additional borings will be required in areas of faulted, sheared, tightly folded, highly weathered, or other potentially detrimental conditions exist.

Hand augers, direct push (i.e., GeoProbe), air-track drills, test pits, geophysical surveys and other alternative exploration techniques can be used to supplement the test borings in proposed cut slopes to determine the elevations of variable bedrock surfaces and depths to bedrock. Air-track drills may also be used to penetrate the bedrock surface to determine and further resolve the location(s) of weathered rock zones and other features within the proposed cut slope.

3.5.2.3 Subgrade Borings

Where relatively unvarying subsurface conditions are predicted and no other foundations or earthworks are expected, the maximum subgrade boring spacing should be 200 feet (61m). In areas where highly variably geology is predicted, the boring spacing should be decreased to 100 feet (30m) and further decreased to 50 feet (15m) in highly erratic conditions. Where critical subgrade conditions exist, the boring spacing may be decreased to 25 feet (8m).

Alternate exploration methods may be used in variable geologic conditions to supplement the borings and further resolve the characteristics and distribution of problematic materials and conditions. Such methods may include hand augers, push-probes, geoprobes, and test pits.

Test pits

Test pits on short intervals (25 feet/8meters) are not recommended due to the potential introduction of soft areas in the subgrade where the pits were located. If necessary, this problem may be alleviated by the use of compacted granular backfill materials to abandon the test pits after exploration. The test pit spoils would then need to be disposed of off-site. Several geophysical survey methods may also be appropriate for subgrade investigations to supplement the test boring information. Seismic reflection and electro-magnetic methods are commonly the best-suited for determining material property boundaries and saturated or water-bearing zones.

3.5.2.4 Tunnel and Trenchless Pipe Installation Borings

Tunnel construction for highway projects in Oregon is rare; however, trenchless pipe installation is common. Tunnels and trenchless pipe installations share many common construction and design issues and are thus treated in a similar manner with respect to subsurface characterization and exploration. Borehole spacing requirements for tunneling and trenchless pipe installation are highly dependent on the site geologic conditions and topography. The soil, rock, or mixed-face conditions predicted will determine the borehole spacing as well as the type of exploration and testing conducted. The depth of the tunnel/trenchless pipe alignment will greatly influence the total amount of drilling required.

The actual borehole spacing selected for tunnel or trenchless pipe installation should be determined by the actual site conditions. These conditions should be identified in advance by preliminary site review, and in the case of larger projects, preliminary site investigations conducted during the Phase I field survey. The recommended general borehole spacing for selected conditions is shown in the following table:

Table 3-1. Tunneling and Trenchless Pipe Installation Recommendations

Recommendations	
Soft Ground Tunneling	
Adverse Conditions	**50-100 feet (15-30m)**
Favorable Conditions	**200-300 feet (61-91m)**
Mixed-Face Tunneling	
Adverse Conditions	**25-50 feet (8-15m)**
Favorable Conditions	**50-75 feet (15-23m)**
Hard Rock Tunneling	
Adverse Conditions	**50-100 feet (15-30m)**
Favorable Conditions	**200-500 feet (61-152m)**
Trenchless Pipe Installation	
Adverse Conditions	**15-30 feet (5-9m)**
Favorable Conditions	**30-50 feet (9-15m)**

In addition to the geologic conditions, other site constraints will equally determine the number and spacing of borings for tunnels and trenchless pipe installations. The location of existing structures with respect to the proposed depths and alignments will necessitate a more detailed investigation at those locations.

Geophysical surveys may also be used in conjunction with the borings to further define the geologic conditions and to help determine the final boring layouts as defined below.

- Wherever possible, horizontal borings should be taken along the proposed tunnel alignment. Current technology and contractor capabilities allow longer and more accurate horizontal borings that provide essential information regarding the expected tunnel face conditions.
- Trenchless pipe installations through existing embankments can and should be fully penetrated by horizontal borings to determine the conditions along the full length of the trenchless installation. Because the horizontal borings do not reveal the conditions above and below the tunnel/trenchless pipe installation horizons, vertical borings are still required.

Clearly, tunnels with horizontal and vertical curves will be difficult to investigate with horizontal borings, but as technology advances, methods may soon be available to steer borings along these alignments.

3.5.2.5 Structure-Specific Borings

The actual number and spacing for borings for specific structures varies greatly depending on the predicted geologic conditions and the complexity of the site. In this regard, nearby features such as streams and environmentally sensitive areas, geologic hazards, and nearby structures will further prescribe the actual amount of exploration required.

Bridges

For all bridges on ODOT projects, at least one boring will be placed at each bent location. Borings should be placed at opposite sides of adjacent bent locations when practical as defined below.

- For bridges that are 100 feet (30m) wide and larger, at least two borings will be placed at each bent.
- When spread footings are proposed, two borings at opposing corners of the footing are advisable. Spread footings located on the banks of rivers and streams should be investigated with at least two borings – one on the down-slope and one on the upslope side of the proposed footing.
- If wingwalls greater than 20 feet long are to be constructed, then a boring should be placed at the end of each wingwall and at 50 foot (15m) intervals from the end of the wingwall to the bridge abutment.
- Trestle-type bridges (usually for detours) should also be investigated at every bent. Preferably, the borings should be staggered from opposite ends of adjacent bents.
- Where highly variable conditions are anticipated, then a boring should be advanced at both ends of each bent.
- For drilled shaft foundations, 1 boring should be placed at the location of each proposed shaft of 6 feet (1.8m) in diameter and larger. **Federal Highway Publication FHWA-IF-99-025** should be consulted for exploration spacing at drilled shaft foundation locations using smaller diameter shafts.

Culverts

All proposed new and replacement culverts require some level of subsurface investigation as defined below:

- Typically, culverts with a diameter of 6 feet (1.8m) and larger are investigated with test borings while smaller culverts are investigated with hand-dug test pits or hand auger holes. However, judgments should be made regarding the actual site conditions and the facility in question to determine the number and spacing of borings.
- Complex geologic conditions merit a more intense investigation, while larger embankments, adjacent facilities, and proximate unstable slopes may result in a more detailed investigation for smaller-diameter culverts.
- At least two borings should be completed for each culvert up to 100 feet (30m) long.
- For culverts longer than 100 feet (30m), borings should have a maximum spacing of 50 feet (15m).
- In complex geologic conditions, boring spacing may be decreased to 20 feet (6m). Borings will typically be located along the axis of the proposed culvert.
- For culvert replacements, the borings should be located immediately outside or partially within the excavation limits of the original culvert installation with particular care to not locate a boring where it will penetrate the existing pipe.
- Borings will typically be located along the axis of any proposed culvert location.

Box culverts 100 feet (30m) and longer require two borings at each end and at the prescribed interval between the ends. Refer to Section 3.5.3.4 Tunnel and Trenchless Pipe Installation Borings for exploration spacing on culverts installed using trenchless technology.

Retaining Walls

Retaining walls higher than 4 feet (1.2m) and any wall with a foreslope and/or backslope angle steeper than horizontal require a subsurface investigation. At least two borings are required for every retaining wall regardless of length with the exception of retaining walls less than 25 feet (8m) long. The maximum borehole spacing along any retaining wall is 100 feet (30m). The preponderance of retaining walls for ODOT projects will require closer spacing due to the typically variable conditions encountered. One boring is required at each end of the proposed wall. Where the proposed wall is longer than 100 feet (30m) long, and less than 200 feet (61m), the third boring may be placed at either the midpoint of the wall, or at the location of the maximum wall height. Embankments supported by retaining walls on each side should be investigated as two separate walls.

Borings are typically located on the wall alignment at the proposed location of the wall face however; they may be staggered to either side of the wall line but should remain within the wall footprint to evaluate the wall foundation conditions. Consider the following:

- For soil nail, tieback, and similarly reinforced walls, additional borings should be completed in the wall reinforcement zones.
- Borings should be located behind the wall in the predicted bond/anchorage zones for tieback walls, or horizontally 1 to 1.5 times the wall height back from the wall face.
- Borings for tiebacks/anchors should be interspersed with the borings along the wall face. Thus, a 200 foot (61m)-long wall would have (at a minimum) 5 borings – 3 along the wall centerline at the ends and the midpoint and 2 in the prescribed locations behind the wall at the 50 foot (15m) and 150 foot (46m) points along the wall centerline.

The preceding recommended borehole spacing should be halved for walls that will be constructed to retain landslides. Landslide retaining walls should have a minimum of 2 borings along the wall line regardless of length. The maximum borehole spacing along such walls is 50 feet (15m) with corresponding holes interspersed between located in the bond/anchorage zone. These boreholes are specifically for characterizing the subsurface conditions at the location of the proposed retaining wall, and are in addition to any borings advanced to characterize the landslide. Landslide investigation borings may suffice for the retaining wall investigation only where they fall within the prescribed locations.

Soundwalls, Traffic Structures and Buildings

Soundwalls and traffic structures, such as mast arm signal poles, strain poles, monotube cantilever sign supports, sign and VMS truss bridges, luminaire poles, high mast luminaire poles, and camera poles are common features on highway transportation projects. Buildings such as maintenance facilities, rest areas, pump stations, water tanks and other unique structures are also sometimes required for ODOT projects.

Standard drawings have been developed for soundwalls and most of the traffic structures and these standard drawings contain standard foundation designs for each of these structures. Every foundation design shown on a standard drawing is based on a certain set of foundation soil properties, groundwater conditions and other factors that are described on the drawings. These soil properties and conditions must be met in order to use the foundation design shown on the standard drawing.

Note:
The subsurface investigation for these structures (with standard foundation designs) should be sufficient to determine whether or not the subsurface and site conditions meet the requirements shown on the standard drawings. If the foundation conditions at the site are determined not to meet

the subsurface and site conditions described on the standard drawings (e.g., "poor" soil conditions or steep slope), then the standard drawings cannot be used and a site specific foundation investigation and design is required.

For buildings and traffic structures without standard foundation designs, the foundation conditions must be investigated sufficiently to determine the soil properties and groundwater conditions required for a site specific foundation design.

All new soundwalls, traffic structures or buildings require some level of subsurface investigation. Considerable judgment is needed to determine which structures will need site-specific field investigations and the extent of those investigations. If the available geotechnical data and information gathered from the site reconnaissance and/or office review is not adequate to make an accurate determination of subsurface conditions, then site specific subsurface data should be obtained through a proper investigation. In these cases, explorations consisting of geotechnical borings, test pits and hand auger holes, or a combination, shall be performed to meet the investigation requirements Most of these structures require site-specific soil and/or rock properties for their foundation design or to determine foundation embedment depths when using a standard drawing. Therefore, subsurface exploration should be anticipated for most soundwall, traffic structure, and building foundation designs.

The extent of the investigation will be largely dependent on the predicted site conditions and the type of structure. At unfavorable locations, drilling and sampling may need to be conducted more frequently while sites with favorable conditions may allow for less frequent and/or less expensive investigation methods such as hand augers holes and test pits.

As a minimum, develop the subsurface exploration and laboratory test program to obtain information to analyze foundation bearing capacity, lateral capacity, stability and settlement.

The following information is generally obtained:

- Geological formation(s)
- Location and thickness of soil and rock units
- Engineering properties of soil and rock units such as unit weight, shear strength and compressibility
- Groundwater conditions (seasonal variations and maximum level over the design life of the structure)
- Ground surface topography
- Local considerations, (e.g., slope instability potential, expansive or dispersive soil deposits, utilities or underground voids from solution weathering or mining activity)

Specific field investigation requirements for soundwalls, traffic structures and buildings are summarized in **Table** 3-2. Specific field investigation requirements Note that the term "borings" in the table refers to conventional geotechnical boreholes while the term "exploration points" may consist of any combination of borings, test pits, hand augers, probes or other subsurface exploration device as required to adequately determine foundation conditions.

Table 3-2. Specific field investigation requirements

Structure Type	Field Investigation Requirements
Mast Arm Signal Poles, Strain Poles, Sign and VMS Truss Bridges, Monotube Cantilever Sign Supports, Luminaire Poles, High Mast Luminiare Supports, and Camera poles.	• **For mast arm signal pole or strain pole foundations within approximately 75 ft of each other or less, such as at small to moderate sized intersections, one geotechnical boring for the foundation group is adequate if conditions are relatively uniform. For more widely spaced foundation locations, or for more variable site conditions, one boring near each foundation should be obtained.** • **Investigate sign and VMS truss bridges with one boring at each footing location unless uniform subsurface conditions are sufficient to justify only a single boring. Where highly variable conditions occur or where the sign bridge footing is proposed on a slope, additional borings or exploration points may be necessary.** • **For single, isolated monotube cantilever signs; one geotechnical boring at each footing location.** • **Luminaires, High Mast Luminaire Supports and Camera Poles; one exploration point at each footing location.** • **The depth of the explorations should be equal to the maximum expected depth of the foundation plus 2 to 5 ft.**
Sound Walls	**For sound walls less than 100 ft in length, a geotechnical boring approximately midpoint along the alignment and should be completed on the alignment of the wall. For sound walls more than 100 ft in length at least 2 borings are required. Borings or exploration points should be spaced every 100 to 400 feet, depending on the uniformity of subsurface conditions. Where adverse conditions are encountered, the exploration spacing can be decreased to 50 feet. Locate at least one exploration point near the most critical location for stability. Exploration points should be completed as close to the alignment of the wall face as possible. For sound walls placed on slopes, an additional boring off the wall alignment to investigate overall stability of the wall-slope combination should be obtained.**
Building Foundations	**The wide variability of these projects often makes the approach to the investigation of their subsurface conditions a case-by-case endeavor. The following minimum guidelines for frequency of explorations should be used. More detailed guidance can be found in the International Building Code (IBC). Borings should be located to allow the site subsurface stratigraphy to be adequately defined beneath the structure. Additional explorations may be required depending on the variability in site conditions, building geometry and expected loading conditions. Water tanks constructed on slopes may require at least two borings to develop a geologic cross-section for stability analysis.**

Table 3-2 (Cont.)

Structure Type	Field Investigation Requirements
	Building surface area (ft^2) / **No. of Borings (minimum)** **<200** — **1** **200 - 1000** — **2** **1000 - 3,000** — **3** **>3,000** — **3 – 4**
	The depth of the borings will vary depending on the expected loads being applied to the foundation and/or site soil conditions. All borings should be extended to a depth below the bottom elevation of the building foundation a minimum of 2.5 times the width of the spread footing foundation or 1.5 times the length of a deep foundation (i.e., piles or shafts). Exploration depth should be great enough to fully penetrate soft highly compressible soils (e.g., peat, organic silt, soft fine grained soils) into competent material suitable for bearing capacity (e.g., stiff to hard cohesive soil, compact dense cohesionless soil or bedrock).

In addition to the exploration requirements in Table 3-2. Specific field investigation requirements, groundwater measurements, conducted in accordance with Chapter 3, should be obtained if groundwater is anticipated within the minimum required depths of the borings as described herein.

3.5.2.6 Critical-Area Investigations

In areas where critical geologic conditions or hazards such as highly irregular bedrock surfaces, extremely weathered or altered rock, compressible materials, and caverns or abandoned underground facilities are predicted from detailed background study or preliminary exploration, it may be necessary to further investigate the area with additional explorations. Such investigations normally involve drilling on a grid pattern over the area in question. An initial, wider grid pattern may be selected to locate the area of most concern with a closer grid pattern used later to further characterize the area of concern. Grid pattern investigations may consist of hand auger holes, direct push holes, or cone penetrometers in addition to the more conventional test borings. Geophysical surveys may also be used to establish or refine the boundaries of the grid pattern investigation.

3.5.2.7 Landslides

The number and layout of test borings for landslide investigation depends upon the size and nature of the landslide itself and on the results of detailed site mapping and initial subsurface models based on the mapping. Since information about the subsurface is unknown initially, landslide investigation largely becomes an iterative process as new data obtained provides information that is used to further develop enough knowledge of the landslide to begin stability analysis.

The approach to landslide investigation is very complex and involves numerous techniques and procedures, and is discussed in greater detail in **Chapter 13**. This chapter is intended to convey a general sense of the layout of the borings needed for a "typical" landslide investigation.

Enough borings must be made initially to fully develop at least one geologic cross-section through the axis of the slide. Consider the following:

- As a minimum, there should be borings near the top, middle, and bottom of a known or potential landslide area. Ideally, the borings would be placed in the toe or passive wedge area (if applicable), at the head or active slide zone, the area of transition between the active and passive zones, and in the areas behind the headscarp and in front of the toe outside of the slide zone.
- For longer slides, space additional borings in the active and/or passive slide zones on 50 foot (15m) intervals.
- Place additional borings on a 50 foot (15m) interval in a line perpendicular to the direction of slide movement at the deepest zone of slide movement.

For investigation of areas of potential slide movement, a grid pattern of explorations are usually selected for preliminary identification and delineation of the affected area. The grid spacing is dependent on several factors. Usually, the predicted size of the landslide, results of remote sensing, availability of previous data, and site access will primarily determine the spacing between borings. Where large areas would potentially be affected by landslide movement, a 200 foot (61m) square or staggered grid spacing is sufficient for preliminary identification.

Subsurface Investigations on Unstable Rock Slopes

Subsurface investigations for unstable rock slopes are necessary when a significant amount of rock excavation is needed to accommodate highway realignment or an increased fallout area.

- Typically, the amount of information available at a large, accessible rock exposure is sufficient for minor slope modification, and of generally greater value than core drilling with respect to information concerning rock conditions.
- However, when significant modification of the slope is considered for realignment and/or rockfall mitigation, subsurface investigation is frequently needed to determine the rock character within the proposed cut, overburden thicknesses, groundwater conditions, three-dimensional character of the units (if unknown), and other important design and construction information.
- Drilling is recommended to assure continuous subsurface conditions throughout the excavated rock material.

The skilled geologist's interpretation of the outcrop generally provides enough information for rock slope design, but the changeable nature of the state's geology, and the need to assure subsurface conditions to prevent construction delays and claims is usually reason enough to gain the additional assurance of further subsurface data. This is not to state that drilling for a rock cut slope modification is automatic. The geotechnical designer must determine the cost-benefit of additional subsurface investigation based on the local geology and the risks involved.

Note:
For the assessment of large block or wedge failures, subsurface investigation should proceed in a similar manner to the approach to landslide investigations as described above. Some of the borings, or additional borings may be needed at prescribed orientations other than vertical to assess the projected failure planes.

For projects where realignment or slope modification to increase the fallout area is needed the investigation should carry on according to the procedures for cut slope investigation described in Section 3.5.3.2.

3.5.3 Exploration Depths

Determining the required depths of subsurface explorations requires the consideration of many variables such as the size, type, and importance of the structure, and most of all, the underlying geology. Consider the following:

- The borings should penetrate any unsuitable or questionable materials and deep enough into strata of adequate bearing capacity where significant settlement or consolidation from the increased loads from the proposed structure is reduced to a negligible amount. The stress at depth added by the structure is usually taken from the appropriate tables and charts or determined using the **Boussingesq** or **Westergaard** solutions.
- All soft, unsuitable, or questionable strata should be fully penetrated by the borings even where they occur below an upper layer of high bearing capacity.
- Test borings should not be terminated in low-strength or questionable materials such as soft silt and clay, organic silt or peat, or any fill materials unless special circumstances arise while drilling.

3.5.3.1 Termination Depths

When competent bedrock is encountered, test borings may generally be terminated after penetrating 15 feet (4.5m) into it. Where very heavy loads are anticipated, test borings may be extended to a considerable depth into the bedrock depending on its characteristics and verification that it is underlain by materials of equal or greater strength. For most structures, it is advisable to extend at least one boring into the underlying bedrock even when the remaining borings are terminated in soils of adequate bearing capacity.

As with all other aspects of subsurface investigation, considerable professional judgment is needed to determine the final depths of planned explorations. Generally, previous subsurface information is needed to determine the approximate depth of the proposed borings on the Exploration Plan. Where this information is unavailable, general guidelines can be used to establish the preliminary exploration depths and quantities. These guidelines are outlined for specific geotechnical features in the following sections.

3.5.3.2 Embankment and Cut Slope Exploration Depths

For embankments of 10 feet (3m) or greater in height, the test borings should penetrate from 2 to 4 times the proposed fill height or more depending on the final width of the roadway and the actual materials encountered. If suitable foundation materials are encountered such as dense granular soils or bedrock, the depth may be decreased up to a minimum depth equaling the height of the embankment. Where confined aquifers with artesian pressures or liquefiable soils are present, the exploration depth should be extended to fully penetrate these units.

Cut slopes with a depth of 10 feet (3m) or more should be explored to a depth that is two times the height of the proposed cut. When bedrock is encountered in a cut slope boring, the boring should extend at least 15 feet below the finish grade of the cut. Cut slope borings should be extended if sheared surfaces or other evidence of landslide susceptibility are encountered that could affect the performance or constructability of the finished slope.

3.5.3.3 Subgrade Borings

Where minor amounts of earthwork (cut slopes less than 10 feet (3m) deep) for the alignment profile are expected, test borings and test pits should extend 15 feet (4.5m) below the proposed final grade elevation. Where bedrock or other hard materials are encountered, coring should be extended 15 feet (4.5m) into the hard stratum to evaluate their conditions. For fill areas less than 10 feet (3m) high, explorations should extend to 15 feet (4.5m) below the original ground surface unless questionable materials are encountered. If soft, organic or other deleterious materials are encountered in subgrade borings, the depth of exploration should be increased as necessary to fully evaluate those materials.

3.5.3.4 Tunnel and Trenchless Pipe Installation Borings

A “rule-of-thumb” for tunnel exploration is the amount of exploration drilling should be 1.5 times the length of the tunnel. This should be considered as a bare minimum for exploration cost estimating for tunnel/trenchless installation projects will shallow alignments in very favorable conditions, and does not include horizontal drilling along the tunnel/pipe profile. Clearly, the amount of drilling for any given length of tunnel/trenchless installation alignment is dependent on several factors that include, among others, the depth of the invert, diameter of the tunnel/pipe, geologic conditions, and contingencies. Typically, tunnel/trenchless installation borings should be extended at least 1.5 tunnel/pipe diameters below the proposed grade of the invert. It may be beneficial to further extend the borings to as much as 3 times the tunnel/pipe diameter as a contingency if the final tunnel alignment has not been determined. The depth of the borings should be increased further to evaluate any unforeseen or unfavorable geologic conditions encountered that may impact the tunnel or pipe design and construction. Wherever practical, horizontal borings should be taken along the tunnel profile because of the advantages of having a full-length representation of the actual tunnel/pipe horizon conditions.

3.5.3.5 Structure-Specific Borings

The guidelines for boring depths presented in **Section 3.5.3** stem from structure-specific boring guidelines developed by AASHTO and other agencies. Follow these guidelines:

- It is highly desirable for all structure-specific borings to penetrate at least 15 feet (4.5m) into bedrock.
- For drilled shaft installations, the test borings should be advanced 1.25 times the total projected shaft length beyond the predicted shaft base elevation.
- If the shaft base is to be founded in soil or rock with an RQD of 50% or less, then the test borings should be extended an additional depth below the proposed bottom of the shaft equal to the larger of 20 feet (6m) or 3 times the shaft base diameter. Shafts are most commonly designed to bear on competent bedrock, thus, where the RQD is greater than 50%, the test boring should also be advanced to the greater of 20 feet (6m) or 3 times the shaft base diameter below the estimated shaft base elevation.

Note:
The geotechnical designer must exercise judgment concerning the nature of the facility with respect to the total and economical amount of drilling needed for the specific structure. Borings for soundwalls, small traffic structures or culverts may not be required to obtain core samples in bedrock, but for bridge foundations, bedrock drilling would certainly be needed.

3.5.3.6 Critical-Area Investigations

In those areas where unfavorable or critical geologic conditions are expected to have an adverse effect on the project design and construction, the explorations should be extended to a depth where those conditions may be fully evaluated. All problematic strata and areas of concern should be fully penetrated by the borings. It is advisable to extend the borings beyond the depths that are strictly necessary rather than terminate them before the desired information is obtained. Borings should never be terminated in soft, organic, or any other deleterious materials that will adversely affect the project design, construction, or performance. Extra drilling in some borings is less expensive than drilling additional borings or even remobilizing equipment to the site to obtain sufficient data for design.

3.5.3.7 Landslides

Considerable flexibility must be built into the Exploration Plan for any landslide, and particularly with respect to the depth of the explorations. Follow these guidelines:

- Typically, the cross-section drawn along the centerline of the landslide is used to develop the preliminary exploration depths.
- Circular, elliptical, or composite curves drawn from the headscarp to the toe bulge are projected onto the cross-section to show the possible depths of slide movement. These curves are commonly exaggerated to conservatively estimate the slide depth.
- The preliminary boring depths should extend 20 feet (6m) or more below the projected slide plane to assure that the zone of movement is fully penetrated, and to secure instruments below the slide plane for the best results.
- Firm, resistant strata, bedrock projections and irregular surfaces will also affect the geometry of the slide plane, and subsequently, the final depths of individual borings.
- Landslide borings should always be extended to a depth that clearly identifies which materials are involved in the current slope movement, which underlying materials are presently stable, and the location of the slide surface(s). This is not only important to the development of a stability analysis, but will become important once again during construction when the precise locations of mitigation efforts will be determined. There is often a possibility that the observed landslide activity is an accelerated portion of a slower, deeper-moving landslide that may only be detected by instrumentation. For this reason, at least one boring should be extended far below the predicted slide surface to divulge such activity. Any Exploration Plan for landslide investigations should contain the flexibility to extend borings to considerable depth during the site exploration.

3.5.4 Sampling Requirements

Since the primary purpose of the subsurface exploration program is the collection of samples that are as closely representative of actual site conditions, the sampling requirements are typically the most stringent in the Exploration Plan. Particular care must be taken in their method of collection, measurement, handling, and preservation since field and laboratory testing results are so greatly dependent on the quality of the sampling. Sampling requirements are also subject to the same variables that affect exploration layout and depth.

- **Sampling interval:** Most Exploration Plans will have a set maximum sampling interval. For most ODOT projects, Standard Penetration Tests (SPTs) are taken, and samples retained, on 2.5-foot (0.76m) intervals in the first 20 feet (6m) of the boring, and on 5-foot

(1.5m) intervals thereafter to the bottom of the hole or until rock coring begins. In addition to this minimum interval, samples should also be taken at each noted change in material or subsurface condition. Where thick, uniform strata exist, a wider sampling interval may be warranted however, this greatly depends on the extent of previous site knowledge and project requirements. Where complex conditions and/or numerous strata exist, the sampling interval may be increased to a shorter sampling interval.

- **Sample collection:** Samples should be collected from each identified stratum, preferably from more than one boring to fully characterize each unit. In addition, undisturbed samples should be obtained from all cohesive soil units encountered. It is frequently warranted to drill additional borings to obtain undisturbed samples in particular units that may have been missed by previous sampling intervals or to further characterize those units. Where a larger volume sample is needed, a variety of sampling methods and techniques can be utilized including oversized split-spoons, various coring methods, and Becker-hammer drills. Sampling techniques are discussed in the next section.
- **Continuous sampling:** Continuous sampling is beneficial in areas of changeable site conditions and underlying geology as well as critical zones for project design. The zones immediately below proposed foundation elevations should be sampled continuously in addition to the zones immediately above, through, and below projected landslide zones of movement. For tunnel/trenchless pipe installations, continuous sampling should be conducted for 1 tunnel diameter above and below the tunnel horizon as well as the tunnel horizon itself. Soil and rock coring is by its nature, a continuous sample, and is the most common method to obtain a continuous representation of the subsurface materials. However, continuous SPTs, Shelby Tubes, or a combination of these and other methods can be used.
- **Observation:** Careful observation and evaluation during drilling and logging of the recovered samples is essential to the entire exploration program. Much information can be recovered even when sample recovery itself is minimal.

3.5.5 Sampling Methods

Various sampling methods are described in this section. Many of the sampling methods are based on ASTM International standards located at www.astm.org (the “ASTM Site”).

3.5.5.1 Standard Penetration Testing

All Standard Penetration Tests must be performed according to ASTM D 1586-99. The Standard Penetration Test (SPT) is the most common method for field testing and sampling of soils. Some variations with respect to standard intervals and refusal criteria occur throughout the industry however the fundamental procedure still adheres to the ASTM standard. The SPT uses the following methods:

- This sampling method uses the standard configuration 2-inch (5cm) outside diameter split spoon sampler at the end of a solid string of drill rods. The split spoon is driven for a 1.5-foot (0.45m) interval using a 140 Lb (63.5 Kg) hammer dropped through a 30-inch (76cm) free fall.
- The number of hammer blows needed to advance the sampler for each 6-inch (15cm) interval is recorded on the boring log and sample container.

- The Standard Penetration Resistance or uncorrected "N"-value is the sum of the blows required for the last two 6-inch (15cm) drives. Refusal is defined as 50 blows in 6 inches (15cm) of penetration and recorded on the log as 50 blows and the distance driven in that number of blows.
- The hole is advanced and cleaned out between sampling intervals for at least the full depth of the previous sample.

This general procedure can be used with larger diameter samplers and heavier hammers for the purpose of obtaining additional sample volumes, but the blow counts do not provide standard resistance values. Prior to the commencement of drilling operations, the hammer energy must be measured to determine the actual hammer efficiency. This information can usually be obtained by the drill manufacturer. If it is not available, a competent technician must be engaged to measure the hammer energy for each drill rig.

3.5.5.2 Thin-Walled Undisturbed Tube Sampling

Undisturbed samples of cohesive soils should be taken with 3-inch (7.6cm) diameter Shelby Tubes according to the standard practice for thin-walled tube sampling of soils in ASTM D 1587-00. This method obtains relatively undisturbed samples by pressing the thin-walled tube into the subject strata at the bottom of the boring. Thin-walled sampling is simply a method for retrieving a sample for laboratory testing. There is no actual field testing involved with thin-walled sampling unless a Torvane or Pocket Penetrometer test is performed on the end of the sample. Pressures exerted by the drill rig while pushing Shelby tubes are frequently recorded for general reference but do not provide repeatable test results. After the unfavorable effects of the sampling procedure, transport, handling, and storage, a truly undisturbed sample cannot be realistically tested in the laboratory. However, with appropriate care, valid samples can be taken for shear strength, density, consolidation, and permeability testing.

Shelby tubes do not utilize a sample retention system to hold the sample in place during retrieval from the borehole, so sample recovery can be unreliable. Thin-walled sampling in general is successful only in soft to stiff cohesive soils. Soils that are very soft are difficult to recover with standard Shelby tube while the upper range of stiff and very stiff soils are difficult to penetrate or bend the tube resulting in a disturbed sample. Oversized clasts and organic fragments in the softer soil matrix can also be detrimental to thin-walled sampling.

Various samplers that use retractable pistons to create a vacuum in the top of the tube can achieve greater success in obtaining undisturbed samples of soft cohesive soils as well as granular materials.

3.5.5.3 Rock Coring

Rock core drilling should be carried out according to ASTM-D 2113-99. Successful core drilling is as much a skill as it is a test procedure. Experienced, conscientious personnel are necessary not only to run the equipment, but also to interpret the results of the drill action as well as the samples recovered. Material recovered may not actually represent the subsurface conditions present if not correctly sampled. Observation and interpretation of the drill action, fluid return, and other characteristics provide indications of the actual validity of the core sample as well as other information concerning the actual conditions in the subsurface.

Note:
ASTM states that the instructions given in D 2113-99 cannot replace education and experience and should be used in conjunction with professional judgment. Qualified professional drillers should be given the flexibility to exercise their judgment on every alternative that can be used within the appropriate economic and environmental limitations.

Triple-tube Core Barrel Systems

Because of the close-jointed, highly fractured nature of many rock formations in Oregon, and the detailed observations desired, rock coring should be performed with triple-tube core barrel systems that are best-suited to such material. These systems provide the best recovery in difficult, highly fractured and/or weathered rock which is extremely important since discontinuity spacing and weathering characteristics usually limit the strength of a rock mass with respect to foundation loading, or the performance of rock excavations. Triple-tube barrels provide direct observation of the rock core specimen in the split-half of the innermost tube as it is extracted from the inner core barrel. This allows accurate measurement of RQD and recovery and discontinuity attitudes prior to further specimen handling. Partial isolation of the sample in the inner split-barrel from the drilling fluids also preserves much of the discontinuity texture and infilling material that is also very important to rock mass characterization.

Most rock coring is performed with "H"-sized systems that provide core specimens with a diameter of $2^{13/32}$ inches (61.1mm).

Note:
Considerable penalties occur with respect to sample quality when using smaller diameter coring systems due primarily to drill action, particularly at greater boring depths; thus, H-sized core should be considered the minimum size for explorations.

Larger diameter cores also provide a better assessment of discontinuity properties. There may be situations where smaller diameter coring is necessary such as difficult access sites where small equipment is needed that may not have the torque required to turn larger diameter casing. Core runs are typically made in 5-foot sections since this is the approximate length of most commonly-available core barrels. Runs may be shortened when difficult drilling conditions are encountered. Longer barrels may also be used in highly favorable conditions such as quarry site investigations or other areas with uncommonly massive rock.

Rock core specimens should be preserved and transported according to the standard practice in ASTM D 5079-02. Core specimens should always be extruded from the inner core barrel using the hydraulic piston system. The inner split barrel should not be manually rammed out of the inner barrel as this will result in sample disturbance. The core should not be dumped out of the end of the barrel either since this will also disturb the sample as well as invalidate some of the information.

3.5.5.4 Bulk Sampling

Bulk sampling should be carried out at all pipe/culvert locations from the actual invert elevation when test borings are not required. The samples collected are submitted for the appropriate chemical testing. Typically, bulk samples of 25 lbs (11Kg) if impermeable bags are used, or 2 gallons (7.5 liters) for jar/bucket samples are collected from each discrete sampling site. Sample receptacles must be sealed to preserve natural moisture conditions. Bulk sampling may also be conducted for material source investigations and other surficial applications. All samples collected should be preserved and transported according to ASTM D 4220-95**.**

3.5.6 Sample Disposition

Soil and rock samples collected during subsurface exploration should be transported to the appropriate ODOT region storage facility upon completion of the investigation. Soil samples are usually retained for only a short period of time after project construction since physical and chemical changes occur that, over time, invalidate the results of further testing regardless of any effort to preserve them. Rock core specimens are typically retained for 3 years after the final acceptance of the project or when the contractors and other concerned parties have been settled with provided that there are no problems with the performance of the facility. Specimens related to future construction activities should be retained. Under no circumstance will soil samples and rock core specimens that may have a bearing on an unsettled claim be disposed of until such claims are finally resolved.

3.5.7 Exploration Survey Requirements

The actual location and elevation of all exploration sites should be surveyed and plotted on the project base map. Once exploration is complete, the actual exploration site should be marked with a survey lath or painted target so that the survey crew can readily measure the intended location. The exploration number should also be marked in the field for accurate reference by the surveyors. Surveys should be completed based on the project coordinates in addition to the **WSG-84** datum. Elevations should be referenced to Mean Sea Level (MSL).

3.6 Subsurface Exploration Methods

3.6.1 General

Many factors influence the applicability and selection of subsurface exploration equipment and methodology for any selected project site investigation. Selection of equipment and methods are usually based entirely on geotechnical data needs and geologic conditions but may also be based on site access, equipment availability, project budget, environmental restrictions, or a combination of any of these.

In many cases, trade-offs between expected results and the exploration method chosen must be evaluated to achieve the needed results within defined time limits and project budget constraints.

Geotechnical designers should be familiar with the exploration methods applied on their projects, and their results and potential limitations or effects on the data they receive from the field.

Most test borings conducted for transportation projects in Oregon are standard diameter vertical borings using rotary or auger drilling methods. Sampling within the boring is typically done by Standard Penetration Tests (SPTs), 3-inch (7.62cm) Undisturbed Shelby Tube samples, HQ3-sized rock coring, and auger coring. Additional, supplementary explorations are conducted using hand augers, direct push (i.e. GeoProbe) rigs, cone penetrometers, and test pits dug either by hand or more commonly with hydraulic excavators. ODOT is currently evaluating and using newer exploration technologies as they are developed or become increasingly available. The use of sonic drilling and geophysical methods are examples.

3.6.2 Test Boring Methods

The most commonly used drilling methods on ODOT projects are auger boring and rotary drilling. Continuous sampling core drilling is employed with both methods. Most modern drill rigs are capable of employing both of these techniques with only minor adjustments to the tooling in the field. Other techniques that are less commonly used are displacement borings using rotosonic or percussion methods. Each drilling method should be selected based on the quality of information obtained in the materials for which the drilling method is best suited for, thus, selection of drilling technique should be carefully considered. Since most test borings penetrate many types of materials, several techniques are commonly employed in any single test boring. Various institutions or individuals have strong preferences for certain types of drilling methods and will tend to use them as a "default" for almost any condition encountered. This behavior should be corrected or avoided. Almost every technique is capable of penetrating the subsurface or "making a hole". The quality of the results is the purpose of subsurface investigation, and different drilling techniques are better suited to certain materials and conditions. Achieving quality results from a drilling program are more important than convenience.

3.6.2.1 Methods Generally Not Used

Cable-tool, wash, jet, and air-rotary methods are generally not used on ODOT projects for many reasons. Cable-tool drilling may be useful for some environmental applications and well installations, but is generally antiquated and not productive for geotechnical investigation. Wash and jet borings cause down-hole disturbance well past the bottom of the boring, and the fluids are difficult to recover making them more of a liability than a source of data. Air-rotary drilling usually causes too much down-hole disturbance to provide reliable SPT data, and difficult to advance in soft soils. Groundwater typically stops further advancement of air-rotary drills, forms large voids, and casts sediment-laden water about the site. Air-rotary drilling may be suited to specific applications where known materials at a site are delineated based on the drill advance rate and obvious changes in the drill cuttings as they are flushed from the hole. In these applications, the air-rotary borings should be supplemental to standard geotechnical exploration borings conducted at the site.

3.6.2.2 Auger Borings

Rotary auger drilling is one of the more rapid and economical methods of advancing exploration borings. Most modern drilling equipment has enough power to turn augers of considerable diameter to a substantial depth. Currently, most augering uses a hollow-stem auger that allows the hole to remain cased while the various sampling or drilling tools are used and withdrawn from the hole with drill rods or wireline retrievers. A central “stinger” bit or plug is placed at the bottom of the auger while the boring is advanced. Solid stem auger use has largely been discontinued due largely to the advent of hollow stem augers and the more powerful equipment that is capable of turning their larger diameter drill string. The standard practice for using hollow-stem augers is described by ASTM D 6151-97. Auger boring has many advantages and disadvantages for various materials encountered as described below.

Auger Boring Advantages

Auger boring has many advantages and disadvantages for various materials encountered. The primary advantages of augers are the preservation of the natural moisture content of the soil and the rapid advancement of the drill through soft to stiff soils. Augers are also useful where drill fluids are difficult to obtain or are an environmental concern, and in freezing conditions where the use of water is problematic. An additional advantage of augers is that they create a large enough hole to install larger-diameter standpipe piezometers or nested piezometers in conformance with Water Resources Department regulations. In addition, the natural piezometric surface is more readily monitored during drilling. Coring tools are also available for auger systems that provide continuous sampling in soils and even weak rock materials. These tools can be placed by either rods or wireline into special auger bits that feed a continuous soil sample into a split barrel that is then retrieved in 2.5 or 5-foot (0.76-1.52m) sampling intervals. Plastic liners that fit in the auger core barrel can also be used to preserve soil cores in their natural moisture conditions.

Auger Boring Advantages and Disadvantages

The disadvantages of augering are the power needed to turn long strings of auger in dense formations, the volume of the hole and the cuttings created, and the disturbance of the natural materials in certain conditions. When hollow-stem augers are used in granular soils below the water table, the hydrostatic pressure differential between the inside and outside of the auger casing will force saturated sands, silts, and fine gravels up into the casing effectively loosening the materials below the auger bit. This can be caused by either the natural differential, or by the pressure induced during retraction of the "stinger" bit or plug. The augers themselves can also affect the conditions of loose granular materials and silts ahead of the bit. In both cases, SPT values obtained will be different than what is true for the natural conditions. To counter this effect, a head of water or other drilling fluid can maintained in the auger casing to counteract these effects. Adding fluids to the auger generally negates their advantages and if such action is necessary, a different drilling technique should be employed. Hollow stem augering should not be employed when assessing liquefaction potential.

A common complaint about augering is the volume of cuttings generated. Where disposal is a concern, this is probably a disadvantage. However, when drilling in an environmentally sensitive area, augering is often preferable because the cuttings are easily contained on site when drilling above the water table. A past complaint has also been the weight of the augers themselves although this has largely been negated by the more powerful equipment and the available wire line systems to assist with moving them around the site.

3.6.2.3 Rotary Drilling

Rotary drilling is the most common, and usually the most versatile drilling method available. Various tools and products available for rotary drilling allow it to be adaptable to most drilling conditions and geologic materials. Rotary boreholes can be uncased holes advanced with a drill bit on rods or cased holes made with a casing, casing advancer and casing shoe. The casing advancer is a driver assembly with latches that fit in the bottom of the casing where it holds the center bit at the bottom of the hole and is subsequently retrieved with a wireline system. This method of drilling involves a relatively fast rotation speed, fluid circulation and variable pressure on the drill bit to penetrate the formation, pulverize the formation particles at the bottom of the borehole. The circulating fluids carry these cuttings away from the bit, up the borehole annulus, and out of the hole.

When the desired sampling depth is reached, the drill rods or casing advancer are retracted from the hole and replaced with the desired sampling tool. The sampling/testing is conducted while the hole is filled with fluid, retrieved from the hole, and then replaced once again with the drilling tool and borehole advancement continues to the next sampling depth. For uncased holes, the drilling fluid is relied upon to stabilize the borehole and prevent it from caving or heaving. In particularly weak or porous formations where drilling fluids are rapidly lost, cased holes are generally used. In uncased holes, the drilling fluid is usually recirculated from a mud tank or pit at the ground surface. Borings that use casing advancers typically use pure water that is not recirculated.

Rotary Drilling Advantages

The advantage of rotary drilling is the relative speed of advancement in deep borings while maintaining borehole stability that best preserves in-situ soil conditions by counteracting soil and pore-water pressures in partially or fully saturated conditions. It is of particular advantage in very soft materials that are very sensitive to disturbance by the drilling equipment. Because of its ability to maintain natural conditions, rotary drilling is usually the best choice when conducting in-situ analysis such as vane shear and pressuremeter testing. The trade-offs for rotary drilling is the introduction of moisture and other minerals that will influence the natural moisture conditions, and the difficulties with installing groundwater monitoring instruments although this later can in some cases be rectified by the use of special drilling fluids and by purging the borehole prior to installation. Special care is needed to contain drilling fluids during exploration, and for ultimate disposal that may involve transport off-site.

Drill Rods

A variety of drilling rods, casings, and drill bits are available for various tasks. Most drilling tools come in standard sizes that are generally adaptable to one another. However, complexities arise when changing from one size to another when various thread sizes and configurations are used. Use the following information relating to drill rods and casing sizing:

- Drill rod and casing sizes are designated from smaller to larger by the letters R, E, A, B, N, and H. Drill rod outside diameters range from $1^{3/32}$ inches (27.8mm) for R-sized rods to 3.5 inches (88.9mm) for H-sized rods.
- Drill casing outside diameter sizes range from $1^{7/16}$ inches (36.5mm) for R-sized casing to 4.5 inches (114.3mm) for H-sized casing. Additional letters such as HW or NWJ designate different thread or coupling configurations.
  Complete tables of drilling tool types, sizes, weights, and volumes are available from the drilling suppliers and manufacturers.
- The important aspects of tool size is that the larger diameter, heavier drill sizes generally provide a more stable hole and allow a greater variety of testing and sampling tools to be used. These larger sizes also help control the eccentric movement of longer drill strings, reduce vibration at the drill bit, and help the driller maintain a straight and plumb boring.

The Diamond Core Drill Manufacturers Association (DCDMA) has standardized the drill rod and casing sizes although any number of other sizes and types remain on the market or are frequently introduced.

Drill Bits

The choice of drill bit greatly influences the test boring quality and speed of completion. Rotary drill bits come in a variety of different types, each suited to a particular soil and/or rock composition. Driller preference is usually what determines what type of bit is used. Experienced drillers can and should normally be relied upon to select the appropriated bit. Certain drill bits are intended for specific geologic materials, but many drillers, through their experience and specific equipment, are able to achieve superb results with bits that are not usually used for that type of material. Follow these guidelines when using drill bits:

- **Soft or loose soils:** Soft or loose soils are usually drilled with drag bits. These bits have two or more wings of either tempered steel or carbide inserts that act as cutting teeth.
- **Hard soils and rock:** Roller bits are used to penetrate hard soils and rock. Roller bits may consist of hardened steel teeth or carbide "buttons". Typically, steel teeth are sufficient for hard soil drilling while carbide button bits are used for bedrock drilling or for drilling in formations with numerous boulders and potential obstructions.

Rotary Drilling Fluids

Various admixtures are available for mixing with the drilling fluids in different applications. Usually, the drilling fluid or "mud" is a mineral solution (usually bentonite and water, thus, a colloidal fluid) with a viscosity and specific gravity that is greater than water. These properties allow the fluid to better stabilize the borehole, cool and lubricate the bit, lift the cuttings out of the hole, and can also increase sample recovery. Various chemical and mineral additives may also be added to the mud mixture for the site-specific conditions. Certain chemical additives, such as pH stabilizers and flocculants, are introduced for common groundwater or mineral conditions that are the source of particular drilling difficulties. Mineral additives, such as barite, may be used to further increase the specific gravity of the mud for unstable boreholes and zones of high artesian pressures. Other additives inhibit corrosion of tools; seal off highly fractured or porous formations to prevent fluid loss, increase the suspension and entrainment of sediments to flush the borehole, and numerous other applications.

Fluids or "mud mixtures" can greatly enhance rotary drilling, and in some very difficult drilling situations, is the only way to complete borings. Mud mixing should be treated with care as improper materials and quantities can actually be detrimental. Volumes and weights should be carefully measured and fluid density and viscosity should be monitored during borehole advancement as these properties will be affected by the formation materials. Several batches may be needed for individual borings depending on the depth of the borehole and other conditions.

The U.S. Bureau of Reclamation and the U.S. Natural Resources Conservation Service have established general guidelines for drilling mud mixtures including amounts of dry materials, volume of water, and fluid densities. ASTM D 4380-84 describes the procedures for determining the density of bentonitic slurries that can be used in rotary drilling.

3.6.2.4 Rock Coring

Rock core sampling is used to obtain a continuous, relatively undisturbed sample of the intact rock mass for evaluation of its geologic and engineering characteristics. When performed appropriately, core drilling produces invaluable subsurface information. Rock coring procedures have generally remained the same since the advent of the technology: a steel tube with a diamond bit rotated into the rock. Advancements in the bits, core barrels for retrieving the samples, and improvements to mechanized equipment overall have greatly enhanced this method.

Note:
Rock core drilling procedures and equipment has largely been standardized by ASTM D2113-99. The Diamond Core Drill Manufacturers Association (DCDMA) has also standardized bit, core barrel, reaming shell, and casing sizes similar to drill rods.

Rock coring almost exclusively involves the use of diamond bits, thus the terms "rock coring" and "diamond drilling" are used interchangeably. Selecting the proper drill bit for the rock coring conditions is essential. Sample recovery and drill production is dependent upon it. The ultimate responsibility for bit selection is the driller's, however, it is important to be familiar with bit types to help determine recovery problems in the field since they may actually be unrelated to the drilling method. The actual configuration of the drill bit is selected based on the actual site conditions. The cross-sectional configuration, kerf, crown, and number of water ports are all determined by the anticipated conditions and characteristics of the rock mass. Consider the following:

- Incorrect bit selection can be extremely detrimental to core recovery, production, and project budget.
- Typically, a surface-set bit consisting of industrial diamonds set in a hardened matrix is used for massive rock bodies.
- Larger and fewer diamonds in the set are used for soft rocks while smaller and more numerous diamonds are used in hard rock. Hard rock bits commonly have a rounded or steeply-angled crown.
- Flat-headed bits are usually for very soft rock. Impregnated bits consist of very fine diamonds in the matrix and are generally used for soft, severely weathered and highly fractured formations. Some carbide blade and button bits are used for soft, sedimentary rocks. These are ideally suited for soft rocks with voluminous cuttings that require a considerable amount bit flushing and cutting extraction.

Core Barrel

The core barrel is the section of the drill string that retains the core specimens and allows them to be retrieved as a whole section. Core barrels may be of different types and sizes, and may consist of numerous components that may be changed depending on the rock mass condition. Core barrels have evolved greatly over time. Single-tube barrels were originally used and required the entire drill string to be retracted to withdraw the sample. These have evolved through double-tube systems of either rigid-types where the inner tube rotates with the outer barrel, or swivel-types where the inner tube remains stationary. Most core barrels used today are triple-tube systems that employ another non-rotating liner to a swivel-mounted double core barrel. This split metal liner retains the sample during extraction that allows minimal sample handling and disturbance prior to measurement and observation. Where desired, a solid, clear plastic tube can be used in place of the split metal tube. Single and even double-tube coring system often require a considerable amount of effort to extract the cores from the barrel that can result in detrimental sample disturbance.

Consider the following:

- Available triple-tube coring systems usually provide specimens that range in diameter from $1^{5/16}$ inches (33.5mm) for “B”-sized core to $3^{9/32}$ inches (83mm) for “P”-sized core.
- Larger core sizes are also available from rather specialized systems.
- A substantial penalty on the quality of rock structural information results from smaller diameter cores. Most rock core taken is “H”-sized ($2^{13/32}$ inches, 61.1mm) in diameter.
- The use of smaller N-sized cores may be necessary in difficult access, or very deep drilling applications.
- The difference in RQD measurements between single, double, and triple tube systems are substantial.

Specialized Methods

These specialized methods are also used:

- **Oriented core barrels:** Orienting core barrels can be used to determine the true attitudes of discontinuities in the rock mass. These specialized core barrels usually scribe a reference mark on the core as it is drilled. Recording devices within the core barrel relate the known azimuth to the reference mark so that the exact orientation of the discontinuities can be determined after the sample has been retrieved.
- **Borehole camera surveys:** Borehole camera surveys are used to determine discontinuity orientations. Several methods for both oriented coring and down-hole surveying have evolved, and highly trained personnel are typically needed to operate them successfully. The 1988 AASHTO Manual is a good source of information on the older core orientation systems while vendors such as the Baker-Hughes Corporation have technical information on the newer magnetic/electronic core alignment systems.

3.6.2.5 Vibratory or Sonic Drilling

Sonic drilling may be called vibratory or rotosonic drilling. This type of drilling is used for continuous sampling in unconsolidated sediments and soft, weathered bedrock. It is best suited for use in oversized unconsolidated deposits enriched with cobbles and boulders such as talus slopes, colluvium, and debris flows or any other formation containing large clasts.

Benefits

- The primary benefit of this method is recovery of oversized materials in a continuous sample, rapid drilling rate, reduced volume of cuttings, and fast monitoring well installation.

- This drilling technique is 8 to 10 times faster than hollow stem augering and produces about 10% of the volume of cuttings.

Drawbacks

- The drawbacks to this method are that it is typically more expensive, and cannot penetrate very far into bedrock.
- The vibration of the drill stem during borehole advancement may disturb the subsurface materials for an unknown distance ahead of the bit, and soft, loose materials can be liquefied during sampling.
- The sample size and speed of extraction will require additional personnel to process, log, and classify in the field.

Sonic drill rigs use hydraulic motors that drive eccentric weights to oscillate the drill head. The oscillation generates a standing sinusoidal wave in the drill stem with a frequency that can be varied depending on the materials encountered. The drill head also rotates the drill stem. An inner and outer casing is advanced so that the hole can be cased at the same time that samples are collected. During drill advancement, the sample is forced into the inner casing from which it is retrieved on a set interval. SPTs and Shelby tube samples can be taken between runs of rotosonic coring.

3.6.2.6 Becker Hammer Drilling

Becker hammer drills are specifically for use in sand, gravel, and boulders. Some Becker hammer drill operators may also have a coring system that can also be run for limited applications. Becker hammer drills use a small diesel-powered pile hammer to drive a special double-walled casing. The casing can be fitted with an array of toothed bits depending on the application. An air compressor forces air through the annulus between the casings to the bottom of the hole where it extracts the materials up through the center of the innermost casing, through a cyclone, and into the sampling bucket. The materials can be extracted on a set interval as the driller engages the air compressor. The Becker drill casings range in size from 5.5-inch (14cm) to 9 inches (23cm) for the outer casing, and 3.3-inch (8.4cm) to 6 inches (15.2cm) respectively for the inner casing. This size of casing allows retrieval of relatively large, unbroken clasts. As the drill is advanced, blow counts are taken along with measurements of the hammer’s bounce chamber pressure. Becker hammer drill data can be correlated to the soil density and strength in coarse-grained soils similarly to the SPT test. In addition, SPTs can be taken through the inner casing of the Becker hammer string.

3.6.2.7 Supplemental Drilling/Exploration Applications

A wide assortment of exploration techniques are available to supplement the subsurface information gathered from test borings at a project site. Typically, any method that can be employed to properly evaluate the subsurface conditions in a supplementary capacity is acceptable on an ODOT project if not constrained by environmental considerations. These methods are usually the most simple and economic to quickly gather subsurface information with minimal cost. In some cases, more extensive and costly methods are required to obtain critical design information. Generally, supplemental investigations consist of simple hand auger borings or backhoe test pits to gather more detailed information and collect additional samples in near-surface or overburden materials.

Hand Tools

Hand augers are available in many forms that allow rapid penetration of near-surface soils and collection of representative samples. Various bits can be used that are suited to general soil conditions that help penetrate and retain samples from certain materials. Extra sections of rods can be added to extend the depth range of these tools. Small engine-powered augers can also be used to increase the depth of penetration and to reduce the physical workload. Most hand augers are of sufficient diameter to permit undisturbed Shelby-tube sampling in the boring where soft soils are encountered. Additional tools such as jacks, cribbing, and extra weights may be needed to retract the tube after sampling. Most field vehicles are equipped with shovels that geotechnical designers can apply to subsurface investigations. Hand-excavated pits can provide essential, detailed information on the near-surface environment.

Various hand probes and penetrometers can be used to make soundings of soft material depths and delineate underground facilities in soft ground conditions. Hand auger borings and hand-excavated test pits are often required for collection of bulk samples.

Cone Penetrometers

Cone penetrometers can be operated from most drill rigs, or they may come as a separate vehicle specially rigged for cone penetration testing. The cone penetration test (CPT) is conducted by pushing an instrumented cylindrical steel probe at a constant rate into the subsurface with some type of hydraulic ram. The cone penetration test is very advantageous in certain (usually soft) soil conditions as it provides a continuous log of stress, pressures, and other measurements without actually drilling a hole. CPTs can be conducted with a transducer to measure penetration pore pressure. Additional instrumentation can be used to measure the propagation of shear waves generated at the surface. Standard cone penetration test procedures are described in ASTM D 3441-98. Electronic CPT testing must be done in accordance with ASTM D 5778**.**

Percussion or Direct push (i.e. GeoProbe®) Borings

Direct push drills are hydraulically-powered, percussion/probing machines originally intended for use in environmental investigations. The direct push method uses the weight of the vehicle combined with percussion to advance the drill string. Drive tools are used to obtain continuous, small-diameter soil cores or discrete samples from specific locations. Direct push drills can obtain continuous samples through the soil column and are capable of penetrating most soils up to about 100 feet (30m). Small-diameter piezometers can also be installed through the direct push tools. Direct push rigs are quick and economical to mobilize and sample the soil column very quickly. Their small diameter and method of penetration produce few if any cuttings that must be disposed of. The percussion advance of the direct push method produces a considerable amount of sample disturbance.

Note:
Direct push advancement rates may provide a relative determination of soil density with respect to material encountered by that particular machine but it is not correlative to SPT data. Direct push rigs are lighter and less powerful than most conventional drill rigs. Thus, they do not have the ability to penetrate certain formations, and because of the effort in doing so, may give a false, overestimation of the formation density.

Test Pits

Backhoe-excavated test pits or trenches are commonly used to provide detailed examination of near surface geologic conditions and to collect bulk samples. Test pits allow examination of larger-scale features that would not be visible in standard borehole samples. Features such as faulting, seepage zones, material contact geometry and others are readily measured in test pit walls. In addition, Torvane and pocket penetrometer tests can be performed in the walls and floor of the test pit. In-place percolation testing can also be carried out in test pits. Test pits have the advantage of the shear bulk of materials that can be observed. In this regard, the overall composition of the materials in a unit are better assessed by the many cubic feet of material excavated and observed opposed to the relatively minute amount of material contained in a split spoon sampler.

Warning:
Under no circumstances will personnel enter a test pit deeper than 4 feet (1.2m) below the ground surface unless the appropriate shoring and bracing is used. If any evidence of instability or seepage is evident in the test pit walls, no entry will be permitted until shoring is complete. Test pits must be filled in as soon as they are completed to prevent passersby from entering or falling in. When a test pit is used for percolation tests or for assessment of trench stability, appropriate barricades and signs must be placed around the site to prevent accidental entry.

ODEX or Air-Track Drilling

Percussive air drilling is typically used in a similar manner to other probing systems with the exception that air-drill holes are used to probe harder materials. A relative rate of advancement coupled with the cuttings retrieved in certain intervals allows basic interpretation of subsurface conditions. ODEX systems using an outer casing allow installation of instruments below the water table that would otherwise be impossible to install with other air-driven equipment. The advantage of this method is the speed of installation and borehole advancement. As previously described, air drilling system are not suited for standard testing methods due to the unknown amount of down-hole disturbance.

3.6.3 Alternative Exploration Methods and Geophysical Surveys

Alternatives to drilling and test pit excavations characteristically involve the use of geophysical methods. For ODOT projects, geophysical survey results are always supplemental to direct observation of subsurface conditions by borings and test pits and should never be considered as a replacement.

Geophysical surveys play an important role in engineering geology and geotechnical engineering however they do not provide all of the information needed for the development of geotechnical design parameters.

Note:
From a liability and construction claims standpoint, direct observation, sampling, and testing are critical. Direct observation and measurement will assure that subsurface conditions not measured by geophysical survey methods are revealed and further support or refute the results of geophysical surveys.

Most of the data obtained from a geophysical survey require an experienced and highly-trained geophysicist to interpret and process before it is of any use to an engineering geologist or geotechnical engineer. Geophysicists can base their interpretation on direct calculations, tabulations, or regression analyses, or they may base it wholly upon their own experience. Any geophysical method used has its own aspects that can result in serious misinterpretation or inappropriate use of the results. Prior knowledge of the actual site conditions and the possible errors of the survey technique are needed to calibrate, or fit the data to the known baseline data.

Geophysical survey results and resolution of the data is dependent upon the density of measurement points, and frequency of measurements. These variables may be set according to the overall project needs and level of detail required. Modern geophysical instruments are sensitive enough to produce measurements at the levels needed for geotechnical investigations. Methods most frequently used are:

- Seismic methods are the most commonly conducted techniques for engineering geologic investigations.

- Seismic refraction provides the most basic geologic data by using the simplest procedures, and commonly available equipment. The data provided is the most readily interpreted and correlated to other known material properties.

3.7 Geotechnical Instrumentation

3.7.1 General – Instrumentation and Monitoring

Of equal importance to site characterization and exploration as sampling and testing data is the information provided by geotechnical instrumentation and monitoring. Sampling and testing of materials provides needed design information concerning the existing site conditions at the time of investigation. Information regarding certain site conditions as they change through time due to the effects of natural variations in the earth's surface and atmosphere or the effects of human activities, such as construction, can be provided by the appropriate selection, installation, and monitoring of geotechnical instruments. Most geotechnical instruments are used to monitor the performance of structures and earthworks during construction and operation of the facility. Some instrumentation programs are planned to provide actual design criteria such as landslide depths of movement and piezometric surfaces. Other programs are intended to verify design assumptions. In any case, considerable design and planning efforts are needed to derive the needed results. Geotechnical instrumentation has become much more "user-friendly" as technologies have developed, but an all-inclusive process beginning with a determination of the instrumentation project objectives that are carried through to completion and use of the data.

3.7.2 Purposes of Geotechnical Instrumentation

A rule of thumb for geotechnical instrumentation programs is: "every instrument installed should be selected and placed to assist in answering a specific question". The point of this rule is to start a geotechnical instrumentation program on the correct course of study to acquire the necessary results with the greatest efficiency. Instruments can have an initially high installation cost, but the time and effort for reading them and making sense of the results is where the most costly inefficiencies occur. Any instrument installed will provide some information; whether or not it is relevant to the immediate project requirements is the issue. Therefore, efforts must be concentrated on the primary questions to gather the most important data from the instrumentation program without time lost to the analysis of extraneous data.

3.7.2.1 Site Investigation and Exploration

Instruments are regularly used to characterize the initial site conditions during the design phase of a project. Landslide remediation projects rely on instruments to determine depths and rates of movement as well as pore water pressures to provide basic information for stability analysis and mitigation design.

Most project sites require some information concerning the actual depth and seasonal fluctuation of groundwater that not only affects the project design, but also its constructability.

3.7.2.2 Design Verification

Instruments are frequently used to verify design assumptions and to check that facility performance is as expected. Instrument data gathered early in a project can be used to modify the design in later phases. Geotechnical instruments are also an inherent part of proof testing to verify design adequacy.

3.7.2.3 Construction and Quality Control

Geotechnical instruments are commonly used to monitor the effects of construction. Construction procedures and schedules can be modified based on actual behavior of the project features for ensuring safety as well as gaining efficiency in the actual construction as determinations can be made regarding how fast construction can proceed without the risk of failure or unacceptable deflections. Instruments can be used to monitor contractor performance to assure that contract requirements and specifications are being met.

3.7.2.4 Safety and Legal Protection

Instruments can be used to provide early warning of impending failures allowing time to isolate the problems and begin implementation of remedial actions. Instrument data provides crucial evidence for legal defense of the agency should owners of adjacent properties claim that construction or operations have caused damage.

3.7.2.5 Performance

Instruments are used for the short and long-term service performance of various facilities. Deformation, slope movement, and piezometric surface measurements in landslides can be used to evaluate the performance of drainage systems installed to stabilize the landslide. Loads on rock bolts and tiebacks may be monitored to assess their long-term performance or evaluate the need for additional supports.

3.7.3 Criteria for Selecting Instruments

For each project, the critical parameters must be identified by the designer that will require instrumentation to determine. The appropriate instruments should then be selected to measure them based on the required range, resolution, and precision of measurements. The ground conditions are another consideration in the choice of instruments. Use the following to help select instruments:

- **Landslides:** Relatively fast-moving landslides may require a larger-diameter inclinometer pipe or TDR cable to determine the zone of slide movement, or Vibrating Wire piezometers may be selected to measure groundwater in low permeability soils where a standpipe would require a large volume of water to flow into it before even small changes in pore-water pressure can be detected.
- **Temperature and humidity:** Temperature and humidity also affect the choice of instruments. Certain instruments may be difficult to use in freezing conditions while warm and humid environments may affect the reliability of electronic instruments unless particular care is taken to isolate their environment.
- **Number of parameters:** The number of parameters to measure is also important for instrument selection since soil and rock masses typically have more than one property that dictates their behavior. Some parameters correlate with one another, and instruments that obtain complementary measurements provide an efficiency gain. In areas with complex problems, several parameters can be measured, and a number of correlations can be found from instrumentation data leading to a better understanding of the site conditions. Strain gages and load cells on a retaining wall and inclinometers behind it are examples where complementary data can be obtained. When relationships can be developed with the data, further data can be obtained even when one set of instruments fail.

- **Instrument performance and reliability:** Instrument performance and reliability are also important considerations. The cost of an instrument generally increases with higher resolution, accuracy, and precision in the instrument. Also, the range of measurements obtained can be reduced by higher-functioning instruments, so the geotechnical designer should have a clear understanding of the scale and level of measurements to be taken.

  Example: An example is the placement of a vibrating wire transducer in a borehole to measure an unknown piezometric surface. The instrument selected would have a wide range of testing, but a lower resolution of values that could be read. Where the piezometric surface is known within a narrower range and small changes are of significance to the design, an instrument capable of reading a smaller range of values but at a higher resolution within the known range.

- **Quality of the instrument:** There are some instances where the use of lower-quality instruments is warranted, but in general, choosing a lower-quality instrument to save on initial costs is a false economy. The difference in cost between a high-quality instrument and a lower-quality instrument is low with respect to the overall cost of installing and monitoring an instrument.

- **Cost:** The cost of drilling a hole and the labor of installing the instrument is usually an order of magnitude higher than the cost of the instrument. The less easily quantifiable loss of data from a failed instrument in terms of monetary cost should also be considered. It is expensive and often impossible to replace failed instruments. Furthermore, essential baseline data is also lost that cannot be replaced.

3.7.3.1 Automatic Data Acquisition Systems (ADAS)

Automatic Data Acquisition Systems (ADAS) can provide significant advantages to a geotechnical instrumentation program. They can provide numerous readings at set and reliable intervals, and they can store and transmit data from remote or difficult access locations. ADAS are necessary for real-time instrument monitoring and relay. They are beneficial at sites where many sensors are present that would require copious staff time to read manually or for large-scale proof tests with many concurrently-read instruments to be monitored throughout the test.

Automatic Data Acquisition Systems come in many forms ranging from the very simple, user-friendly devices to systems requiring significant programming and electronics to install and run. Project requirements usually dictate what system is selected, but the simplest, most inexpensive, and easiest to connect to the chosen instruments are best. Follow these guidelines:

- Simple dataloggers connected to individual instruments that are retrieved and downloaded periodically are sufficient for most projects.
- Large, complex problems may require a more intelligent system that can be programmed to change monitoring routines in response to site or environmental changes.
- Most instrumentation companies also have companion dataloggers to go with their products while several independent companies also manufacture easy-to-use dataloggers. Other companies, such as Campbell Scientific Incorporated, produce more complex systems that can read multiple installations of different types of instruments as well as store and transmit data.

- In addition to the data collection devices, these firms also produce software for processing and displaying the data. The software is another consideration if export to other systems is desired. Compatibility between programs can create problems and errors in the end product of an instrumentation project.

3.7.3.2 Instrument Use and Installation

Instruments have been developed to monitor many specific geologic conditions and engineering parameters. In many cases, a single instrument can be used or adapted for use on other applications. For this, the manufacturer and other professionals should be consulted to assure that the results obtained are valid, or, they may have insights and case histories that are of use for the situation. The manufacturer's literature, installation procedures, and other guidance documents should be followed for proper installation of their products as procedures can vary for different manufacturers same instrument products. Detailed discussions of instrument installation and initialization procedures, function, and operation can be found in manufacturer's documents such as Slope Indicator Company (SINCO) Applications Guide or in published literature such as Dunnicliff (1988).

3.7.3.3 Inclinometers

Inclinometers are used on transportation projects mainly to detect and monitor lateral earth movements in landslides and embankments. They are also used to monitor deflections in laterally loaded piles and retaining walls. Horizontally installed inclinometers can also be used to monitor settlement. Inclinometer systems are composed of:

- grooved casing installed in a borehole, embedded in a fill or concrete, or attached to structures,
- probe and cable for taking measurements at set intervals in the casing, and
- a digital readout unit and/or data storage device.

The installed casing is for single installation use, and the probe, cable and data storage unit are used for almost all installations.

Note:
It is important to use the same probe for each reading in any particular installation since each probe must be independently calibrated.

Inclinometers are manually read by a trained technician on a set schedule or in response to environmental changes such as increased rainfall in the area or observation of surficial signs of slope movement. In-place inclinometers spanning known or highly suspected zones of movement can be installed for continuous, automatic monitoring. These usually remain in the hole permanently if significant slope movement occurs.

- Inclinometer casing installation is essential to successful performance of the instrument. Shortcuts taken during installation will frequently result in poor performance of the instrument or render it completely useless.
- Inclinometers should be installed according to the procedures described in the SINCO Applications guide with the exception of the grout valve.
- Borings should be initially drilled or later reamed to a sufficient diameter that will accommodate the inclinometer casing and an attached tremie tube.

- The tremie tube should be attached to the inclinometer casing approximately 6 inches above the bottom and along the casing at a close enough interval to prevent it from getting tangled or constricted in the borehole.
- One of the four grooves in the inclinometer casing should be aligned to the direction of slide movement as the casing is assembled and lowered into the hole to prevent spiraling.
- If the borehole walls are unstable, the drill casing may need to remain in the borehole, and withdrawn as the grout level rises. Generally, the grout should be maintained at a visible level in the casing as the drill string is withdrawn.

Initial readings should be taken as soon as the grout has sufficiently set up. This is usually 3 to 5 days after grouting. During installation, some grout is naturally lost to fractures and voids in the formation. This may occur to the extent that additional grouting is required. Usually, this only entails topping off the hole with a small batch of grout to stabilize the uppermost portion of the casing. In more severe cases, the grout pump may be reconnected to the tremie tube to re-grout the remaining voids.

3.7.3.4 Piezometers

Piezometers used to measure pore-water pressure and groundwater levels can range from simple standpipes to complex electronic devices or pneumatic systems. Piezometers are typically installed in selected layers to measure the piezometric pressures in that layer. The layout and target depths of piezometer installation are determined by actual site conditions and project requirements.

Note:
*All piezometers must be installed according to **Oregon Water Resources Department** regulations defined by **ORS 690.240** and **ORS 537.747** through **ORS 737.799 (appropriation of water generally)**. Specifications for a properly operating instrument are usually more stringent than these rules apart from the requirements for abandonment.*

The various types of piezometers are generally used for different applications as described below.

- Standpipe piezometers are general-purpose instrument for monitoring piezometric water levels and are best-suited for granular materials. Standpipe piezometers require a water level indicator to obtain readings.
- Vibrating Wire piezometers utilize a pressure transducer to convert water pressure to a frequency signal that is read by an electronic device. Vibrating Wire piezometers can be automated by electronic systems.
- Pneumatic piezometers are typically used to measure pore water pressure in saturated conditions. Both Pneumatic and vibrating wire piezometers are used for all soil types and are better suited to fine-grained soils than the standpipe variety due to the response time and volume of water needed to record changes in water level in that type.

Piezometers should be placed at the desired sensing zone in a porous medium and sealed with the appropriate materials above and below this zone to assure measurement of the piezometric pressure in the desired location. Porous mediums or filter packs should be composed of pre-screened commercial-grade silica sand. All piezometers should be installed and initialized according to their manufacturer's specifications.

3.7.3.5 Other Instruments

A vast array of geotechnical instruments is available for most applications. Strain gauges, extensometers and load cells of all types and configurations for structural as well as geotechnical applications are obtainable from numerous vendors. Most vendors have prescribed applications as well as installation and monitoring procedures that should be followed when using their products on transportation projects. Professional knowledge, experience, and judgment must be applied to the use of all instruments to assure appropriate use of these instruments and the adequacy of data obtained.

3.8 Environmental Protection during Exploration

Compliance with all State, Federal, and Local ordinances, laws and regulations concerning environmental protection at all work locations is **mandatory** for any activity that may disturb the ground surface or vegetation. All environmental permits, clearances, or any other documentation needed for compliance with the pertinent environmental regulations must be ready prior to mobilization of exploration equipment.

The ODOT Programmatic Biological Opinion for Drilling, Surveying, and Hydraulic Engineering Activities may be applicable for some sites. This document can be referenced on the ODOT Geo-Environmental web page.

Note:
Every precaution necessary to minimize environmental impacts during site investigation must be taken, and every effort made to restore the site to its original condition. All drilling fluids and cuttings must be disposed of safely and legally. In no circumstance should sediment-laden water or other pollutants be allowed to enter streams or other bodies of water. In the event where there is a potential for pollutants to contaminate such, all operations will be suspended until the situation can be rectified. Violation of Federal, State, and Local environmental protection laws can result in personal penalties, including arrest and incarceration.

3.8.1 Protection of Fish, Wildlife, and Vegetation

Compliance with the Laws of the Oregon Department of Fish and Wildlife**,** National Marine Fisheries Service**,** United States Fish and Wildlife Service, and the rules and practices developed through the Oregon Plan for Salmon and Watersheds is also **mandatory**. All subsurface investigation activities shall be conducted to avoid any hazard to the safety and propagation of fish and shellfish in the waters of the State.

Unless specifically authorized by the State and by permit, the Contractor shall not:

- Use water jetting
- Release petroleum or other chemicals into the water, or where they may eventually enter the water
- Disturb spawning beds or other wildlife habitat
- Obstruct streams
- Cause silting or sedimentation of water
- Use chemically treated timbers or platforms
- Impede fish passage

The permitted work area boundaries will be defined by the permit for the project from the regulatory agencies.

3.8.2 Forestry Protection

All necessary permits must be obtained prior to exploration in accordance with ORS 477.625 and ORS 527.670, and comply with the laws of any authority having jurisdiction for protection of forests. At certain times of the year, the exploration activities will be subject to IFPL constraints, and operational schedules must be adjusted accordingly. Fire-suppression equipment may be required on site as well as a designated fire watch.

3.8.3 Wetland Protection

All operations shall comply with the Clean Water Act Section 404 (33 U.S.C. 1344)**;** Federal Rivers and Harbors Act of 1899, Section 10 (33 U.S.C. 403 et seq.)**;** Oregon Removal-Fill law (ORS 196.800 - 196.990)**;** Oregon Removal and Filling in Scenic Waterways law (ORS 390.805 - 390.925), and other applicable Laws governing preservation of wetland resources.

Note:
The terms "wetland", or "wetlands" are defined as "Areas that are inundated or saturated by surface or groundwater at a frequency and duration sufficient to support, and that under normal circumstance do support, vegetation typically adapted for life in saturated Soil conditions. Wetlands generally include swamps, marshes, bogs, and similar areas". Wetlands also include all other jurisdictional waters of the U.S. and/or the State.

If wetlands are known to be on the project site, they should be delineated by the region's wetland specialist or their contractor to prevent accidental entry by the exploration operation. Wetlands to be temporarily impacted should also be identified at this time. Wetlands to be protected will be considered as "no work zones".

Subsurface exploration operations must also comply with Clean Water Act Section 404 permits issued by the U.S. Army Corps of Engineers, and Fill/Removal permits issued by DSL. These permits allow specified quantities of fill and excavation, including soil and rock samples within specifically identified areas of wetlands.

3.8.4 Cultural Resources Protection

The exploration crew is also required to comply with all Laws governing preservation of cultural resources. Cultural resources may include, but are not limited to, dwellings, bridges, trails, fossils, and artifacts. Known locations of cultural resources will be considered as "no work zones".

If cultural resources are encountered in the project area, and their disposition is not addressed in the contract, the exploration crew shall:

- Immediately cease operations or move to another area of the project site
- Protect the cultural resource from disturbance or damage
- Notify the region's cultural resource specialist

The region's cultural resource specialist will:

- Arrange for immediate investigation
- Arrange for disposition of the cultural resources
- Notify the exploration crew when to begin or resume operations in the affected area

3.9 REFERENCES

AASHTO, 2007, *LRFD Bridge Design Specifications*, American Association of State Transportation and Highway Officials, 17th Edition (with current Interims), Washington, D.C., USA.

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C.H. Dowding, Ed., *Site Characterization & Exploration, ASCE Specialty Workshop Proceedings*, Northwestern University, 1978.

Dunnicliff, John 1988. *Geotechnical Instrumentation For Monitoring Field Performance*, John Wiley & Sons, New York.

U.S. Department of Transportation, *Federal Highway Administration Evaluation of Soil and Rock Properties, Geotechnical Engineering Circular No. 5, FHWA-IF-02-034*, April, 2002.

U.S. Department of Transportation, *Federal Highway Administration Subsurface Investigation Participant's Manual, Publication No. FHWA HI-97-201*, November, 1997.

U.S. Department of Transportation, Federal Highway Administration *Subsurface Investigations - Geotechnical Site Characterization Reference Manual*, Publication No. FHWA NHI-01-031, May, 2002.

Turner, Keith A., and Schuster, Robert L., Eds., *LANDSLIDES Investigation and Mitigation, Transportation Research Board Special Report 247*, 1996, Pages 140-163.

Geophysical Exploration for Engineering and Environmental Investigations, U.S. Army Corps of Engineers Engineering and Design Manual, EM 1110-1-1802, August 1995**.**

Geotechnical Investigations, U.S. Army Corps of Engineers Engineering and Design Manual, EM 1110-1-1804, January 2001.

U.S. Department of the Interior, Bureau of Reclamation, 1994, *Engineering Geology Field Manual*.

References are made to various ASTM standards. The ASTM International standards located at www.astm.org (the "ASTM Site").

3.10 Appendix 3-A Permit of Entry Form

Oregon Department of Transportation
RIGHT-OF-ENTRY for EXPLORATION
REGION 3 GEOLOGY
Phone: (541) 957-3602 FAX: (541) 957-3604
3500 NW Stewart Parkway
Roseburg, OR 97470

(1) (We) ______________ and __________________________ hereinafter referred to as "grantor", do hereby grant to the STATE OF OREGON, by and through the Oregon Department of Transportation, and its officers, agents, and employees, the right and license to go upon the following described real property to drill or to gain access to highway Right-of -Way for exploration core drilling at:

Township 37 South, Range 2 West, Section 28
77 Hanley Road
Central Point, Oregon 97502

Property Description:

D-89-16328
37-2W-28 TL 800

IT IS UNDERSTOOD AND AGREED: That this right and license shall be valid until all exploration is completed unless revoked by grantor before completion. It is further understood that the Oregon Department of Transportation shall, to the extent permitted by Oregon law, be responsible for any unnecessary damage done, in connection with said exploration, this will include any crops or other improvements on said property.

Grantor hereby represents and warrants that He/She is the owner of said property or otherwise has the right to grant this permit of entry.

Date____________ Day________, 2003

Permission Acquired by: _____________________

Signature:__________________________________

Title: Project Geologist

Owner(s)
Signature(s):____

3.11 Appendix 3-B Utility Notification Worksheet

Memo to File

UTILITY LOCATE DATA SHEET
Region Geology Unit
Oregon Department of Transportation

Project Name:
Highway and Mile Point:

Utility Locate Called By:

Locators Called (When):

Required Information	
Caller ID #:	
Type of Work:	
County/City	
Highway:	
Mile Point:	
Township/Range/ Quarter Section:	
Distance from Nearest Cross Street:	
Overhead Lines:	
Special Markings:	
Date to Be Located:	
Ticket#:	
Name of Person Called:	
Utilities Notified:	

Utilities Field Marked:	
Gas	
Electric	
Sewer	
Water	
Telephone	
Cable Television	
Irrigation	
Signals/Illumination	
Other	

Chapter

4

4 Soil and Rock Classification and Logging

4.1 General

The ODOT Soil and Rock Classification Manual (1987) should be used for the description and classification of all soil and rock materials. This manual is available on the Geo-Environmental web page at the following address:

Soil_Rock_Classification_Manual.pdf on ftp.odot.state.or.us

Chapter

5

5 Engineering Properties of Soil and Rock

5.1 General

The purpose of this chapter is to identify appropriate methods of soil and rock property assessment and describe how to use soil and rock property data to establish engineering parameters for geotechnical design. Soil and rock design parameters should be based on the results of a geotechnical investigation which includes in-situ field testing and a laboratory testing program, used separately or in combination. The geotechnical designer's responsibility is to determine which parameters are critical to the design of the project and then determine the parameters to an acceptable level of accuracy. See Chapter 2 and the individual chapters that cover each geotechnical design element area for further information on how to plan and obtain soil and rock parameters.

The detailed measurement and interpretation of soil and rock properties should be consistent with the guidelines provided in *Sabatini, et al, April, 2002, U.S. Department of Transportation, Federal Highway Administration Evaluation of Soil and Rock Properties, Geotechnical Engineering Circular No. 5, FHWA-IF-02-034.*

The focus of geotechnical design property assessment and final selection should be on the individual geologic strata identified at the project site. A geologic stratum is characterized as having the same geologic depositional history and stress history, and generally has similarities throughout the stratum in terms of density, source material, stress history, and hydrogeology. It should be recognized that the properties of a given geologic stratum at a project site are likely to vary significantly from point to point within the stratum. In some cases, a measured property value may be closer in magnitude to the measured property value in an adjacent geologic stratum than to the measured properties at another point within the same stratum. However, soil and rock properties for design should not be averaged across multiple strata. It should also be recognized that some properties (e.g., undrained shear strength in normally consolidated clays) may vary as a predictable function of a stratum dimension (e.g., depth below the top of the stratum). Where the property within the stratum varies in this manner, the design parameters should be developed taking this variation into account, which may result in multiple values of the property within the stratum as a function of a stratum dimension such as depth.

5.2 Influence of Existing and Future Conditions on Soil and Rock Properties

Many soil properties used for design are not intrinsic to the soil type, but vary depending on conditions. In-situ stresses, the presence of water, rate and direction of loading can all affect the behavior of soils. Prior to evaluating the properties of a given soil, it is important to determine the existing conditions as well as how conditions may change over the life of the project. Future construction, such as new embankments, may place new surcharge loads on the soil profile or the groundwater table could be raised or lowered. Often it is necessary to determine how subsurface conditions or even the materials themselves will change over the design life of the project. Normally, consolidated clays can gain strength with increases in effective stress and over-consolidated clays may lose strength with time when exposed in cuts. Some construction materials such as weak rock may loose strength due to weathering within the design life of the embankment.

5.3 Methods of Determining Soil and Rock Properties

Subsurface soil or rock properties are generally determined using one or more of the following methods:

- In-situ testing during the field exploration program,
- Laboratory testing, and
- Back analysis based on site performance data.

The two most common in-situ test methods for use in soil are the Standard Penetration Test, (SPT) and the Cone Penetrometer Test (CPT). Other in-situ tests, such as pressuremeter and vane shear are used less frequently, but are important tests in specific instances. In-situ tests for rock are sometimes performed for the design of major structures but generally are not common for highway applications.

The laboratory soil and rock testing program generally consists of index tests to obtain general information or to use with correlations to estimate design properties, and performance tests to directly measure specific engineering properties. A wide array of index and performance tests for soil, rock and groundwater measurement are discussed in Sabatini, et al, April, 2002, U.S. Department of Transportation, *Federal Highway Administration Evaluation of Soil and Rock Properties, Geotechnical Engineering Circular No. 5, FHWA-IF-02-034*.

The observational method, or use of back analysis, to determine engineering properties of soil or rock is often used with slope failures, embankment settlement or excessive settlement of existing structures.

- **Landslides or slope failures:** With landslides or slope failures, the process generally starts with determining the geometry of the failure and then determining the soil/rock parameters or subsurface conditions that cause the safety factor to approach 1.0. Often the determination of the back-calculated properties is aided by correlations with index tests or experience on other projects.
- **Embankment settlement:** For embankment settlement, a range of soil properties is generally determined based on laboratory performance testing on undisturbed samples. Monitoring of fill settlement and pore pressure in the soil during construction allows the soil properties and prediction of the rate of future settlement to be refined.

- **Structure settlement:** For structures such as bridges that experience unacceptable settlement or retaining walls that have excessive deflection, the engineering properties of the soils can sometimes be determined if the magnitudes of the loads are known. As with slope stability analysis, the geometry of the subsurface soil must be adequately known, including the history of the groundwater level at the site.

5.4 In-Situ Field Testing

Standards and details regarding field tests such as the Standard Penetration Test (SPT), the Cone Penetrometer Test (CPT), the vane shear test, and other tests and their applications in geotechnical design are provided in Sabatini, et al. (2002). Standards for sampling and testing of materials are in general accordance with ASTM (www.astm.org).

In general, correlations between N-values and soil properties should only be used for cohesionless soils and sand, in particular. Caution should be used when using N-values obtained in gravelly soil. Gravel particles can plug the sampler, resulting in higher blow counts and estimates of friction angles than actually exist. Caution should also be used when using N-values to determine silt or clay parameters due to the dynamic nature of the test and resulting rapid changes in pore pressures and disturbance within the deposit. Correlations of N-values with cohesive soil properties should generally be considered as preliminary. N-values can also be used for liquefaction analysis. See Chapter 6 for more information regarding the use of N-values for liquefaction analysis.

A discussion of field measurement of permeability is presented in Sabatini, et al. (2002), and ASTM D 4043 presents a guide for the selection of various field methods.

Note:

If in-situ test methods are utilized to determine hydraulic conductivity, one or more of the following methods should be used:

- Well pumping tests
- Packer permeability tests
- Seepage Tests
- Slug tests
- Piezocone tests

5.4.1 Correction of Field SPT Values

The N-values obtained are dependent on the equipment used and the skill of the operator, and should be corrected to standard N60 values (an efficiency of 60 percent is typical for rope and cathead systems). This correction is necessary because many of the correlations developed to determine soil properties are based on N60-values. SPT N-values should be corrected for hammer efficiency in accordance with section 4.4.3 of Sabatini, et al. (2002).

ODOT requires that all hammers have an energy measurement performed at the time of drilling of a boring or that the hammer efficiency of each hammer be supplied with the boring log. Caution must be used when noting N-value correlations and the notation "N(uncorr)" or "N'(60)" must be indicated.

The following values for energy ratios (ER) may be assumed if hammer specific data are not available:

- ER = 60% for conventional drop hammer using rope and cathead

- ER = 80% for automatic trip hammer

Hammer efficiency (ER) for specific hammer systems used in local practice may be used in lieu of the values provided. If used, specific hammer system efficiencies shall be developed in general accordance with ASTM D-4945 for dynamic analysis of driven piles or another accepted procedure.

Corrections for rod length, hole size, and use of a liner may also be made, if appropriate. In general, these are only significant in unusual cases or where there is significant variation from standard procedures. These corrections may be significant for evaluation of liquefaction. Information on these additional corrections may be found in: "*Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils"; Publication Number: MCEER-97-0022*; T.L. Youd, I.M. Idriss (1997) and in "Cetin, K., Seed, R., et al.

N-values are also affected by overburden pressure, and in general should be corrected for that effect, if applicable to the design method or correlation being used. Corrections for overburden pressure are normally only applied to cohesionless soils where the resulting N-values will be used in correlation with the angle of internal friction or liquefaction analysis or to obtain other design parameters. N-values corrected for both overburden and the efficiency of the field procedures used shall be designated as (N1)60 as stated in Sabatini, et al. (2002).

5.5 Laboratory Testing of Soil and Rock

Laboratory testing is a fundamental element of a geotechnical investigation. The ultimate purpose of laboratory testing is to measure physical soil and rock properties utilizing standard repeatable procedures. Laboratory test data is also used to refine the visual observations and field testing data from the subsurface field exploration program, and to determine how the soil or rock will behave under the proposed loading conditions. The ideal laboratory program will provide sufficient data to complete an economical design without incurring excessive tests and costs. Depending on the project issues, testing may range from simple soil classification testing to complex strength and deformation testing. Details regarding specific types of laboratory tests and their use are provided in Sabatini, et al. (2002).

5.5.1 Quality Control for Laboratory Testing

Improper storage, transportation and handling of samples can significantly alter the material properties and result in misleading test results. The requirements provided in *FHWA-HI-97-021,Subsurface Investigations, NHI course manual #132031*, Mayne, et al., (1997) for these issues and laboratory testing of soils should be followed. Laboratories conducting geotechnical testing shall be either AASHTO accredited or fulfill the requirements of AASHTO R18 for qualifying testers and calibrating/verifications of testing equipment for those tests being performed.

5.5.2 Developing the Testing Plan

The amount of laboratory testing required for a project will vary depending on availability of preexisting data, the character of the soils and the requirements of the project. Laboratory tests should be selected to provide the desired and necessary data as economically as possible. Geotechnical information requirements are provided in Sabatini, et al. (2002) that address design of geotechnical features. Laboratory testing should be performed on both representative and critical test specimens obtained from geologic layers across the site. Critical areas correspond to locations where the results of the laboratory tests could result in a significant change in the proposed design. In general, a few carefully conducted tests on samples selected to cover the range of soil properties with the results correlated by classification and index tests is the most efficient use of resources. The following should be considered when developing a testing program:

- Project type (bridge, embankment, rehabilitation, buildings, etc.)
- Size of the project
- Loads to be imposed on the foundation soils
- Types of loads (i.e., static, dynamic, etc.)
- Whether long-term conditions or short-term conditions are in view
- Critical tolerances for the project (e.g., settlement limitations)
- Vertical and horizontal variations in the soil profile as determined from boring logs and visual identification of soil types in the laboratory
- Known or suspected peculiarities of soils at the project location (i.e., swelling soils, collapsible soils, organics, etc.)
- Presence of visually observed intrusions, slickensides, fissures, concretions, etc in sample – how will it affect results
- Project schedules and budgets

5.6 Engineering Properties of Soil

5.6.1 Laboratory Performance Testing

Laboratory performance testing of soil is mainly used to estimate strength, compressibility, and permeability characteristics. Shear strength may be determined on either undisturbed specimens of fine- grained soil (undisturbed specimens of granular soils are very difficult, if not impossible, to obtain), or disturbed or remolded specimens of fine or coarse grained soil. There are a variety of shear strength tests that can be conducted, and the specific type of test selected depends on the specific application. See Sabatini, et al. (2002) for specific guidance on the types of shear strength tests needed for various applications.

5.6.1.1 Disturbed Shear Strength Testing

Disturbed soil shear strength testing is less commonly performed, and is primarily used as supplementary information when performing back-analysis of existing slopes, or for fill material and construction quality assurance when minimum shear strength is required. It is difficult to obtain accurate shear strength values through shear strength testing of disturbed (remolded) specimens since the in-situ density and soil structure is quite difficult to accurately recreate, especially considering the specific in-situ density may not be known. The accuracy of this technique in this case must be recognized when interpreting the results. However, for estimating the shear strength of compacted backfill, more accurate results can be obtained, since the soil placement method, as well as the in-situ density and moisture content, can be recreated in the laboratory with some degree of confidence. The key in the latter case is the specimen size allowed by the testing device, as in many cases, compacted fills have a significant percentage of gravel sized particles, requiring fairly large test specimens (i.e., minimum 3 to 4 inch diameter, or narrowest dimension specimens of 3 to 4 inches).

Typically, a disturbed sample of the granular backfill material (or native material in the case of obtaining supplementary information for back-analysis of existing slopes) is sieved to remove particles that are too large for the testing device and test standard, and is compacted into a mold to simulate the final density and moisture condition of the material. The specimens may or may not be saturated after compacting them and placing them in the shear testing device, depending on the condition that is to be simulated. In general, a drained test is conducted, or if it is saturated, the pore pressure during shearing can be measured (possible for triaxial testing; generally not possible for direct shear testing) to obtained drained shear strength parameters. Otherwise, the test is run slow enough to be assured that the specimen is fully drained during shearing (note that estimating the testing rate to assure drainage can be difficult). Multiple specimens tested using at least three confining pressures should be tested to obtain a shear strength envelope. See Sabatini, et al. (2002) for additional details.

5.6.1.2 Other Laboratory Tests

Tests to evaluate compressibility or permeability of existing subsurface deposits must be conducted on undisturbed specimens, and sample disturbance must be kept to a minimum. See Sabatini, et al. (2002) for additional requirements regarding these and other types of laboratory performance tests that should be followed.

5.6.2 Correlations to Estimate Engineering Properties of Soil

Correlations that relate in-situ index test results such as the SPT or CPT or laboratory soil index testing may be used in lieu of, or in conjunction with, performance laboratory testing and back-analysis of site performance data to estimate input parameters for the design of the geotechnical elements of a project. Since properties estimated from correlations tend to have greater variability than measurement using laboratory performance data (see Phoon, et al., 1995), properties estimated from correlation to in-situ field index testing or laboratory index testing should be based on multiple measurements within each significant geologic unit (if the geologic unit is large enough to obtain multiple measurements). A minimum of 3 to 5 measurements should be obtained from each geologic unit as the basis for estimating design properties.

The drained friction angle of granular deposits should be determined based on the correlation provided in Table 5-1.

Table 5-1. Correlation of SPT N values to drained friction angle of granular soils (modified after Bowles, 1977)

N1(60) from SPT (blows/ft)	Φ' (deg)
<4	25-30
4	27-32
10	30-35
30	35-40
50	38-43

Experience should be used to select specific values within the ranges. In general, finer materials or materials with significant silt-sized material will fall in the lower portion of the range. Coarser materials with less than 5% fines will fall in the upper portion of the range.

Care should be exercised when using other correlations of SPT results to soil parameters. Some published correlations are based on corrected values (N1(60)) and some are based on uncorrected values (N). The designer should ascertain the basis of the correlation and use either N1(60) or N as appropriate. Care should also be exercised when using SPT blow counts to estimate soil shear strength if in soils with coarse gravel, cobbles, or boulders. Large gravels, cobbles, or boulders could cause the SPT blow counts to be unrealistically high.

Correlations for other soil properties (other than as specifically addressed above for the soil friction angle) as provided in Sabatini, et al. (2002) may be used if the correlation is well established and if the accuracy of the correlation is considered regarding its influence if the estimate obtained from the correlation in the selection of the property value used for design. Local geologic formation-specific correlations may also be used if well established by data comparing the prediction from the correlation to measured high quality laboratory performance data, or back-analysis from full scale performance of geotechnical elements affected by the geologic formation in question.

5.7 Engineering Properties of Rock

Engineering properties of rock are generally controlled by the discontinuities within the rock mass and not the properties of the intact material. Therefore, engineering properties for rock must account for the properties of the intact pieces and for the properties of the rock mass as a whole, specifically considering the discontinuities within the rock mass. A combination of laboratory testing of small samples, empirical analysis, and field observations should be employed to determine the engineering properties of rock masses, with greater emphasis placed on visual observations and quantitative descriptions of the rock mass.

Rock properties can be divided into two categories: intact rock properties and rock mass properties.

- **Intact rock**: Intact rock properties are determined from laboratory tests on small samples typically obtained from coring, outcrops or exposures along existing cuts. Engineering properties typically obtained from laboratory tests include specific gravity, unit weight, ultrasonic velocity, compressive strength, tensile strength, and shear strength.
- **Rock mass properties:** Rock mass properties are determined by visual examination and measurement of discontinuities within the rock mass, and how these discontinuities will affect the behavior of the rock mass when subjected to the proposed construction.

The methodology and related considerations provided by Sabatini, et al. (2002) should be used to assess the design properties for the intact rock and the rock mass as a whole.

However, the portion of Sabatini, et al. (2002) that addresses the determination of fractured rock mass shear strength parameters (Hoek and Brown, 1988) is outdated. The original work by Hoek and Brown has been updated and is described in Hoek, et al. (2002).

The updated method uses a Geological Strength Index (GSI) to characterize the rock mass for the purpose of estimating strength parameters, and has been developed based on re-examination of hundreds of tunnel and slope stability analyses in which both the 1988 and 2002 criteria were used and compared to field results. While the 1988 method has been more widely published in national (e.g., FHWA) design manuals than has the updated approach provided in Hoek, et al. (2002), considering that the original developers of the method have recognized the short-comings of the 1988 method and have reassessed it through comparison to actual rock slope stability data, the Hoek, et al. (2002) is considered to be the most accurate methodology. Therefore the Hoek, et al. (2002) method should be used for fractured rock mass shear strength determination. Note that this method is only to be used for highly fractured rock masses in which the stability of the rock slope is not structurally controlled.

5.8 Final Selection of Design Values

5.8.1 Overview

After the field and laboratory testing is completed, the geotechnical designer should review the quality and consistency of the data, and should determine if the results are consistent with expectations. Once the lab and field data have been collected, the process of final material property selection begins. At this stage, the geotechnical designer generally has several sources of data consisting of that obtained in the field, laboratory test results and correlations from index testing. In addition, the geotechnical designer may have experience based on other projects in the area or in similar soil/rock conditions. Therefore, if the results are not consistent with each other or previous experience, the reasons for the differences should be evaluated, poor data eliminated and trends in data identified. At this stage it may be necessary to conduct additional performance tests to try to resolve discrepancies.

Geotechnical Design Property Assessment

As stated in Section 5.1, the focus of geotechnical design property assessment and final selection is on the individual geologic strata identified at the project site. A geologic stratum is characterized as having the same geologic depositional history and stress history, and generally has similarities throughout the stratum in its density, source material, stress history, and hydrogeology. All of the information that has been obtained up to this point including preliminary office and field reconnaissance, boring logs, CPT soundings etc., and laboratory data are used to determine soil and rock engineering properties of interest and develop a subsurface model of the site to be used for design. Data from different sources of field and lab tests, from site geological characterization of the site subsurface conditions, from visual observations obtained from the site reconnaissance, and from historical experience with the subsurface conditions at or near the site must be combined to determine the engineering properties for the various geologic units encountered throughout the site.

However, soil and rock properties for design should not be averaged across multiple strata, since the focus of this property characterization is on the individual geologic stratum. Often, results from a single test (e.g. SPT N-values) may show significant scatter across a site for a given soil/rock unit. Data obtained from a particular soil unit for a specific property from two different tests (e.g. field vane shear tests and lab UU tests) may not agree. Techniques should be employed to determine the validity and reliability of the data and its usefulness in selecting final design parameters. After a review of data reliability, a review of the variability of the selected parameters should be carried out. Variability can manifest itself in two ways: 1) the inherent in-situ variability of a particular parameter due to the variability of the soil unit itself, and 2) the variability associated with estimating the parameter from the various testing methods. From this step, final selection of design parameters can commence, and from there completion of the subsurface profile.

5.8.2 Data Reliability and Variability

Inconsistencies in data should be examined to determine possible causes and assess any mitigation procedures that may be warranted to correct, exclude, or downplay the significance of any suspect data. Chapter 8 of Sabatini, et al. (2002) outlines step-by-step procedures for analyzing data and resolving inconsistencies.

5.8.3 Final Property Selection

The final step is to incorporate the results of the previous section into the selection of values for required design properties. Recognizing the degree of variability discussed in the previous section, the potential impact of that variability (or uncertainty) on the level of safety in the design, and on potential cost and constructability impacts, should be assessed. If the impact of this uncertainty is likely to be significant, parametric analyses should be conducted, or more data could be obtained to help reduce the uncertainty. Since the sources of data that could be considered may include measured laboratory data, field test data, performance data, and other previous experience with the geologic unit(s) in question, it will not be possible to statistically combine all this data together to determine the most likely property value.

Engineering judgment, combined with parametric analyses as needed, will be needed to make the final assessment and determination of each design property. This assessment should include a decision as to whether the final design value selected should reflect the interpreted average value for the property, or a value that is somewhere between the most likely average value and the most conservative estimate of the property. Design property selection should achieve a balance between the desire for design safety and the cost effectiveness and constructability of the design. In some cases, the selection of conservative design properties could result in very conservative designs that are un-constructible (e.g., using very conservative design parameters resulting in a pile foundation that must be driven deep into a very dense soil unit that in reality is too dense to penetrate with available equipment).

Note that in **Chapter 8**, where reliability theory was used to establish load and resistance factors, the factors were developed assuming that mean values for the design properties are used. However, even in those cases, design values that are more conservative than the mean may still be appropriate, especially if there is an unusual amount of uncertainty in the assessment of the design properties due, for example to highly variable site conditions, lack of high quality data to assess property values, or due to widely divergent property values from the different methods used to assess properties within a given geologic unit.

Depending on the availability of soil or rock property data and the variability of the geologic strata under consideration, it may not be possible to reliably estimate the average value of the properties needed for design. In such cases, the geotechnical designer may have no choice but to use a more conservative selection of design parameters to mitigate the additional risks created by potential variability or the paucity of relevant data. Note that for those resistance factors that were determined based on calibration by fitting to allowable stress design, this property selection issue is not relevant, and property selection should be based on the considerations discussed previously.

The process and examples to make the final determination of properties to be used for design provided by Sabatini, et al. (2002) should be followed.

5.8.4 Development of the Subsurface Profile

While **Section 5.8** generally follows a sequential order, it is important to understand that the selection of design values and production of a subsurface profile is more of an iterative process. The development of design property values should begin and end with the development of the subsurface profile. Test results and boring logs will likely be revisited several times as the data is developed and analyzed before the relation of the subsurface units to each other and their engineering properties are finalized.

The ultimate goal of a subsurface investigation is to develop a working model that depicts major subsurface layers exhibiting distinct engineering characteristics.

The end product is the subsurface profile, a two dimensional depiction of the site stratigraphy. The following steps outline the creation of the subsurface profile:

1. Complete the field and lab work and incorporate the data into the preliminary logs.
2. Lay out the logs relative to their respective field locations and compare and match up the different soil and rock units at adjacent boring locations, if possible. However, caution should be exercised when attempting to connect units in adjacent borings, as the geologic stratigraphy does not always fit into nice neat layers. Field descriptions and engineering properties will aid in the comparisons.
3. Group the subsurface units based on engineering properties.
4. Create cross sections by plotting borings at their respective elevations and positions horizontal to one another with appropriate scales. If appropriate, two cross sections should be developed that are at right angles to each other so that lateral trends in stratigraphy can be evaluated when a site contains both lateral and transverse extents (i.e. a building or large embankment).
5. Analyze the profile to see how it compares with expected results and knowledge of geologic (depositional) history. Have anomalies and unexpected results encountered during exploration and testing been adequately addressed during the process? Make sure that all of the subsurface features and properties pertinent to design have been addressed.

5.8.5 Selection of Design Properties for Engineered Materials

This section provides guidelines for the selection of properties that are commonly used on ODOT projects such as engineered fills. The engineering properties are based primarily on gradation and compaction requirements, with consideration of the geologic source of the fill material typical for the specific project location. For materials such as common borrow where the gradation specification is fairly broad, a wider range of properties will need to be considered.

5.8.5.1 Borrow Material

The standard specification for Borrow Material, section 00330.12, states it may be virtually any soil or aggregate either naturally occurring or processed which is free of unsuitable materials. Follow these guidelines:

- On ODOT projects, Borrow Material which meets the criteria for Moisture-Density Testable Material is compacted to at least 95 percent of maximum density based on the Standard Proctor in accordance with 00330.43(b) (2-b). Borrow Material which is a Non-Moisture Density Testable Material is typically compacted in accordance with the procedure described in 00330.43(c).
- Because of the variability of the materials that may be used as Borrow Material, the estimation of an internal friction angle and unit weight should be based on the actual material used.
- For non-plastic materials, the friction angle may be in the 30 to 34 degree range, and the unit weight may be in the 115 to 130 pcf range.
- Lower range values should be used for finer grained materials compacted to 90 percent of maximum density.

In general during design, the specific source of borrow is not known. Therefore, it is not prudent to select a design friction angle that is near or above the upper end of the range unless the geotechnical designer has specific knowledge of the source(s) likely to be used, or unless quality assurance shear strength testing is conducted during construction. Borrow material will likely have a high enough fines content to be moderately to highly moisture sensitive. This moisture sensitivity may affect the design property selection if it is likely that placement conditions are likely to be marginal due to the timing of construction.

5.8.5.2 Select Granular Backfill

The standard specification for Select Granular Backfill, section 00330.14, ensures that the mixture will be granular and contain at least a minimal amount of gravel size material. The materials are likely to be poorly graded sand and contain enough fines to be moderately moisture sensitive. The following applies:

- Select Granular Backfill is not an all-weather material. Select Granular Backfill gradation indicates that drained friction angles of 34 to 38 degrees are possible when the soil is well compacted.
- Relatively clean sands in a loose state will likely have drained friction angles of 30 to 35 degrees. Unit weights will be in the 120 to 130 pcf range for all the Select Granular Backfill materials. However, these values are highly dependent on the geologic source of the material. Windblown, beach, or alluvial sands that have been rounded through significant transport could have significantly lower shear strength values.
- Reject and scalped materials from processing could also have relative low friction angles depending on the uniformity of the material and the degree of rounding in the soil particles.

In general, during design, the specific source of borrow is not known. Therefore, it is not prudent to select a design friction angle that is near or above the upper end of the range unless the geotechnical designer has specific knowledge of the source(s) likely to be used or unless quality assurance shear strength testing is conducted during construction. Select Granular Backfill with significant fines content may sometimes be modeled as having a temporary or apparent cohesion value from 50 to 200 psf. If a cohesion value is used, the friction angle should be reduced so as not to increase the overall strength of the material. For long term analysis, all the Granular Materials should be modeled with no cohesive strength.

5.8.5.3 Select Stone Backfill

The standard specification for Select Stone Backfill, section 00330.15, should ensure reasonably well graded sand and gravel. Maximum fines content is not specified, so the material may be moisture sensitive. In very wet conditions, material with lower fines content should be used. The Select Stone Backfill specification indicates that internal angles of friction up to 40 degrees are possible, and that shear strength values less than 36 degrees are not likely. However, lower shear strength values are possible for Select Stone Backfill from naturally occurring materials obtained from non-glacially derived sources such as wind blown or alluvial deposits. In many cases, processed materials are used for Select Stone Backfill, and in general, this processed material has been crushed, resulting in rather angular particles and high soil friction angles. Unit weights of 130 to 140 pcf are possible if very well graded. In general, during design, the specific source of borrow is not known. Therefore, it is not prudent to select a design friction angle that is near or above the upper end of the range unless the geotechnical designer has specific knowledge of the source(s) likely to be used or unless quality assurance shear strength testing is conducted during construction.

5.8.5.4 Stone Embankment Material

Stone Embankment Material, standard specification section 00330.16, is considered an all-weather material. Compactive effort is based on a method specification. Because of the nature of the material, compaction testing is generally not feasible. The specification allows for a broad range of material and properties such that the internal friction angle and unit weight can vary considerably based on the amount and type of rock in the fill. For compacted rock embankments constructed with Stone Embankment Material:

- Internal friction angles of up to 45 degrees may be reasonable.
- Unit weights for rock embankments generally range from 130 to 140 pcf.

Durability is major issue with this material. Rock excavated from cuts consisting of siltstone, sandstone and claystone may break down during the compaction process, resulting in less coarse material. Also, if the rock is weak, failure may occur through the rock fragments rather than around them. In these types of materials, the strength parameters may resemble those of embankments constructed from Borrow Materials. For existing embankments, the soft rock may continue to weather with time, if the embankment materials continue to become wet. Inadequate slope stability and excessive settlement of embankments with non-durable materials are the long term effects of using weak rock materials without proper placement and compaction.

5.8.5.5 Wood Fiber

Wood fiber fills have been used by ODOT for fill heights up to about 20 feet. The wood fiber has generally been used as lightweight fill material in emergency repair situations because wet weather does not affect the placement and compaction of the embankment. Only fresh wood fiber should be used to prolong the life of the fill, and the maximum particle size should be 6 inches or less. The wood fiber is generally compacted in lifts of about 12 inches with two or more passes of a track dozer. Presumptive design values of 50 pcf for unit weight and an internal angle of friction of about 40 degrees may be used for the design of the wood fiber fills (Allen et al., 1993).

To mitigate the effects of leachate, the amount of water entering the wood should be minimized. Generally, topsoil caps of about 2 feet in thickness are used. The pavement section should be a minimum of 2 feet (a thicker section may be needed depending on the depth of wood fiber fill). Wood fiber fill will experience creep settlement for several years and some pavement distress should be expected during that period. Additional information on the properties and durability of wood fiber fill is provided in Kilian and Ferry (1993).

5.8.5.6 Geofoam

Geofoam has not been used as lightweight fill on ODOT projects, but there may be projects that will incorporate it in the future. In contrast, WSDOT has had about 10 years of experience with Geofoam in embankment construction. Geofoam ranges in unit weight from about 1 to 2 pcf. The Geofoam material is made from expanded polystyrene (EPS) and is manufactured according to ASTM standards for minimum density (ASTM C 303), compressive strength (ASTM D 1621) and water absorption (ASTM C 272). Type I and II Geofoam are generally used in highway applications. Bales of recycled industrial polystyrene waste are also available. These bales have been used to construct temporary haul roads over soft soil. However, these bales should not be used in permanent applications.

5.9 References

Allen, T. M., Kilian, A. P., 1993, "Use of Wood Fiber and Geotextile Reinforcement to Build Embankment Across Soft Ground," Transportation Research Board Record 1422.

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Chapter

6

6 Seismic Design

6.1. General

This chapter describes ODOT's standards and policies regarding the geotechnical aspects of the seismic design of ODOT projects. The purpose is to provide geotechnical engineers and engineering geologists with specific seismic design guidance and recommendations not found in other standard design documents used for ODOT projects. Complete design procedures (equations, charts, graphs, etc.) are usually not provided unless necessary to supply, or supplement, specific design information, or if they are different from standards described in other references. This chapter also describes what seismic recommendations should typically be provided by the geotechnical engineer in the Geotechnical Report.

6.1.1 Seismic Design Standards

The seismic design of ODOT bridges shall follow methods described in the most currently adopted edition of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* (AASHTO, 2009) supplemented by the Bridge Design and Drafting Manual (BDDM) and the recommendations supplied in this chapter.. Refer to the *ODOT BDDM* for additional design criteria and guidance regarding the use of the new Guide Specification on bridge projects. The term "AASHTO" as used in this chapter refers to AASHTO LRFD design methodology. For seismic design of new buildings the 2003 International Building Code (IBC) (International Code Council, 2002) should be used. In addition to these standards, the manuals listed below may be referenced for additional design guidance in seismic design for issues and areas not addressed in detail in the AASHTO specifications or this chapter. Unless otherwise noted, the standards and policies described in this chapter supersede those described in the referenced documents.

- "*Design Guidance: Geotechnical Earthquake Engineering For Highways*", Geotechnical Engineering Circular No. 3, Volumes I & II, FHWA-SA-97-076&077.

This FHWA document provides design guidance for geotechnical earthquake engineering for highways. Specifically, this document provides guidance on earthquake engineering fundamentals, seismic hazard analysis, ground motion characterization, site characterization, seismic site response analysis, seismic slope stability, liquefaction, and seismic design of foundations and retaining walls. The document also includes design examples (Volume II) for typical geotechnical earthquake engineering analyses.

For liquefaction analysis, embankment deformation estimates and bridge damage assessment the following two documents should be referenced:

- "*Assessment and Mitigation of Liquefaction Hazards to Bridge Approach Embankments in Oregon*", Dickenson, S., et al., Oregon State University, Department of Civil, Construction and Environmental Engineering, SPR Project 361, November, 2002.

- "*Recommended Guidelines For Liquefaction Evaluations Using Ground Motions From Probabilistic Seismic Hazard Analysis*", Dickenson, S., Oregon State University, Department of Civil, Construction and Environmental Engineering, Report to ODOT, June, 2005.

The above two documents are available on the ODOT Geo-Environmental web page. In light of the continuous advances being made in evaluating the impact of liquefaction hazards and ground failures on structures the above two references should be supplemented with the most up to date technical information and guidelines.

- **NCHRP Report 472:** The National Cooperative Highway Research Program Report 472 (2002), "*Comprehensive Specifications for the Seismic Design of Bridges*", is a report containing the findings of a study completed to develop recommended specifications for seismic design of highway bridges. The report covers topics including design earthquakes and performance objectives, foundation design, liquefaction hazard assessment and design, and seismic hazard representation.

- United States Geological Survey (**USGS**) Website.

  The USGS National Seismic Hazard Maps website is a valuable tool for characterizing the seismic hazard for a specific site. This site provides the results of Probabilistic Seismic Hazard Analyses (PSHA) in the form of the Uniform Seismic Hazard, which reflects the contribution of all seismic sources in the region on the ground motion parameters. The website provides ground motion parameters (Peak Ground Acceleration (PGA), and acceleration response spectral ordinates between 0.1 and 5.0 seconds) on Site Class B rock for various return periods, specified as a percentage probability of exceedance in a given exposure interval, in years. The website provides PGA and spectral acceleration ordinates at periods of 0.2 and 1.0 second for risk levels of 5 and 10 percent probabilities of exceedance (PE) in 50 years for use in the development of response spectra, in accordance with AASHTO. This risk level of 5 and 10 percent PE corresponds to approximately 7 and 14 percent PE in 75 years, respectively. The website also provides interactive deaggregation of a site's probabilistic seismic hazard. The deaggregation is useful for demonstrating the relative contribution of regional seismic sources, in terms of magnitude and source-to-site distance, on the seismic hazard at a site. De-aggregation is particularly useful for demonstrating how the ground motion parameters generated by individual sources compare to the mean motions determined for the Uniform Seismic Hazard.

- WSDOT Geotechnical Design Manual, M46-03.01, November, 2008
  - **http://www.wsdot.wa.gov/publications/manuals/fulltext/M46-03/Geotech.pdf**

6.1.1 Background

In light of the complexity of seismic foundation design, continuous enhancements to analytical and empirical methods of evaluation are being made as more field performance data is collected and research advances the state of knowledge. New methods of analysis and design are continuously being developed and therefore it is considered prudent to not be overly prescriptive in defining specific design methods for use in the seismic design process. However, a standard of practice needs to be established within the geotechnical community regarding minimum required design criteria for seismic design. It is well recognized that these standards are subject to change in the future as a result of further research and studies. This chapter will be continually updated as more information is obtained, new design codes are approved and better design methods become available.

Significant engineering judgment is required throughout the entire seismic design process. The recommendations provided herein assume the geotechnical designer has a sound education and background in basic earthquake engineering principles. These recommendations are not intended to be construed as complete or absolute. Each project is different in some way and requires important decisions and judgments be made at key stages throughout the design process. The applicability of these recommended procedures should be continually evaluated throughout the design process. Peer review may be required to assist the design team in various aspects of the seismic hazard and earthquake-resistant design process.

Earthquakes often result in the transfer of large axial and lateral loads from the bridge superstructure into the foundations. At the same time, foundation soils may liquefy, resulting in a loss of soil strength and foundation capacity. Under this extreme event condition it is common practice to allow the foundations to be loaded up to the nominal (ultimate) foundation resistances (allowing resistance factors as high as 1.0). This design practice requires an increased emphasis on quality control during the construction of bridge foundations since we are now often relying on the full, unfactored nominal resistance of each foundation element to support the bridge during the design seismic event.

In addition to seismic foundation analysis, seismic structural design also involves an analysis of the soil-structure interaction between foundation materials and foundation structure elements. Soil-structure interaction is typically performed in bridge design by modeling the foundation elements using equivalent linear springs. Some of the recommendations presented herein relate to bridge foundation modeling requirements and the geotechnical information the structural designer needs in order to do this analysis. Refer to *Section 1.1.4* of the *ODOT Bridge Design and Drafting Manual (BDDM)* for more information on bridge foundation modeling procedures.

6.1.2 Responsibility of the Geotechnical Designer

The geotechnical designer is responsible for providing geotechnical/seismic input parameters to the structural engineers for their use in design of the transportation infrastructure. Specific elements to be addressed by the geotechnical designer include the design ground motion parameters, site response, geotechnical design parameters and geologic hazards. The geotechnical designer is also responsible for providing input for evaluation of soil-structure interaction (foundation response to seismic loading), earthquake induced earth pressures on retaining walls, and an assessment of the impacts of geologic hazards on the structures. Refer to **Chapter 21** for geotechnical seismic design reporting requirements.

The seismic geologic hazards to be evaluated include fault rupture, liquefaction, ground failure including flow slides and lateral spreading, ground settlement, and instability of natural slopes and earth structures. The seismic performance of tunnels is a specialized area of geotechnical earthquake engineering not specifically addressed in this guidance document; however the ground motion parameters determined in the seismic hazard analyses outlined herein may form the basis for tunnel stability analyses (e.g., rock fall adjacent to portals and in unlined tunnels, performance of tunnel lining). The risk associated with seismic geologic hazards shall be evaluated by the geotechnical designer following the methods described in this chapter.

6.2 Seismic Design Performance Requirements

6.2.1 New Bridges

A two-level approach is used in ODOT for the seismic design of all new bridges. The seismic design of ODOT bridges is evaluated in terms of performance requirements for ground motions having average return periods of approximately 500-years and 1000-years. The 500-year and 1000-year return period ground motions have probabilities of exceedance of approximately 14% and 7% in 75 years respectively. For a 50 year time period, the probabilities of exceedance of the 500 and 1000 year ground motions are approximately 10% and 5% respectively. The seismic foundation design requirements, including approach embankments, shall be consistent with meeting the current ODOT Bridge Engineering Section seismic design criteria. Excerpts from those criteria are summarized as follows from the *BDDM*:

> **1000-year "No-Collapse" Criteria**: Design all bridges for 1000-year return period ground motions (7% probability of exceedance in 75 years) under "No Collapse" criteria.
>
> Under this level of shaking, the bridge and approach structures, bridge foundation and approach fills must be able to withstand the forces and displacements without collapse of any portion of the structure. In general, bridges that are properly designed and detailed for seismic loads can accommodate relatively large deflections without the danger of collapse. If large embankment displacements (lateral spread) or overall slope failure of the end fills are predicted, the impacts on the bridge end bent, abutment walls and interior piers should be evaluated to see if the impacts could potentially result in collapse of any part of the structure. Slopes adjacent to a bridge or tunnel should be evaluated if their failure could result in collapse of a portion or all of the structure.
>
> **500-year "Serviceability" Criteria**: In addition to the 1000-year "No Collapse" criteria, design all bridges to remain "Serviceable" after subjected to 500-year return period ground motions (14% probability of exceedance in 75 years).
>
> Under this level of shaking, the bridge and approach fills, are designed to remain in service shortly after the event (after the bridge has been properly inspected) to provide access for emergency vehicles. In order to do so, the bridge is designed to respond semi-elastically under seismic loads with minimal damage. Some structural damage is anticipated but the damage should be repairable and the bridge should be able to carry emergency vehicles immediately following the earthquake. This holds true for the approach fills leading up to the bridge.
>
> Approach fill settlement and lateral displacements should be minimal to provide for immediate emergency vehicle access for at least one travel lane. For mitigation purposes approach fills are defined as shown on **Figure 6-12**. As a general rule of thumb, an estimated lateral embankment displacement of up to 1 foot is considered acceptable in many cases as long as the "serviceable" performance criteria described above can be met. Vertical settlements on the order of 6" to 12" may be acceptable depending on the roadway geometry and anticipated performance of the bridge end panels. Bridge end panels are required on all state highway bridge projects (per *BDDM*) and should be evaluated for their ability to withstand the anticipated embankment displacements and settlement and still provide the required level of performance. These displacement criteria are to serve as general guidelines only and engineering judgment is required to determine the final amounts of acceptable displacement that will meet the desired criteria. It should be noted that these estimated displacements are not at all precise values and may easily vary by factors of 2 to 3 depending on the analysis method(s) used. The amounts of allowable vertical and horizontal displacements should be decided on a case-by-case basis, based on discussions and consensus between the bridge designer and the geotechnical designer and perhaps other project personnel.

In addition to bridge and approach fill performance, embankments through which cut-and-cover tunnels are constructed should be designed to remain stable during the design seismic event because of the potential for damage or possible collapse of the structure should they fail.

Approach embankments and structure foundations should be designed to meet the above performance requirements. Unstable slopes such as active or potential landslides, and other seismic hazards such as liquefaction, lateral spread, post-earthquake settlement and downdrag may require mitigation measures to ensure that the structure meets these performance requirements. Refer to **Chapter 11** for guidance on approved ground improvement techniques to use in mitigating these hazards.

6.2.2 Bridge Widenings

For the case where an existing bridge is to be widened, the foundations for the widened portion of the bridge and bridge approaches should be designed to remain stable and meet the same bridge performance criteria as new bridges. In addition, if the existing bridge foundation is not stable and could cause collapse of the bridge widening, or if liquefiable soils are present, to the extent practical, measures should be taken to prevent collapse of the existing bridge during the design seismic event. If foundation retrofit or liquefaction mitigation is necessary to meet the performance criteria, these designs shall be reviewed and approved by the HQ Bridge Section. The foundations for the widening should be designed in a way that the seismic response of the bridge widening can be made compatible with the seismic response of the existing bridge as stabilized in terms of foundation deformation and stiffness. If it is not feasible to stabilize the existing bridge such that it will not cause collapse of the bridge widening during the design seismic event, consideration should be given to replacing the existing bridge rather than widening it.

6.2.3 Bridge Abutments and Retaining Walls

Seismic design performance objectives for retaining walls depend on the function of the retaining wall and the potential consequences of failure.

There are four retaining wall categories, as defined in **Section 15.2.1**. The seismic design performance objectives for these four categories are listed below:

- **Bridge Abutments**: Bridge Abutments are considered to be part of the bridge, and shall meet the seismic design performance objectives for the bridge see **Section 6.2.1.**

- **Bridge Retaining Walls**: Design all Bridge Retaining Walls for 1000-year return period ground motions under the “No Collapse” bridge criteria. Under this level of shaking, the Bridge Retaining Wall must be able to withstand seismic forces and displacements without failure of any part of the wall or collapse of any part of the bridge which it supports. Bridge Retaining Walls shall be designed for overall stability under these seismic loading conditions, including anticipated displacements associated with liquefaction. Mitigation to achieve overall stability may be required.

  In addition, design all Bridge Retaining Walls for 500-year return period ground motions under the “Serviceability” bridge criteria. Under this level of shaking, Bridge Retaining Wall movement must not result in unacceptable performance of the bridge or bridge approach fill, as described under the 500-Year “Serviceability” criteria in **Section 6.2.1**.

- **Highway Retaining Walls**: Design all Highway Retaining Walls for 1000-year return period ground motions. Under this level of shaking, the Highway Retaining Wall must be able to withstand seismic forces and displacements without failure of any part of the Highway Retaining Wall. Highway Retaining Walls shall be designed for overall stability under these seismic loading conditions, including anticipated displacements associated with liquefaction. Mitigation to achieve overall stability may be required

- **Minor Retaining Walls**: Minor Retaining Wall systems have no seismic design requirements.

The policy to design all Highway Retaining Walls to meet overall stability requirements for seismic design may not be practical at all wall locations. Where it is not practical to design a Highway Retaining Wall for overall stability under seismic loading, and where a failure of this type would not endanger the public, impede emergency and response vehicles along essential lifelines, or have an adverse impact on another structure, the local Region Tech Center will evaluate practicable alternatives for improving the seismic resistance and performance of the retaining wall.

In general, retaining walls and bridge abutments should not be built on or near landslides or other areas that are marginally stable under static conditions. However, if site conditions and project constraints provide no cost effective or technical alternative, the local Region Tech Center will evaluate, on a case-by-case basis, the possible placement of these structures in these locations, as well as requirements for global (overall) instability of the landslide during the design seismic event.

6.2.4 Embankments and Cut Slopes

Cut slopes, fill slopes, and embankments are generally not evaluated for seismic instability unless they directly affect a bridge, highway retaining wall or other structure. Bridge approach fills should always be evaluated for stability and settlement, especially if they are relied upon to provide passive soil resistance behind the abutment (Earthquake-Resisting System). Seismic instability associated with routine cuts and fills are typically not mitigated due to the high cost of applying such a design policy uniformly to all slopes statewide. If failure and displacement of existing slopes, embankments or cut slopes, due to seismic loading, could adversely impact an adjacent structure, these areas should be considered for stabilization. Such impacts should be evaluated in terms of meeting the performance criteria described in **Section 6.2**.

6.3 Ground Motion Parameters

The ground motion parameters to be used in design are currently based on the 2002 USGS National Seismic Hazard maps for the Pacific Northwest region. The USGS seismic hazard mapping project provides the results of probabilistic seismic hazard analysis (PSHA) at the regional scale. The ground motion maps for the 500 and 1000-year return periods are available in the *ODOT Bridge Design and Drafting Manual (BDDM)*.

Ground motion parameters for the 2002 USGS hazard maps are also available on the USGS website at:

http://earthquake.usgs.gov/hazmaps/

The designer should review the basis of these hazard maps and have a thorough understanding of the data they represent and the methods used for their development. The 2008 USGS National Seismic Hazard maps are currently under review by the ODOT Bridge Section and are not approved for use on ODOT projects at this time.

The BDDM maps provide Peak Ground Acceleration (PGA), 0.20 sec. and 1.0 sec. spectral accelerations scaled in contour intervals of 0.01g. Ground motion values can be obtained from the USGS website by selecting the **“Interactive Deaggregations”** link under the **“Seismic Hazard Analysis Tools”** heading. The PGA and spectral accelerations can then be obtained by entering the latitude and longitude of the site and the desired probability of exceedance (i.e., 5% in 50 years for the 1000 year return event). It should be noted that the PGA obtained from these maps is actually the

Peak "Bedrock" Acceleration (i.e., Site Class B), and does not include, or take into account, any local soil amplification effects. See **Section 6.5.1** for the development of design ground motion data.

6.3.1 Site Specific Probabilistic Seismic Hazard Analysis

Ground motion parameters are also sometimes determined from a site specific Probabilistic Seismic Hazard Analysis (PSHA). A site specific probabilistic hazard analysis may be considered when:

- New information about one or more active seismic sources that affect the site has become available since the most recent USGS/AASHTO Seismic Hazard Maps were developed (2006), and the new seismic source information will result in a significant change to the seismic hazard at the site. The existence of the Next Generation Attenuation (NGA) relationships shall not be the sole justification for conducting a PSHA.
- The site is located within 6 miles of a known active fault capable of producing at least a magnitude 5 earthquake. For these cases, near-fault ground motion effects (directivity, directionality) were not explicitly modeled in the development of national ground motion maps, and the code/specification based hazard level may be significantly unconservative. These "near-fault" effects are normally only considered for essential or critical structures.
- The size or importance of the bridge is such that a lower probability of exceedance (and therefore a longer return period) should be considered.

It should be noted that the site-specific PSHA is often uncoupled from subsequent site-specific ground response analyses, which are described in **Section 6.5.1.3.** A site specific probabilistic hazard analysis focuses on the spatial and temporal occurrence of earthquakes, and evaluates all of the possible earthquake sources contributing to the seismic hazard at a site with the purpose of developing ground motion data consistent with a specified uniform hazard level. The analysis takes into account all seismic sources that may affect the site and quantifies the uncertainties associated with the seismic hazard, including the location of the source, extent and geometry, maximum earthquake magnitudes, rate of seismicity, and estimated ground-motion parameters. The result of the analysis is a uniform hazard acceleration response spectrum that is based on a specified uniform hazard level or probability of exceedance within a specified time period (i.e., 7% probability of exceedance in 75 years). The PSHA is routinely performed to yield ground motion parameters for bedrock (Site Class B) sites. The influence of the soil deposits at the site on the ground motion characteristics is subsequently evaluated using the results of the PSHA for bedrock conditions. A ground response analysis commonly utilizes computer programs such as SHAKE or D-MOD to model the response of a soil column subjected to bedrock ground motion. The input bedrock motion is in the form of time-histories that are typically matched (or scaled) to the bedrock response spectrum developed from the probabilistic hazard analysis.

A site specific probabilistic hazard analysis is typically not performed on routine ODOT projects. If such an analysis is desired for the design of ODOT bridge projects the HQ Bridge Engineering Section must approve the justification and procedures for conducting the analysis and the analysis must be reviewed by an independent source approved by the HQ Bridge Engineering Section. The review and approval of the PSHA will be coordinated with the region geotechnical engineer.

6.3.2 Magnitude and PGA for Liquefaction Analysis

Earthquake engineering evaluations that address repeated (cyclic) loading and failure of soils must include estimates of the intensity and duration of the earthquake motions. In soils, liquefaction and cyclic degradation of soil stiffness/strength represent fatigue failures that often impact bridge structures. In practice-oriented liquefaction analysis, the intensity of the cyclic loading is related to the PGA and/or cyclic stress ratio, and the duration of the motions is correlated to the magnitude of the causative event. The PGA and magnitude values selected for the analysis should represent realistic ground motions associated with specific, credible scenario earthquakes. The PGA values obtained from the USGS web site represent the "mean" values of all of the sources contributing to the hazard at the site for a particular recurrence interval. These "mean" PGA values should not typically be used for liquefaction analysis unless the ground motions at the site are dominated by a single source, as demonstrated in the PSHA deaggregation. Otherwise, the "mean" PGA values may not represent realistic ground motions resulting from known sources affecting the site. Additionally, the mean magnitude provided by PSHA should not be used as the causative event as this often averages the magnitude of large Cascadia Subduction Zone earthquakes and the magnitude of the smaller, local crustal events with a resulting magnitude that is not representative of any seismic source in the region. For this reason the modal event(s), designated as Magnitude and Distance (M-R) pairs, should be evaluated individually.

6.3.3 Deaggregation of Seismic Hazard

A deaggregation of the total seismic hazard should be performed to find the principal individual sources contributing to the seismic hazard at the site. As a general rule of thumb, all sources that contribute more than about 5% to the hazard should be evaluated. However, sources that contribute less than 5% may also be sources to consider since they may still significantly affect the liquefaction analysis or influence portions of the site's response spectra. The relative contribution of all considered sources, in terms of magnitude and distance, on PGA and on spectral accelerations at 9 different frequencies (or periods) of structural vibration can be readily evaluated using the results of the USGS seismic hazard mapping tools and deaggregation capabilities available on-line.

It is recommended that the relative contributions of all of the following sources be considered when performing liquefaction and ground deformation hazards:

1. Cascadia Subduction Zone – mega-thrust earthquakes
2. Deep, intraslab Benioff Zone earthquakes such as the 1949 and 1965 Puget Sound, and 2001 Nisqually earthquakes
3. Shallow crustal earthquakes associated with mapped faults
4. Regional background seismicity and 'randomly" occurring earthquakes that are not associated with mapped faults (gridded seismicity)

Deaggregation of the seismic hazard will provide the Magnitude (M) and Distance (R) of each source contributing to the hazard at the site. These M & R values can then be utilized with ground motion attenuation relationships to obtain bedrock PGA values at the site due to the individual sources. It is recommended that more than one attenuation relationship be used to estimate ground motion parameters for each of the primary seismic sources in Oregon (i.e., Cascadia Subduction Zone events, and shallow crustal events). The use of three to four attenuation relationships is common in practice. In order to facilitate direct comparison of the ground motion parameters with the USGS seismic hazard mapping results it is necessary to employ the same attenuation relationships that were used in developing the 2002 USGS seismic hazard maps. These attenuation relationships are summarized below. Additional attenuation relationships, appropriate for the style of faulting, can be used at the discretion of the geotechnical engineer.

<u>Crustal Faults:</u>

Extensional Areas; Equal weight for all:

Boore et al., (1997), Sadigh et al. (1997), Abrahamson and Silva (1997), Spudich et al., 1999, and Campbell and Bozorgnia (2003).

Non-Extensional Areas; Equal weight for all:

Boore et al., (1997), Sadigh et al. (1997), Abrahamson and Silva (1997), and Campbell and Bozorgnia (2003).

Extensional and non-extensional areas are defined in Figure 5 of the USGS report:

"*Documentation for the 2002 Update of the National Seismic Hazard Maps*", Open-File Report 02-420, U.S. DEPARTMENT OF THE INTERIOR U.S. GEOLOGICAL SURVEY, 2002

<u>Cascadia Subduction Zone</u>:

Youngs et al. (1997) Youngs, R.R., S.J. Chiou, W.J. Silva, and J.R. Humphrey (1997). Strong ground motion attenuation relationships for subduction zone earthquakes, Seism. Res. Letts., v. 68, no. 1, pp. 58-73.

Sadigh, K., C.Y. Chang, J. Egan, F. Makdisi, and R. Youngs (1997). Attenuation relationships for shallow crustal earthquakes based on California strong motion data, Seism. Res. Letts., v. 68, pp. 180-18

Magnitude 9.0:

Use equal weighting for both methods for distances where the Sadigh et al. (1997) PGA values for M8.5 exceed those of Youngs et al. (1997) for M9.0. For larger distances (R > 60 km), where the Youngs et al. (1997) PGA values are the higher of the two, use only the Youngs et al. (1997) relations.

Magnitude 8.3:

Use equal weighting for both methods for distances up to 70km. For distances larger than 70km, apply full weight to Youngs et al. (1997).

The source distances for the subduction zone events reported from the USGS deaggregation web site are the closest distances to the fault or slab (R_{rup}). Review the following document for more information on the proper applications and usage of these attenuation relationships.

"*Documentation for the 2002 Update of the National Seismic Hazard Maps*", Open-File Report 02-420, U.S. DEPARTMENT OF THE INTERIOR U.S. GEOLOGICAL SURVEY, 2002

It is important to note that the ground motion values (PGA, $S_{0.2}$, $S_{1.0}$) obtained for the primary M-R pairs obtained in this fashion will not likely be the same as the "mean" values developed for the Uniform Seismic Hazard (USH), which are used as the basis for structural analysis. Also, it is likely that the average value of a specific ground motion parameter obtained for the principal M-R pairs will also vary from the mean value provided by the USGS USH. The difference will reflect the number M-R pairs considered and the relative contributions of the sources to the overall hazard.

This deaggregation process will likely yield more than one M-R pair, and therefore more than one magnitude and peak ground acceleration, for liquefaction analysis in some areas of the state where the hazard is dominated by two or more seismic sources. In most of western Oregon, this will include both shallow crustal sources and the Cascadia Subduction Zone. In this case, each M-R (i.e., M-PGA) pair should be evaluated individually in a liquefaction analysis. If liquefaction is estimated for any given M-PGA pair, the evaluation of that pair is continued through the slope stability and lateral deformation evaluation processes. In some areas in the state where the seismic hazard is dominated by a single source, such as the Cascadia Subduction Zone along the coast, a single pair of M-R values (largest magnitude (M) and closest distance (R)) may be appropriate for defining and assessing the worst case liquefaction condition.

The steps involved in a simplified deaggregation application and liquefaction analysis are described in Dickenson, 2005. Four example problems are provided in Dickenson, 2005 for different areas of the state, demonstrating the deaggregation procedure. A recommended procedure for estimating lateral embankment deformations is also included in this paper along with two example problems. A flow chart of this process, extracted from this paper, is attached as **Appendix 6-A**.

6.4 Site Characterization for Seismic Design

The geotechnical site investigation should identify and characterize the subsurface conditions and all geologic hazards that may affect the seismic analysis and design of the proposed structures or features. The goal of the site characterization for seismic design is to develop the subsurface profile and soil property information needed for seismic analyses. The geotechnical designer should review and discuss the project objectives with the project engineering geologist and the structural designer, as seismic design is a cooperative effort between the geotechnical and structural engineering disciplines. The geotechnical designer should do the following as a minimum:

- Identify potential geologic hazards, areas of concern (e.g., deep soft soils or liquefiable soils), and potential variability of local geology.
- Identify engineering analyses to be performed (e.g., ground response analysis, liquefaction susceptibility, lateral spreading/slope stability assessments, seismic-induced settlement/ downdrag, dynamic earth pressures).
- Identify engineering properties required for these analyses.
- Determine methods to obtain the required design parameters and assess the validity of such methods for the soil and rock material types.

Develop an integrated investigation of in-situ testing, soil sampling, and laboratory testing. This includes determining the number of tests/samples needed and appropriate locations to obtain them.

6.4.1 Subsurface Investigation for Seismic Design

Refer to Section 6.0 of AASHTO, 2009, for guidance regarding subsurface investigation and site characterization for seismic foundation design. With the possible exception of geophysical explorations associated with obtaining seismic shear wave velocities in soil and rock units, the subsurface data required for seismic design is typically obtained concurrently with the data required for static design of the project (i.e., additional exploration for seismic design over and above what is required for foundation design is typically not necessary). However, the exploration program may need to be adjusted to obtain the necessary parameters for seismic design. For example, the use of the seismic cone penetration test, SCPT, is recommended in order to supplement tip resistance and friction data with shear wave velocity. Also, for Site Class determination, subsurface investigations must extend to a depth of at least 100 feet unless bedrock is encountered before reaching that depth.

The selection of field drilling equipment and sampling methods will reflect the goals of the investigation. If liquefaction potential is a significant issue, mud rotary drilling with SPT sampling is the preferred method of investigation. Hollow-stem auger (HSA) drilling may be utilized for SPT sampling and testing if precautionary measures are taken. Soil heaving and disturbance in HSA borings can lead to unreliable SPT “N” values. Therefore care must be taken if using HSA methods to maintain an adequate water head in the boring at all times and to use drilling techniques that minimize soil disturbance. Non-standard samplers shall not be used to collect data used in liquefaction analysis and mitigation design.

In addition to standard subsurface investigation methods, the following equipment calibration, soil testing, and/or sampling should be considered depending upon site conditions.

- **<u>SPT Hammer Energy:</u>** This value (usually termed hammer efficiency) should be noted on the boring logs or in the Geotechnical Report. The hammer efficiency should be obtained from the hammer manufacturer, preferably through field testing of the hammer system used to conduct the test. This is needed to determine the hammer energy correction factor, C_{er}, for liquefaction analysis.

- **<u>Soil Samples for Gradation Testing:</u>** Used for determining the amount (percentage) of fines in the soil for liquefaction analysis. Also useful for scour estimates.

- **<u>Undisturbed Samples:</u>** Laboratory testing for S_u, $e_{50,}$ E, G, and other parameters for both foundation modeling and seismic design.

- **<u>Shear Wave Velocity Measurements:</u>** For use in determining soil Site Class. Also used to develop a shear wave velocity profile of the soil column and to obtain low strain shear modulus values to use in analyses such as dynamic soil response.

- **<u>Seismic Piezocone Penetrometer:</u>** For use in determining soil Site Class. Also used to develop a shear wave velocity profile and obtain low strain shear modulus values to use in a ground response analysis.

- **<u>Piezocone Penetrometer Test:</u>** Used for liquefaction analysis and is even preferred in some locations due to potential difficulties in obtaining good quality SPT results. Pore pressure measurements and other parameters can be obtained for use in foundation design and modeling. Also useful in establishing the pre-construction subsurface soil conditions prior to conducting ground improvement techniques and the post-construction condition after ground improvement.

- **<u>Depth to Bedrock:</u>** If a ground response analysis is to be performed, the depth to bedrock must be known. "Bedrock" material for this purpose is defined as a material unit with a shear wave velocity of at least 2500 ft/sec.

- **<u>Pressuremeter Testing:</u>** For development of p-y curves if soils cannot be adequately characterized using standard COM624P or LPile parameters. Testing is typically performed in soft clays, organic soils, very soft or decomposed rock and for unusual soil or rock materials. The shear modulus, G, for shallow foundation modeling and design can also be obtained.

<u>Table 6-1</u> provides a summary of site characterization needs and testing considerations for geotechnical/seismic design.

Table 6-1. Summary of site characterization needs and testing considerations for seismic design (adapted from Sabatini, et al., 2002)

• Geotechnical Issues	• Engineering Evaluations	• Required Information For Analyses	• Field Testing	• Laboratory Testing
Site Response	• source characterization and attenuation • site response spectra • time history	• subsurface profile (soil, groundwater, depth to rock) • shear wave velocity • bulk shear modulus for low strains • relationship of shear modulus with increasing shear strain • equivalent viscous damping ratio with increasing shear strain • Poisson's ratio • unit weight • relative density • seismicity (PGA, design earthquakes)	• SPT • CPT • seismic one • geophysical testing (shear wave velocity) • piezometer	• cyclic triaxial tests • Atterberg Limits • specific gravity • moisture content • unit weight • resonant column • cyclic direct simple shear test • torsional simple shear test
Geologic Hazards Evaluation (e.g. liquefaction, lateral spreading, slope stability)	• liquefaction susceptibility • liquefaction induced settlement • settlement of dry sands • lateral spreading • slope stability and deformations	• subsurface profile (soil, groundwater, rock) • shear strength (peak and residual) • unit weights • grain size distribution • plasticity characteristics • relative density • penetration resistance • shear wave velocity • seismicity (PGA, design earthquakes) • site topography	• SPT • CPT • seismic cone • Becker penetration test • vane shear test • piezometers • geophysical testing (shear wave velocity)	• soil shear tests • triaxial tests (including cyclic) • grain size distribution • Atterberg Limits • specific gravity • organic content • moisture content • unit weight

Table 6-1 Summary of site characterization needs and testing considerations for seismic design (cont'd) (adapted from Sabatini, et al., 2002).

• Geotechnical • Issues	• Engineering • Evaluations	• Required • Information • For Analyses	• Field Testing	• Laboratory Testing
Input for Structural Design	• shallow foundation springs • p-y data for deep foundations • down-drag on deep foundations • residual strength • lateral earth pressures • lateral spreading/ slope movement loading • post-earthquake settlement	• subsurface profile (soil, groundwater, rock) • shear strength (peak and residual) • seismic horizontal earth pressure coefficients • shear modulus for low strains or shear wave velocity • relationship of shear modulus with increasing shear strain • unit weight • Poisson's ratio • seismicity (PGA, design earthquake) • site topography	• CPT • SPT • seismic cone • piezometers • geophysical testing (shear wave velocity) • vane shear test	• triaxial tests • soil shear tests • unconfined compression • grain size distribution • Atterberg Limits • specific gravity • moisture content • unit weight • resonant column • cyclic direct simple shear test • torsional simple shear test

For routine designs, in-situ or laboratory testing for parameters such as the dynamic shear modulus at small strains, equivalent viscous damping, shear modulus and damping ratio versus shear strain, and residual shear strength are generally not directly obtained. Instead, index properties and correlations based on in-situ field measurements (such as the SPT and CPT) are generally used in lieu of in-situ or laboratory measurements for routine design to estimate these values.

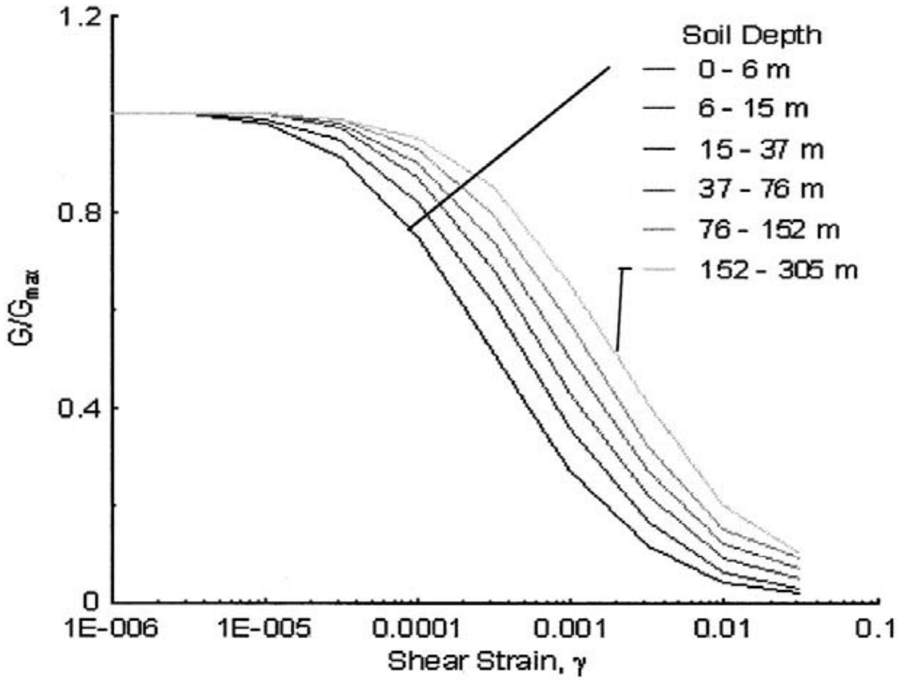
If correlations are used to obtain seismic soil design properties, the following correlations are recommended. Other acceptable correlations can be found in Dickenson et al. (2002), Kramer (1996), and other technical references. Region and site-specific correlations developed by practitioners are acceptable with adequate supporting documentation and approval by ODOT.

- **ODOT** Table 6-2, which presents correlations for estimating initial shear modulus (G_{max}) based on relative density, penetration resistance or void ratio.
- **ODOT** Figure 6-1, which presents shear modulus reduction curves and equivalent viscous damping ratio for cohesionless soils (sands) as a function of shear strain and depth.
- **ODOT** Figure 6-2 and Figure 6-3, which present shear modulus reduction curves and equivalent viscous damping ratio, respectively, as a function of cyclic shear strain and plasticity index for fine grained (cohesive) soils.

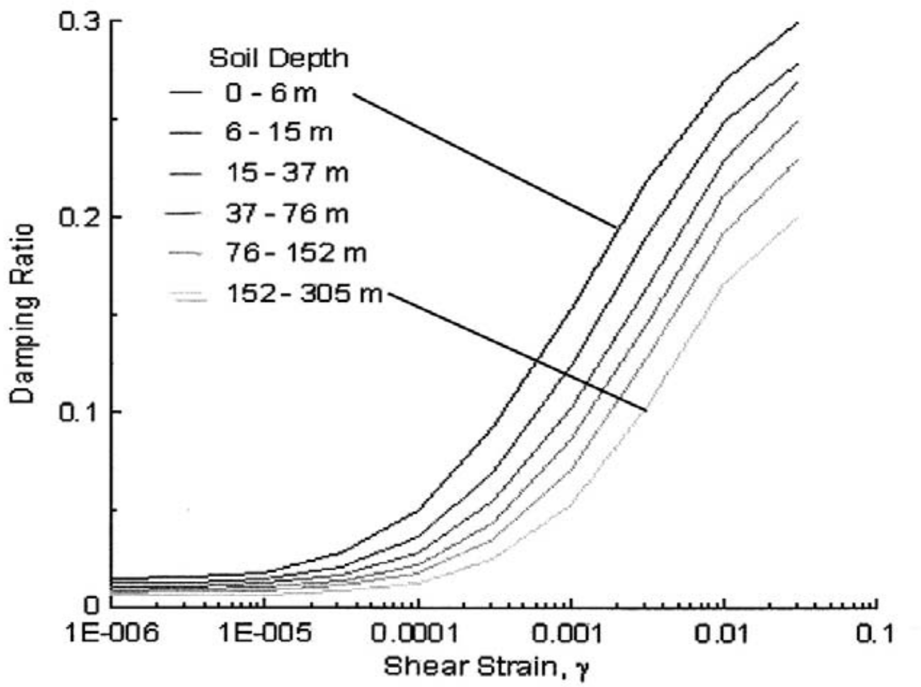
- ODOT **Figure 6-4**, **Figure 6-5** and **Figure 6-6** which presents a chart for estimating undrained residual shear strength for liquefied soils as a function of SPT blow counts (N'_{60}) and vertical effective stress.

Table 6-2. Correlations for estimating initial shear modulus (Kavazajjian, et al., 1997).

Reference	Correlation	Units (1)	Limitations
• Seed et al. (1984)	• $G_{max} = 220\ (K_2)_{max}\ (\sigma'_m)^{½}$ • • • $(K_2)_{max} = 20(N_1)_{60}^{⅓}$	• kPa	• $(K_2)_{max}$ is about 30 for very loose sands and 75 for very dense sands; about 80 to 180 for dense well graded gravels; Limited to cohesionless soils
• Imai and Tonouchi • (1982)	• $G_{max} = 15{,}560\ N_{60}^{0.68}$	• kPa	• Limited to cohesionless soils
• Mayne and Rix • (1993)	• $G_{max} = 99.5(P_a)^{0.305}(q_c)^{0.695}/(e_0)^{1.13}$	• kPa (2)	• Limited to cohesive soils; P_a = atmospheric pressure
• Notes:	• (1) 1 kPa = 20.885 psf • (2) P_a and q_c in kPa	•	•



Shear Modulus Reduction Curves



Damping Ratio Curves

Figure 6-1. Shear modulus reduction and damping ratio curves for sand (EPRI, 1993).

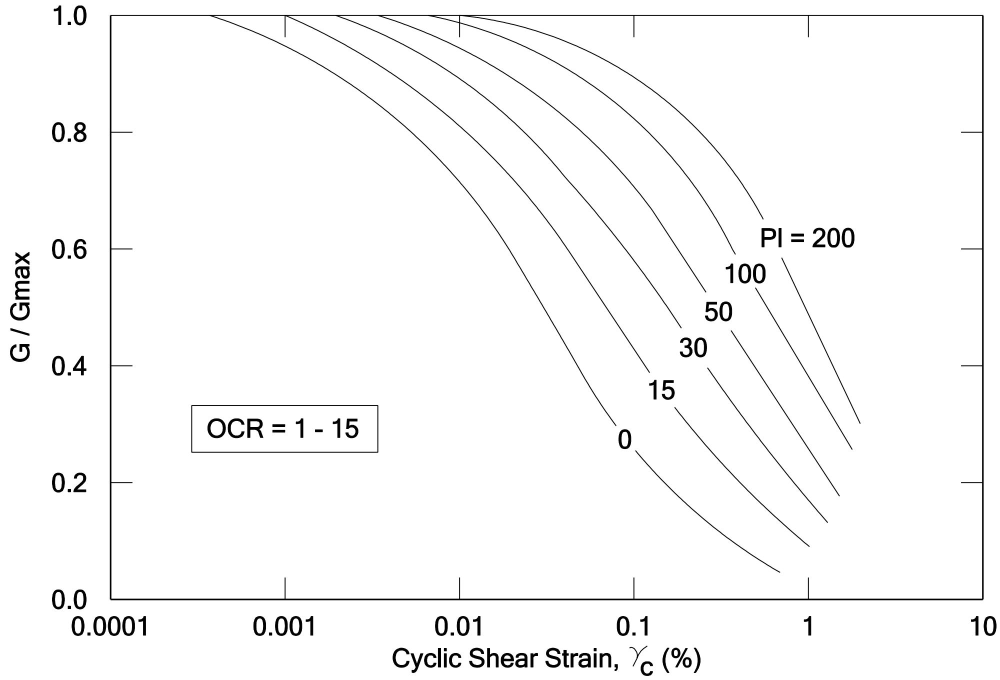


Figure 6-2. Variation of G/Gmax vs. cyclic shear strain for fine grained soils (redrafted from Vucetic and Dobry, 1991).

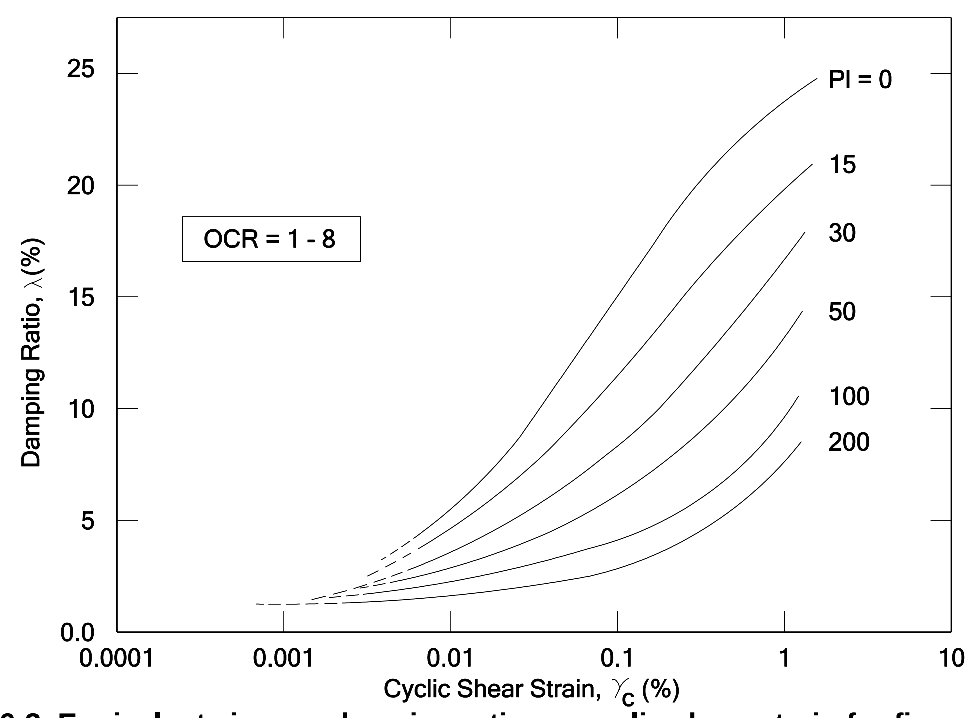


Figure 6-3. Equivalent viscous damping ratio vs. cyclic shear strain for fine grained soils (redrafted from Vucetic and Dobry, 1991).

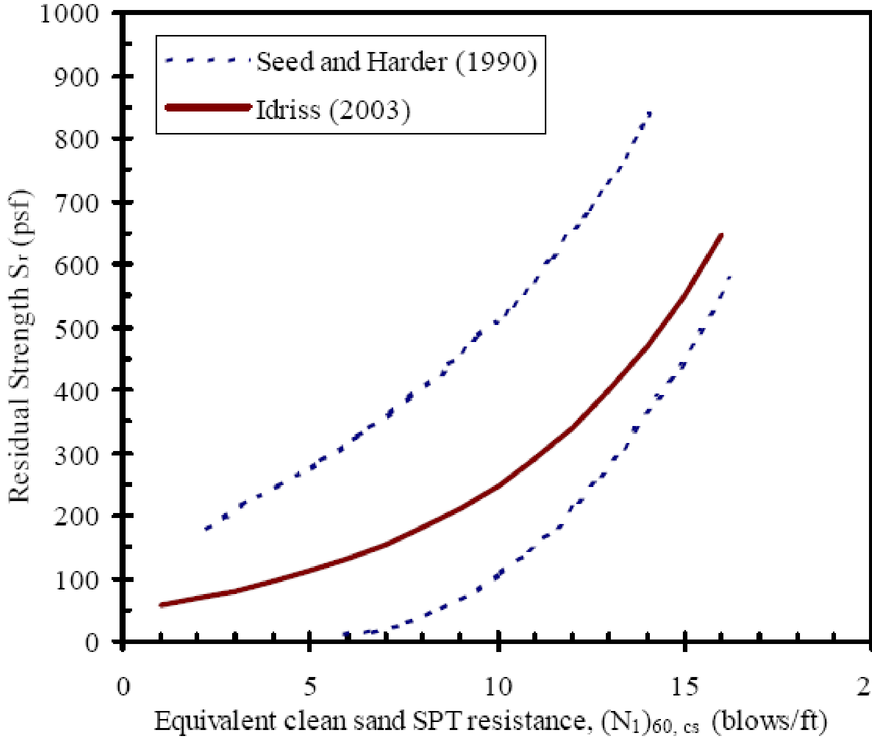


Figure 6-4. Residual undrained shear strength for liquefied soils as a function of SPT blow counts (Seed and Harder, 1990 and Idriss, 2003).

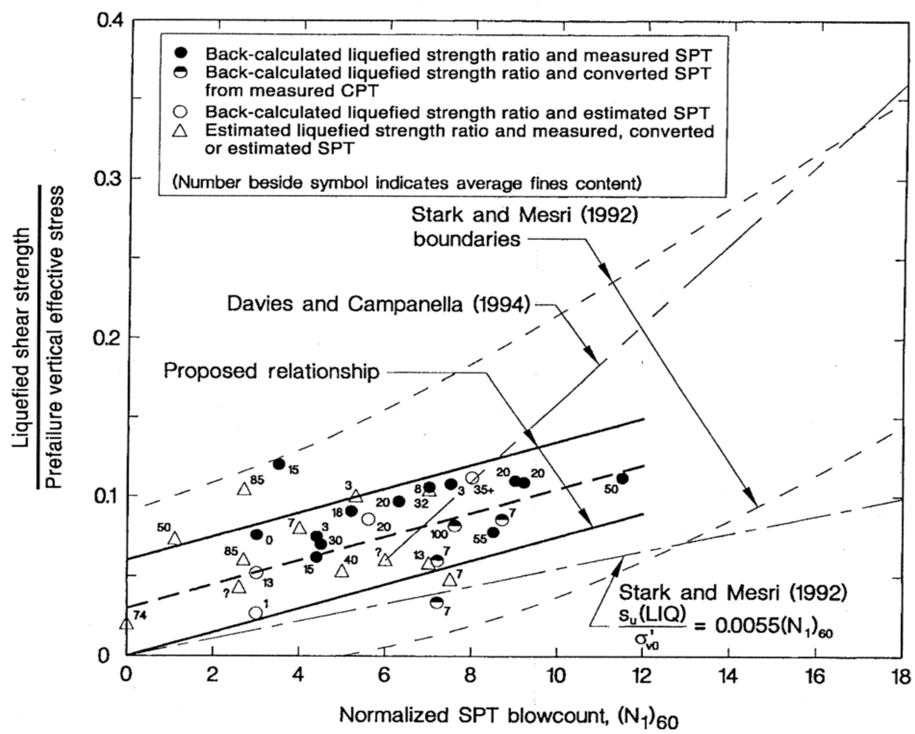


Figure 6-5. Estimation of residual strength ratio from SPT resistance (Olson and Stark, 2002).

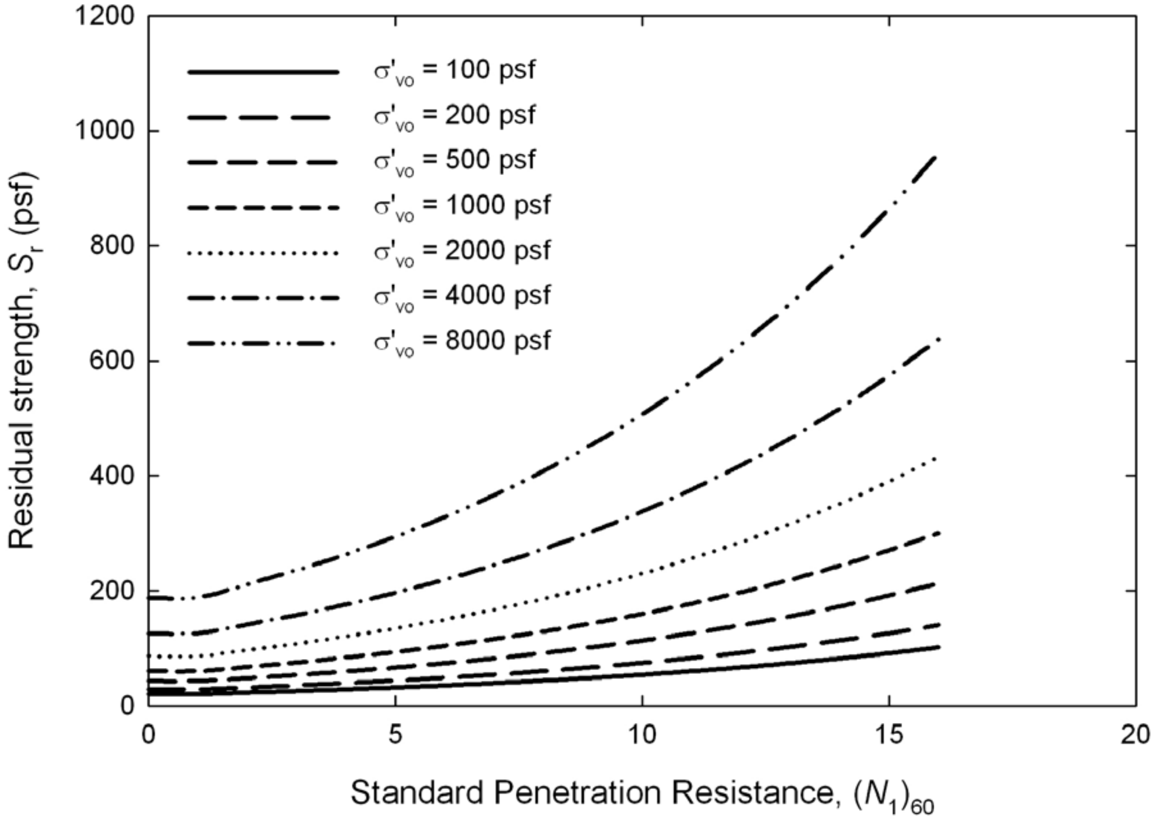


Figure 6-6. Variation of residual strength ratio with SPT resistance and initial vertical effective stress using Kramer-Wang model (Kramer, 2008).

6.5 Geotechnical Seismic Design Procedures

The geotechnical designer shall evaluate the site and subsurface conditions to the extent necessary to provide the following assessments and recommendations:

- An assessment of the seismic hazard,
- Determination of design ground motion values,
- Site characterization,
- Seismic analysis of the foundation materials and
- An assessment of the effects of the foundation response on the proposed structure.

Specific aspects of seismic foundation design generally consist of the following procedures:

- Determine the Peak Bedrock Acceleration (PGA), 0.2 and 1.0 second spectral accelerations for the bridge site from the 2002 USGS National Seismic Hazard Maps for the 500 and 1000-year return periods,
- Determine the Site Class and Site Coefficients based on the properties of the soil profile,
- Develop the Design Response Spectrum for the site or conduct ground response analysis if necessary,

- Determine liquefaction potential of foundation soils,
- If liquefaction is predicted:
    - o Estimate embankment deformations (lateral spread), bridge damage potential and approach fill performance for both the 500 and 1000-year events.
    - o Determine seismic fill settlement (potential downdrag and bridge damage if applicable).
    - o Provide soil properties for both the liquefied and non-liquefied soil conditions for use in the lateral load analysis of deep foundations.
    - o Determine reduced foundation resistances and their effects on proposed bridge foundation elements.
- Evaluate slope stability and settlement for non-liquefied soil conditions.
- Evaluate impacts of seismic geologic hazards including liquefaction, lateral spreading and slope instability on infrastructure, including estimated loads and deformations acting on the structure.
- Develop foundation spring values for dynamic loading (liquefied and non-liquefied soil conditions). Also recommendations regarding lateral springs for use in modeling abutment backfill soil resistance.
- Determine earthquake induced earth pressures (active and passive) and provide stiffness values for equivalent soil springs (if required) for retaining structures and below grade walls.
- Evaluate options to mitigate seismic geologic hazards, such as ground improvement, if appropriate.

Note that separate analysis and recommendations will be required for the 500 and 1000 year seismic design ground motions. A general design procedure is described in the flow chart shown in **Figure** 6-7 along with the information that should be supplied in the final geotechnical report.

STEP 1; Ground Motion Data

- Identify the seismic sources in the region affecting the site for the given return period (500 and 1000 yrs):
- Determine Peak Ground Accelerations (PGA), S_s, and S_1, from the 2002 USGS Seismic Hazard Maps for bedrock (Site Class B) conditions
- Determine Site Class and Site Coefficients, then develop the Design Response Spectrum representing the Uniform Seismic Hazard

STEP 2; Site Response Analysis

- Decide whether a site response analysis is warranted.
- If so, perform deaggregation of seismic hazard to determine principal M & R pairs.
- Select appropriate acceleration time histories and establish scaling factors or perform spectral matching.
- Generate the following:
  - PGA and 5% damped smoothed response spectra at the depth(s) of interest (e.g., ground surface, depth of pile/pier fixity).
  - Profiles of PGA, cyclic shear strain, and cyclic shear stress with depth.

STEP 3; Evaluate Liquefaction Potential & Effects (PGA≥0.10g)

- Estimate the cyclic resistance of the soils as a function of depth from in situ and/or lab data.
- Specify the cyclic loading at each depth from Step 2.
- Using the ratio of the cyclic resistance to the cyclic loading determine the potential for significant excess pore pressure generation and cyclic degradation of soil stiffness and strength.

Liquefaction Potential

STEP 3a; For foundation soils susceptible to liquefaction:

- Estimate post-liquefaction soil strengths
- Evaluate embankment stability and est. deformations
- Develop mitigation designs if required
- Assess the effects of liquefaction on foundation resistances and provide reduced foundation resistances under liquefied soil conditions. (CHECK DOWNDRAG)

No Liquefaction Potential

STEP 3a; Evaluate Non-liquefied Soil Response

- Dynamic settlement of foundation soils and downdrag potential
- Evaluate approach fill slope stability
- Estimate lateral approach fill displacements

STEP 4; Provide seismic foundation modeling parameters as appropriate (see Section 1.1.4 of BDDM):

Spread Footings

- Shear modulus; 'G' is dependent on the shear strain; generally a 'G' corresponding to a shear strain in the range of 0.20% to 0.02% is appropriate. For large magnitude events (M>7.5) and very high PGA (>0.6g), a 'G' corresponding to a shear strain of 1% is recommended. A ground response analysis may also be conducted to determine the appropriate shear strain value to use.
- Poisons ratio, ν
- Kp, Su, μ, γ

Piles

- p-y curve data for non-liquefied and liquefied soils
- p-y multipliers
- Designation as "end bearing" or "friction" piles for modeling axial stiffness

Shafts

- p-y curve data for non-liquefied and liquefied soils
- p-y multipliers

Figure 6-7. General Geotechnical Seismic Design Procedures

6.5.1 Design Ground Motion Data

6.5.1.1 Development of Design Ground Motion Data

In general, there are two options for the development of design ground motion parameters (response spectral ordinates) for seismic design. Both procedures are based on the USGS 2002 PSHA maps. These are described as follows:

1. **AASHTO General Procedure:** Use specification/code based hazard (2002 USGS Maps) with specification/code based site coefficients.

2. **Ground Response Analysis:** Use specification/code based hazard (2002 USGS Maps) with site specific ground response analysis.

Both methods take local site effects into account. For most routine structures at sites with competent soils (i.e., no liquefiable, sensitive, or weak soils), the first method (General Procedure), described in Article 3.4 of the *AASHTO Guide Specification for LRFD Seismic Bridge Design*, is sufficient to account for site effects. However, the importance of the structure, the ground motion levels and the soil and geological conditions of a site may dictate the need for a Ground Response Analysis (second method). The geotechnical engineer is responsible for developing and providing the design response spectra for the project.

6.5.1.2 AASHTO General Procedure

The standard method of developing the acceleration response spectrum is described in AASHTO, 2009. First, the peak ground acceleration (PGA), the short-period spectral acceleration (S_s) and the long-period spectral acceleration (S_1) are obtained from the 2002 USGS Seismic Hazard Maps for the location of the bridge. PGA, S_s, and S_1 are obtained for both the 500-year and 1000-year return periods. Then the soil profile is classified as one of six different site classes (A through F). This Site Class designation is then used to determine the “Site Coefficients”, F_{pga}, F_a and F_v**,** except for sites classified as Site Class F, which required a site-specific ground response analysis **Section 6.5.3.** These site coefficients are then multiplied by the peak ground acceleration (***Fpga*** x PGA), the short-period spectral acceleration (***Fa*** x Ss) and the long period spectral acceleration (***Fv*** x S_1) respectively and used to develop the site response spectrum. A program to develop the response spectra using the general procedure has been developed by the Bridge Section and can be accesses through the ODOT Bridge Section web page.

Once the response spectrum is developed the structural engineer can determine the Response Spectral Acceleration (per *AASHTO 2009 Guide Specifications*) for use in the seismic design of the structure.

6.5.1.3 Response Spectra and Analysis for Liquefied Soil Sites

Site coefficients have not been developed for liquefied soil conditions. For this case site-specific analysis is required to estimate ground motion characteristics. The *AASHTO Guide Specifications for LRFD Seismic Bridge Design* states that at sites where soils are predicted to liquefy the bridge shall be analyzed and designed under two configurations, the nonliquefied condition and liquefied soil condition described as follows:

- **Nonliquefied Configuration:** The structure is analyzed and designed, assuming no liquefaction occurs by using ground response spectrum and soil design parameters based on nonliquefied soil conditions.

- **Liquefied Configuration:** The structure is reanalyzed and designed under liquefied soil conditions assuming the appropriate residual resistance for lateral and axial deep foundation response analyses consistent with liquefied soil conditions (i.e., modified P-Y curves, modulus of subgrade reaction, T-Z curves, axial soil frictional resistance). The design spectrum should be the same as that used in nonliquefied configuration.

A site-specific response spectrum may be developed for the "Liquefied Configuration" based on a ground response analysis that utilizes non-linear, effective stress methods, which properly account for pore pressure buildup and stiffness degradation of the liquefiable soil layers see **Section 6.5.1.4**. The decision to complete a ground response analysis where liquefaction is anticipated should be made by the geotechnical designer based on the site geology and characteristics of the bridge being designed. The design response spectrum resulting from the ground response analyses shall not be less than two-thirds of the spectrum developed using the general procedure for the non-liquefied soil condition.

6.5.1.4 Ground Response Analysis

For most projects, the General Procedure as described in Article 3.4.1 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* is appropriate and sufficient for determining the seismic hazard and site response spectrum. However, it may be appropriate to perform a site-specific evaluation for cases involving special aspects of seismic hazard (e.g., near fault conditions, high ground motion values, coastal sites located in relatively close proximity to the CSZ source), specific soil profiles, and essential bridges. The results of the site-specific response analysis may be used as justification for a reduction in the spectral response ordinates determined using the standard AASHTO design spectrum (General Procedure) representing the Uniform Seismic Hazard.

Site specific ground response analyses (GRA) are required for Site Class "F" soil profiles, and may be warranted for other site conditions or project requirements. Site Class "F" soils are defined as follows:

- Peat or highly organic clays, greater than 10 ft in thickness,
- Very high plasticity clays (H > 25 ft with PI > 75),
- Very thick soft/medium stiff clays (H >120 ft),

Other conditions under which a ground response analysis should be considered are listed below:

- Very important or critical structures or facilities,
- Liquefiable Soil Conditions. For liquefiable soil sites, it may be desirable to develop response spectra that take into account increases in pore water pressure and soil softening. This analysis results in a response spectra that is generally lower than the nonliquefied response spectra except for spectral accelerations in the higher period range (above 1.0 second). A nonlinear effective stress analysis may also be necessary to refine the standard liquefaction analysis based on Seed's Simplified (SPT) Method (or others) with information from a GRA. This is especially true if liquefaction mitigation designs are proposed. The cost of liquefaction mitigation is sometimes very large and a more detailed analysis to verify the potential, and extent, of liquefaction is usually warranted.
- Very deep soil deposits or thin (<40 – 50 feet) soil layers over bedrock.
- To obtain better information for evaluating lateral deformations, near surface soil shear strain levels or deep foundation performance.

- To obtain ground surface PGA values for abutment wall or other design.

Procedures for conducting a site specific ground response analysis are described in Article 3.4.3. of the AASHTO guide specifications and in Chapter 4 of Kavazanjian, et al. (1997).

A ground response analysis evaluates the response of a layered soil deposit subjected to earthquake motions. One-dimensional, equivalent-linear models are commonly utilized in practice. This model uses an iterative total stress approach to estimate the nonlinear elastic behavior of soils. Modified versions of the numerical model SHAKE (e.g., ProSHAKE, SHAKE91, SHAKE2000) are routinely used to simulate the propagation of seismic waves through the soil column and generate output consisting of ground motion time histories at selected locations in the soil profile, plots of ground motion parameters with depth (e.g., PGA, cyclic shear stress, cyclic shear strain), and acceleration response spectra at depths of interest. The program calculates the induced cyclic shear stresses in individual soil layers which may be used in liquefaction analysis.

The equivalent linear model provides reasonable results for small to moderate cyclic shear strains (less than about 1 to 2 percent) and modest accelerations (less than about 0.3 to 0.4g) (Kramer and Paulsen, 2004). Equivalent linear analysis cannot be used where large strain incompatibilities are present, to estimate permanent displacements, or to model development of pore water pressures in a coupled manner. Computer programs capable of modeling non-linear, effective stress soil behavior are recommended for sites where high ground motion levels are indicated and it is anticipated that moderate to large shear strains will be mobilized. These are typically sites with soft to medium stiff fine-grained soils or saturated deposits of loose to medium dense cohesionless soils.

Basically, the input parameters required for site specific seismic response analysis include soil layering (thickness), standard geotechnical index properties for the soils, dynamic soil properties for each soil layer, the depth to bedrock or firm soil interface, and a set of ground motion time histories representative of the primary seismic hazards in the region.

Soil parameters required by the equivalent linear models include the shear wave velocity or initial (small strain) shear modulus and unit weight for each soil layer, and curves relating the shear modulus and damping ratio as a function of shear strain (see **Section 6.4.1** and **Figure 6-1**, **Figure** 6-2 and **Figure 6-3** for examples).

6.5.1.5 Selection of Time Histories for Ground Response Analysis

AASHTO (2009) allows two options for the selection of time histories to use in ground response analysis. The two options are:

a) Use a suite of 3 response-spectrum-compatible time histories with the design response spectrum developed enveloping the maximum response, or
b) Use of at least 7 time histories and develop the design spectrum as the mean of the computed response spectra.

For both options, the time histories shall be developed from the representative recorded earthquake motions, or in special instances synthetic ground motions may be used with approval of ODOT. The time histories for these applications shall have characteristics that are representative of the seismic environment of the site and the local site conditions, including the response spectrum for the site.

Analytical techniques used for spectral matching shall be demonstrated to be capable of achieving seismologically realistic time series. The time histories should be scaled to the approximate level of the design response spectrum in the period range of significance (i.e., $0.5 < T < 2.0$).

The procedures for selecting and scaling time histories for use in ground motion response analysis can be summarized as follows:

1. Identify the target response spectra to be used to develop the time histories. The target spectra are obtained from the 2002 USGS Seismic Hazard Maps for top-of-rock locations (base of soil column). Two spectra are required, one for the 500-yr return event and one for the 1000-yr event.
2. Identify the seismic sources that contribute to the seismic hazard for the site, considering the desired probability of exceedance (i.e., 500 and 1000-yr return periods). Use the deaggregation information for the 2002 USGS Seismic Hazard maps to obtain information on the primary sources that affect the site. All seismic sources (M-R pairs) that contribute more than 5% to the hazard in the period range of interest should be considered.
3. Select time histories to be considered for the analysis, considering tectonic environment and style of faulting (subduction zone, Benioff zone, or shallow crustal faults), seismic source-to-site-distance, earthquake magnitude, duration of strong shaking, peak acceleration, site subsurface characteristics, predominant period, etc. In areas where the hazard has a significant contribution from both the Cascadia Subduction Zone (CSZ) and from crustal sources (e.g., Portland and much of the Western part of the state) both earthquake sources need to be included in the analysis and development of a site specific response spectra. In cases such as this, it is recommended that the ground response analysis be conducted using a collection of time histories that include at least 3 motions representative of subduction zone events and 3 motions appropriate for shallow crustal earthquakes with the design response spectrum developed considering the mean spectrum of each of these primary sources.

At sites where the uniform hazard is dominated by a single source, three (3) time histories, representing the seismic source characteristics, may be used and the design response spectrum determined by enveloping the caps of the resulting response spectra.

4. Scale the time histories to match the target spectrum as closely as possible in the period range of interest prior to spectral matching. Match the response spectra from the recorded earthquake time histories to the target spectra using methods that utilize either time series adjustments in the time domain or adjustments made in the frequency domain. See *AASHTO Guide Specifications for LRFD Seismic Bridge Design* and Kramer (1996) for additional guidance on these techniques.
5. Once the time history(ies) have been spectrally matched, they can be used directly as input into the ground response analysis programs to develop response spectra and other seismic design parameters. Five percent (5%) damping is typically used in all site response analysis.

The results of the dynamic soil response modeling should be presented as the “mean”, “average” and “85th percentile” curves from all of the output response spectra. A “smoothed” response spectra may be obtained by enveloping the peaks of the 85% percentile response curve. The resulting design spectrum may be lower than the 85th percentile curve outside of the period range of interest. Engineering judgment will be required to account for possible limitations of the response modeling. For example, equivalent linear analysis methods may overemphasize spectral response where the predominant period of the soil profile closely matches the predominant period of the bedrock motion. Final modification of the design spectrum must provide representative constant velocity and constant displacement portions of the response. Site-specific response spectra may be used for design however the lower limit shall be no less than 2/3rd of the AASHTO response spectrum using the General Procedure. An example response spectrum is attached in **Appendix 6-B**.

At some bridge sites, the subsurface conditions (soil profile) may change dramatically along the length of the bridge and more than one response spectrum may be required to represent segments of the bridge with different soil profiles. If the site conditions dictate the need for more than one response spectrum for the bridge, the design response spectrum should be developed by combining the individual spectra into a composite spectrum that envelopes the spectral acceleration values of the individual spectra.

Nonlinear effective stress analysis methods such as D-MOD, DESRA and others may be used to develop response spectra especially at sites where liquefaction of foundation soils is likely. All non-linear, effective stress modeling and analysis will require an independent peer reviewer with expertise in this type of analysis. In some cases, the response spectra resulting from a nonlinear effective stress analysis may result in spectral acceleration values exceeding the 2/3 AASHTO general procedure criteria in the range of higher periods. If this is the case, the higher response spectra values of the two methods shall be used. For non-linear analysis methods the lower limit shall be no less than 2/3rd of the AASHTO response spectrum developed using the General Procedure.

6.5.1.6 Ground Motion Parameters for Other Structures

For buildings, restrooms, shelters, and other non-transportation structures, specification based seismic design parameters required by the 2003 IBC should be used. The seismic design requirements of the 2003 IBC are based on a risk level of 2 percent PE in 50 years. The 2 percent PE in 50 years risk level corresponds to the maximum considered earthquake. The 2003 IBC identifies procedures to develop a maximum considered earthquake acceleration response spectrum, and defines the design response spectrum as two-thirds of the value of the maximum considered earthquake acceleration response spectrum.

Site response shall be in accordance with the 2003 IBC. As is true for transportation structures, for critical or unique structures or for sites characterized as soil profile Type F (thick sequence of soft soils or liquefiable soils), site response analysis may be required.

6.5.1.7 Bedrock versus Ground Surface Acceleration

Soil amplification factors that account for the presence of soil over bedrock with regard to the estimation of peak ground acceleration (PGA) are directly incorporated into the development of the general procedure for developing response spectra for structural design of bridges and similar structures in the *AASHTO LRFD Bridge Design and Guide Specifications* and for the structural design of buildings and non-transportation related structures in the 2003 IBC. Additional amplification factors should not be applied to peak bedrock accelerations when code based response spectra are used. However, amplification factors should be applied to the peak bedrock acceleration to determine the peak ground acceleration (PGA) for liquefaction assessment, such as for use with the Simplified Method **Section 6.5.5.2**, and for the estimation of seismic earth pressures and inertial forces for retaining wall and slope design. For liquefaction assessment and retaining wall and slope design, the Site Factors (F_{pga}) presented in AASHTO 3.10.3.2 may be applied to the bedrock PGA used to determine the ground surface acceleration, unless a site specific evaluation of ground response is conducted.

6.5.2 Liquefaction Analysis

Liquefaction has been one of the most significant causes of damage to bridge structures during past earthquakes. Liquefaction can damage bridges and structures in many ways including:

- Bearing failure of shallow foundations founded above liquefied soil;
- Liquefaction induced ground settlement;
- Lateral spreading of liquefied ground;
- Large displacements associated with low frequency ground motion;
- Increased earth pressures on subsurface structures;
- Floating of buoyant, buried structures; and
- Retaining wall failure.

Liquefaction refers to the significant loss of strength and stiffness resulting from the generation of excess pore water pressure in saturated, cohesionless soils. Liquefaction can occur in sand and non-plastic to low plasticity silt-rich soils, and in confined gravel layers; however, it is most common in sands and silty sands. For a detailed discussion of the effects of liquefaction, including the types of liquefaction phenomena, liquefaction-induced bridge damage, evaluation of liquefaction susceptibility, post liquefaction soil behavior, deformation analysis and liquefaction mitigation techniques refer to Dickenson, et al. (2002).

Liquefaction hazard assessment includes identifying soils susceptible to liquefaction on the basis of composition and cyclic resistance, evaluating whether the design earthquake loading will initiate liquefaction, and estimating the potential effects of liquefaction on the planned facility. Potential effects of liquefaction on soils and foundations include the following:

- Loss in strength in the liquefied layer(s)
- Liquefaction-induced ground settlement
- Flow failures, lateral spreading, and slope instability.

Due to the high cost of liquefaction mitigation measures, it is important to identify liquefiable soils and the potential need for mitigation measures early on in the design process (during the DAP (TS&L) phase) so that appropriate and adequate funding decisions are made. The following sections provide ODOT's policies regarding liquefaction and a general overview of liquefaction hazard assessment and its mitigation. .

6.5.2.1 Liquefaction Design Policies

All new bridges, bridge widening projects and retaining walls in areas with seismic acceleration coefficients, or PGA, greater than or equal to 0.10g should be evaluated for liquefaction potential. The maximum considered liquefaction depth shall be limited to 75 feet. The potential for liquefaction and limited strength and stiffness reductions due to pore pressure increase caused by ground shaking may be considered below this depth on the basis of cyclic laboratory test data and/or the use of non-linear, effective stress analysis techniques. All non-linear, effective stress modeling and analysis will require an independent peer reviewer with expertise in this type of analysis.

Bridges scheduled for seismic retrofit should also be evaluated for liquefaction potential if they are in a seismic zone with an acceleration coefficient (or PGA) ≥ 0.10g.

In general, liquefaction is conservatively predicted to occur when the factor of safety against liquefaction (FS_L) is less than 1.1. A factor of safety against liquefaction of 1.1 or less also indicates the potential for liquefaction-induced ground movement (lateral spread and settlement). Soil layers with FS_L between 1.1 and 1.4 will have reduced soil shear strengths due to excess pore pressure generation. For soil layers with FS_L greater than 1.4, excess pore pressure generation is considered negligible and the soil does not experience appreciable reduction in shear strength.

If liquefaction is predicted based on the Simplified Method **Section 6.5.2.2**, and the effects of liquefaction require mitigation measures, a more thorough ground response analysis (e.g. SHAKE, DMOD) is recommended to verify and substantiate the predicted, induced ground motions. This procedure is especially recommended for sites where liquefaction potential is marginal ($0.9 < FS_L < 1.10$). It is also important to determine whether the liquefied soil layer is stratigraphically (laterally) continuous and oriented in a manner that will result in lateral spread or other adverse impact to the bridge.

Groundwater: The groundwater level to use in the liquefaction analysis should be determined as follows:

- **Static Groundwater Condition:** Use the estimated, average annual groundwater level. Perched water tables should only be used if water is estimated to be present in these zones more than 50% of the year.

- **Tidal Areas:** Use the mean high tide elevation.

- **Adjacent Stream, Lake or Standing Water Influence:** Use the estimated, annual, average elevation for the wettest (6 month) seasonal period.

Note that groundwater levels measured in borings advanced using water or other drilling fluids may not be indicative of true static groundwater levels. Water in these borings should be allowed to stabilize over a period of time to insure measured levels reflect true static groundwater levels. Groundwater levels are preferably measured and monitored using piezometers, taking measurements throughout the climate year to establish reliable static groundwater levels taking seasonal effects into account.

6.5.2.2 Methods to Evaluate Liquefaction Potential

Evaluation of liquefaction potential should be based on soil characterization using in-situ testing methods such as Standard Penetration Tests (SPT) and Cone Penetration Tests (CPT). Liquefaction potential may also be evaluated using shear wave velocity (V_s) testing and Becker Penetration Tests (BPT); however, these methods are not preferred and are used less frequently than SPT or CPT methods. V_s and BPT testing may be appropriate in soils difficult to test using SPT and CPT methods such as gravelly soils though, in the absence of fine grained soil layers that may act as poorly drained boundaries, these soils often have a low susceptibility to liquefaction potential due to high permeability and rapid drainage. If the CPT method is used, SPT sampling and soil gradation testing shall still be conducted to obtain direct information on soil type and gradation parameters for use in liquefaction susceptibility assessment.

Preliminary Screening: A detailed evaluation of liquefaction potential is not required if any of the following conditions are met:

- The bedrock PGA (or Acceleration Coefficient, As) is less than 0.10g,
- The ground water table is more than 75 feet below the ground surface,
- The soils in the upper 75 feet of the profile have a minimum SPT resistance, corrected for overburden depth and hammer energy ($N1_{60}$), of 25 blows/ft, or a cone tip resistance q_{ciN} of 150 tsf.
- All soils in the upper 75 feet are classified as "cohesive", and have a PI ≥ 18. Note that cohesive soils with PI ≥ 18 may still be very soft or exhibit sensitive behavior and could therefore undergo significant strength loss under earthquake shaking. This criterion should be used with care and good engineering judgment. Recent advances in the screening and evaluation of fine-grained soils for strength loss during cyclic loading can be found in Bray and Sancio, (2006) and Boulanger and Idriss, (2006, 2007).

Simplified Procedures: Simplified Procedures should always be used to evaluate the liquefaction potential even if more rigorous methods are used to supplement or refine the analysis. The Simplified Procedure was originally developed by Seed and Idriss (1971) and has been periodically modified and improved since. It is routinely used to evaluate liquefaction resistance in geotechnical practice.

The procedures described in Section 3.4 of the report "*Assessment and Mitigation of Liquefaction Hazards to Bridge Approach Embankments in Oregon*", (Dickenson et al, 2003) should be followed for assessing the liquefaction potential of soil by the Simplified Procedures.

The paper titled "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils" (Youd et al., (2001) also provides a state of the practice summary of the Simplified Procedures for assessment of liquefaction susceptibility. This paper resulted from a 1996 workshop of liquefaction experts sponsored by the National Center for Earthquake Engineering Research and the National Science Foundation with the objective being to gain consensus on updates and augmentation of the Simplified Procedures. Youd et al. (2001) provide procedures for evaluating liquefaction susceptibility using SPT, CPT, V_s, and BPT criteria.

The Simplified Procedures are based on the evaluation of both the cyclic resistance ratio (CRR) of a soil layer (i.e., the cyclic shear stress required to cause liquefaction) and the earthquake induced cyclic shear stress ratio (CSR). The resistance value (CRR) is estimated based on empirical charts relating the resistance available to specific index properties (i.e. SPT, CPT, BPT or shear wave velocity values) and corrected to an equivalent magnitude of 7.5 using a magnitude scaling factor. Youd et al. (2001) provide the empirical liquefaction resistance charts for both SPT and CPT data to be used with the simplified procedures.

The basic form of the simplified procedures used to calculate the earthquake induced CSR for the Simplified Method is shown in the following equation:

Equation 6.1

$$CSR_{eq} = \frac{\tau_{av}}{\sigma_{vo}'} = 0.65 \left(\frac{a_{max}}{g}\right)\left(\frac{\sigma_{vo}}{\sigma_{vo}'}\right) r_d$$

Where: $'_{av}$ = average or uniform earthquake induced cyclic shear stress

a_{max} = peak horizontal acceleration at the ground surface accounting for site amplification effects (ft/sec^2)

g = acceleration due to gravity (ft/sec^2)

σ_o = initial total vertical stress at depth being evaluated (lb/ft^2)

σ_o' = initial effective vertical stress at depth being evaluated (lb/ft^2)

r_d = stress reduction coefficient

The factor of safety against liquefaction is defined by:

FS_{liq} = CRR/CSR

The use of the SPT for the Simplified Procedure has been most widely used and has the advantage of providing soil samples for fines content and gradation testing. The CPT provides the most detailed soil stratigraphy, is less expensive, can simultaneously provide shear wave velocity measurements, and is more reproducible. If the CPT is used, soil samples shall be obtained using the SPT or other methods so that detailed gradational and plasticity analyses can be conducted. The use of both SPT and CPT procedures can provide the most detailed liquefaction assessment for a site.

Where SPT data is used, the sampling and testing procedures should include:

- Documentation on the hammer efficiency (energy measurements) of the system used.
- Correction factors for borehole diameter, rod length and sampler liners should be used, where appropriate.
- Where gravels or cobbles are present, the use of short interval adjusted SPT N values may be effective for estimating the N values for the portions of the sample not affected by gravels or cobbles.
- Blowcounts obtained using non-standard samplers such as the Dames and Moore or modified California samplers shall not be used for liquefaction evaluations.

Limitations of the Simplified Procedures: The limitations of the Simplified Procedures should be recognized. The Simplified Procedures were developed from empirical evaluations of field observations. Most of the case history data was collected from level to gently sloping terrain underlain by Holocene-age alluvial or fluvial sediment at depths less than 50 feet. Therefore, the Simplified Procedures are applicable to only these site conditions. Caution should be used for evaluating liquefaction potential at depths greater than 50 feet using the Simplified Procedure. In addition, the Simplified Procedures estimate the trend of earthquake induced cyclic shear stress ratio with depth based on a coefficient, r_d, which becomes highly variable at depths below about 40 feet.

As an alternative to the use of Equation 6.1, one dimensional ground response analyses should be used to better determine the maximum earthquake induced shear stresses at depths greater than about 50 feet. Equivalent linear, total stress computer programs (Shake2000, ProShake or other equivalent program) may be used for this purpose.

Nonlinear Effective Stress Methods: An alternative to the simplified procedures for evaluating liquefaction susceptibility is to perform a nonlinear, effective stress site response analysis utilizing a computer code capable of modeling pore water pressure generation and dissipation (D-MOD2000, DESRA, FLAC). These are more rigorous analyses and they require additional soil parameters, validation by the practitioner, and additional specialization.

The advantages of this method of analysis include the ability to assess liquefaction at depths greater than 50 feet, the effects of liquefaction and large shear strains on the ground motion, and the effects of higher accelerations that can be more reliably evaluated. In addition, seismically induced deformation can be estimated, and the timing of liquefaction and its effects on ground motion at and below the ground surface can be assessed.

Several non-linear, effective stress analysis programs can be used to estimate liquefaction susceptibility at depth. However, few of these programs are being used by geotechnical designers at this time. In addition, there has been little verification of the ability of these programs to predict liquefaction at depths greater than 50 feet because there are few well documented sites of deep liquefaction.

Due to the highly specialized nature of these more sophisticated liquefaction assessment approaches, an independent peer review by an expert in this type of analysis is required to use nonlinear effective stress methods for liquefaction evaluation.

Magnitude and PGA for Liquefaction Analysis: The procedures described in **Section 6.3.2**, and in Dickenson et al. (2002), should be used to determine the appropriate earthquake magnitude and peak ground surface acceleration to use in the simplified procedure for liquefaction analysis. If a site specific ground response analysis is used to determine the peak ground surface acceleration(s) for use in liquefaction analyses, this value should be representative of the cyclic loading induced by the M-R pair(s) of interest. It is anticipated that PGA values obtained from site-specific ground response analysis will differ from the PGA determined by the *AASHTO General Procedure for the Uniform Seismic Hazard*. The PGA and magnitude values used in the liquefaction hazard analysis shall be tabulated for all considered seismic sources.

Magnitude Scaling Factors (MSF): Magnitude scaling factors are required to adjust the cyclic stress ratios (either CRR or CSR) obtained from the Simplified Method (based on M = 7.5) to other magnitude earthquakes. The range of Magnitude Scaling Factors recommended in the 1996 NCEER Workshop on Evaluation of Liquefaction Resistance of Soils (Youd, et. al., 2001) is recommended. Below magnitude 7.5, a range is provided and engineering judgment is required for selection of the MSF. Factors more in line with the lower bound range of the curve are recommended. Above magnitude 7.5 the factors recommended by Idriss are recommended. This relationship is presented in the graph (**Figure 6-8**) and the equation of the curve is: $MSF = 10^{2.24} / M^{2.5}$

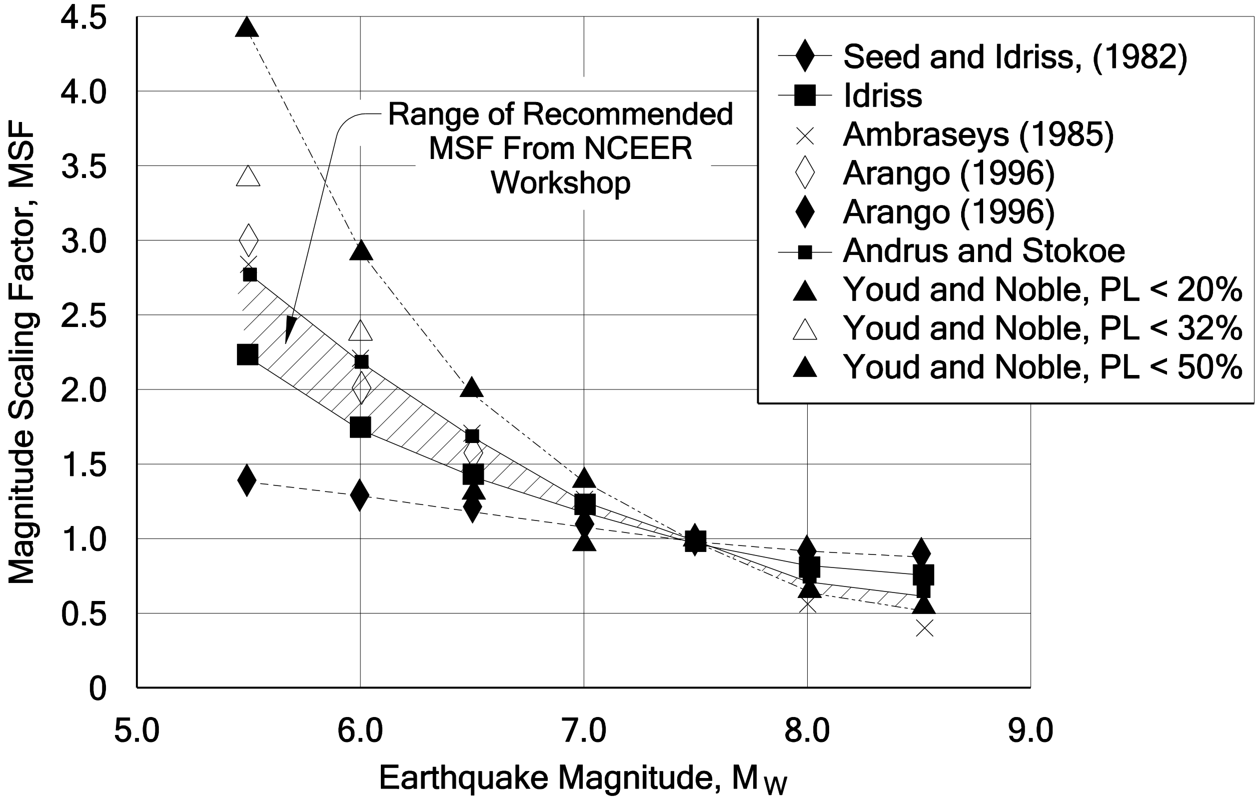


Figure 6-8. Magnitude Scaling Factors Derived by Various Investigators (redrafted from 1996 NCEER Workshop Summary Report)

6.5.2.3 Liquefaction Induced Settlement

Both dry and saturated deposits of loose granular soils tend to densify and settle during earthquake shaking. Settlement of unsaturated (dry) granular deposits is discussed in **Section 6.5.4**. If the Simplified Procedure is used to evaluate liquefaction potential, liquefaction induced ground settlement of saturated granular deposits should be estimated using the procedures by Tokimatsu and Seed (1987) or Ishihara and Yoshimine (1992). The Tokimatsu and Seed (1987) procedure estimates the volumetric strain as a function of earthquake induced CSR and corrected SPT blowcounts. The Ishihara and Yoshimine (1992) procedure estimates the volumetric strain as a function of factor of safety against liquefaction, relative density, and corrected SPT blowcounts or normalized CPT tip resistance. Example charts used to estimate liquefaction induced settlement using the Tokimatsu and Seed procedure and the Ishihara and Yoshimine procedure are presented as **Figure 6-9** and **Figure 6-10**, respectively.

Non-plastic to low plasticity silts (PI ≤ 12) have also been found to be susceptible to volumetric strain following liquefaction. In cases where saturated silt is liquefiable the post-cyclic loading volumetric strain should be estimated from cyclic laboratory testing, or approximately from the relationship developed by Ishihara and Yoshimine.

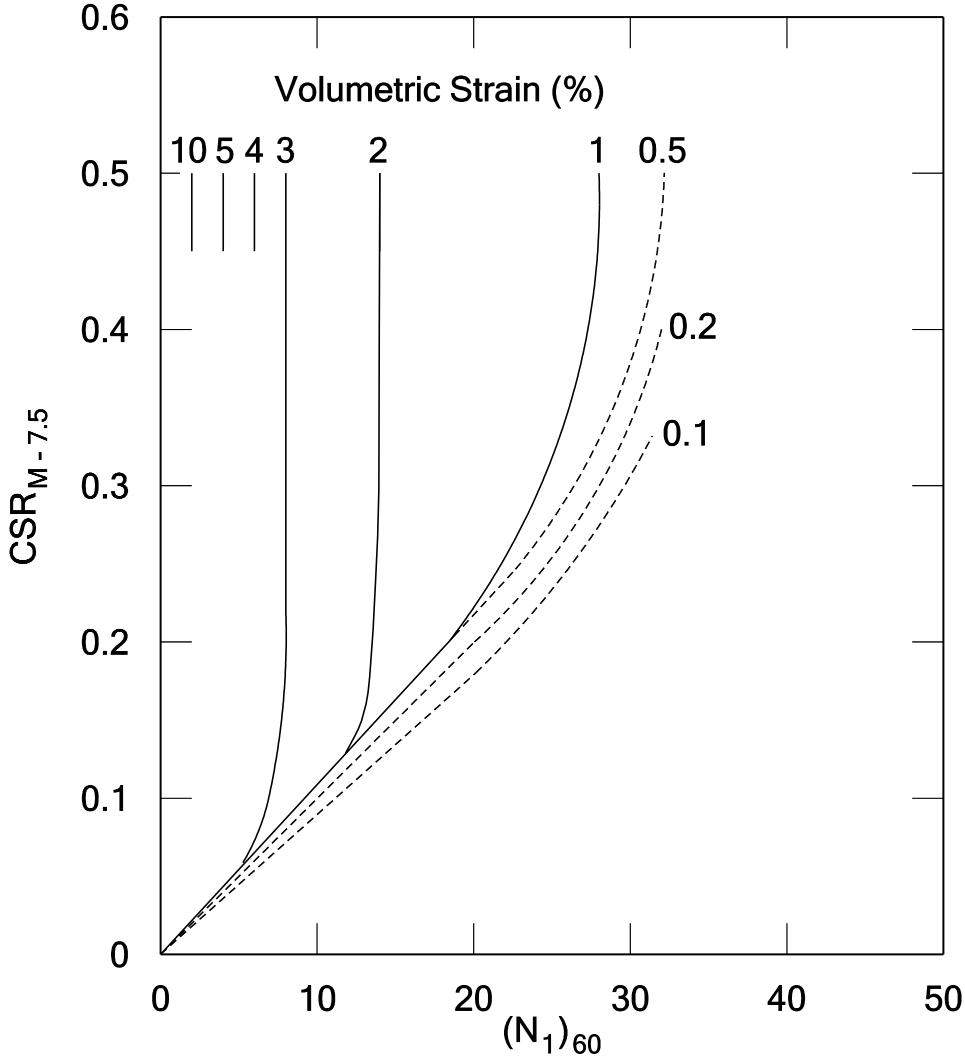


Figure 6-9. Liquefaction induced settlement estimated using the Tokimatsu & Seed procedure (redrafted from Tokimatsu and Seed, 1987).

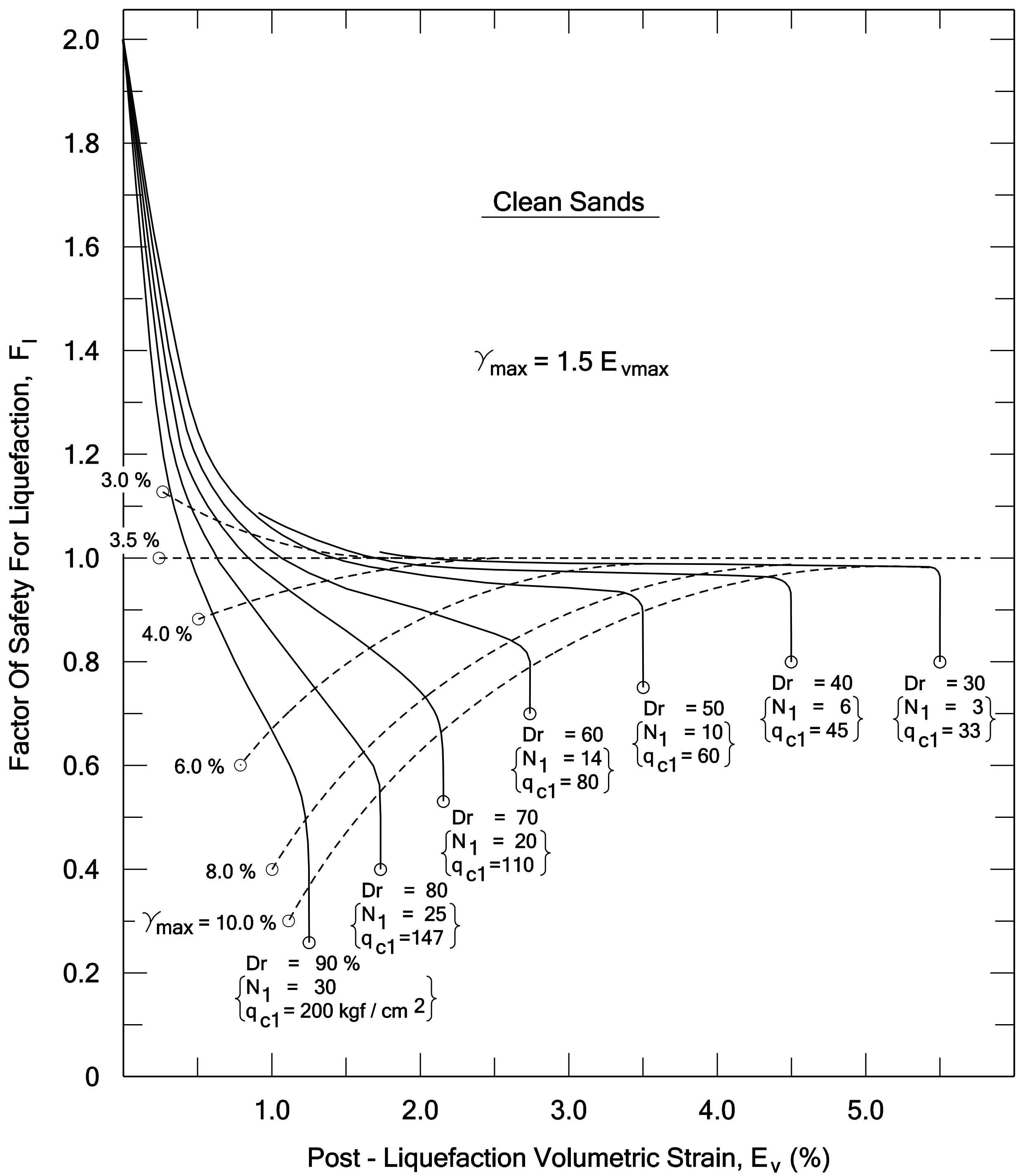


Figure 6-10. Liquefaction induced settlement estimated using the Ishihara and Yoshimine procedure. (redrafted from Ishihara and Yoshimine, 1992).

6.5.2.4 Residual Strength Parameters

Liquefaction induced instability is strongly influenced by the residual strength of the liquefied soil. Instability occurs when the shear stresses required to maintain equilibrium exceed the residual strength of the soil deposit. Evaluation of residual strength of a liquefied soil deposit is one of the most difficult problems in geotechnical practice. A variety of methods are available to estimate the residual strength of liquefied soils. The procedures recommended in Section 6.4.1, include Seed and Harder (1990), Olson and Stark (2002), Idriss (2003) and Kramer (2008). Other methods as described in Dickenson, et. al., (2002), as well as the results of cyclic laboratory testing, may also be used.

All of these methods estimate the residual strength of a liquefied soil deposit based on an empirical relationship between residual undrained shear strength and equivalent clean sand SPT blowcounts using the results of back-calculation of the apparent shear strengths from case histories, including flow slides. All of these methods should be used to calculate the residual undrained shear strength and an average value selected based on engineering judgment, taking into consideration the basis and limitations of each correlation method.

6.5.3 Slope Stability and Deformation Analysis

Sloping earth structures and native slopes can become unstable due to: 1) liquefaction, or increased pore pressures in soils, associated with a seismic event, 2) inertial effects associated with ground accelerations, or 3) both. The methods described in this section, in Dickenson et. al (2002), and the reference, should be used to assess seismic slope stability and for estimating ground displacements. The slopes and conditions requiring such assessments and analysis are described in **Section 6.2.4**.

If liquefaction is not present, ground accelerations may produce inertial forces within the slope or embankment that could exceed the strength of the foundation soils and result in slope failure and large displacements. At these sites a pseudo-static analysis, which includes earthquake induced inertia forces, is conducted to determine the general stability of the slope or embankment under these conditions, as described in **Section 6.5.3.1**. The pseudo-static analysis is also used to determine the yield acceleration which is then used in estimating slope or embankment displacements.

If soils vulnerable to cyclic degradation (liquefiable soils, sensitive soils, brittle soils) are present slope instability may develop in the form of flow failures, lateral spreading or other large embankment deformations. Conventional slope stability analysis methods are typically conducted for liquefiable soil sites using residual strength parameters to model the liquefied soils. The results of the analysis are used to assess the potential for flow failures (FOS<1.0) and for use in displacement analysis. Under liquefied soil conditions, slope stability is usually modeled in the "post-earthquake" condition without including any inertial force from the earthquake ground motions (a de-coupled analysis) as described in **Section 6.5.3.2**.

6.5.3.1 Pseudo-static Analysis

Pseudo-static slope stability analyses should be used to evaluate the seismic stability of slopes and embankments. The pseudo-static analysis shall consist of conventional limit equilibrium static slope stability analysis, using horizontal and vertical pseudo-static acceleration coefficients (kh and kv) that act upon the critical failure mass. Refer to Dickenson et al. (2002) for a discussion and detailed guidance on pseudo-static analysis.

Non-liquefied soil conditions: For non-liquefied soil conditions, a horizontal pseudo-static coefficient, k_h, of 0.5As and a vertical pseudo-static coefficient, k_v, equal to zero should be used when seismic stability of slopes is evaluated. For these conditions, the minimum allowable factor of safety is 1.0. Pseudo-static analyses do not result in predictions or estimates of slope deformation and therefore are not sufficient for evaluation of bridge approach fill performance or for evaluating foundations at the service limit state. The pseudo-static analysis is generally used to determine a yield acceleration for use in the Newmark (or other) analysis for estimating ground displacements, as described in **Section 6.5.3.2**.

Liquefiable soil conditions: For liquefiable soil conditions, the potential for flow failures should be assessed. Flow failures are driven by large static stresses that lead to large deformations or flow following triggering of liquefaction. Such failures are similar to debris flows. Flow failures typically occur near the end of strong shaking or shortly after shaking. However, delayed flow failures caused by post-earthquake redistribution of pore water pressures can occur—particularly if liquefiable soils are capped by relatively impermeable layers. For flow failures, both stability and deformation should be assessed and mitigated if stability failure or excessive deformation is predicted.

Conventional limit equilibrium slope stability analysis methods are most often used to assess flow failure potential. Residual undrained shear strength parameters are used to model the strength of the liquefied soil. When using liquefied soil strengths, the horizontal and vertical pseudo-static coefficients, kh and kv, respectively, should be set equal to zero (de-coupled analysis). Alternatively, a site-specific, non-linear effective stress ground response analysis may be conducted to more thoroughly assess liquefaction effects and determine appropriate acceleration values to use in the pseudo-static analysis.

Where the factor of safety is less than 1.0, flow failure shall be considered likely. In these instances, the magnitude of deformation is usually too large to be acceptable for design of bridges or structures, and some form of mitigation may be appropriate. The exception is where the liquefied material and crust flow past the structure and the structure can accommodate the imposed loads see **Section 6.5.6**. Where the factor of safety is greater than 1.0, deformations can be estimated using the methods described in **Section 6.5.3.2**.

6.5.3.2 Deformation Analysis

Deformation analyses should be employed where estimates of the magnitude of seismically induced slope deformation are required. This is especially important for bridge approach fills where the deformation analysis is a crucial step in evaluating whether or not the bridge performance requirements described in **Section 6.2** will be met. The procedures for estimating ground deformations and examples are provided in Dickenson et al. (2002) and Dickenson (2005) along with a discussion of which procedures are appropriate for specific conditions. It is recommended that several of the methods described in these reports be used as appropriate and engineering judgment applied to the results to determine the most reasonable range of predicted displacements.

Acceptable methods of estimating the magnitude of seismically induced lateral slope deformation include:

- Empirically-based displacement estimates for lateral spreading (Youd et al. (2002),
- Newmark-type analyses using acceleration time histories generated from site-specific soil response modeling.
- Simplified charts based on Newmark-type analyses (Makdisi and Seed, 1978)
- Simplified procedures based on refined Newmark-type analyses (Bray and Travasarou 2007, Saygili and Rathje 2008)
- Simplified charts based on nonlinear, effective stress modeling (Dickenson et al, 2002)
- Two-dimensional numerical modeling of dynamic slope deformation.

The Newmark sliding block methods should not be employed to estimate displacements associated with liquefaction or cyclic strength loss if the static factor of safety with the reduced (residual) strength parameters is less than 1.0.

Brief summaries of each method are described below.

Youd et al. (2002); Lateral Spreading: If the slope stability factor of safety from the flow failure analysis **Section 6.5.3.1**, assuming liquefied conditions, is 1.0 or greater, a lateral spreading/deformation analysis should be conducted. Lateral spreading results when the shear strength of the liquefied soil is incrementally exceeded by the inertial forces induced during an earthquake. The result of lateral spreading is typically horizontal movement of nonliquefied soils located above liquefied soils, in addition to the liquefied soils themselves.

The potential for liquefaction induced lateral spreading on gently sloping sites or where the site is located near a free face may be evaluated using empirical relationships such as the procedure of Youd et al. (2002). This procedure uses empirical relationships based on case histories of lateral spreading. Input into the Youd et al. model includes earthquake magnitude, source-to-site distance, and site geometry/slope, cumulative thickness of saturated soil layers and their characteristics (e.g. SPT "N" values, average fines content and average grain size). This method is based on a regression analysis of several independent variables correlated to field measurements of lateral spread. Therefore it is best applied to site conditions that fit within the range of the variables used in the models. Care should be taken when applying this method to sites with conditions outside the range of the model variables. The Youd et al. procedure can provide a useful approximation of the potential magnitude of deformation for sites with liquefiable soils.

Newmark Analysis: Newmark (1965) proposed a seismic slope stability analysis that provides an estimate of seismically induced slope deformation. The advantage of the Newmark analysis over pseudo-static analysis is that it provides an index of permanent deformation. The Newmark analysis treats the unstable soil mass as a rigid block on an inclined plane. The procedure for the Newmark analysis consists of three steps that can generally be described as follows:

- Identify the yield acceleration of the slope by completing limit equilibrium stability analyses. The yield acceleration is the horizontal pseudo-static coefficient, kh, required to bring the factor of safety to unity (1.0). Note that if the yield acceleration applied to the entire acceleration time history is based on residual soil strengths consistent with fully liquefied conditions, the estimated lateral deformation will likely be overly conservative since the liquefied, residual soil strength condition (and associated yield acceleration) will only be in effect over a portion of the entire time history.
- Select earthquake time histories representative of the design earthquakes. A minimum of three time histories representative of the predominant earthquake source zone(s) should be selected for this analysis. Note that these time histories need to be propagated through the soil column to the ground surface to adjust them for local site effects.
- Double integrate all relative accelerations (i.e., the difference between acceleration and yield acceleration) in the earthquake time histories.

A number of commercially available computer programs are available to complete Newmark analysis, such as Shake2000 or Java Programs for using Newmark's Method and Simplified Decoupled Analysis to Model Slope Performance during Earthquakes (Jibson, 2003).

Makdisi-Seed Analysis: Makdisi and Seed (1978) developed a simplified procedure for estimating seismically induced slope deformations based on Newmark sliding block analysis. The Makdisi-Seed procedure provides an estimated range of permanent seismically induced slope deformation as a function of the ratio of yield acceleration over maximum acceleration and earthquake magnitude as shown on **Figure 6-11**. The Makdisi-Seed procedure provides a useful index of the magnitude of slope deformation. Because the Makdisi-Seed procedure includes the dynamic effects of the seismic response of dams, its results should be interpreted with caution when applied to other slopes.

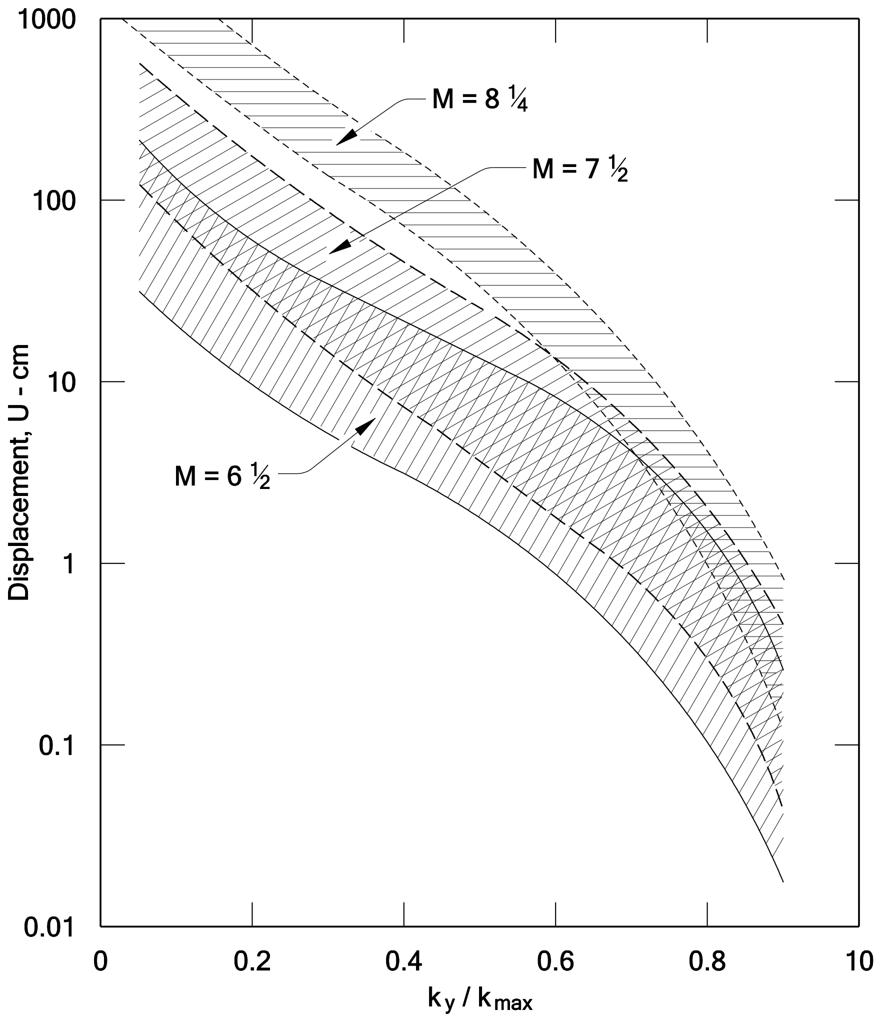


Figure 6-11. The Makdisi-Seed procedure for estimating the range of permanent seismically induced slope deformation as a function of the ratio of yield acceleration over maximum acceleration (redrafted from Makdisi and Seed, 1978).

Refined Newmark-Type Analysis-Bray and Travasarou (2007): This method is another modification, or enhancement, of the original Newmark sliding block model. It consists of a simplified, semiempirical approach for estimating permanent displacements due to earthquake-induced deviatoric deformations using a nonlinear, fully coupled, stick-slip sliding block model. In addition to estimating permanent displacements from rigid body slippage (basic Newmark approach) it also includes estimates of permanent displacement (volumetric staining) from shearing within the sliding mass itself. The model can be used to predict the probability of exceeding certain permanent displacements or for estimating the displacement for a single deterministic event. This procedure is available in EXCEL spreadsheet form.

Refined Newmark-Type Analysis – Saygili and Rathje (2008): This method is another modification, or enhancement, of the original Newmark sliding block model, suitable for shallow sliding surfaces that can be approximated by a rigid sliding block. The model predicts displacements based on multiple ground motion parameters in an effort to reduce the standard deviation of the predicted displacements.

Bracketed Intensity Method: The bracket intensity method is a modification of the Arias Intensity procedure developed by Jibson (1993). The primary difference between the two methods is that the bracketed intensity is a measure of only the ground motion intensity that is actually contributing to the displacement of the sliding block. This method is quite similar to the Newmark-type methods. A step-by-step procedure for calculating the estimated displacement is presented in Dickenson et al. (2002).

Numerical Modeling Correlations (GMI): This is a simplified method for estimating lateral deformations of embankments over liquefied soils. The method is presented Dickenson et al. (2002)) and is based on two dimensional numerical modeling of typical approach embankments using a finite difference computer code (FLAC). In the procedure developed, limit equilibrium methods are used to first calculate the post-earthquake factor of safety, using residual shear strengths in liquefied soils as appropriate. The resulting FOS is then used in combination with a Ground Motion Intensity (GMI) parameter to estimate embankment displacements. The GMI was developed to account for the intensity and duration of the ground motions used in the FLAC analysis and is equal to the PGA divided by the MSF (magnitude scaling factor). This procedure is also useful for estimating the amount, or area, of ground improvement needed to limit displacements to acceptable levels.

Numerical Modeling of Dynamic Slope Deformation: Seismically induced slope deformations can be estimated through a variety of dynamic stress-deformation computer models such as PLAXIS, DYNAFLOW, and FLAC. The accuracy of these models is highly dependent upon the quality of the input parameters. As the quality of the constitutive models used in dynamic stress-deformation models improves, the accuracy of these methods will improve. Another benefit of these models is their ability to illustrate mechanisms of deformation, which can provide useful insight into the proper input for simplified analyses.

Dynamic stress deformation models should not be used for routine design due to their complexity, and due to the sensitivity of the accuracy of deformation estimates from these models on the constitutive model selected and the accuracy of the input parameters.

6.5.4 Settlement of Dry Sand

Seismically induced settlement of unsaturated granular soils (dry sands) is well documented. Factors that affect the magnitude of settlement include the density and thickness of the soil deposit and the magnitude of seismic loading. The most common means of estimating the magnitude of dry sand settlement are through empirical relationships based on procedures similar to the Simplified Procedure for evaluating liquefaction susceptibility. The procedures provided by Tokimatsu and Seed (1987) for dry sand settlement should be used. The Tokimatsu and Seed approach estimates the volumetric strain as a function of cyclic shear strain and relative density or normalized SPT N'_{60} values. The step-by-step procedure is presented in Section 8.5 of Geotechnical Engineering Circular No. 3 (Kavazanjian, et al., 1997).

6.5.5 Liquefaction Effects on Structure Foundations

6.5.5.1 Bridge Approach Fills

All bridge approach fills should be assessed for the potential of excessive embankment deformation (lateral displacement and settlement) due to seismic loading and the effects of these displacements on the stability and functional performance requirements of the bridge. This is true whether liquefaction of the foundation soils is predicted or not. As a general rule, for the 500-year event, up to one (1) foot of lateral and 6 to 12 inches of vertical embankment displacement can be used as a general guideline for determining adequate performance of the approach fill. This range of displacements should be considered only as a general guideline for evaluating the final condition of the roadway surface and the ability to provide one-lane access to the bridge for emergency response vehicles following the earthquake. Always keep in mind the accuracy of the methods used to predict embankment deformations. Lateral displacement and fill settlement will also produce loads on the bridge foundation elements which also have to be evaluated in terms of providing the required overall bridge stability and performance. Specific embankment displacement limits are not provided for the 1000-year event since under this level of shaking the bridge and approach fills are evaluated only in terms of meeting the “No-Collapse” criteria.

6.5.5.2 General Liquefaction Policies Regarding Bridge Foundations

If liquefaction is predicted under either the 500 or 1000 year return events, the effects of liquefaction on foundation design and performance must be evaluated. Soil liquefaction and the associated effects of liquefaction on foundation resistances and stiffness is generally assumed, in standard analyses, to be concurrent with the peak loads in the structure (i.e. no reduction in the transfer of seismic energy due to liquefaction and soil softening). This applies except for the case where a site-specific nonlinear effective stress ground response analysis is performed which takes into account pore water pressure increases (liquefaction) and soil softening.

Liquefaction effects include:

- reduced axial and lateral capacities and stiffness in deep foundations,
- lateral spread, global instabilities and displacements of slopes and embankments,
- ground settlement and possible downdrag effects

The following design practice, related to liquefied foundation conditions, should be followed:

- **Spread Footings:** Spread footings are not recommended for bridge or abutment wall foundation support over liquefiable soils unless ground improvement techniques are employed that eliminate the potential liquefaction condition.
- **Piles and Drilled Shafts:** The tips of piles and drilled shafts shall be located below the deepest liquefiable soil layer. Friction resistance from liquefied soils should not be included in either compression or uplift resistance recommendations for the Extreme Event Limit I state loading condition. As stated above, liquefaction of foundation soils, and the accompanying lose of soil strength, is assumed to be concurrent with the peak loads in the structure. If applicable, reduced frictional resistance should also be applied to partially liquefied soils either above or below the predicted liquefied layer. Methods for this procedure are presented in Seed and Idriss, (1971) and Dickenson et al. (2002).

Pile Design Alternatives: Obtaining adequate lateral pile resistance is generally the main concern at pier locations where liquefaction is predicted. Battered piles are not recommended. Prestressed concrete piles have not been recommended in the past due to problems with excessive bending stresses at the pile-footing connection. Vertical steel piles are generally recommended in high seismic areas to provide the most flexible, ductile and cost-effective pile foundation system. Steel pipe piles often are preferred over H-piles due to their uniform section properties, versatility in driving with either closed or open ended and their potential for filling with reinforced concrete. The following design alternatives should be considered for increasing group resistance or stiffness and the most economical design selected:

- Increase pile size, wall thickness (section modulus) and/or strength.
- Increase numbers of piles.
- Increase pile spacing to reduce group efficiency effects.
- Deepen pile cap and/or specify high quality backfill around pile cap for increase capacity and stiffness.
- Design pile cap embedment for fixed conditions.
- Ground improvement techniques.

Liquefied P-y Curves: Studies have shown that liquefied soils retain a reduced, or residual, shear strength and this shear strength may be used in evaluating the lateral capacity of foundation soils. In light of the complexity of liquefied soils behavior (including progressive strength loss, strain mobilization, and possible dilation and associated increase in soil stiffness) computer programs commonly used for modeling lateral pile performance under liquefied soil conditions often rely on simplified relationships for soiLPile interaction. At this time, no consensus exists within the professional community on the preferred approach to modeling lateral pile response in liquefied soil. Peer review is recommended for projects involving deep foundations in liquefiable soils.

One simplified procedure for modeling pile performance utilizes the static sand model(s) in the LPILE program, modified using the residual shear strength and the effective overburden stress, at the depth at which the residual strength was calculated (or measured), to estimate a reduced soil friction angle (Φr) and initial soil modulus (k). The reduced soil friction angle is calculated using the inverse tangent (i.e., Tan-1) of the residual undrained shear strength divided by the effective vertical stress at the depth where the residual shear strength was determined or measured. Other procedures can be used with approval by ODOT.

The reduced soil parameters may be applied to either the LPile static model or the strain wedge static model (i.e., in DFSAP). DFSAP has an option built in to the program for estimating liquefied lateral stiffness parameters and lateral spread loads on a single pile or shaft. However, it should be noted the accuracy of the liquefied soil stiffness and predicted lateral spread loads using strain wedge theory, in particular the DFSAP program, has not been well established and is not recommended at this time,

For pile or shaft groups, for fully liquefied conditions, P-y curve group reduction factors may be set to 1.0. For partially liquefied conditions, the group reduction factors shall be consistent with the group reduction factors used for static loading.

Additional liquefied P-y curve recommendations are provided in the research report titled: "TILT: The Treasure Island Liquefaction Test: Final Report", (Ashford, S. and Rollins, K., 2002) available from the HQ Bridge Engineering Section. This full scale pile load test study produced P-y curves for liquefied sand conditions that are fundamentally different than those derived from the standard static P-y curve models. These liquefied P-y curves are available in the LPile computer program, however the use of these liquefied p-y curves is not recommended at this time until further studies are completed and a consensus is reached on the standard of practice for P-y curves to use in modeling liquefied soils.

T-Z curves: Modify either the PL/AE method or APILE Plus program as follows:

- For the PL/AE method, if the liquefied zone reduces total pile skin friction to less than 50% of ultimate bearing capacity, use "end bearing" condition (i.e. full length of pile) in stiffness calculations. Otherwise use "friction" pile condition.
- For the APile program, assume sand layer for liquefied zone with modified soil input parameters similar to methods for P-y curve development.

Settlement and Downdrag Loads: Settlement of foundation soils due to the liquefaction or dynamic densification of unsaturated cohesionless soils could result in downdrag loads on foundation piling or shafts. Refer to Section 3.11.8 of the *AASHTO LRFD Bridge Design Specifications* for guidance on designing for liquefaction-induced downdrag loads. Refer to **Chapter 8** for guidance on including seismic-induced settlement and downdrag loads on the seismic design of pile and shaft foundations.

6.5.5.3 Lateral Spread / Slope Failure Loads on Structures

In general, there are two different approaches to estimate the induced load on deep foundations systems due lateral spreading — a displacement based method and a force based method. Displacement based methods are more prevalent in the United States. The force based approach has been specified in the Japanese codes and is based on case histories from past earthquakes, especially the pile foundation failures observed during the 1995 Kobe earthquake. Overviews of both approaches are presented in the following sections.

6.5.5.4 Displacement Based Approach

The recommended displacement based approach for evaluating the impact of liquefaction induced lateral spreading loads on deep foundation systems is presented in the NCHRP Report 472 titled "*Comprehensive Specification for Seismic Design of Bridges*" (NCHRP, 2002) and supporting documentation by Martin et al., (2002). The general procedure is as follows:

- **Evaluate the Liquefaction Potential:** Evaluate the liquefaction potential of the site for the design risk levels (500 and 1000-yr. return periods). Assign residual and reduced strength parameters to liquefied and partially liquefied soils layers.
- **Conduct Slope Stability Analyses:** If liquefaction is predicted, conduct slope stability analyses using residual strength parameters for the liquefied soil layers and reduced strength parameters for partially liquefied soil layers. If the static factor of safety is less than 1.0, a flow failure is predicted. If the static factor of safety is greater than 1.0, conduct pseudo-static stability analyses to determine the yield acceleration K_y.
- **Check Zone of Influence:** Assess whether or not the estimated failure surface could impact the bridge foundation system. If the bridge foundations are expected to be within the zone of influence, estimate the ground deformations.
- **Slope Deformations:** Estimate lateral displacements based on the procedures described in **Section 6.5.3.2**. Use all appropriate methods that could apply to the site conditions and use judgment to determine the most reasonable amount of predicted displacement that could occur.

- **Induced Loads on Foundation Elements:** Assess whether the soil will displace and flow around a stable foundation or whether foundation movement will occur in concert with the soil. This assessment requires a comparison between the estimated passive soil forces that can be exerted on the foundation and the ultimate resistance that can be provided by the structure.

The magnitudes of moment and shear induced in the foundations by the ground displacement can be estimated using soiLPile structure interaction programs, such as LPile. The process is to apply the assumed displacement field to the interface springs whose properties are represented by P-y curves. The liquefied soil layers are typically modeled in the LPile or DFSAP programs using the modified sand P-y model and the undrained residual strength of the liquefied soil see **Section 6.5.5.2**. Partially liquefied soil layers are typically adjusted by reducing their friction angle (see Dickenson et al. 2003 for methods to reduce the friction angle based on increased pore pressures). The strength parameters of non-liquefied layers above and/or below the liquefied zones are not reduced.

The estimated induced loads are then checked against the ability of the foundation system to resist those loads. The ultimate foundation resistance is based in part on the resistance provided by the portion of the pile/shaft embedded in non-liquefiable soils below the lateral spread zone and the structural capacity of the pile/shaft. Large pile deformations may result in plastic hinges forming in the pile/shaft. If foundation resistance is greater than that applied by the lateral spreading soil, the soil will flow around the structure. If the potential load applied by the soil is greater than the ultimate foundation system resistance, the pile/shaft is likely to move in concert with the soil. Also, the passive pressure generated on the pile cap by the spreading soil needs to be considered in the total load applied to the foundation system. In cases where a significant crust of non-liquefiable material may exist, the foundation is likely to continue to move with the soil. Since large-scale structural deformations may be difficult and costly to accommodate in design, mitigation of foundation subsoils will likely be required.

In-ground hinging and plastic failure of piles or shafts due to lateral spread and slope failures is not permitted on ODOT bridge projects for either the 500 or 1000 year design events.

Similar approaches to those outlined above can be used to estimate loads that other types of slope failure may have on the bridge foundation system.

6.5.5.5 Force Based Approaches

A force-based approach to assess lateral spreading induced loads on deep foundations is specified in the Japanese codes. The method is based on back-calculations from pile foundation failures caused by lateral spreading. The pressures on pile foundations are simply specified as follows:

- The liquefied soil exerts a pressure equal to 30 percent of the total overburden pressure (lateral earth pressure coefficient of 0.30 applied to the total vertical stress).
- Non-liquefied crustal layers exert full passive pressure on the foundation system.

Data from simulated earthquake loading of model piles in liquefiable sands in centrifuge tests indicate that the Japanese force method is an adequate design method (Finn, 2004).

Another force-based approach to estimate lateral spreading induced foundation loads is to use a limit equilibrium slope stability program to determine the load the foundation must resist to achieve a target safety factor of 1.1. This force is distributed over the foundation in the liquefiable zone as a uniform stress. This approach may be utilized to estimate the forces that foundation elements must withstand if they are to act as shear elements stabilizing the slope.

6.5.6 Mitigation Alternatives for Lateral Spread

The two basic options to mitigate the lateral spread induced loads on the foundation system are to design the structure to accommodate the loads or improve the ground such that the hazard does not occur.

Structural Options (design to accommodate imposed loads): The general structural approach to design for the hazard is outlined below.

- **Step 1:** If the soil is expected to displace around the foundation element, the foundation is designed to withstand the passive force exerted on the foundation by the flowing soil and any overlying layers, or crust, of resistant soil. In this case, the maximum loads determined from the P-y springs for large deflections are applied to the pile/shaft, and the pile/shaft is evaluated using a soil structure interaction program similar to LPile. The pile/shaft stiffness, strength, and embedment are adjusted until the desired structural response to the loading is achieved.

  Note that it is customary to evaluate the lateral spread/slope failure induced loads independently from the inertial forces caused by the shaking forces (i.e. the shaking force loads and the lateral spread loads are typically not assumed to act concurrently). In most cases this is reasonable since peak vibration response is likely to occur in advance of maximum ground displacement, and displacement induced maximum shear and moments will generally occur at deeper depths than those from inertial loading.

- **Step 2:** If the assessment indicates that movement of the foundation is likely to occur in concert with the soil, then the structure is evaluated for the maximum expected ground displacement. In this case the soil loads are generally not the maximum possible (loads at large displacements), but instead some fraction thereof. Again the P-y data for the soils in question are used to estimate the loading.

If the deformations determined in Step 2 are beyond tolerable limits for structural design, the options are to a) re-evaluate the deformations based on the "pinning" or "doweling" action that foundations provide as they cross a potential failure plane (with consideration of the foundation strength; or b) re-design the foundation system to accommodate the anticipated loads. Simplified procedures for evaluating the available resistance to slope movements provided by the foundation "pinning" action are presented in (NCHRP, 2002) and (Martin, et al., 2002) and require knowledge of the plastic moment and location of plastic hinges in the foundation elements. This information should be provided by the bridge designer or structural consultant. The concept of considering a plastic mechanism or hinging in the piles/shafts is tantamount to accepting foundation damage.

With input from the structural designer regarding "pinning" resistance provided by the foundation system, recalculate the estimated displacement based on the revised resistance levels. If the structure's behavior is acceptable under the revised displacement estimate, the design for liquefaction induced lateral spreading is complete. If the performance is not acceptable, then the foundation system should be redesigned or ground improvement should be considered.

It is sometimes cost prohibitive to design the bridge foundation system to resist the loads imposed by liquefaction induced lateral loads, especially if the depth of liquefaction extends more than about 20 feet below the ground surface and if a non-liquefied crust is part of the failure surface. Ground improvement to mitigate the liquefaction hazard is the likely alternative if it is not practical to design the foundation system to accommodate the lateral loads.

Ground Improvement: The need for ground improvement techniques to mitigate liquefaction effects depends, in part, upon the type and amount of anticipated damage to the structure and approach fills due to the effects of liquefaction and embankment deformation (both horizontal and vertical). The performance criteria described in **Section 6.2** should be followed. Ground Improvement methods are described in Elias et al. (2000) and **Chapter 11**. All ground improvement designs required to mitigate the effects of soil liquefaction shall be reviewed by the HQ Bridge Section.

If, under the 500-year event, the estimated bridge damage is sufficient to render the bridge out of service for one lane of emergency traffic then ground improvement measures should be undertaken. If, under the 1000-year event, estimated bridge damage results in the possible collapse of a portion or all of the structure then ground improvement is recommended. A flow chart of the ODOT Liquefaction Mitigation Procedures is provided in **Appendix 6-C**.

Ground improvement techniques should result in reducing estimated ground and embankment displacements to acceptable levels. Mitigation of liquefiable soils beneath approach fills should extend a distance away, in both longitudinal and transverse directions, from the bridge abutment sufficient enough to limit lateral embankment displacements to acceptable levels. As a general rule of thumb, foundation mitigation should extend at least from the toe of the end slope to a point where a 1:1 slope extending from the back of the bridge end panel intersects the original ground (**Figure 6-12**). The final limits of the mitigation area required should be determined from iterative slope stability analysis and consideration of ground deformations. Practice-oriented procedures have been described in Dickenson et al (2002).

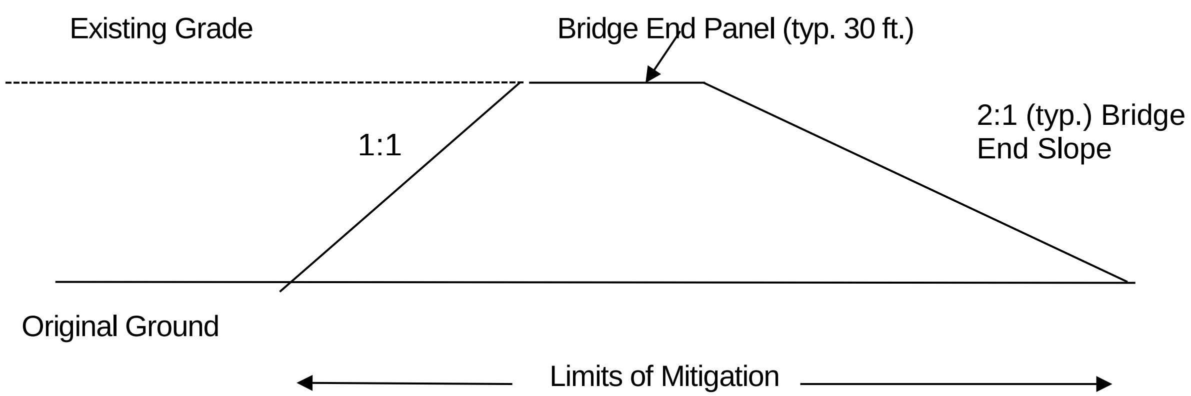


Figure 6-12. Lateral Extent of Ground Improvement for Liquefaction Mitigation

Ground improvement techniques should also be considered as part of any Phase II (substructure & foundation) seismic retrofit process. All Phase II retrofit structures should be evaluated for liquefaction potential and mitigation needs. The cost of liquefaction mitigation for retrofitted structures should be assessed relative to available funding.

The primary ground improvement techniques to mitigate liquefaction fall into three general categories, namely densification, altering the soil composition, and enhanced drainage. A general discussion regarding these ground improvement approaches is provided below.

- **Densification and Reinforcement:** Ground improvement by densification consists of sufficiently compacting the soil such that it is no longer susceptible to liquefaction during a design seismic event. Densification techniques include vibro-compaction, vibro-flotation, vibro-replacement (stone columns), deep dynamic compaction, blasting, and compaction grouting. Vibro-replacement and compaction grouting also reinforce the soil by creating columns of stone and grout, respectively. The primary parameters for selection include grain size distribution of the soils being improved, depth to groundwater, depth of improvement required, proximity to settlement/vibration sensitive infrastructure, and access constraints.

- **Altering Soil Composition:** Altering the composition of the soil typically refers to changing the soil matrix so that it is no longer susceptible to liquefaction. Examples of ground improvement techniques include permeation grouting (either chemical or micro-fine cement), jet grouting, and deep soil mixing. These types of ground improvement are typically more costly than the densification/reinforcement techniques, but may be the most effective techniques if access is limited, construction induced vibrations must be kept to a minimum, and/or the improved ground has secondary functions, such as a seepage barrier or shoring wall.
- **Drainage Enhancements:** By improving the drainage properties of sandy soils susceptible to liquefaction, it may be possible to reduce the build-up of excess pore water pressures, and thus liquefaction during seismic loading. However, drainage improvement is not considered adequately reliable by ODOT to prevent excess pore water pressure buildup due to the length of the drainage path, the time for pore pressure to dissipate, the influence of fines on the permeability of the sand, and due to the potential for drainage structures to become clogged during installation and in service. In addition, with drainage enhancements some settlement is still likely. Therefore, drainage enhancements alone shall not be used as a means to mitigate liquefaction.

Geotechnical engineers are encouraged to work with ground treatment contractors having regional experience in the development of soil improvement strategies for mitigating hazards due to permanent ground deformation.

6.6 Input for Structural Design

6.6.1 Foundation Springs

Structural dynamic response analyses incorporate the foundation stiffness into the dynamic model of the structure to capture the effects of soil structure interaction. The foundation stiffness is typically represented as a system of equivalent springs placed in a foundation stiffness matrix. The typical foundation stiffness matrix incorporates a set of six springs, namely a vertical spring, horizontal springs in the orthogonal plan dimensions, rocking about each horizontal axis, and torsion around the vertical axis.

The primary parameters for calculating the individual springs are the foundation type (shallow spread footings or deep foundations), foundation geometry, and dynamic soil shear modulus. The dynamic soil shear modulus is a function of the shear strain (foundation displacement), so determining the appropriate foundation springs can be an iterative process. Refer to the *ODOT BDDM* for additional information on foundation modeling methods and the soil/rock design parameters required by the structural designer for the analysis.

6.6.1.1 Shallow Foundations

For evaluating shallow foundation springs, the structure designer requires values for the dynamic shear modulus, G, Poisson's ratio, and the unit weight of the foundation soils. The maximum, or low-strain, shear modulus can be estimated using index properties and the correlations presented in **Table 6-2**. Alternatively, the maximum shear modulus can be calculated using Equation 6.2, if the shear wave velocity is known:

$$G_{max} = \gamma / g (V_s)^2$$

Where:

G_{max} = maximum dynamic shear modulus

γ = soil unit weight

V_s = shear wave velocity

g = acceleration due to gravity

The maximum dynamic shear modulus is associated with small shear strains (less than 0.0001 percent). As shear strain level increases, dynamic shear modulus decreases. At large cyclic shear strain (1 percent), the dynamic shear modulus approaches a value of approximately 10 percent of G_{max} (Seed et al., 1986). As a minimum, shear modulus values for 0.2 percent shear strain and 0.02 percent shear strain to simulate large and small magnitude earthquakes should be provided to the structural engineer. Shear modulus values at other shear strains could also be provided as needed for the design. Shear modulus values may be estimated using **Figure 6-1**, **Figure 6-2** and **Figure** 6-3. Alternatively, laboratory tests, such as the cyclic triaxial or direct simple shear, or resonant column tests may be used to determine the shear modulus values at intermediate shear strains. The results of in situ tests such as the CPT and DMT have also been used to develop non-linear relationships for soil stiffness (Mayne, 2001).

Poisson's Ratio can be estimated based on soil type, relative density/consistency of the soils, and correlation charts such as those presented in Foundation Analysis and Design (Bowles, 1996).

6.6.1.2 Deep Foundations

Lateral soil springs for deep foundations shall be determined in accordance with **Chapter 8**. Refer to **Section 6.5.5.2** for guidance on modifying t-z curves and the soil input required for P-y curves representing liquefied or partially liquefied soils.

6.6.1.3 Downdrag Loads on Structures

Downdrag loads on foundations shall be determined in accordance with **Chapter 8**.

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Appendix 6-A

FLOW CHART FOR EVALUATION OF LIQUEFACTION HAZARD AND GROUND DEFORMATION AT BRIDGE SITES

STEP 1

Identify Seismic Sources in the Region

CSZ interplate, deep intraplate, shallow crustal earthquakes refer to USGS Seismic Hazard Mapping Project Web Site
Obtain M-R pairs from de-aggregation tables for 475 and 975 mean return periods
Consider the following sources:

CSZ Interplate Earthquakes

M 8.3 and M 9.0
as defined by the USGS

Deep Intraplate Earthquake

- Very small contribution to PGA hazard in most of Oregon
- Confirm on De-Aggregation tables by checking for representative M-R pairs

Crustal, Areal, or "Gridded" Seismicity

- Obtain M-R pairs from USGS de-aggregation tables for all regional
- Define criteria for selecting all M-R pairs that significantly contribute to the overall seismic hazard

STEP 2

Select Appropriate Ground Motion Attenuation Relationships for each Source and Style of Faulting

Calculate the bedrock PGA values for each M-R pair

STEP 3

Select Appropriate Acceleration Time Histories for Bedrock Motions

- Three, or more, records from different earthquakes are recommended per M-R pair
- Consider style of faulting, magnitude, and the characteristics of the candidate motions (duration, frequency content, and energy)

STEP 4

Perform Dynamic Soil Response Analysis

- Develop profiles of cyclic stress ratio (CSR) versus depth for each M-R pair (3 or more time histories per M-R pair)
- Compute the average CSR profile with depth for each M-R pair
- Compute suite of Acceleration Response Spectra (ARS) if needed for structural engineering

STEP 5

Compute the Factor of Safety against Liquefaction for each M-R Pair

- Use the averaged CSR profile for each M-R pair
- Utilize standard methods for liquefaction susceptibility evaluation based on penetration resistance or shear wave velocity

STEP 6

Establish Post-Cyclic Loading Shear Strengths of Embankment and Foundation Soils

- This is performed for each M-R pair
- Focus on sensitive soils, weak fine-grained soils, loose to medium dense sandy soils *(potentially liquefiable soils are addressed as follows)*

If FSliq ≥ 1.4

Use drained shear strengths

If 1.4 > FSliq > 1.0

- Estimate the residual excess pore pressure
- Compute the equivalent friction angle

If FSliq ≤ 1.0

Estimate the residual undrained strength using two or more methods

STEP 7

Perform Slope Stability Analysis

- Static analysis using post-cyclic loading shear strengths for each M-R pair
- Calculate the FOS against sliding and determine the critical acceleration values for each M-R pair
- Focus trial slip surfaces on weak soil layers

STEP 8

Perform Deformation Analysis for each M-R pair

- Rigid-body, sliding block analysis (Newmark Method)
- Simplified chart solutions
- Numerical modeling

STEP 9

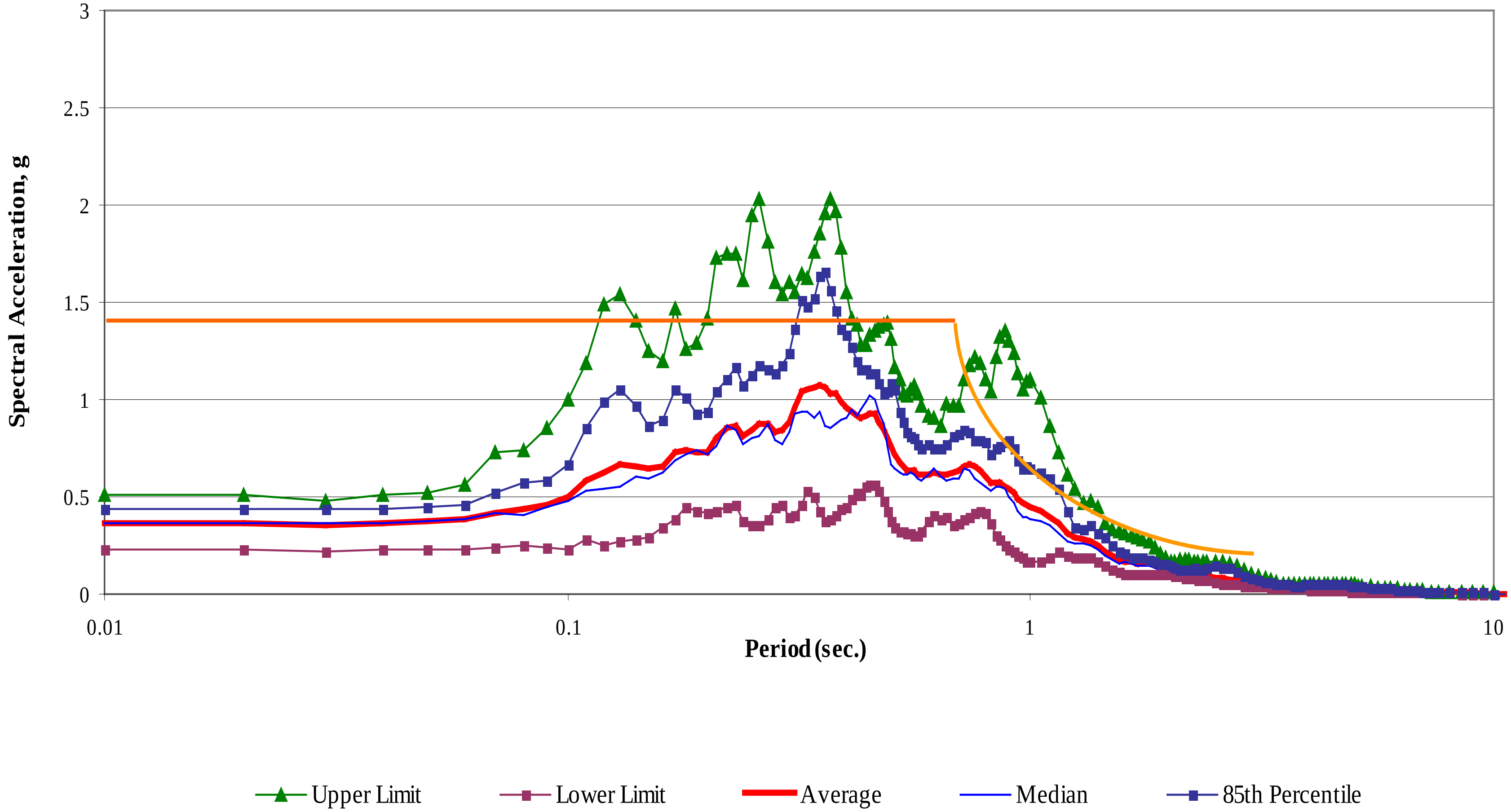
Evaluate Computed Deformations in Terms of Tolerable Limits

Permanent Deformations are Acceptable

- Computed displacements are less than defined limits
- Continue with structural design

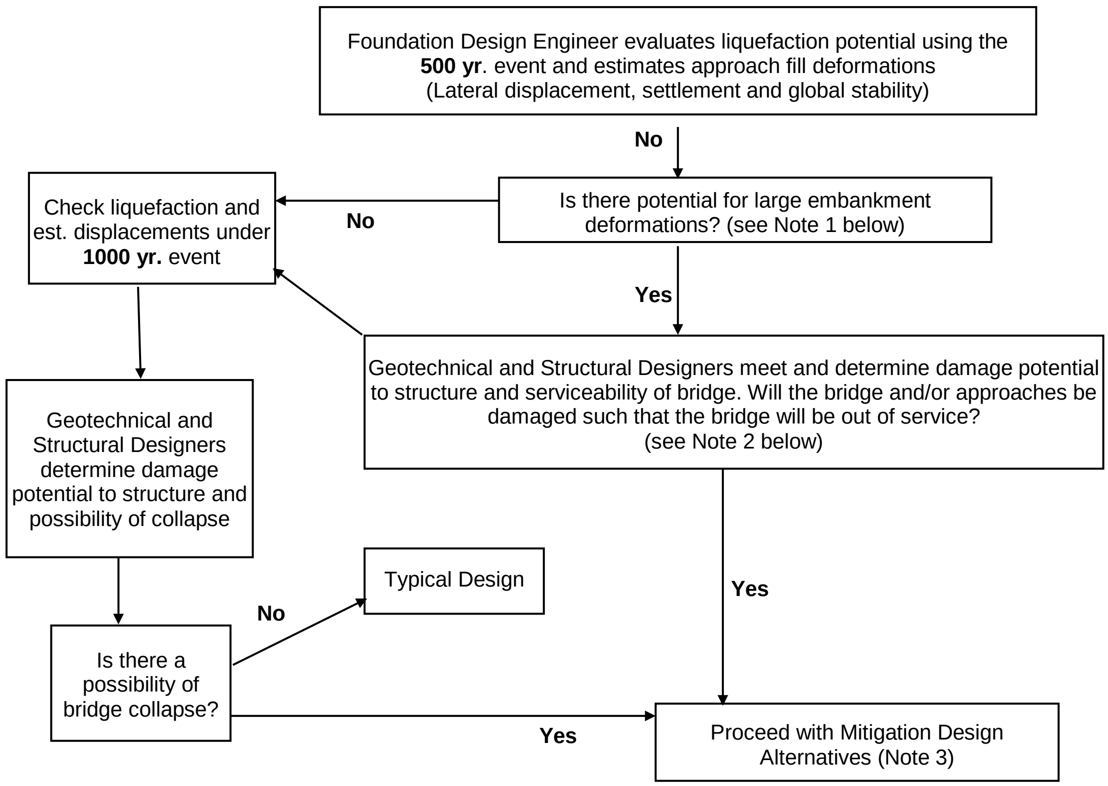
Permanent Deformations are Unacceptable

- Computed displacements exceed defined limits repeat analysis incorporating the effects of remedial ground treatment
- Return to Step 4 if the soil improvement does not significantly change the anticipated dynamic response of the soil column (e.g., isolated soil improvement)
- Return to Step 3 if the ground treatment substantially alters the dynamic response of the site (e.g., extensive soil improvement in the vertical and lateral direction, extensive treatment including grouting or deep soil mixing)
- A reduced number of input time histories are acceptable for each M-R pair (bracket the problem using trends from the initial analysis)



Spectral Acceleration, g
3
2.5
2
1.5
1
0.5
0
0.01
0.1
1
10
Period (sec.)
Upper Limit
Lower Limit
Average
Median
85th Percentile

Appendix 6-C: ODOT Liquefaction Mitigation Procedures

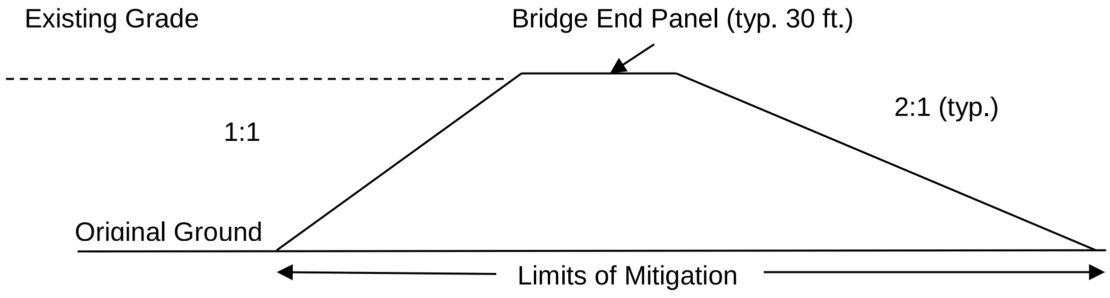


Note 1: For meeting the performance requirements of the 500 year return event (serviceability), lateral deformation of approach fills of up to 12" are generally considered acceptable under most circumstances pending an evaluation of this amount of lateral deformation on abutment piling. Larger lateral deformations and settlements may be acceptable under the 1000 year event as long as the "no-collapse" criteria are met.

Note 2: The bridge should be open to emergency vehicles after the 500-year design event, following a thorough inspection. If the estimated embankment deformations (vertical or horizontal or both) are sufficient enough to cause concerns regarding the serviceability of the bridge mitigation is recommended.

Note 3: Refer to ODOT research report SPR Project 361: "*Assessment and Mitigation of Liquefaction Hazards to Bridge Approach Embankments in Oregon*", Nov. 2002 and FHWA Demonstration Project 116; "*Ground Improvement Technical Summaries, Volumes I & II*", (Pub. No. FHWA-SA-98-086) for mitigation alternatives and design procedures.

As a general guideline, the foundation mitigation should extend from the toe of the end slope to a point that is located at the base of a 1:1 slope which starts at the end of the bridge end panel:



Chapter

7

7 Slope Stability Analysis

7.1 General

Slope stability analysis is used in a wide variety of geotechnical engineering problems, including, but not limited to, the following:

- Determination of stable cut and fill slopes
- Assessment of overall stability of retaining walls, including global and compound stability (includes permanent systems and temporary shoring systems)
- Assessment of overall stability of shallow and deep foundations for structures located on slopes or over potentially unstable soils, including the determination of lateral forces applied to foundations and walls due to potentially unstable slopes
- Stability assessment of landslides (mechanisms of failure, and determination of design properties through back-analysis), and design of mitigation techniques to improve stability
- Evaluation of instability due liquefaction

Types of slope stability analyses include rotational slope failure, sliding block analysis, irregular surfaces of sliding, and infinite slope failure. Stability analysis techniques specific to rock slopes, other than highly fractured rock masses, that can in effect be treated as soil, are described in **Chapter 12**. Detailed stability assessment of landslides is described in **Chapter 13**.

7.2 Development of Design Parameters and Input Data for Slope Stability Analysis

The input data needed for slope stability analysis is described in **Chapter 2** for site investigations in general, **Chapter 9**, **Chapter 10** for fills and cuts, and **Chapter 13** for landslides. **Chapter 5** provides requirements for the assessment of design property input parameters. Detailed assessment of soil and rock stratigraphy is critical to the proper assessment of slope stability, and is in itself a direct input parameter for slope stability analysis. It is important to define any thin weak layers present, the presence of slickensides, etc., as these fine details of the stratigraphy could control the stability of the slope in question. Knowledge of the geologic nature of the units present at the site and knowledge of past performance of such units may also be critical factors in the assessment of slope stability. Whether long-term or short-term stability is in view, and which will control the stability of the slope, will affect the selection of soil and rock shear strength parameters used as input in the analysis. For short-term stability analysis, undrained shear strength parameters should be obtained. For long-term stability analysis, drained shear strength parameters should be obtained. For assessing the stability of landslides, residual shear strength parameters will be needed, since the soil has in such cases already deformed enough to reach a residual value. For highly over-consolidated clays, if the slope is relatively free to deform after the cut is made or is otherwise unloaded, residual shear strength parameters should be obtained and used for the stability analysis.

Detailed assessment of the groundwater regime within and beneath the slope is also critical. Detailed piezometric data at multiple locations and depths within and below the slope will likely be needed, depending on the geologic complexity of the stratigraphy and groundwater conditions. Potential seepage at the face of the slope must be assessed and addressed. In some cases, detailed flow net analysis may be needed. If seepage does exit the slope face, the potential for soil piping should also be assessed as a slope stability failure mechanism, especially in highly erodable silts and sands.

7.3 Design Requirements

Limit equilibrium methodologies are usually used to assess slope stability. The Modified Bishop, simplified Janbu, Spencer, or other widely accepted slope stability analysis methods should be used for rotational and irregular surface failure mechanisms. In cases where the stability failure mechanisms anticipated are not well modeled by limit equilibrium techniques, or if deformation analysis of the slope is required, more sophisticated analysis techniques (e.g., finite difference methodologies such as is used by the computer program FLAC) may be used in addition to the limit equilibrium methodologies. Since these more sophisticated methodologies are quite sensitive to the quality of the input data and the details of the model setup, including the selection of constitutive models used to represent the material properties and behavior, limit equilibrium methods should also be used in such cases. If the differences in the results are significant, engineering judgment should be applied in conjunction with any available field observations to assess the correctness of the design model used. If the potential slope failure mechanism is anticipated to be relatively shallow and parallel to the slope face, with or without seepage affects, an infinite slope analysis should be conducted. Typically, slope heights of 15 to 20 ft or more are required to have this type of failure mechanism. For infinite slopes which are either above the water table or which are fully submerged, the factor of safety for slope stability is determined as follows:

- **Seepage:** Considering that the buoyant unit weight is roughly one-half of the saturated unit weight, seepage on the slope face can reduce the factor of safety by a factor of two, a condition which should obviously be avoided through some type of drainage if at all possible; otherwise much flatter slopes will be needed.
- **Slopes:** When using the infinite slope method, if the FS is near or below 1.0 to 1.15, severe erosion or shallow slumping is likely. Vegetation on the slope can help to reduce this problem, as the vegetation roots add cohesion to the surficial soil, improving stability. Note that conducting an infinite slope analysis does not preclude the need to check for deeper slope failure mechanisms, such as would be assessed by the Modified Bishop or similar methods listed above. For very simplified cases, design charts to assess slope stability are available. Examples of simplified design charts are provided in NAVFAC DM-7. These charts are for a c-φ soil, and apply only to relatively uniform soil conditions within and below the cut slope. They do not apply to fills over relatively soft ground, as well as to cuts in primarily cohesive soils. Since these charts are for a c-φ soil, a small cohesion will be needed to perform the calculation.
- If these charts are to be used, it is recommended that a cohesion value of 50 to 100 psf be used in combination with the soil friction angle obtained from SPT correlation for relatively clean sands and gravels.
- **Soil parameters:** For silty to very silty sands and gravels, the cohesion could be increased to 100 to 200 psf, but with the friction angle from SPT correlation (see

Chapter 5) reduced by 2 to 3 degrees, if it is not feasible to obtain undisturbed soil samples suitable for laboratory testing to measure the soil shear strength directly. This should be considered general guidance, and good engineering judgment should be applied when selecting soil parameters for this type of an analysis. Simplified design charts should only be used for final design of non-critical slopes that are 10 ft in height or less and that are consistent the simplified assumptions used by the design chart. Simplified design charts may be used as applicable for larger slopes for preliminary design. The detailed guidance for slope stability analysis provided by Abramson, et al. (1996) should be used.

7.4 Resistance Factors and Safety Factors for Slope Stability Analysis

For overall stability analysis of walls and structure foundations, design shall be consistent with **Chapter 6**, **Chapter 8** and **Chapter 15** and the *AASHTO LRFD Bridge Design Specifications*. For slopes adjacent to but not directly supporting structures, a maximum resistance factor of 0.75 should be used. For foundations on slopes that support structures such as bridges and retaining walls, a maximum resistance factor of 0.65 should be used. Exceptions to this could include minor walls that have a minimal impact on the stability of the existing slope, in which the 0.75 resistance factor may be used. Since these resistance factors are combined with a load factor of 1.0 (overall stability is assessed as a service limit state only), these resistance factors of 0.75 and 0.65 are equivalent to a safety factor of 1.3 and 1.5, respectively. For general slope stability analysis of cuts, fills, and landslide repairs, a minimum safety factor of 1.25 should be used. Larger safety factors should be used if there is significant uncertainty in the slope analysis input parameters. For seismic analysis, if seismic analysis is conducted see **Chapter 6** for policies on this issue, a maximum resistance factor of 0.9 should be used for slopes involving or adjacent to walls and structure foundations. This is equivalent to a safety factor of 1.1. For other slopes (cuts, fills, and landslide repairs), a minimum safety factor of 1.05 should be used.

7.5 References

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Chapter

8

8 Foundation Design

8.1 General

This chapter covers the geotechnical design of bridge foundations, retaining wall foundations and cut-and-cover tunnel foundations. Both shallow and deep foundation types are addressed. Foundation design work entails assembling all available foundation information for a structure, obtaining additional information as required, performing foundation analyses and compiling the information into a report that includes the specific structure foundation recommendations. An adequate site inspection, office study, appropriate subsurface exploration program and comprehensive foundation analyses that result in foundation recommendations are all necessary to construct a safe, cost-effective structure. See **Chapter 21** for guidance on the foundation information that should be included in Geotechnical Reports. See **Chapter 2** for guidance on foundation information available through office studies and the procedures for conducting a thorough site reconnaissance.

Unless otherwise stated in this manual, the Load and Resistance Factor Design approach (LRFD) shall be used for all foundation design projects, as prescribed in the most current version of the *AASHTO LRFD Bridge Design Specifications*. The ODOT foundation design policies and standards described in this chapter supersede those in the AASHTO LRFD specifications. FHWA design manuals are also acceptable for use in foundation design and preferable in cases where foundation design guidance is not adequately provided in AASHTO. Structural design of bridge foundations, and other structure foundations, is addressed in the *ODOT Bridge Design and Drafting Manual (BDDM)*.

It is important to establish and maintain close communication between the geotechnical designer and the structural designer at all times throughout the entire foundation design process and continuing through construction.

8.2 Project Data and Foundation Design Requirements

The scope of the project, project requirements, project constraints and the geology and subsurface conditions of the site should be analyzed to determine the type and quantity of geotechnical investigation work to be performed. Project information such as a vicinity map, a project narrative, preliminary structure plans/layout (pre-Type, Size & Location) and hydraulics information (if applicable) should be obtained to allow for proper planning of the subsurface exploration program. Keep abreast of changes to the project scope that might impact the geotechnical investigation and design work required. Proposed retaining wall and bridge bent locations should be obtained from the bridge designer prior to the beginning of field work to properly locate bore holes.

Anticipated foundation loads, structure settlement criteria and the heights of any proposed fills should be determined or estimated to insure that the exploration boreholes are advanced to the proper depth and the proper information is obtained.

Refer to AASHTO Article 10.4.1 for more details of the information needed at this stage.

The foundation type(s) selected for each structure will each require specific subsurface investigation methods, materials testing, analysis and design. **Table 8-1** provides a summary of information needs and testing considerations for foundation design.

Table 8-1. Summary of information needs and testing considerations (modified after Sabatini, et. al. 2002)

Foundation Type	Engineering Evaluations	Required Information For Analyses	Field Testing	Laboratory Testing
Shallow Foundations	• bearing capacity • settlement (magnitude & rate) • shrink/swell of foundation soils (natural soils or embankment fill) • frost heave • scour (for water crossings) • liquefaction	• subsurface profile (soil, groundwater, rock) • shear strength parameters • compressibility parameters (including consolidation, shrink/swell potential, and elastic modulus) • frost depth • stress history (present and past vertical effective stresses) • depth of seasonal moisture change • unit weights • geologic mapping including orientation and characteristics of rock discontinuities	• SPT (granular soils) • CPT • PMT • dilatometer • rock coring (RQD) • plate load testing • geophysical testing	• 1-D Oedometer tests • soil/rock shear tests • grain size distribution • Atterberg Limits • specific gravity • moisture content • unit weight • organic content • collapse/ swell potential tests • intact rock modulus • point load strength test

Table 8-1 (Continued)

Driven Pile Foundations	• pile end-bearing • pile skin friction • settlement • down-drag on pile • lateral earth pressures • chemical compatibility of soil and pile • drivability • presence of boulders/very hard layers • scour (for water crossings) • vibration/heave damage to nearby structures • liquefaction	• subsurface profile (soil, groundwater, rock) • shear strength parameters • horizontal earth pressure coefficients • interface friction parameters (soil and pile) • compressibility parameters • chemical composition of soil/rock (e.g., potential corrosion issues) • unit weights • presence of shrink/swell soils (limits skin friction) • geologic mapping including orientation and characteristics of rock discontinuities	• SPT (granular soils) • pile load test • CPT • PMT • vane shear test • dilatometer • piezometers • rock coring (RQD) • geophysical testing	• soil/rock shear tests • interface friction tests • grain size distribution • 1-D Oedometer tests • pH, resistivity tests • Atterberg Limits • specific gravity • organic content • collapse/ swell potential tests • intact rock modulus • point load strength test
Drilled Shaft Foundations	• shaft end bearing • shaft skin friction • constructability • down-drag on shaft • quality of rock socket • lateral earth pressures • settlement (magnitude & rate) • groundwater seepage/ deepwatering/ potential for caving • presence of boulders/very hard layers • scour (for water crossings) • liquefaction	• subsurface profile (soil, groundwater, rock) • shear strength parameters • interface shear strength • friction parameters (soil and shaft) • compressibility parameters • horizontal earth pressure coefficients • chemical composition of soil/rock • unit weights • permeability of water-bearing soils • presence of artesian conditions • presence of shrink/swell soils (limits skin friction) • geologic mapping including orientation and characteristics of rock discontinuities • degradation of soft rock in presence of water and/or air (e.g., rock sockets in shales)	• installation technique test shaft • shaft load test • vane shear test • CPT • SPT (granular soils) • PMT • dilatometer • piezometers • rock coring (RQD) • geophysical testing	• 1-D Oedometer tests • soil/rock shear tests • grain size distribution • interface friction tests • pH, resistivity tests • permeability tests • Atterberg Limits • specific gravity • moisture content • unit weight • organic content • collapse/ swell potential tests • intact rock modulus • point load strength test • slake durability

8.3 Field Exploration for Foundations

Subsurface explorations shall be performed in accordance with Article 10.4.2 of the AASHTO LRFD Bridge Design Specifications, supplemented by the *FHWA Geotechnical Engineering Circular No. 5, "Evaluation of Soil and Rock Properties" (FHWA-IF-02-034*). The procedures outlined in the ODOT "Soil and Rock Classification Manual" are used to describe and classify subsurface materials. The explorations shall provide the information needed for the design and construction of foundations. Accurate and adequate subsurface information at, or as near as possible to, each structure support is extremely important, especially for drilled shaft and spread footing designs.

The minimum exploration requirements specified in AASHTO Section 10, and as supplemented in **Chapter 3**, should be considered the standard of practice with regards to subsurface investigation requirements. It is understood that engineering judgment will need to be applied by a licensed and experienced geotechnical professional to adapt the exploration program to the foundation types and depths needed and to the variability in the subsurface conditions observed. The extent of exploration shall be based on the variability in the subsurface conditions, structure type, foundation loads, and any project requirements that may affect the foundation design or construction. The exploration program should be extensive enough to reveal the nature and types of soil deposits and/or rock formations encountered the engineering properties of the soils and/or rocks, the potential for liquefaction, and the groundwater conditions. The exploration program should be sufficient to identify and delineate problematic subsurface conditions such as deep, very soft soil deposits, bouldery deposits, swelling or collapsing soils, existing fill or waste areas, etc.

For cut-and-cover tunnels, culverts, arch pipes, etc., spacing of exploration locations shall be consistent with the requirements described in **Chapter 3**.

The groundwater conditions at the site are very important for both the design and construction of foundations. Groundwater conditions are especially important in the construction of drilled shafts, spread footings or any other excavation that might extend below the water table or otherwise encounter groundwater. Piezometer data adequate to define the limits and piezometric head in all unconfined, confined, and locally perched groundwater zones should be obtained at each foundation location. The measured depth and elevations of groundwater levels, and dates measured, should be noted on the exploration logs and discussed in the final Geotechnical Report. It is important to distinguish between the groundwater level and the level of any drilling fluid. Also, groundwater levels encountered during exploration may differ from design groundwater levels. Any artesian groundwater condition or other unusual groundwater condition should be identified and reported as this often has important impacts on foundation design and construction.

8.4 Field and Laboratory Testing for Foundations

Conduct subsurface investigations and materials testing in conformance with AASHTO Articles 10.4.3 and 10.4.4. **Table 8-1** provides a summary of field and laboratory testing considerations for foundation design. Foundation design will typically rely upon the Standard Penetration Test (SPT), Cone Penetrometer Test (CPT) and rock core samples obtained during the field exploration. Visual descriptions of the soil and rock materials are recorded. Correlations are usually made between these field tests to shear strength and compressibility of the soil. Groundwater and other hydraulic information needed for foundation design and constructability evaluation is typically obtained during the exploration using field instrumentation (e.g., piezometers) and in-situ tests (e.g., slug tests, pump tests, etc.).

ODOT owns the following equipment:

- A Texam Pressuremeter which is available for use on Agency designed projects. The pressuremeter requires predrilled boreholes. The pressuremeter is stored in Region 2. Contact the Region 2 Bridge/Geo-Hydro Section for assistance in obtaining the use of this equipment.
- A Vane Shear device, a Point Load Tester and a Geoprobe. Contact the Pavements Unit to schedule use of the Geoprobe equipment.

In general, for foundation design, laboratory testing should be used to augment the data obtained from the field investigation program and to refine the soil and rock properties selected for design. Index tests such as soil gradation, Atterberg limits, water content, and organic content are used to confirm the visual field classification of the soils encountered, but may also be used directly to obtain input parameters for some aspects of foundation design (e.g., soil liquefaction, scour, degree of over-consolidation, and correlation to shear strength or compressibility of cohesive soils). Laboratory tests conducted on undisturbed soil samples are used to assess shear strength or compressibility of finer grained soils, or to obtain seismic design input parameters such as shear modulus.

8.5 Material Properties for Design

The selection of soil and rock design properties should be in conformance with those described in **Chapter 5** with additional reference to "*Evaluation of Soil and Rock Properties", Geotechnical Engineering Circular No. 5, (FHWA-IF-02-034*).

8.6 Bridge Approach Embankments

The embankments at bridge ends should be evaluated for stability and settlement. The FHWA publication *"Soils and Foundations Reference Manual", 2006, (FHWA NHI-06-088)* should be referenced for guidance in the analysis and design of bridge approach embankments. New embankment placed for bridge approaches should be evaluated for short term (undrained) and long term (drained) conditions.

Bridge end slopes are typically designed at 2(H):1(V). If steeper end slopes such as 1½: 1 are desired, they should be evaluated for stability and designed to meet the required factors of safety. If embankment stability concerns arise, consider the use of staged construction, wick drains, flatter slopes, soil reinforcement, lightweight materials, subexcavation/replacement, counterbalances, or other measures depending on site conditions, costs and constraints. The embankment stability analysis, any recommended stabilization measures, instrumentation or other embankment monitoring needs, should be described in detail in the Geotechnical Report.

For overall stability, the static factor of safety for bridge approach embankments should be at least 1.30. A factor of safety of at least 1.5 must be provided against overall stability for abutment spread footings supported directly on embankments or abutment retaining walls. The programs XSTABL5.2 and Slope/W are available for evaluating slope stability. Dynamic (seismic) slope stability, settlement and lateral displacements are discussed in **Chapter 6**.

The FHWA program "EMBANK" (Urzua, A., 1993) is available for use in estimating embankment settlement. If the estimated post-construction settlement is excessive, consider the use of waiting periods, surcharges, wick drains or other ground improvement methods to expedite or minimize embankment settlement and allow for bridge construction. Consider relocating the bridge end if embankment settlement and stability concerns result in extreme and costly measures to facilitate embankment construction. Also, evaluate long term embankment settlement potential and possible downdrag effects on piles or drilled shafts and provide downdrag mitigation recommendations, such as wait periods, if necessary. In general, design for the long term settlement of approach embankments to not exceed 1" in 20 years. Refer to the *BDDM* for additional approach fill settlement limitations regarding integral abutments.

8.6.1 Abutment Transitions

ODOT standard practice is to provide bridge end panels at each end bent location for bridges constructed on the State Highway system. Embankment settlement often occurs at this transition point after construction is completed and the end panels are necessary to eliminate a potentially dangerous traffic hazard and reduce the impact of traffic loads to the bridge. The settlement is sometimes the result of poorly placed and compacted embankment material or abutment backfill it or might be due to long-term settlement of the foundation soils. Guidance for proper detailing and material requirements for abutment backfill is provided in the "*Soils and Foundations Reference Manual", 2006, (FHWA NHI-06-088).*

End panels may be considered for deletion if the following geotechnical conditions are met:

- Foundation materials are characterized as "incompressible" (e.g., bedrock or very dense granular soils)
- Post-construction settlement estimates are negligible (<0.25"),
- Provisions are made to insure the specifications for embankment and backfill materials, placement and compaction are adhered to (increased inspection and testing QC/QA)

A geotechnical and structural evaluation is required for considering the deletion of end panels and approval of a deviation from standard *ODOT BDDM* practice is required. The final decision on whether or not to delete end panels shall be made by the ODOT HQ Bridge Section Engineer with consideration to the geotechnical and structural evaluation.

In addition to geotechnical criteria, other issues such as average daily traffic (ADT), design speed, or accommodation of certain bridge structure details may supersede the geotechnical reasons for deleting end panels. End panels shall be used for all ODOT bridges with stub, or integral abutments to accommodate bridge expansion and contraction. End panels shall also be used in all cases where seismic loads could result in excessive dynamic fill settlement and the failure to meet the performance criteria described in the *BDDM*.

8.6.2 Overall Stability

The evaluation of overall stability of earth slopes with or without a foundation unit shall be investigated at the service limit state as specified in Article 11.6.2.3 of the AASHTO *LRFD Bridge Design Specifications*. Overall stability should be evaluated using limiting equilibrium methods such as modified Bishop, Janbu, Spencer, or other widely accepted slope stability analysis methods. Article 11.6.2.3 recommends that overall stability be evaluated at the Service I limit state (i.e., a load factor of 1.0) and a resistance factor, φ_{os}, of 0.65 for slopes which support a structural element. This corresponds to a factor of safety of 1.5.

Most slope stability programs produce a single factor of safety, FS. Overall slope stability shall be checked to insure that foundations designed for a maximum bearing stress equal to the specified service limit state bearing resistance will not cause the slope stability factor of safety to fall below 1.5. This practice will essentially produce the same result as specified in Article 11.6.2.3 of the AASHTO *LRFD Bridge Design Specifications*. The foundation loads should be as specified for the Service I limit state for this analysis. If the foundation is located on the slope such that the foundation load contributes to slope instability, the designer shall establish a maximum footing load that is acceptable for maintaining overall slope stability for Service, and Extreme Event limit states (see **Figure** 8-1 for example). If the foundation is located on the lower portion of the slope such that the foundation load increases slope stability, overall stability of the slope shall be evaluated ignoring the effect of the footing on slope stability.

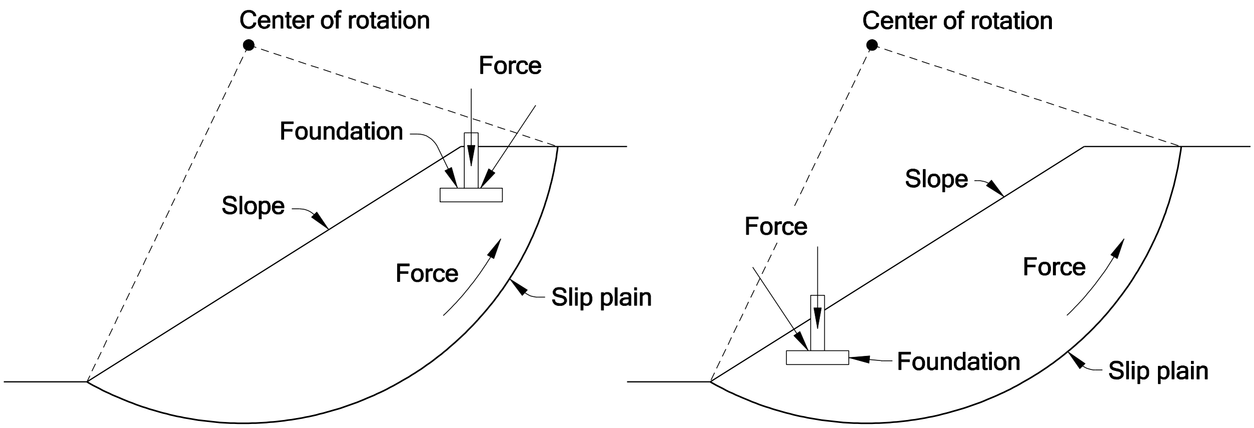

Figure 8-1. Example where footing contributes to instability of slope (left figure) vs. example where footing contributes to stability of slope (right figure)

8.7 Foundation Selection Criteria

The foundation type selected for a given structure should result in the design of the most economical bridge, taking into account any constructability issues and constraints. The selection of the most suitable foundation for the structure should be based on the following considerations:

- The ability of the foundation type to meet performance requirements (e.g., deformation, bearing resistance, uplift resistance, lateral resistance/deformation) for all limit states including scour and seismic conditions,
- The constructability of the foundation type (taking into account issues like traffic staging requirements, construction access, shoring required, cofferdams)
- The cost of the foundation,
- Meeting the requirements of environmental permits (e.g. in-water work periods, confinement requirements, noise or vibration effects from pile driving or other operations, hazardous materials)
- Constraints that may impact the foundation installation (e.g., overhead clearance, access, surface obstructions, and utilities)
- The impact of foundation construction on adjacent structures, or utilities, and the post-construction impacts on such facilities,
- The impact of the foundation installation (in terms of time and space required) on traffic and right-of-way

This is the most important step in the foundation design process. These considerations should be discussed (as applicable) with the structural designer. Bridge bent locations may need to be adjusted based on the foundation conditions, construction access or other factors described above to arrive at the most economical and appropriate design.

Spread Footings

Spread footings are typically very cost effective, given the right set of conditions. Footings work best in hard or dense soils that have adequate bearing resistance and exhibit tolerable settlement under load. Footings can get rather large in less dense soils such as medium dense sand or stiff clays depending on the structure loads and settlement requirements. Structures with tall columns or with high lateral loads which result in large eccentricities and footing uplift loads may not be suitable candidates for footing designs. Footings are not allowed or cost-effective where soil liquefaction can occur at or below the footing level. Other factors that affect the cost feasibility of spread footings include:

- The need for a cofferdam and seals when placed below the water table,
- The need for significant over-excavation and replacement of unsuitable soils,
- The need to place footings deep due to scour, liquefaction or other conditions,
- The need for significant shoring to protect adjacent existing facilities, and
- Inadequate overall stability when placed on slopes that have marginally adequate stability.

Settlement (service limit state criteria) often controls the feasibility of spread footings. The amount of footing settlement must be compatible with the overall bridge design. The superstructure type and span lengths usually dictate the amount of settlement the structure can tolerate and footings may still be feasible and cost effective if the structure can be designed to tolerate the estimated settlement (e.g., flat slab bridges, bridges with jackable abutments, etc.). Footings may not be feasible where expansive or collapsible soils are present near the bearing elevation. Refer to the *FHWA Geotechnical Engineering Circular No. 6, Shallow Foundations*, and the FHWA publication*, Selection of Spread Footings on Soils to Support Highway Bridge Structures* (FHWA-RC/TD-10-001) for additional guidance on the selection and use of spread footings.

Deep Foundations

Deep foundations are the next choice when spread footings cannot be founded on competent soils or rock at a reasonable cost. Deep foundations are also required at locations where footings are unfeasible due to extensive scour depths, liquefaction or lateral spread problems. Deep foundations may be installed to depths below these susceptible soils to provide adequate foundation resistance and protection against these problems. Deep foundations should also be used where an unacceptable amount of spread footing settlement may occur. Deep foundations should be used where right-of-way, space limitations, or other constraints as discussed above would not allow the use of spread footings.

Two general types of deep foundations are typically considered: pile foundations, and drilled shaft foundations. The most economical deep foundation alternative should be selected unless there are other controlling factors. Shaft foundations are most advantageous where very dense intermediate strata must be penetrated to obtain the desired bearing, uplift, or lateral resistance, or where materials such as boulders or logs must be penetrated. Shafts may also become cost effective where a single shaft per column can be used in lieu of a pile group with a pile cap, especially when a cofferdam, seal and/or shoring is required to construct the pile cap. However, shafts may not be desirable where contaminated soils are present, since the contaminated soil removed would require special handling and disposal. Constructability is also an important consideration in the selection of drilled shafts. Shafts can be used in lieu of piles where pile driving vibrations could cause damage to existing adjacent facilities.

Piles may be more cost effective than shafts where pile cap construction is relatively easy, where the depth to the foundation layer is large (e.g., more than 100 ft), or where the pier loads are such that multiple shafts per column, requiring a shaft cap, are needed. The tendency of the upper loose soils to flow, requiring permanent shaft casing, may also be a consideration that could make pile foundations more cost effective. Artesian pressure in the bearing layer could preclude the use of drilled shafts due to the difficulty in keeping enough head inside the shaft during excavation to prevent heave or caving under slurry.

When designing pile foundations keep in mind the potential cost impacts associated with the use of large pile hammers. Local pile driving contractors own hammers typically ranging up to about 80,000 ft.-lbs of energy. When larger hammers are required to drive piles to higher pile bearing resistance they have to rent the hammers and the mobilization cost associated with furnishing pile driving equipment may increase sharply. Larger hammers may also impact the design and cost work bridges due to higher hammer and crane loads.

For situations where existing substructures must be retrofitted to improve foundation resistance, where there is limited headroom available for pile driving or shaft construction, or where large amounts of boulders must be penetrated, micropiles may be the best foundation alternative, and should be considered.

Augercast piles can be very cost effective in certain situations. However, their ability to resist lateral loads is minimal, making them undesirable to support structures where significant lateral loads must be transferred to the foundations. Furthermore, quality assurance of augercast pile integrity and capacity needs further development. Therefore, it is ODOT current policy not to use augercast piles for bridge foundations.

8.8 Overview of LRFD for Foundations

The basic equation for load and resistance factor design (LRFD) states that the loads multiplied by factors to account for uncertainty, ductility, importance, and redundancy must be less than or equal to the available resistance multiplied by factors to account for variability and uncertainty in the resistance per the *AASHTO LRFD Bridge Design Specifications*. The basic equation, therefore, is as follows:

$$\Sigma \eta_I \gamma_i Q_i \leq \varphi R_n \qquad (8\text{-}1)$$

η_I = Factor for ductility, redundancy, and importance of structure

γ_i = Load factor applicable to the i'th load Qi

Q_i = Load

φ = Resistance factor

R_n = Nominal (predicted) resistance

For typical ODOT practice, η_I is set equal to 1.0 for use of both minimum and maximum load factors.

The product, φRn, is termed the “factored resistance”. This term is analogous to the term “allowable capacity” previously used in Allowable Stress Design. AASHTO Article 10.5.5 provides the resistance factors to use in foundation design. Resistance factors for a given foundation type are a function of the design method used, soil type/condition and other factors. AASHTO Article 10.5.5, and its associated commentary, should be reviewed for information on the development of the specified resistance factors used in foundation design and provides guidance in the selection and use of these factors. Foundations shall be proportioned so that the factored resistance is always greater than or equal to the factored loads. The loads and load factors to be used in pile foundation design shall be as specified in *Section 3* of the *AASHTO LRFD Bridge Design Specifications*.

8.9 Foundation Design Policies

8.9.1 Downdrag Loads

Downdrag loads on piles, shafts, or other deep foundations shall be evaluated as described in AASHTO Article 3.11.8. If a downdrag condition exists, the resulting downdrag loads (DD) are included with the permanent load combinations used in structure design and an appropriate load factor is applied to the downdrag loads. In addition to applying the downdrag loads on the load side of the LRFD equation, the downdrag loads must also be subtracted from the resistance side of the equation since this resistance will not be available for foundation support.

If the settlement cannot be mitigated, consideration should be given to reducing the effects of downdrag loads on the foundations by the use of bitumen coating or pile sleeves. The NCHRP Report "*Design and Construction Guidelines for Downdrag on Uncoated and Bitumen-Coated Piles*" (Briaud, J. et al., 1997) should be referenced for more guidance on downdrag mitigation methods.

Earthquakes may also produce foundation settlement and downdrag loads due to either liquefaction of saturated sandy soils or dynamic compaction of unsaturated sandy soils resulting from seismic ground motions. **Chapter 6** presents methods for calculating liquefaction potential and dynamic settlement estimates. Downdrag loads resulting from seismic loading conditions should not be combined with downdrag loads resulting from static long-term foundation settlement.

8.9.2 Scour Design

Structures crossing waterways may be subject to damage by scour and erosion of the streambed, stream banks, and possibly the structure approach fills. Bents placed in the streambed increase the potential for scour to occur. The degree and depth of scour will have a significant affect on the selection of the most appropriate foundation type. The Hydraulic Report should be consulted for scour predictions.

Scour depths are typically calculated for both the 100-year ('base flood") and 500-year ("check flood") events. However, if overtopping of the roadway can occur, the incipient roadway overtopping condition is then the worst case for scour because it will usually create the greatest flow contraction and highest water velocities at the bridge. This overtopping condition may occur less than every 100 years and therefore over-ride the base flood (100-yr) design condition or it could occur between 100 and 500 years and over-ride the 500-year (check flood) condition. All bridge scour depths are calculated for the following flood conditions, depending on the recurrence interval for the overtopping flood:

- $Q_{overtopping} > Q_{500}$: Both the 100-year and 500-year flood scour depths are analyzed
- $Q_{100} < Q_{overtopping} < Q_{500}$: The 100-year flood and the overtopping flood scour depths are analyzed
- $Q_{overtopping} < Q_{100}$: Only the overtopping flood scour depth is analyzed

The top of the footing should be set below the potential scour elevation for the 100-year scour or the roadway-overtopping flood, whichever is the deepest. The bottom of the footing should be set below the potential scour elevation for the Check Flood, which will be either the roadway-overtopping flood or the 500-year flood.

Minimum pile and drilled shaft tip elevations and spread footing elevations should be based on providing the nominal bearing resistance (resistance factor equal to 1.0) with the estimated 500-year flood scour depths or with the scour depths from the overtopping flood if the recurrence interval of the overtopping flood is greater than 100-years. A resistance factor of 0.70 may be used in foundation design with the estimated 100-year flood scour depths. However, if the recurrence interval of the overtopping flood is less than 100 years, the resistance factor should be evaluated on a case by case basis using engineering judgment and assessing the long term hydraulics and scour potential of the site. Overtopping recurrence intervals that are much less than every 100 years are not considered extreme events and therefore resistance factors associated with the no-scour condition may be more appropriate to use.

For footings constructed on bedrock, provide recommendations regarding the scour potential of the bedrock to the Hydraulics designer. Some types of “bedrock” are very weak and extremely susceptible to erosion and scour. At present, there are no specific recommendations or guidelines to use to determine the scour potential of bedrock types typically found in Oregon. Good engineering judgment should be used in estimating the scour potential of marginally “good” quality rock, taking into account rock strength, RQD, joint spacing, joint filling material, open fractures, weathering, degradation characteristics and other factors. See if any exposed bedrock at the site shows signs of erosion or degradation or if there is a history of bedrock scour in the past. Signs of bedrock scour may include the undermining of existing footings, steeply incised stream banks or scour holes in the bedrock streambed. If any doubts remain, drilled shafts should be considered.

Spread footings supporting bridge abutments should generally be constructed assuming the contraction and degradation scour depths calculated for the main channel are present at the abutment location. Exceptions to this policy include bridge abutment footings that are constructed on non-erodable rock and/or located sufficiently far away from the main channel (e.g. long approach ramps or viaduct). Refer to the *ODOT Hydraulics Manual* for more guidance regarding scour, riprap protection and footing depth requirements. Loose riprap is not considered permanent protection. Design riprap protected abutments according to the guidance and recommendations outlined in *FHWA HEC-18 manual, “Evaluating Scour at Bridges*” (Richardson, E. et al., 2001).

8.9.3 Seismic Design

Chapter 6 describes ODOT seismic practices regarding design criteria, performance requirements, ground motion characterization, liquefaction analysis, ground deformation and mitigation. Once the seismic analysis is performed the results are applied to foundation design in the Extreme Event I limit state analysis as described in AASHTO Section 10. Also refer to, and be familiar with, *Section 1.1.4; “Foundation Modeling*”, of the *ODOT Bridge Design and Drafting Manual*.

This section describes the various methods bridge designers use to model the response of bridge foundations to seismic loading and also the geotechnical information required to perform the analysis.

In general, nominal resistances are used in seismic design except for pile and shaft uplift conditions (see AASHTO Article 10.5.5.3).

If the foundation soils are determined to be susceptible to liquefaction, then spread footings should not be recommended for foundation support of the structure unless proven ground improvement techniques are employed to stabilize the foundation soils and eliminate the liquefaction potential. Otherwise, a deep foundation should be recommended.

Deep foundations (piles and drilled shafts) supporting structures that are constructed on potentially liquefiable soils are normally structurally checked for two separate loading conditions; i.e. with and without liquefaction. Nominal (unfactored) resistances, downdrag loads (if applicable) and soiLPile interaction parameters should be provided for both nonliquefied and liquefied foundation conditions. Communication with the structural designer is necessary to insure that the proper foundation design information is provided.

If the predicted amount of earthquake-induced embankment deformation (lateral deformation and/or settlement) is excessive then assessments should be made of approach fill performance and the potential for bridge and approach fill damage. The need for possible liquefaction mitigation measures should then be evaluated. Refer to the "ODOT Liquefaction Mitigation Policy", in **Chapter 6**, for more guidance on ODOT liquefaction mitigation policies.

8.10 Soil Loads on Buried Structures

For tunnels, culverts and pipe arches, the soil loads to be used for design shall be as specified in Sections 3 and 12 of the AASHTO LRFD Bridge Design Specifications.

8.11 Spread Footing Design

Refer to AASHTO Article 10.6 for spread footing design requirements.

Once footings are selected as the preferred design alternative, the general spread footing design process can be summarized as follows. Close communication and interaction is required between the structural and geotechnical designers throughout the footing design phase.

- Determine footing elevation based on location of suitable bearing stratum and footing dimensions (taking into account any scour requirements, if applicable)
- Determine foundation material design parameters and groundwater conditions
- Calculate the nominal bearing resistance for various footing dimensions (consult with structural designer for suitable dimensions)
- Select resistance factors depending on design method(s) used; apply them to calculated nominal resistances to determine factored resistances
- Determine nominal bearing resistance at the service limit state
- Check overall stability (determine max. bearing load that maintains adequate slope stability)

For footings located in waterways, the bottom of the footing should be below the estimated depth of scour for the check flood (typically the 500 year flood event or the overtopping flood). The top of the footing should be below the depth of scour estimated for the design flood (either the overtopping or 100-year event). As a minimum, the bottom of all spread footings should also be at least 6 feet below the lowest streambed elevation unless they are keyed full depth into bedrock that is judged not to erode over the life of the structure. Spread footings are not permitted on soils that are predicted to liquefy under the design seismic event.

8.11.1 Nearby Structures

Refer to AASHTO, Article 10.7.1.6.4. Issues to be investigated include, but are not limit to, settlement of the existing structure due to the stress increase caused by the new footing, decreased overall stability due to the additional load created by the new footing, and the effect on the existing structure of excavation, shoring, and/or dewatering to construct the new foundation.

8.11.2 Service Limit State Design of Footings

Footing foundations shall be designed at the service limit state to meet the tolerable movements for the structure in accordance with AASHTO Article 10.5.2. The nominal unit bearing resistance at the service limit state, q_{serve}, shall be equal to or less than the maximum bearing stress that results in settlement that meets the tolerable movement criteria for the structure.

8.12 Driven Pile Foundation Design

Refer to AASHTO, Article 10.7 for pile design requirements. Pile design should meet or exceed the requirements specified for each limit state. ODOT standards and policies regarding pile foundation design and construction shall also be followed.

All driven piles shall be accepted based on bearing resistance determined from dynamic formula, wave equation, dynamic measurements with signal matching (PDA/CAPWAP) or full-scale load testing. Pile acceptance shall not be accepted based solely on static analysis.

For piles requiring relatively low nominal resistances (<600 kips) and without concerns about high driving stresses, the dynamic formula is typically used for determining pile driving acceptance criteria. In cases where piles are driven to higher resistances or where high pile driving stresses are a concern, such as short, end bearing piles, the wave equation is typically used for both drivability analysis and in determining the final driving acceptance criteria.

Pile acceptance based on the pile driving analyzer (PDA) is typically reserved for projects where it is economically advantageous to use, or for cases where high pile driving stresses are predicted and require monitoring. The PDA (with signal matching) method can be most cost effective on projects that have a large number of long, high capacity, friction piles.

Full-scale static pile load tests are less common in practice due to their inherent expense. However, they may be economically justified in cases where higher bearing resistances can be verified through load testing and applied in design to reduce the cost of the pile foundation. If static load testing is considered for a project it should be conducted early on in the design stage so the results may be utilized in the design of the structure. Also, the pile load test should be taken to complete failure if at all possible. Refer to AASHTO Section 10 for descriptions on how to use the results of the static load tests results to determine driving criteria. Static load test results should be used in combination with either PDA testing or wave equation analysis to develop final driving criteria for the production piles.

Once the pile (bent) locations and foundation materials and properties are defined, the pile foundation design process for normal bridge projects typically consists of the following:

- Determine scour depths (if applicable)
- Determine liquefaction potential and depths; estimate seismic induced settlement (if applicable)
- Evaluate long-term embankment settlement and downdrag potential
- Select most appropriate pile type
- Select pile dimension (size) based on discussions with structural designer regarding preliminary pile loading requirements (axial and lateral)
- Establish structural nominal resistance of the selected pile(s)
- Conduct static analysis to calculate nominal single pile resistance as a function of depth for the strength and extreme limit states (or a pile length for a specified resistance)
- Select resistance factors based on the field method to be used for pile acceptance (e.g. dynamic formula, wave equation, PDA/CAPWAP, etc.)
- Calculate single pile factored resistance as a function of depth
- Estimate downdrag loads; consolidation and/or seismic-induced (if applicable)

- Calculate pile/pile group settlement or pile lengths required to preclude excessive settlement
- Determine nominal uplift resistance as a function of depth
- Determine p-y curve parameters for lateral load analysis
  - o Modify parameters for liquefied soils (if applicable)
  - o Provide p-y multipliers as appropriate for pile groups. P-Y multipliers are not required for pile (or shafts) groups installed in rock sockets where calculated lateral displacements are minimal (i.e., <0.50”).
- Determine required pile tip elevation(s) based on geotechnical design requirements including the effects of scour, downdrag, or liquefaction
- Obtain and verify final pile tip elevations and required resistances (factored and unfactored loads) from the structural designer, finalize required pile tip elevations and assess the following:
  - o Determine the need to perform a pile drivability analysis to obtain required tip elevation
  - o Evaluate pile group settlement (if applicable). If settlement exceeds allowable criteria, adjust pile lengths or the size of the pile layout and/or lengths
- Determine the need for pile tip reinforcement

8.12.1 Required Pile Tip Elevation

Required pile tip elevations should typically be provided for all pile foundation design projects. The required pile tip elevation is provided to ensure the constructed foundation meets the design requirements of the project, which may include any or all of the following conditions and criteria:

- Pile tip reaches the designated bearing layer
- Scour
- Downdrag
- Uplift
- Lateral loads

A general note is included on the bridge plans designating the “Pile Tip Elevation for Minimum Penetration” for each bent.

The required tip elevation may require driving into, or through, very dense soil layers resulting in potentially high driving stresses. Under these conditions a wave equation driveability analysis is necessary to make sure the piles can be driven to the required embedment depth (tip elevation). Higher grade steel (ASTM A252, Grade 3 or A572, Grade 50) are sometimes specified if needed to meet driveability criteria. If during the structural design process, adjustments in the required tip elevations are necessary, or if changes in the pile diameter are necessary, the geotechnical designer should be informed so that pile drivability can be re-evaluated.

8.12.2 Pile Drivability Analysis and Wave Equation Usage

High pile stresses often occur during pile driving operations and, depending on subsurface and loading conditions, a Wave Equation analysis should always be considered to evaluate driving stresses and the probability of pile damage. A pile driveability analysis is typically used in most pile foundation designs to determine the nominal geotechnical resistance that a pile can be driven to without damage. Foundation piles should typically be driven to the highest geotechnical axial resistance feasible based on wave equation analysis so the maximum structural resistance of the pile is utilized, resulting in the most cost-effective pile design.

All piles driven to nominal resistances greater than 600 kips should be driven based on wave equation criteria. Piles driven to nominal resistance less than or equal to 600 kips may also require a wave equation analysis depending on the subsurface conditions (such as very short end bearing piles) and the pile loads. Engineering judgment is required in this determination. Pile driving stresses should be limited to those described in AASHTO Article 10.7.8.

8.12.3 Pile Setup and Restrike

Using a waiting period and restrike after initial pile driving may be advantageous in certain soil conditions to optimize pile foundation design. After initially driving the piles to a specified tip elevation, the piles are allowed to “set up” for a specified waiting period, which allows pore water pressures to dissipate and soil strength to increase. The piles are then restruck to confirm the required nominal resistance.

The length of the waiting period depends primarily on the strength and drainage characteristics of the subsurface soils (how quickly the soil can drain) and the required nominal resistance. The minimum waiting period specified in the Standard Specifications is 24 hours. If needed, this waiting period may be extended in the contract special provisions to provide additional time for the soils to gain strength and the piles to gain resistance. However, consideration should be given to increased contractor standby costs that may be incurred by extended waiting periods. The pile design should compare the cost and risk of extending the standard waiting period to gain sufficient strength versus designing and driving the piles deeper to achieve the required bearing.

For projects with piles that require restrike, at least 1 pile per bent or 1 in 10 piles in a group (whichever is more) should typically be restruck for pile acceptance. Additional restrike verification testing should be conducted on any piles that indicate lower resistance at the end of initial driving or if subsurface conditions vary substantially within a pile group. Restrike should be performed using a warm pile hammer.

If pile acceptance from restrike is based on measured blow counts (dynamic formula or wave equation methods), the restrike resistance (blow count) should be determined by measuring the total pile set in the first 5 blows of driving and in successive 5 blow increments thereafter up to a total of at least 20 blows or until refusal driving conditions are reached (>20 blows per inch). The driving resistance reported (in blows per inch) is then determined by taking the inverse of the set per each 5 blow increment.

8.12.4 Driven Pile Types, and Sizes

The pile types generally used on most permanent structures are steel pipe piles (driven both open and closed-end) and steel H-piles. Either H-pile or open-end steel pipe pile can be used for end bearing conditions. For friction piles, steel pipe piles are often preferred because they can be driven closed-end (as full displacement piles) and because of their uniform cross section properties, which provides the same structural bending capacity in any direction of loading. This is especially helpful under seismic loading conditions where the actual direction of lateral loading is not precisely known. Uniform section properties also aid in pile driving. Pipe piles are available in a variety of diameters and wall thickness; however there are some sizes that are much more common than others and therefore usually less expensive. The most common pipe pile sizes used on ODOT projects are:

- PP 12.75 x 0.375
- PP 16 x 0.500
- PP 20 x 0.500
- PP 24 x 0.500

Timber piles are occasionally used for temporary detour structures and occasionally on specialty bridges, for retrofit or repair, and, on rare occasions, "in-kind" widening projects. ODOT standard prestressed concrete piles are rarely used due to the following reasons:

- They typically have less bending capacity than steel piles for a given size
- They are difficult to connect to the pile cap for uplift resistance
- They are inadequately reinforced for plastic hinge formation
- Pile driving damage potential
- Splicing difficulties
- Cost, (typically more expensive than steel for a given capacity)

Prestressed concrete piles may however be appropriate in some areas like low seismic zones or highly corrosive environments. The use of prestressed concrete piles is not prohibited in ODOT if they are properly designed and cost effective.

- The typical ASTM steel specifications and grades used in ODOT are as follows:
- Steel Pipe Piles: ASTM A 252, Grade 2 & 3
- Steel H-piles: ASTM A 572, Grade 50

The higher-grade steel may be required in some cases due to predicted high driving stresses or due to high lateral bending stresses. Refer to AASHTO for ASTM requirements for other pile types.

Reinforced pile tips may be warranted in some cases where piles may encounter, or are required to penetrate through, very dense cobbles and/or boulders. Pile tips are useful in protecting the tip of the pile from damage. However, installing a reinforced pile tip does not eliminate all potential for pile damage. High driving stresses may occur at these locations and still result in pile damage located just above the reinforce pile tip. A driveability analysis should be performed in these cases where high tip resistance is anticipated. All reinforced tips are manufactured from high strength (A27) steel.

Tip reinforcement for H-piles are typically called pile points. These come in a variety of shapes and designs. H-pile tips are listed on the ODOT QPL. For pipe piles tip reinforcement are typically termed “shoes”, although close-end “points”, like conical points, are also available. Pipe pile shoes may be either inside or outside-fit. Besides protecting the pile tip, inside-fit shoes are sometimes specified to help in delaying the formation of a pile “plug” inside the pipe pile so the pile may penetrate further into, or even through, a relatively thin dense soil layer. If outside-fit shoes are specified, the outside lip of the shoe may affect (reduce) the pile skin friction and this effect should be taken into account in the pile design.

8.12.5 Extreme Event Limit State Design

For the applicable factored loads for each extreme event limit state, the pile foundations shall be designed to have adequate factored axial and lateral resistance.

8.12.5.1 Scour Effects on Pile Design

The effects of scour, where scour can occur, shall be evaluated in determining the required pile penetration depth. The pile foundation shall be designed so that the pile penetration after the design scour events satisfies the required nominal axial and lateral resistance. The pile foundation shall also be designed to resist debris loads occurring during the flood event in addition to the loads applied from the structure. The resistance factors for scour shall be those described in **Section 8.10.** The axial resistance of the material lost due to scour should be determined using a static analysis. The axial resistance of the material lost due to scour should not be factored. The piles will need to be driven to the required nominal axial resistance plus the skin friction resistance that will be lost due to scour.

From Equation 8-1:

$$\Sigma\eta_i\gamma_i Q_i \leq \varphi R_n \qquad (8\text{-}1)$$

The summation of the factored loads ($\Sigma\gamma iQi$) must be less than or equal to the factored resistance (φR_n). Therefore, the nominal resistance needed, R_n, must be greater than or equal to the sum of the factored loads divided by the resistance factor φ:

$$R_n \geq (\Sigma\gamma iQi)/\varphi_{dyn}$$

For scour conditions, the resistance that the piles must be driven to needs to account for the resistance in the scour zone that will not be available to contribute to the resistance required under the extreme event (scour) limit state. The total driving resistance, Rndr, needed to obtain Rn, is therefore:

$$R_{ndr} = R_n + R_{scour}$$

Note that R_{scour} remains unfactored in this analysis to determine R_{ndr}.

Pile design for scour is illustrated further in **Figure 8-2**, where,

- R_{scour} = skin friction which must be overcome during driving through scour zone (KIPS)
- Q_p = ($\Sigma\gamma iQi$) = factored load per pile (KIPS)
- D_{est}.= estimated pile length needed to obtain desired nominal resistance per pile (FT)
- φ_{dyn} = resistance factor

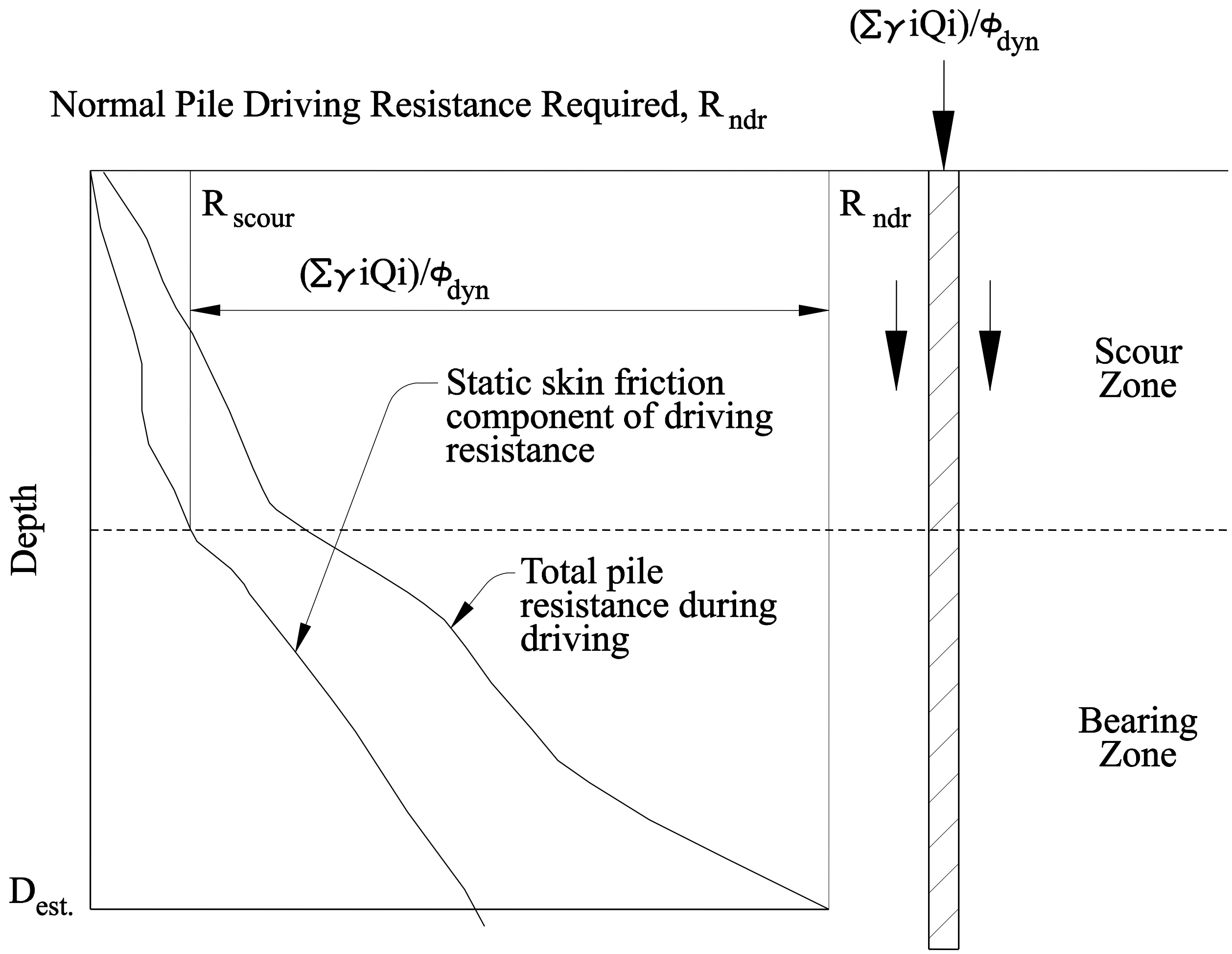

Figure 8-2. Design of pile foundations for scour

8.12.5.1.1 Seismic Design for Pile Foundations

For seismic design, all soil within and above liquefiable zones, shall not be considered to contribute axial compressive resistance. Downdrag resulting from liquefaction induced settlement shall be determined as specified in AASHTO and included in the loads applied to the foundation. Static downdrag loads should not be combined with seismic downdrag loads due to liquefaction.

In general, the available factored geotechnical resistance should be greater than the factored loads applied to the pile, including the downdrag, at the extreme event limit state. The pile foundation shall be designed to structurally resist the downdrag plus structure loads. Pile design for liquefaction downdrag is illustrated in **Figure 8-3**, where,

- R_{Sdd} = skin friction which must be overcome during driving through downdrag zone
- Q_p = ($\Sigma\gamma iQi$) = factored load per pile, excluding downdrag load
- DD = downdrag load per pile
- D_{est}. = estimated pile length needed to obtain desired nominal resistance per pile
- φ_{seis} = resistance factor for seismic conditions
- γ_p = load factor for downdrag

The nominal bearing resistance of the pile needed to resist the factored loads, including downdrag, is therefore,

- $R_n = (\Sigma\gamma iQi)/\varphi_{seis} + \gamma_p DD/\varphi_{seis}$

The total driving resistance, R_{ndr}, needed to obtain R_n, must account for the skin friction that has to be overcome during pile driving that does not contribute to the design resistance of the pile. Therefore:

- $R_{ndr} = R_n + R_{Sdd}$

Note that R_{Sdd} remains unfactored in this analysis to determine R_{ndr}.

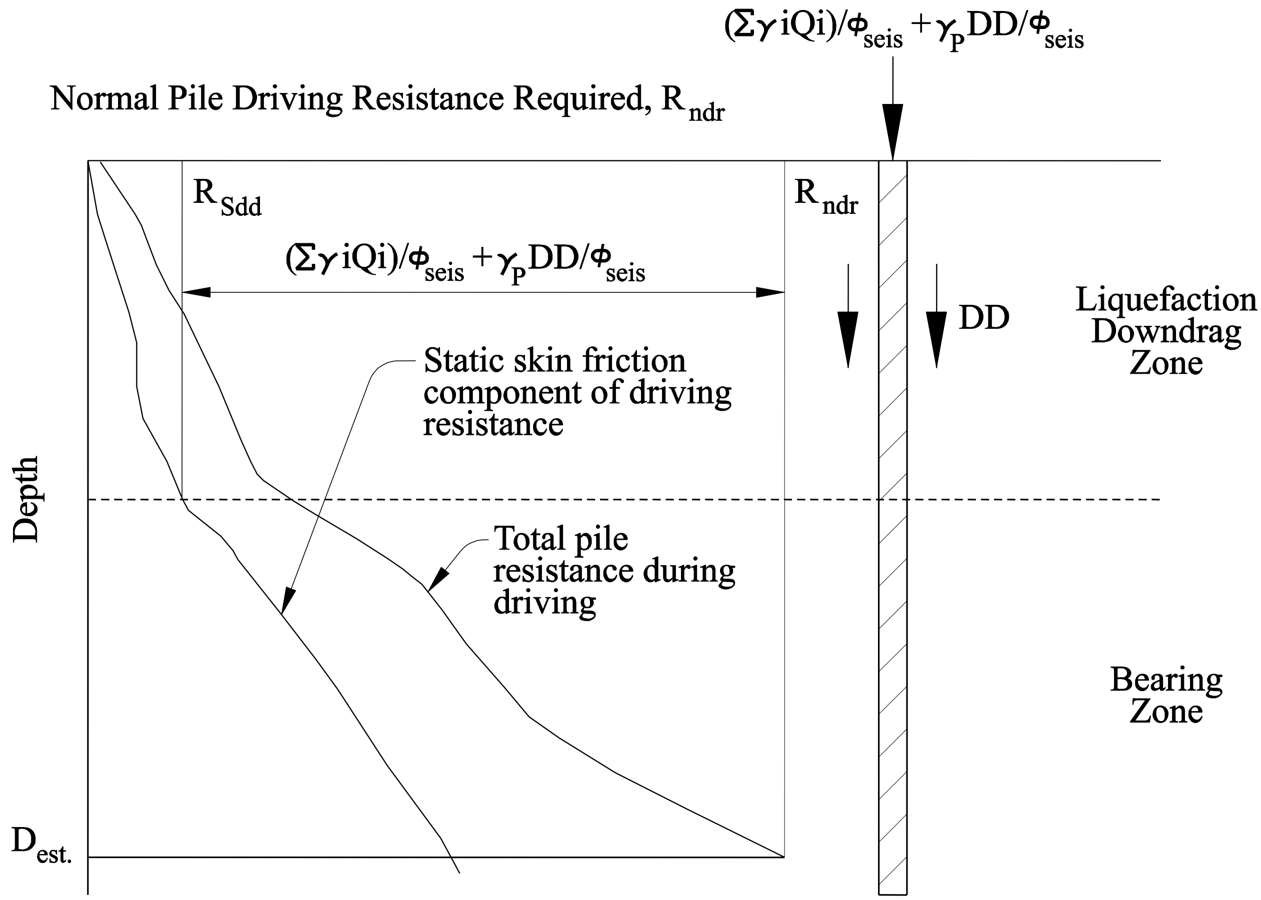

Figure 8-3. Design of pile foundations for liquefaction downdrag (WSDOT, 2006)

In the instance where it is not possible to obtain adequate geotechnical resistance below the lowest layer contributing to downdrag (e.g., friction piles) to fully resist the downdrag, or if it is anticipated that significant deformation will be required to mobilize the geotechnical resistance needed to resist the factored loads including the downdrag load, the structure should be designed to tolerate the settlement resulting from the downdrag and the other applied loads.

The static analysis procedures in AASHTO should be used to estimate the skin friction within, above and below, the downdrag zone and to estimate pile lengths required to achieve the required bearing resistance. For this calculation, it should be assumed that the soil subject to downdrag still contributes overburden stress to the soil below the downdrag zone.

The pile foundation shall also be designed to resist the horizontal force resulting from lateral spreading, if applicable, or the liquefiable soil shall be improved to prevent liquefaction and lateral spreading. For lateral soil resistance of the pile foundation, the P-y curve soil parameters should be reduced to account for liquefaction. To determine the amount of reduction, the duration of strong shaking and the ability of the soil to fully develop a liquefied condition during the period of strong shaking should be considered.

The force resulting from lateral spreading should be calculated as described in **Chapter 6**. In general, the lateral spreading force should not be combined with the seismic forces. See **Chapter 6** , "Seismic Design" for additional guidance regarding this issue.

8.13 Drilled Shaft Foundation Design

Refer to AASHTO Article 10.8 for drilled shaft design requirements. Common shaft sizes range from 3 feet to 8 feet in diameter in 6 inch increments. Larger shaft diameters are also possible. The minimum shaft diameter is 12 inches.

Once the shaft locations and foundation materials and properties are known, the drilled shaft design process for normal bridge projects typically consists of the following:

- Determine scour depths (if applicable),
- Determine liquefaction potential and depths; estimate seismic induced settlement (if applicable),
- Evaluate long-term embankment settlement and downdrag potential,
- Select most appropriate shaft diameter(s) in consultation with structure designer,
- Determine (in consult with the structure designer) whether or not permanent casing will be used,
- Calculate nominal single shaft resistance as a function of depth,
- Select and apply resistance factors to nominal resistance
- Estimate downdrag loads (if applicable),
- Estimate shaft or shaft group settlement and adjust shaft diameter or lengths if necessary to limit settlement to service state limits,
- Determine p-y curve parameters for lateral load analysis; modify parameters for liquefied soils (if applicable),

The diameter of shafts will usually be controlled by the superstructure design loads and the configuration of the structure but consideration should also be given to the foundation materials to be excavated. If boulders or large cobbles are anticipated, attempt to size the shafts large enough so the boulders or cobbles can be more easily removed if possible. Shaft diameters may also need to be increased to withstand seismic loading conditions. The geotechnical engineer and the bridge designer should confer and decide early on in the design process the most appropriate shaft diameter(s) to use for the bridge, given the loading conditions, subsurface conditions at the site and other factors. Also decide early on with the bridge designer if permanent casing is desired since this will affect both structural and geotechnical designs. Specify each shaft as either a “friction” or “end bearing” shaft since this dictates the final cleanout requirements in the specifications.

When the drilled shaft design calls for a specified length of shaft embedment into a bearing layer (rock socket) and the top of the bearing layer is not well defined, an additional length of shaft reinforcement should be added to the length required to reach the estimated tip elevation. This extra length is required to account for the uncertainty and variability in the final shaft length. This practice is much preferred instead of having to splice on additional reinforcement in the field during which time the shaft excavation remains open. Any extra reinforcement length that is not needed can be easily cut off prior to steel placement once the final shaft tip elevation is known. CSL tubes would also need to be either cut off and recapped or otherwise adjusted. This additional reinforcement length should be determined by the geotechnical engineer based on an evaluation of the site geology, location of borehole information and the potential variability of the bearing layer surface at the plan location off the shaft. The additional recommended length should be provided in the Geotechnical Report and included in the project Special Provisions. Refer to the *Standard Special Provisions for Section 00512* for further guidance and details of this application.

If a minimum rock embedment (socket) depth is required, specify the reason for the rock embedment. Try to minimize hard rock embedment depths if possible since this adds substantially to the cost of drilled shafts.

Settlement may control the design of drilled shafts in cases where side resistance (friction) is minimal, loads are high and the shafts are primarily end bearing on compressible soil. The shaft settlement necessary to mobilize end bearing resistance may exceed that allowed by the bridge designer. Confer with the bridge designer to determine shaft service loads and allowable amounts of shaft settlement. Refer to the AASHTO methods to calculate the settlement of individual shafts or shaft groups. Compare this settlement to the maximum allowable settlement and modify the shaft design if necessary to reduce the estimated settlement to acceptable levels.

8.13.1 Nearby Structures

Where shaft foundations are placed adjacent to existing structures, the influence of the existing structure(s) on the behavior of the foundation, and the effect of the foundation on the existing structures, including vibration effects due to casing installation, should be investigated. In addition, the impact of caving soils during shaft excavation on the stability of foundations supporting adjacent structures should be evaluated. At locations where existing structure foundations are adjacent to the proposed shaft foundation, or where a shaft excavation cave-in could adversely affect an existing foundation, the design should require that casing be advanced as the shaft excavation proceeds.

8.13.2 Scour

The effect of scour shall be considered in the determination of the shaft penetration. The shaft foundation shall be designed so that the shaft penetration and resistance remaining after the design scour events satisfies the required nominal axial and lateral resistance. For this calculation, it shall be assumed that the soil lost due to scour does not contribute to the overburden stress in the soil below the scour zone. The shaft foundation shall be designed to resist debris loads occurring during the flood event in addition to the loads applied from the structure.

Resistance factors for use with scour are described in **Section 8.9.2**. The axial resistance of the material lost due to scour shall not be included in the shaft resistance.

8.13.3 Extreme Event Limit State Design of Drilled Shafts

The provisions of **Section 8.12.5** shall apply. The nominal shaft resistance available to support structure loads plus downdrag shall be estimated by considering only the positive skin and tip resistance below the lowest layer contributing to the downdrag. For this calculation, it shall be assumed that the soil contributing to downdrag does contribute to the overburden stress in the soil below the downdrag zone.

In general, the available factored geotechnical resistance should be greater than the factored loads applied to the shaft, including the downdrag, at the extreme limit state. The shaft foundation shall be designed to structurally resist the downdrag plus structure loads.

In the instance where it is not possible to obtain adequate geotechnical resistance below the lowest layer contributing to downdrag (e.g., friction shafts) to fully resist the downdrag, the structure should be designed to tolerate the settlement resulting from the downdrag and the other applied loads.

8.14 Micropiles

Micropiles shall be designed in accordance with Article 10.9 of the *AASHTO LRFD Bridge Design Specifications*. Additional information on micropile design may be found in the FHWA Reference Manual; *Micropile Design and Construction* (Publication No. FHWA NHI-05-039).

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Chapter

9

9 Embankments – Analysis and Design

9.1 General

This chapter addresses the analysis and design of rock and earth embankments. Also addressed briefly are foundation improvement (ground improvement), the use of lightweight fill and settlement and stability mitigation techniques. Bridge approach embankments, defined as fill under bridge ends, are not covered in this chapter, but are addressed in **Chapter 8** and in **Chapter 6**. The primary geotechnical issues that impact embankment performance are overall (global) stability, internal (slope) stability, settlement, material selection, compaction, and constructability. For the purposes of this chapter, embankments include the following:

- Rock embankments, also known as all-weather embankments, are defined as fills in which the material is non-moisture-density testable and is composed of durable granular materials.
- Earth embankments are fills that are typically composed of onsite or imported borrow, and could include a wide variety of materials from fine to coarse grain. The material is usually moisture-density testable.
- Lightweight fills contain lightweight fill or recycled materials as a significant portion of the embankment volume, and the embankment construction is specified by special provision. Lightweight fills are most often used as a portion of the bridge approach embankment to mitigate settlement or in landslide repairs to reestablish roadways.

Embankments under 10 feet high in areas of stable ground and with slopes flatter than 2:1 generally do not require a detailed geotechnical investigation and analysis. These embankments can be designed based on past experience with similar soils and on engineering judgment. Embankments over 10 feet high, with steeper slopes, constructed in problem soil areas, or from specially designed or unique materials will require a detailed geotechnical analysis, development of special provisions and possibly details included in the contract plans.

9.2 Design Considerations

9.2.1 Typical Embankment Materials and Compaction

New embankments and embankment widenings require suitable fill materials be used and properly compacted with the right equipment for the type of material. Compaction control of soil embankments requires the development of moisture-density relationships to allow measurement of in-place compaction during construction. Tamping foot rollers and specified passes of the rollers are used to achieve the required density of the fill. Non-durable rock materials may require additional compactive effort beyond the usual soil construction methods to prevent long term settlement of an embankment. The *ODOT Standard Specifications for Construction* identifies the acceptable embankment construction methods for soil, non-durable rock and rock materials. The geotechnical designer should determine during the exploration program if any of the material from planned earthwork excavations will be suitable for embankment construction. Consideration should be given as to whether the material is moisture sensitive and difficult to compact during wet weather.

9.2.1.1 All-Weather Embankment Materials

ODOT projects are increasingly being constructed within shorter time frames that may require fill placement occurring at any time of the year. Clean, granular, all-weather embankment materials allow the contractor the ability to properly place and compact fill materials year round. Clean, granular fill material use also provides better access to work areas, and facilitates construction staging and traffic detouring. The *ODOT Standard Specifications* identify 2 materials considered to be suitable for all-weather construction: Selected Stone Backfill (*section 00330.15*), and Stone Embankment Material (*section 00330.16*). Both of these materials have in common the use of “durable” material, as defined in *section 00110.20 Durable Rock*. Compaction tests cannot be applied to coarse material with any degree of accuracy; therefore, a method specification approach is typically specified for granular embankments, as described in *section 00330.43 Non-Moisture Density Testable Materials*.

9.2.1.2 Durable and Non-Durable Rock Materials

Special consideration should be given during design to the type of material that will be used in rock embankments. In some areas of the state, moderately weathered or very soft rock may be encountered in cuts and used as embankment fill. Follow these guidelines:

- Degradable fine grained sandstone and siltstone are often encountered in the cuts and the use of this material in embankments can result in significant long term settlement and stability problems as the rock degrades, unless properly compacted with heavy tamping foot rollers (Machan, et al., 1989). The slake durability test (ASTM D4644) should be performed if the geologic nature of the rock source proposed indicates that poor durability rock is likely to be encountered.
- When the rock is found to be non-durable, it should be physically broken down and compacted as earth embankment provided the material meets or exceeds common borrow requirements. Special compaction requirements, defined by method specification, may be needed for these materials. In general, tamping foot rollers work best for breaking down the rock fragments. The minimum size roller should be about 30 tons. Specifications should include the maximum size of the rock fragments and maximum lift thickness. These requirements will depend on the hardness of the rock, and a test section should be incorporated into the contract to verify that the Contractor’s methods

will achieve compaction and successfully break down the material. In general, both the particle size and lift thickness should be limited to 12 inches.

9.2.1.3 Earth Embankments

Embankments constructed with common borrow materials must be placed in accordance with the procedures of the *ODOT Standard Specifications, section 00330 Earthwork*. These specifications are intended for use where it is not necessary to strictly control the strength properties of the embankment material and where all-weather construction is not required.

9.2.2 Embankment Stability Assessment

In general, embankments 10 feet or less in height with 2H:1V or flatter side slopes, may be designed based on past precedence and engineering judgment provided there are no known problem soil conditions such as liquefiable sands, organic soils, soft/loose soils, or potentially unstable soils such as clay, estuarine deposits, or peat. Embankments over 10 feet in height or any embankment on soft and/or unstable soils or those comprised of light weight fill require more in depth stability analyses, as do any embankments with side slope inclinations steeper than 2H:1V. Moreover, any fill placed near or against a bridge abutment or foundation, or that can impact a nearby buried or above-ground structure, will likewise require stability analyses by the geotechnical designer. Slope stability analysis, discussed in **Chapter 7**, are to be conducted in accordance with the standard of practice for geotechnical engineering.

The geotechnical designer should determine key issues that need to be addressed to perform stability analysis. These include:

- Is the site underlain by soft silt, clay or peat? If so, a staged stability analysis (staged construction of fill with stability analysis at each stage) may be required.
- Are site constraints such that slopes steeper than 2H:1V are required? If so, a detailed slope stability assessment is needed to evaluate the various alternatives.
- Is the embankment temporary or permanent? Factors of safety for temporary embankments may be lower than for permanent ones, depending on the site conditions and the potential for variability.
- Will the new embankment impact nearby structures or bridge abutments? If so, more thorough sampling, testing and analysis are required.
- Are there potentially liquefiable soils at the site? If soil, seismic analysis to evaluate this may be warranted see **Chapter 6** and ground improvement may be needed.

Several methodologies for analyzing the stability of slopes are detailed or identified by reference in **Chapter 7** and are directly applicable to earth embankments.

9.2.2.1 Safety Factors

All embankments not supporting or potentially impacting structures shall have a minimum safety factor of 1.25. Embankments supporting or potentially impacting non-critical structures shall have a minimum factor of safety of 1.3. As discussed in **Section 8.7**, all bridge approach embankments and embankments supporting critical structures shall have a safety factor of 1.5.

Under seismic conditions, only those portions of the new embankment that could impact an adjacent structure such as bridge abutments and foundations or nearby buildings require seismic analyses and an adequate overall stability resistance factor (i.e., a maximum resistance factor of 0.9 or a minimum factor of safety of 1.1). See **Chapter 6** for specific requirements regarding seismic design of embankments.

9.2.2.2 Strength Parameters

Strength parameters are required for any stability analysis. Strength parameters appropriate for the different types of stability analyses are determined based on **Chapter 5** and by reference to *FHWA Geotechnical Engineering Circular No. 5 (Sabatini, et al., 2002*). If the critical stability is under drained conditions, such as in sand or gravel, then effective stress analysis using a peak friction angle is appropriate and should be used for stability assessment. In the case of over-consolidated fine grained soils, a friction angle based on residual strength may be appropriate. This is especially true for soils that exhibit strain softening or are particularly sensitive to shear strain. If the critical stability is under undrained conditions, such as in most clays and silts, a total stress analysis using the undrained cohesion value with no friction is appropriate and should be used for stability assessment.

For staged construction, both short (undrained) and long term (drained) stability need to be assessed. At the start of a stage the input strength parameter is the undrained cohesion. The total shear strength of the fine-grained soil increases with time as the excessive pore water dissipates, and friction starts to contribute to the strength.

9.2.3 Embankment Settlement Assessment

New embankments and embankment widenings should be analyzed using the methods discussed in the *FHWA Soils and Foundation Reference Manual,* (Samtani, N. and Nowatzki, E. 2006). Laboratory test results of foundation soil samples obtained should be used as a basis for determining the primary and secondary settlement amounts and rates. Because primary consolidation and secondary compression can continue to occur long after the embankment is constructed (post construction settlement), they represent the major settlement concerns for embankment design and construction. Post construction settlement can damage structures and utilities located within the embankment, especially if those facilities are also supported by adjacent soils or foundations that do not settle appreciably, leading to differential settlements. If the primary consolidation is allowed to occur prior to placing utilities or building structures that would otherwise be impacted by the settlement, the impact is essentially mitigated. However, it can take weeks to years for primary settlement to be essentially complete, and significant secondary compression of organic soils can continue for decades. Many construction projects cannot absorb the scheduling impacts associated with waiting for primary consolidation and/or secondary compression to occur. Therefore, estimating the time rate of settlement is often as important as estimating the magnitude of settlement.

9.2.3.1 Settlement Analysis

The key parameters for evaluating the amount of settlement below an embankment include knowledge of:

- The subsurface profile including soil types, layering, groundwater level and unit weights;
- The compression indexes for primary, rebound and secondary compression from laboratory test data, correlations from index properties, and results from settlement monitoring programs completed for the site or nearby sites with similar soil conditions.
- The geometry of the proposed fill embankment, including the unit weight of fill materials and any long term surcharge loads.

9.2.3.2 Analytical Tools

The primary consolidation and secondary settlement can be calculated by hand or by using computer programs such as EMBANK (FHWA, 1993). Alternatively, spreadsheet solutions can be easily developed. The advantage of computer programs such EMBANK are that multiple runs can be made quickly, and they include subroutines to estimate the increased vertical effective stress caused by the embankment or other loading conditions.

9.3 Stability Mitigation

A variety of techniques are available to mitigate inadequate slope stability for new embankments or embankment widenings. These techniques include staged construction to allow for the underlying soils to gain strength, base reinforcement, ground improvement, use of lightweight fill, and construction of toe berms (counterweights) and shear keys. A summary of these instability mitigation techniques is presented below along with the key design considerations.

9.3.1 Staged Construction

Where soft compressible soils are present below a new embankment location and it is not economical to remove and replace these soils with compacted fill, the embankment can be constructed in stages to allow the strength of the compressible soils to increase under the weight of new fill. Construction of the second and subsequent stages commences when the strength of the compressible soils is sufficient to maintain stability. In order to define the allowable height of fill for each stage and maximum rate of construction, detailed geotechnical analysis is required. The analysis to define the height of fill placed during each stage and the rate at which the fill is placed is typically completed using a limit equilibrium slope stability program along with time rate of settlement analysis to estimate the percent consolidation required for stability. Field monitoring of settlement and pore water pressures are usually required during construction.

9.3.2 Base Reinforcement

Base reinforcement may be used to increase the factor of safety against slope failure. Base reinforcement typically consists of placing a geotextile or geogrid at the base of an embankment prior to constructing the embankment. Base reinforcement is particularly effective where soft/weak soils are present below a planned embankment location. The base reinforcement can be designed for either temporary or permanent applications. Most base reinforcement applications are temporary, in that the reinforcement is needed only until the underlying soil's shear strength has increased sufficiently as a result of consolidation under the weight of the embankment, see **Section 9.3.1**. Therefore, the base reinforcement does not need to meet the same design requirements as permanent base reinforcement regarding creep and durability. The design of base reinforcement is similar to the design of a reinforced slope in that limit equilibrium slope stability methods are used to determine the strength required to obtain the desired safety factor. The detailed design procedures provided by Holtz, et al. (1995) should be used for embankments utilizing base reinforcement.

Base reinforcement materials should be placed in continuous longitudinal strips in the direction of main reinforcement. Joints between pieces of geotextile or geogrid in the strength direction (perpendicular to the slope) should be avoided. All seams in the geotextiles should be sewn and not lapped. Likewise, geogrids should be linked with mechanical fasteners or pins and not simply overlapped. Where base reinforcement is used, the use of Select Stone Backfill or Stone Embankment Material, instead of common or select borrow, may also be needed to increase the embankment shear strength.

9.3.3 Ground Improvement

Refer to **Chapter 11** for references and information on ground improvement design.

9.3.4 Lightweight Fills

Lightweight embankment fill may be used to improve embankment stability. Lightweight fill materials are generally used to reduce driving forces contributing to instability, and reduce potential settlement resulting from consolidation of compressible foundation soils. Situations where lightweight fill may be appropriate include conditions where the construction schedule does not allow the use of staged construction, where existing utilities or adjacent structures are present that cannot tolerate the magnitude of settlement induced by placement of typical fill, and at locations where post-construction settlements may be excessive under conventional fills. Lightweight fill can consist of a variety of materials including polystyrene blocks (geofoam), light weight aggregates (rhyolite, expanded shale, blast furnace slag, fly ash), wood fiber, shredded rubber tires, and other materials. Lightweight fills are infrequently used due to either high costs or other disadvantages with using these materials.

9.3.5 Toe Berms and Shear keys

Toe berms and shear keys are methods to improve the stability of an embankment by increasing the resistance along potential failure surfaces. Toe berms are typically constructed of granular materials that can be placed quickly, do not require much compaction, and have relatively high shear strength. ODOT would typically specify the use of Stone Embankment Material when toe berms and shear keys are required.

9.4 Settlement Mitigation

9.4.1 Acceleration Using Wick Drains

Wick drains, or prefabricated drains, are in essence, vertical drainage paths that can be installed into compressible soils to decrease the overall time required for completion of primary consolidation. Wick drain design considerations, example designs, guideline specifications, and installation considerations are provided by reference in **Chapter 11**. Section 00435 of the *ODOT Standard Specifications* addresses installation of wick drains.

9.4.2 Acceleration Using Surcharges

Surcharge loads are additional loads placed on the fill embankment above and beyond the finish grades. The primary purpose of a surcharge is to speed up the consolidation process. Two significant design and construction considerations for using surcharges include embankment stability and re-use of the additional fill materials. New embankments over soft soils can result in stability problems. Adding additional surcharge fill could exacerbate the stability problem. Furthermore, after the settlement objectives have been met, the surcharge will need to be removed. If the surcharge material cannot be moved to another part of the project site for use as site fill or as another surcharge, it is often not economical to bring the extra surcharge fill to the site only to haul it away again. Also, when fill soils must be handled multiple times (such as with a “rolling” surcharge), it is advantageous to use gravel borrow to reduce workability issues during wet weather conditions.

9.4.3 Lightweight Fills

Lightweight fills can also be used to mitigate settlement issues as indicated in **Section 9.3.4.** Lightweight fills reduce the new loads imposed on the underlying compressible soils, thereby reducing the magnitude of the settlement.

9.4.4 Subexcavation

Subexcavation refers to excavating the soft compressible or unsuitable soils from below the embankment footprint and replacing these materials with higher quality, less compressible material. Because of the high costs associated with excavating and disposing of unsuitable soils as well as the difficulties associated with excavating below the water table, subexcavation and replacement typically only makes economic sense under certain conditions. Some of these conditions include, but are not limited to:

- The area requiring overexcavation is limited;
- The unsuitable soils are near the ground surface and do not extend very deep (typically, even in the most favorable of construction conditions, subexcavation depths greater than about 10 ft are in general not economical);
- Temporary shoring and dewatering are not required to support or facilitate the excavation and;
- Suitable materials are readily available to replace the over-excavated unsuitable soils.

9.5 References

Federal Highway Administration, 1992, ***"EMBANK, Computer Program, User's Manual Publication***," *Publication No. FHWA-SA-92-045*.

Holtz, R. D., Christopher, B. R., and Berg, R. R., 1995, ***Geosynthetic Design and Construction Guidelines***, Federal Highway Administration, FHWA HI-95-038.

Machan, G., Szymoniak, T. and Siel, B., 1989, ***Evaluation of Shale Embankment Construction Criteria,*** *Experimental Feature Final Report OR 83-02*," Oregon State Highway Division, GeotechnicalEngineering Group.

Sabatini, P.J, Bachus, R.C, Mayne, P.W., Schneider, J.A., Zettler, T.E. (2002), ***Geotechnical Engineering Circular No. 5, Evaluation of Soil and Rock Properties***, Report No FHWA-IF-02-034.

Samtani, N. and Nowatzki, E. (2006), *Soils and Foundation Reference Manual, Volumes I and II*, Report No. FHWA NHI-06-088.

Chapter

10

10 Soil Cuts - Analysis and Design

10.1 General

Soil cut slope design must consider many factors such as the materials and conditions present in the slope, materials available or required for construction on a project, space available to make the slopes, minimization of future maintenance and slope erosion. Soil slopes less 10 feet high are generally designed based on past experience with similar soils and on engineering judgment. Cut slopes greater than 6 to 10 feet in height usually require a more detailed geotechnical analysis. Relatively flat (2H:1V or flatter) cuts in granular soil when groundwater is not present above the ditch line, will probably not require rigorous analysis. Any cut slope where failure would result in large rehabilitation costs or threaten public safety should obviously be designed using more rigorous techniques. Situations that will warrant more in-depth analysis include:

- Large cuts,
- Cuts with irregular geometry,
- Cuts with varying stratigraphy (especially if weak zones are present),
- Cuts where high groundwater or seepage forces are likely,
- Cuts involving soils with questionable strength, or
- Cuts in old landslides or in formations known to be susceptible to landsliding.

A major cause of cut slope failure is related to reduced confining stress within the soil upon excavation. Undermining the toe of the slope, increasing the slope angle, and cutting into heavily overconsolidated clays have also resulted in slope failures. Careful consideration should be given to preventing these situations by surcharging or buttressing the base of the slope, choosing an appropriate slope angle (i.e., not oversteepening), and by keeping drainage ditches a reasonable distance away from the toe of slope. Cutslopes in heavily overconsolidated clays may require special mitigation measures, such as retaining walls rather than an open cut in order to prevent slope deformation and reduction of soil strength to a residual value. Consideration should also be given to establishing vegetation on the slope to prevent long-term erosion. It may be difficult to establish vegetation on slopes with inclinations steeper than 2H:1V without the use of erosion mats or other stabilization methods.

10.1.1 Design Parameters

The major cutslope design parameters are slope geometry, soil shear strength and predicted or measured groundwater levels. For cohesionless soil, stability of a cut slope is independent of height and therefore slope angle becomes the key parameter of concern. For cohesive (φ= 0) soils, the height of the cut becomes the critical design parameter. For c'-φ' and saturated soils, slope stability is dependent on both slope angle and height of cut. Also critical to the proper design of cut slopes is the incorporation of adequate surface and subsurface drainage facilities to reduce the potential for future stability or erosional problems.

Establishment of design parameters is done by a thorough site reconnaissance, sufficient exploration and sampling, and a laboratory testing program designed to identify the material soil strength properties to be used in analysis. Backanalysis methods may also be used to determine the appropriate shear strength for design. The geotechnical designer should be familiar with the state of the practice in determining the design parameters for analysis. References are presented in **Section 10.3.**

10.2 Soil Cut Design

10.2.1 Design Approach and Methodology

Safe design of cut slopes is typically based on past experience or on more in-depth analysis. Both approaches require accurate sire specific information regarding geologic conditions obtained from standard field and laboratory classification procedures. Design guidance for simple projects is provided in the *ODOT Highway Design Manual*, located on the ODOT website, and can be used unless indicated otherwise by the geotechnical designer. Slopes less than 6 to 10 feet high, with slopes flatter than 2:1, may be used without in-depth analysis if no special concerns are noted by the geotechnical designer. If the geotechnical designer determines that a slope stability study is necessary, information that will be needed for analysis includes:

- An accurate cross section showing topography,
- Proposed grade,
- Soil unit profiles,
- Unit weight and strength parameters (c',φ'), (c,φ), or Su (depending on soil type and drainage and loading conditions) for each soil unit, and
- Location of the water table and flow characteristics.

The design factor of safety for static slope stability is 1.25. This safety factor should be increased to a minimum of 1.30 for slopes where failure would cause significant impact to adjacent structures. For pseudo-static seismic analysis the factor of safety can be decreased to 1.1. Cut slopes are generally not designed for seismic conditions unless slope failure could impact adjacent structures. These factors of safety should be considered as minimum values. The geotechnical designer should decide on a case by case basis whether or not higher factors of safety should be used based the consequences of failure, past experience with similar soils, and uncertainties in analysis related to site and laboratory investigation.

Preliminary slope stability analysis can be performed using simple stability charts. See Abramson, et al. (1994) for example charts. These charts can be used to determine if a proposed cut slope might be subject to slope failure. If slope instability appears possible, or if complex conditions exist beyond the scope of the charts, more rigorous computer methods such as XSTABL, PCSTABL, and SLOPE/W can be employed see **Chapter 7**. Effective use of these programs requires accurate determination of site geometry including surface profiles, soil unit boundaries, and location of the water table, as well as unit weight and strength parameters for each soil type.

10.2.2 Seepage Analysis and Impact on Design

The introduction of groundwater to a slope is a common cause of slope failures. The addition of groundwater often results in a reduction in the shear strength of soils. A higher groundwater table results in higher pore pressures, causing a corresponding reduction in effective stress and soil shear strength. A cutslope below the groundwater table results in destabilizing seepage forces, adds weight to the soil mass, increasing driving forces for slope failures. It is important to identify and accurately model seepage within proposed cut slopes so that adequate slope and drainage designs are employed. suring the phreatic (water table) surface with open standpipes or observation wells. Piezometric data from piezometers can be used to estimate the phreatic surface or piezometric surface if confined flow conditions exist. A manually prepared flow net or a numerical method such as finite element analysis can be used provided sufficient boundary information is available. The pore pressure ratio (ru) can also be used. However, this method is generally limited to use with stability charts or for determining the factor of safety for a single failure surface.

10.2.3 Surface and Subsurface Drainage Considerations and Design

The importance of adequate drainage cannot be overstated when designing cut slopes. Surface drainage can be accomplished through the use of drainage ditches and berms located above the top of the cut, around the sides of the cut, and at the base of the cut. Surface drainage facilities should direct surface water to suitable collection facilities.

Subsurface drainage should be employed to reduce driving forces and increase soil shear strength by lowering the water table, thereby increasing the factor of safety against a slope failure. Subsurface conditions along cut slopes are often heterogeneous. Thus, it is important to accurately determine the geologic and hydrologic conditions at a site in order to place drainage systems where they will be the most effective. Subsurface drainage techniques available include:

- Cut-off trenches (French drains)
- Horizontal drains
- Relief wells

Cut-off trenches: Cut-off trenches, also known as French drains, are a gravel filled trench near the top of the cut slope to intercept groundwater and convey it around the slope. They are effective for shallow groundwater depths from 2 to 15 feet deep.

Horizontal drains: If the groundwater table needs to be lowered to a greater depth, horizontal drains can be installed, if the soils are noncohesive and granular in nature. Horizontal drains are generally not very effective in finer grained soils. Horizontal drains consist of small diameter holes drilled at slight angles into a slope face and backfilled with perforated pipe wrapped in drainage geotextile. Installation might be difficult in soils containing boulders, cobbles or cavities. Horizontal drains require periodic maintenance as they tend to become clogged over time.

Relief wells: Relief wells can be used in situations where the water table is at a great depth. They consist of vertical holes cased with perforated pipe connected to a disposal system such as submersible pumps or discharge channels similar to horizontal drains. They are generally not common in the construction of cut slopes.

Whatever subsurface drainage system is used, monitoring should be implemented to determine its effectiveness. Typically, piezometers or observation wells are installed during exploration. These should be left in place and periodic site readings should be taken to determine groundwater levels or pore pressures depending on the type of installation. High readings would indicate potential problems that should be mitigated before a failure occurs.

Surface drainage, such as brow ditches at the top of the slope, and controlling seepage areas as the cut progresses and conveying that seepage to the ditch at the toe of the cut, should be applied to all cut slopes. Subsurface drainage is more expensive and should be used when stability analysis indicates pore pressures need to be lowered in order to provide a safe slope. The inclusion of subsurface drainage for stability improvement should be considered in conjunction with other techniques outlined below to develop the most cost effective design meeting the required factor of safety.

10.2.4 Stability Improvement Techniques

There are a number of options that can be used in order to increase the stability of a cut slope. Techniques include:

- Flattening slopes
- Benching slopes
- Lowering the water table (discussed previously)
- Structural systems such as retaining walls or reinforced slopes.

Changing the geometry of a cut slope is often the first technique considered when looking at improving stability. For flattening a slope, enough right-of-way must be available. As mentioned previously, stability in purely dry cohesionless soils depends on the slope angle, while the height of the cut is often the most critical parameter for cohesive soils. Thus, flattening slopes usually proves more effective for granular soils with a large frictional component.

Structural systems are generally more expensive than the other techniques, but might be the only option when space is limited. Shallow failures and sloughing can be mitigated by placing a 2 to 3-foot thick rock drainage blanket over the slope in seepage areas. Moderate to high survivability permanent erosion control geotextile should be placed between native soil and drain rock to keep fines from washing out and/or clogging the drain rock. In addition, soil bioengineering can be used to stabilize cut slopes against shallow failures (generally less than 3 feet deep), surface sloughing and erosion along cut faces.

10.2.5 Erosion and Piping Considerations

Surface erosion and subsurface piping are most common in clean sands, nonplastic silts and dispersive clays. Loess and volcanic ash are particularly susceptible. However, all cut slopes should be designed with adequate drainage and temporary and permanent erosion control facilities to limit erosion and piping as much as possible. The amount of erosion that occurs along a slope is a factor of soil type, rainfall intensity, slope angle, length of slope, and vegetative cover. The first two factors cannot be controlled by the designer, but the last three factors can. Longer slopes can be terraced at approximate 15- to 30-foot intervals with drainage ditches installed to collect water. Best Management Practices (BMPs) for temporary and permanent erosion and stormwater control as outlined in the *ODOT Highway Design Manual* should always be used. Construction practices should be specified that limit the extent and duration of exposed soil. For cut slopes, consideration should be given to limiting earthwork during the wet season and requiring that slopes be covered as they are exposed, particularly for the highly erodable soils mentioned above.

10.2.6 Sliver Cuts

A sliver cut is defined as slope excavation less than 10 feet wide over some or all of its height. Sliver cuts in soils should be avoided because they are difficult to build. Cuts at least 10 feet wide over the full height of the cut require the use of conventional earth moving machinery to maximize production. Cuts less than 10 feet wide and up 25 feet high measured along the slope can be excavated with a large backhoe but at the expense of production. If a sliver cut is used, consider how it will be built and be sure to account for the difficulty in the cost estimate.

10.3 References

Abramson, L., Boyce, G., Lee, T., and Sharma, S., 1994, *Advanced Technology for Soil Slope Stability*,. Vol. 1, FHWA-SA-94-005.

FHWA, 1993, *Soils and Foundations Workshop Manual*, Second Edition, FHWA-HI-88-009.

***ODOT Highway Design Manual*,** 2003.

Turner, A., and Schuster, R., 1996, *Landslides: Investigation and Mitigation*, TRB Special Report 247.

Chapter

11

11 Ground Improvement

11.1 General

Ground improvement is used to address a wide range of geotechnical engineering problems, including, but not limited to, the following:

- Improvement of soft or loose soil to reduce settlement, increase bearing resistance, and/or to improve overall stability of bridge foundations, retaining walls, and/or for embankments.
- To mitigate liquefiable soils.
- To improve slope stability for landslide mitigation.
- To retain otherwise unstable soils.
- To improve workability and usability of fill materials.
- To accelerate settlement and soil shear strength gain.

Types of ground improvement techniques include the following:

- Vibrocompaction techniques such as stone columns and vibroflotation, and other techniques that use vibratory probes that may or may not include compaction of gravel in the hole created to help densify the soil.
- Deep dynamic compaction.
- Blast densification.
- Geosynthetic reinforcement of embankments.
- Wick drains, sand columns, and similar methods that improve the drainage characteristics of the subsoil and thereby help to remove excess pore water pressure that can develop under load applied to the soil.
- Grout injection techniques and replacement of soil with grout, such as compaction grouting, jet grouting, and deep soil mixing.
- Lime or cement treatment of soils to improve their shear strength and workability characteristics.
- Permeation grouting and ground freezing (temporary applications only).

Each of these methods has limitations regarding their applicability and the degree of improvement that is possible.

11.2 Development of Design Parameters and Other Input Data for Ground Improvement Analysis

In general, the geotechnical investigation conducted to design the cut, fill, structure foundation, retaining wall, etc., that the improved ground is intended to support will be adequate for the design of the soil improvement technique proposed. However, specific soil information may need to be emphasized depending on the ground improvement technique selected. For example, for vibro-compaction techniques, deep dynamic compaction, and blast densification, detailed soil gradation information is critical to the design of such methods, as minor changes in soil gradation characteristics could affect method feasibility. Furthermore, the in-situ soil testing method used (e.g., SPT testing, cone testing, etc.) will need to correspond to the technique specified in the contract to verify performance of the ground improvement technique, as the test data obtained during design will be the baseline to which the improved ground will be compared. Other feasibility issues will need to be addressed if these types of techniques are used. Ground vibrations caused by the improvement technique may have critical impacts on adjacent structures. Investigation of the foundation and soil conditions beneath adjacent structures and utilities may be needed, (in addition to standard precondition surveys of the structures) to enable evaluation of the risk of damage caused by the ground improvement technique.

- **Wick Drains:** For wick drains, the ability to penetrate the soil with the wick drain mandrel, in addition to obtaining rate-of-settlement information, must be assessed. Atterberg limit and water content data should be obtained, as well as any other data that can be useful in assessing the degree of overconsolidation of the soil present, if any.
- **Grout Injection Techniques:** Grout injection techniques (not including permeation grouting) can be used in a fairly wide range of soils, provided the equipment used to install the grout can penetrate the soil. The key is to assess the ability of the equipment to penetrate the soil, assign soil density and identify potential obstructions such as boulders.
- **Permeation Grouting:** Permeation grouting is more limited in its application, and its feasibility is strongly dependent on the ability of the grout to penetrate the soil matrix under pressure. To evaluate the feasibility of these two techniques, detailed grain size characterization and permeability assessment must be conducted, as well as the effect groundwater may have on these techniques. An environmental assessment of such techniques may also be needed, especially if there is potential to contaminate groundwater supplies.
- **Ground Freezing:** Similarly, ground freezing is a highly specialized technique that is strongly depending on the soil characteristics and groundwater flow rates present.

11.3 Design Requirements

The following design manuals and references shall be used for specific ground improvement applications:

- **General Ground Improvement Design Requirements:**

  FHWA manual No. FHWA-SA-98-086, "*Ground Improvement Technical Summaries*", (Elias, et al., 2000)

- **Stone Column Design:**

  FHWA Report No. FHWA/RD-83/O2C, "*Design and Construction of Stone Columns*", (Barkdale and Bachus, 1983)

- **Deep Dynamic Compaction:**

  FHWA manual No. FHWA-SA-95-037, *Geotechnical Engineering Circular No. 1, "Dynamic Compaction*", (Lukas, 1995)

- **Wick Drain Design:**

  FHWA manual FHWA/RD-86/168, "*Prefabricated Vertical Drains*", (Rixner, et al., 1986)

- **Blast Densification:**

  Blast Densification for Mitigation of Dynamic Settlement and Liquefaction, Kimmerling, R. E., 1994, *WSDOT Research Report WA-RD 348.1*

  Soil Improvement: State-of-the-Art Report, Mitchell, J. K., 1981, Proceedings of the 10th International Conference on Soil Mechanics and Foundation Engineering, Stockholm, Sweden, pp. 509-565.

- **Lime and Cement Soil Treatment:**

  Alaska DOT/FHWA Report No. FHWA-AK-RD-01-6B, "*Alaska Soil Stabilization Design Guide*", (Hicks, 2002)

11.4 References

Barkdale, R. D., and Bachus, R. C., 1983, *Design and Construction of Stone Columns – Vol. 1*, Federal Highway Administration, FHWA/RD-83/02C.

Elias, V., Welsh, J., Warren, J., and Lukas, R., 2000, *Ground Improvement Technical Summaries – Vol. 1 and 2*, Demonstration Project 116, Federal Highway Administration, FHWA-SA-98-086.

Hicks, R. G., 2002, *Alaska Soil Stabilization Design Guide*, Alaska Department of Transportation and Federal Highway Administration Report No. FHWA-AK-RD-01-6B.

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Chapter

12

12 Rock Cuts – Analysis, Design and Mitigation

12.1 General

This chapter discusses the analysis, design guidelines and standards for rock slopes adjacent to highways. Rock slope design for material sources is discussed in **Chapter 20.**

12.2 ODOT Rock Slope Design Policy

The purpose of the policy is to establish slope design standards for rock cuts and to encourage the active involvement of geologists and geotechnical engineers in the rock slope design process. This involvement is intended to ensure that rock slopes are safe to construct and economical and will optimize safety for the public. In general, the policy includes four sections that deal with rock slopes. These sections cover the rockslope design, rock fallout area requirements, the use of benches, and rock slope stabilization and mitigation techniques.

12.2.1 Rock Slope Design

The purpose of the rock slope design is to develop rock cuts that will be safe to construct and will provide long term safety for the public. The inclination of rock slopes should be based on the structural geology and stability of the rock units, as described in the Geology or Geotechnical Report. Rock unit slopes of vertical, 0.25:1, 0.5:1, 0.75:1 and 1:1 are commonly considered. The design rock cut slope should be the steepest continuous slope (without benches) that satisfies physical and stability considerations. Controlled blasting (using presplit and trim blasting techniques) is normally required for rock cut slopes from vertical to 0.75:1. The purpose of controlled blasting is to minimize blast damage to the rock backslope to help insure long-term-stability, improve safety, and lessen maintenance. See **Section 12.5** for more details regarding rock slope design.

12.2.2 Rockslope Fallout Areas

Fallout areas should be used where hazardous rockfall could occur. The fallout area is a non-traveled area between the highway and the cutslope with minimum width, depth and slope requirements. The minimum dimensions should be determined based on rock cut slope inclination and height. The depth of the fallout area varies with the slope configuration. A preliminary determination of the fallout area or catch ditch dimensions can be obtained from the Ritchie Rockfall Catch Ditch Design Chart located in the *ODOT Highway Design Manual, section 10.4*.

Final catch ditch dimensions should be determined using the *Rockfall Catchment Area Design Guide* (FHWA Final Report SPR-03(032).

As noted in the 2003 *ODOT Highway Design Manual, section 10.4.4*, a goal of 90% retention of rock in the catchment area has been adopted for all new and reconstructed rock slopes. This goal may not be achievable in all cases due to cost, environmental reasons, or other factors. The catchment area depth may be achieved in a number of ways, including excavation and/or placing suitable retaining structures at the highway shoulder. Where the slopes are inclined at flatter than 0.75:1, and where the anticipated size of a single rock is less than 2 feet in diameter, chain link catch fences may be considered as a substitute for depth of fallout. Slopes less than 40 feet high and flatter than 1:1 generally have a ditch and recoverable slope equal to or greater than a fallout ditch shown in the Rockfall Catchment Area Design Guide. In that case, the standard roadway ditch will serve as adequate rockfall catchment.

Temporary detours may require the construction of rockslopes and fallout areas. If the site has previously been an area of rockfall activity, and the detour will reduce the fallout area, thereby putting motorists in increased risk, the rockslope and fallout area must be designed to, at a minimum, not increase the risk to the public. Fallout areas should then be designed to capture or retain at least as much rockfall as was previously available prior to construction. Additional mitigation measures, along with one way travel, reduced travel speed in the rockfall zone, and increased sight distances may be required to reduce risk to the public. The designer should be prepared to address all of these issues in the design process.

12.2.3 Benches

For most rock slope designs, benches should be avoided. The need for benches will be evaluated in the geology and geotechnical investigations and described in the resulting reports. The minimum bench design should satisfy the requirements outlined in the Rockfall Catchment Area Design Guide. The bench configuration may be controlled by the need to perform periodic maintenance which requires access to the bench. Soil and rock slopes may need a modification with benches to conform to the environment or for safety and economic concerns. Following are some appropriate bench applications.

- Benching may improve slope stability where continuous slopes are not stable.
- Where maintenance due to sloughing of soil overburden may be anticipated, a bench will provide access and working room at the overburden rock contact.
- Developing an access bench may facilitate construction where the top of cut begins at an intermediate slope location.
- On very high cuts, benches may be included for safety where rockfall is expected during construction.
- Where necessary, benches may be located to intercept and direct surface water runoff and groundwater seepage to an appropriate collection facility.
- All benches should be constructed to allow for maintenance access.

12.2.4 Rock Slope Stabilization and Rockfall Mitigation Techniques

Rock slope stabilization techniques may be required to accommodate special geologic features. Stabilization techniques include rock bolts and dowels, wire mesh and cable net slope protection, reinforced shotcrete, trim and production blasting. Specific stabilization techniques with appropriate design will be recommended in the Geotechnical Report as necessary. Refer to **Section 12.4** for more detail.

12.3 Rockslope Stability Analysis

Slope stability analysis for rock slopes involves a thorough understanding of the structural geology and rock mechanics. For most rock cuts on highway slopes, the stresses in the rock are much less than the rock strength so there is little concern with the fracturing of the intact rock. Therefore, stability is concerned with the stability of rock blocks formed by the discontinuities. Field data collection of the dip, dip direction, nature and type of joint infilling, joint roughness and spacing are important for the stability analysis of planar, wedge and toppling failure modes. Slope height, angle, presence of potential rock launching features, block size, and block shape are important for the analysis and design of rockfall mitigation techniques. Hand-calculation methods can be used to analyze potential planar and wedge failures and computer programs such as ROCKPACK are also available. Rockfall simulation programs, such as CRSP (Colorado Rockfall Simulation Program), are used to analyze for rockfall catchment size and the prediction of rock kinetic energy. Only geotechnical practitioners experienced in using these programs should perform the analysis. Refer to Wyllie and Mah, 1998, for details on design, excavation, and stabilization of rock slopes.

12.4 Design Guidelines

General design guidelines are found in the references listed in **Section 12.7**. Design of rock slopes adjacent to ODOT highways must also include consideration of additional factors such as environmental issues, history of rockfall hazards, cost, risk/benefit, and needs of the project. The following guidelines provide information on ODOT rock slope design.

12.4.1 Geologic Investigation and Mapping

For projects that include rock cuts, the geotechnical designer should contact the local Maintenance district office to discuss the history of past rockfall events and consult the Region Geologist for the project area to determine the RHRS (Rockfall Hazard Rating System) score and priority for that highway and for the Region. The designer should also discuss the geologic hazard potential with the Region Geologist so that a consensus on the degree of rockfall potential is reached. The discussions will serve to highlight concerns regarding construction, local environmental needs, and feasible options for mitigation of the hazard. The development and implementation of the geologic investigation can then be completed.

Field data collection is generally done on a project site specific basis. Wiley and Mah, 1998, discussed joint mapping techniques, stereographic projection, and types of subsurface exploration that may be performed on rockslopes. Full scale tests of rockfall at the site may also be performed, however, the cost and practicality of traffic control generally prevents this type of work.

12.4.2 Analysis and Design

As previously stated, analysis of planar, wedge and toppling failure modes can be performed by hand or with some available computer programs. Wiley and Mah, 1998, discusses the analysis in detail.

Simulation of rockfall using the CRSP computer program may be needed to determine the minimum required dimensions of a rockfall catch ditch and the kinetic energy of rocks that may need to be restrained by barriers, wire mesh, screens or walls. As a rule of thumb, draped gabion wire mesh slope protection and screens are capable of withstanding impacts from rocks up to 2 feet in diameter. For larger rocks, proprietary rockfall net systems or retaining walls will likely be needed. Experience with the Rockfall Catchment Areas Design Guide study indicates that rockfall catch areas wider that 30 to 35 feet are not typically cost effective to construct, and additional barriers, fences or walls to gain ditch depth become more cost effective than wider ditches.

12.4.3 Construction Issues

Construction of rock slopes near highways frequently must consider traffic control during blasting and scaling operations. The traffic control may include adjacent railroad facilities where trains are running next to the highway or other adjacent structures and facilities. The cost of traffic control for a busy highway can potentially result in a doubling of the project cost. Therefore, careful consideration of staging, detours, work zones and blast-produced flyrock control must be done during design. It may even be necessary to choose another mitigation option than the preferred one because of these issues.

Environmental concerns in scenic highway corridors have made construction of rock slopes more difficult. Presplit hole half-casts that are visible after blasting may be regarded as a visual concern and a bid item may be needed to partially or completely removed them. This issue has been most notable in the Columbia River Gorge Scenic Corridor, and in a few USFS forest highways. Rock coloration has also been a concern and a bid item for Permeon, a rock coloration product, has been included on several projects contracts.

12.4.4 Blasting Consultant

A Blasting Consultant may need to be retained to assist a contractor in designing a safe blast if there are nearby structures, if the site is particularly challenging, or otherwise has the potential to result in undesired consequences. Guidelines for determining when a Blasting Consultant is needed are located on the ODOT website. ODOT keeps a list of preapproved blasting consultants and has a method of approving new blasting consultants and the HQ Geotechnical Group should be contacted.

12.4.5 Gabion Wire Mesh Slope Protection/ Cable Net Slope Protection

For gabion wire mesh slope protection, the designer must choose either galvanized or PVC coated wire in order to place the correct Standard Detail in the construction plans. Anchor spacing for Wire Mesh, Cable Net, and Post-Supported Wire Mesh Slope Protection are based on the weight of the mesh alone. Narrower spacing may be required where snow and ice loads will add a significant amount of stress to the anchors.

The WashDOT research report, *Design Guidelines for Wire Mesh/Cable Net Slope Protection, WA-RD 612.1*, should be used to determine anchor spacing in snow/ice load situations. If mesh is use in a coastal environment, stainless steel fasteners and hardware or heavy galvanizing should be used to inhibit corrosion.

12.4.6 Rock Reinforcing Bolts and Rock Reinforcing Dowels

The designer must identify the installation area, size and strength of steel, pattern or spacing, inclination, minimum length, and design loads of the bolts or dowels and this information must be included in the Geotechnical Report. Since rock reinforcing bolts are considered to be permanent, acceptable materials for bonding the bolt into rock are non-shrink cement grouts, while polyester resin or cement grout is acceptable for the semi-permanent rock reinforcing dowels. Mechanical anchorage bolts and non-shrink cement grout are included in the *ODOT Qualified Products List (QPL)*. Split set and bail set type anchorage systems are considered temporary or low stress installations and are not acceptable for use on ODOT projects.

12.4.7 Proprietary Rockfall Net Systems

High capacity rockfall net systems are available from two accepted manufacturers, GeoBrugg and ROTEC International. Full scale tests on these systems have been performed by the manufacturers. The systems are generally capable of withstanding impact kinetic energies up to 735 ft-tons and can be constructed with breakaway post base connections and post heights up to 20 to 25 feet. These systems are expensive and can raise objections about their highly visible nature. However, they can be a viable alternative to high barriers and MSE walls in rockfall situations.

12.5 Standard Details

The **ODOT GeoEnvironmental webpage** includes a link to Standard Details normally used in the mitigation of rockfall hazards. These details are also found in the *Roadway Contract Plans Development Guide*. The following details, in English and Metric units, are presented:

- Det 2200 - Cable Net Slope Protection Detail
- Det 2201 - Wire Mesh/Cable Net Anchors Detail
- Det 2202 - Shotcrete Slope Detail
- Det 2203 - Wire Mesh Slope Protection Detail
- Det 2204 - Barrier Mounted Rock Protection Screen Detail
- Det 2205 - Post Supported Wire Mesh Slope Protection Detail
- Det 2206 - Post Supported Wire Mesh Slope Protection Detail
- Det 2207 - Post Supported Wire Mesh Slope Protection and Post Supported Rock Protection Screen Anchor Details
- Det 2208 - Rock Protection Screen Behind Concrete Barrier or Guardrail Detail
- Det 2209 - Rock Protection Screen Behind Concrete Barrier or Guardrail Detail

12.6 Specifications

The location of Standard Specifications and Special Provisions for items pertaining to rockslopes and rockslope mitigation are listed in the next sections.

12.6.1 Blasting

Specifications for general excavation of rock slopes flatter than 0.75:1, where presplit (controlled blasting) of the backslope is not required, are located in *Section 00330.41(e) - Blasting of the Standard Specifications*.

Specifications for rock excavation where slopes are 0.75:1 or steeper are located in *Section 00335 - Blasting Methods and Protection of Excavation Backslopes of the Standard Specifications*. A per foot bid item quantity for Controlled Blast Holes is required if this specification is used.

Special Provisions for retaining a Blasting Consultant (see *Section 00335.44 Blasting Consultant*), Vibration Control (see *Section 00335.45 Vibration Control*), and Blasting Noise Control (see *Section*

00335.46 Airblast and Noise Control) are located in the Special Provisions section of the ODOT Specifications Webpage.

12.6.2 Rockslope Mitigation Methods

The following rockslope mitigation methods are located in a new section of the *Standard Specifications, Section 00398 – Rockslope Stabilization and Reinforcement*.

- Wire Mesh Slope Protection
- Post Supported Wire Mesh Slope Protection
- Rock Protection Screen Behind Barrier or Guardrail
- Barrier Mounted Rock Protection Screen
- Rock Reinforcing Bolts/Rock Reinforcing Dowels
- Proprietary Rockfall Net System

12.7 References

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Chapter

13

13 Slope Stability Analysis

13.1 General

(This Chapter to be completed at a later date.)

Chapter

14

14 Geosynthetic Design

14.1 General

(This Chapter to be completed at a later date.)