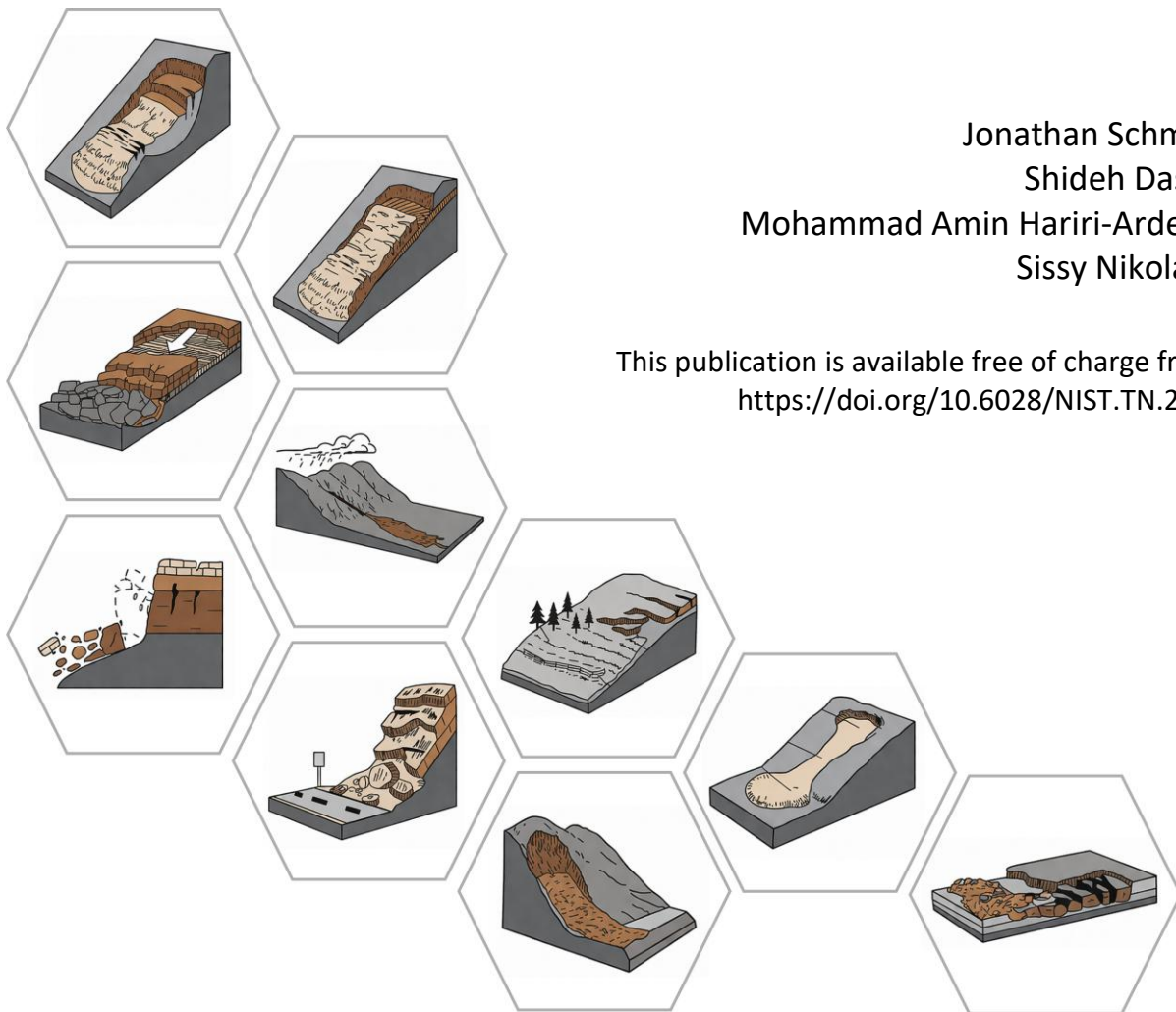




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NIST TN 2371

Implications of Future Hazards and Conditions on Geotechnical Engineering Practice:

A Review of the U.S. Codes, Standards, and Guidelines



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Abstract

Geotechnical engineers are routinely faced with challenging problems relating to soil, water, and the built environment - these problems will become more challenging and complex due to future hazards and conditions. There is a need to understand the provisions of existing design standards and available technical guideline in geotechnical engineering to incorporate and mitigate the impacts of future extreme weather, sea level change, and groundwater depletion. Without clear and consistent guidelines across the nation, a situation is unfolding that will likely result in a patchwork of recommendations and requirements that may vary by state and level of engineering rigor. This report assesses the current state of geotechnical engineering practice to address these needs. First, a summary of relevant national standards and guidelines, grouped based on different types of geotechnical work, is presented. Next, areas where state/local documents substantially modify or expand on national-level guidelines are identified for several selected states. The report finds that existing standards have three levels of coverage for the potential effects of future hazards. Explicit guidelines is rarely present. However, many existing design requirements may implicitly encompass future hazards and conditions. Even so, when consideration of the potential effects of hazards is required in a given code or standard, the accompanying quantitative analysis procedures are often lacking. The report concludes with some avenues for future research to aid in developing forward-thinking design procedures that integrate recent advances in climate science and soil mechanics.

Keywords

Future hazard, geotechnical engineering, soil mechanics, climate, design.

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Executive Summary

This report presents a review of the state of practice in geotechnical engineering regarding incorporating future hazards and conditions in routine design procedures. This review focuses on aspects of the design procedures that may be affected by future hazards rather than giving a complete technical summary (which can be found in the corresponding references).

Additionally, this review is intended as a factual summary of the engineering procedures involved in the state of practice rather than as an opinion on which procedures are the most appropriate for the problem at hand or incorporate the most current research. Furthermore, the review is not intended to describe how practitioners interpret or apply the methods of these documents, which will be addressed in future research efforts.

Section 1 begins by giving an overview of the typical roles and responsibilities for a geotechnical engineer on various types of infrastructure projects, which may be helpful for readers who have less experience with geotechnical work and how it differs from other components of civil engineering (e.g., structural, water resources, or transportation engineering). It also describes the authors' interpretation of what defines the state of practice. The state of practice is characterized by three aspects. The first of these are national-level guidelines, typically in the form of model building/transportation codes, design standards, and engineering manuals. These national-level codes and standards are adopted, amended, and expanded upon by various state and local (i.e., city and county) entities. However, there are several challenges in defining the state of practice. The most relevant of these is the sheer amount of potential material to review – there are multiple national-level standards (which may or may not apply to a given project depending on the work to be performed), and each will be adopted/amended at the state and local level. The section concludes with a detailed overview of the potential interactions between future hazards and aspects of geotechnical engineering design. These can be summarized as changes in loads/hazards, changes in soil thermo-hydro-mechanical properties, and changes in the performance and vulnerability of geotechnical structures.

Section 2 details the review methodology and approach to addressing the challenges described in Section 1. First, three primary categories of geotechnical engineering work are identified: buildings (intended for human occupancy), transportation structures, and hydraulic geostructures (e.g., embankment dams and levees). For these categories, a detailed review is provided of national-level documents that specify the “minimal” level of design effort required for a given project. Next, several states (Alaska, California, Florida, Illinois, and Texas) are selected for their climatic and administrative diversity, and their state/local building codes, Department of Transportation geotechnical manuals, hazard mitigation guidelines, and other relevant documents. For these, the focus is to identify notable modifications to or deviations from the national-level guidelines.

Section 3 comprises most of the report. It describes the guidelines provided by the national and state/local-level documents previously identified for the following selected geotechnical hazards:

- Expansive/collapsible soils

- Temperature effects (e.g., permafrost and desiccation cracking)
- Corrosion
- Seismic soil liquefaction
- Scour and erosion
- Tsunami
- Slope stability
- Groundwater/seepage

For each geotechnical hazard, an overview is provided on how it may be affected by future hazards and long-term weather patterns. Next, an overview is provided about the design guidelines, separated into national and state/local components. Each subsection is concluded by identifying potential gaps in the guidelines relative to the state of practice. Finally, some ongoing research is highlighted in the identified areas.

Section 4 presents conclusions and an evaluation of the state of practice. First, it discusses how the guidelines provided by national and state/local documents result in two levels of coverage for the future effects of hazards and conditions:

- Explicitly prescriptive guidelines are rarely present in the national and local codes. This work defines explicitly prescriptive guidelines as (reasonably) unambiguous requirements in geotechnical standards or statements that specifically identify expected consequences of future hazards as a required aspect of a detailed and quantitative geotechnical design procedure. These requirements may be based on a factor of safety against a dominant failure mode, or they may be performance-based.
- Implicit guidelines are far more common, which indirectly consider the impacts of future hazards and conditions on the safety and performance of geotechnical infrastructure. This work reviews implicit guidelines for geotechnical factors, such as groundwater levels that will be affected to some degree by future hazards or when the geotechnical design relies on hydrologic or structural inputs (such as maximum water levels in a reservoir) that could incorporate the effects of future conditions.

However, how practitioners or reviewing agencies interpret implicit guidelines is yet to be determined, and further targeted research is required. The document-to-document variability in the level of detail contained in hazard-specific design requirements is also highlighted. The section is concluded by identifying future research needs. These are grouped into improving inputs to geotechnical models, better understanding of soil thermo-hydro-mechanical behavior, and the need for performance-based predictive models.

As a general note, official code documents use language very intentionally -- “should”, “shall”, “recommend”, and “may” all have precise meanings in those codes. For readability, this report may not always directly replicate the language in documents, as it is intended to be a general summary of the design guideline provided, not an official interpretation or companion/commentary to the standards and codes. The original document language must be

followed in design. Furthermore, terms such as “reasonable worst case”, “appropriate”, or “sufficient” may appear in this report without corresponding definitions. This usage is deliberate and is intended to reflect a similar lack of clear definitions in the original documents.

1. Introduction

The performance and failure potential of geotechnical infrastructure (e.g., foundations, engineered fills, slopes, embankments, retaining walls) are strongly influenced by factors such as precipitation, temperature, humidity, and wind/air flow. Soil-atmosphere interactions (resulting from variations in soil moisture and temperature) can change the soil's infiltration rate, pore water pressures, suction, swelling and shrinkage, effective stress and strength, potential for liquefaction, and frost heaving. These factors can alter performance and cause slope failure, flooding, soil erosion, desiccation cracks, excessive movements, and more. For example, foundation failure could result from freeze-thaw cycles, changes in the groundwater depth, accelerated scouring, corrosion, creep, or permafrost melting. Slope stability and damage to a range of infrastructure types could result from heavy rainfall, rise in water level, wet-dry cycles, drought-induced cracks and stresses ([1], [2]). Therefore, if these factors are not considered in the design, they can lead to a catastrophic failure of the geotechnical infrastructure, with severe impacts on the lives and livelihoods of communities. Examples include the failure of the New Orleans levee system due to flooding in 2005 [3]; the failure of the Oroville Dam spillway in Northern California due to heavy rainfall [4]; and the combined destruction from the 2021 Nippes, Haiti earthquake, and subsequent tropical storms [5].

Long-term patterns in extreme weather events are expected to amplify the frequency and severity of extreme weather events, including sea level change, rainfall storms, high-intense-low-duration and low-intense-high-duration floods, droughts, heatwaves, and abrupt changes in freezing/thawing cycles due to a range of factors [6]. This will further increase the impact of soil-atmosphere interactions on the response and failure potential of geotechnical infrastructure (e.g., the projected failure of California's levee system under prolonged drought [2]). In addition, the increasing intensity and frequency of these extreme events have increased the likelihood of multi-hazards with compound and cascading impacts on physical infrastructure (e.g., flooding after a fire, heatwaves followed by droughts and heavy precipitation, or extreme weather events coupled with earthquakes in seismically active areas). Most of these interactions pose multi-physics processes involving the thermo-hydro-mechanical properties of soils that are variable in degree of saturation. This report investigates the extent to which soil-atmosphere interactions in realistically variable soils and long-term weather patterns are included in the existing codes and regulations that address geotechnical infrastructure across the U.S. (at the national level and a few representative states) and areas where additional research and guidelines are needed to improve the geotechnical standard of practice.

Many federal agencies, states, and municipalities across the nation have adopted some form of resilience or similar initiatives (e.g. [7], [8], [9], [10], [11], [12], [13], [14], [15], [16]). These often include some means of providing additional financial support to infrastructure projects that aim to improve community resilience to various aspects of future hazards, such as flooding or sea level changes. However, to produce actionable geotechnical designs that fulfill these resilience requirements, national-level guidelines and standards are needed.

1.1. Terminology

Some of the language used in this report may differ from other contexts (depending on the source material). The sections below provide this report’s working definitions for a few key terms, which are adopted or slightly modified from [17], [18], [19], and [20] or as otherwise noted.

1.1.1. Risk

The probability of occurrence for a given hazard event (during a specified time interval) and the associated consequences of failure. Consequences are defined as adverse effects (sometimes in monetary terms) on health, property, communities, and the environment.

1.1.2. Resilience

[21] defines resilience as “the ability to prepare for and adapt to changing conditions and to withstand and recover rapidly from disruptions. Resilience includes the ability to withstand and recover from deliberate attacks, accidents, or naturally occurring threats or incidents. The overarching purpose of community resilience is to maintain the long-term viability of a community following an earthquake or other natural hazard event.” [22] and [23] provide similar definitions.

1.1.3. Vulnerability

Quantification of the susceptibility to damage (i.e., fragility) or disruption should a hazard of a given intensity occur.

1.1.4. Hazard

An event or condition that is a source of potential damage, harm, or adverse effects on people or infrastructure. Hazards are typically classified into realized (those that have happened in the past) and unrealized (those that have not yet happened but could potentially occur in the future).

1.1.5. Geohazard

A natural or man-made hazard primarily characterized by or related to soil, rock, and groundwater behavior (geological processes). Such behavior can impact physical infrastructure and human communities. Note that, per the previous definitions, geohazards that are relevant to engineering projects are more accurately regarded as “georisks” because design is ultimately concerned with their potential consequences (not just the probability of their occurrence). We retain the terminology of geohazard for parsimony with the language often used in geotechnical manuals (e.g., [20]). The geotechnical hazards considered in this report include material behavior hazards (expansive/collapsible soils, permafrost/frost susceptible soils,

desiccation cracking, and corrosion), stability hazards (slope stability, seismic soil liquefaction, scour, and erosion), and other water-related hazards (tsunamis and groundwater/seepage). A complete description for each is given in Sec. 3.

1.1.6. Future Hazards and Conditions

Future hazards and conditions are defined as “significant anomalies in temperature, precipitation, and winds that may manifest as heavy precipitation and flooding, heatwaves, drought, wildfires, and windstorms” [20]. Table 1 provides definitions for the future hazards relevant to this report.

Table 1. Definitions of hazards discussed in this report.

Hazard	Definition ¹
Weather/climate	Weather is the current state of the atmosphere (and its short-term variations) in terms of temperature, humidity, precipitation, cloud cover, and wind. Climate is a statistical description (i.e., mean, variability, cyclical patterns) of the weather in a specific region over a defined time frame. In summary, weather refers to short-term changes in the atmosphere, while climate describes the weather over a long time in a given region.
Drought	An exceptional period of water shortage due to some combination of rainfall, high temperatures, and regional wind patterns.
Extreme heat/heatwave	Temperatures that are much hotter than average. A heatwave is when extreme heat occurs for an abnormally long period, ranging from several days to months.
Extreme weather	A rare weather-related event of high magnitude (e.g., greater than the 90th percentile of historical data) for a given location. High magnitude is also relative to the duration of interest (e.g., hourly, daily, or multiday). This can include precipitation, such as rain or snow, or wind. Specific examples of extreme weather include blizzards, hurricanes/tropical cyclones, monsoons, and tornadoes.
Flooding	Overflow of water beyond the normal limits of a body of surface water (e.g., stream, river, or ocean) or the accumulation of water over areas that are not normally submerged.
Relative sea level change	An increase in sea height relative to a fixed point on the coast. This incorporates both changes in the ocean water levels from melting of glaciers and ice sheets, water circulation patterns, changes in ocean water density, and local subsidence or tectonic uplift.
Saltwater intrusion	Displacement of fresh surface or groundwater by advancing saltwater. This can occur because of both

	drawdown of aquifers (from e.g., pumping or reduced recharge) and/or relative local sea level changes.
Wildfire	A wildland fire originating from an unplanned ignition (e.g., lightning, volcanoes, or human activities) or a prescribed burn that is designated as a wildfire.

1.1.7. Compound, Cascading, and Multi Hazards

Concurrent and/or sequential occurrence of multiple hazards (e.g., drought followed by flooding, or fire or flood following an earthquake) where the initial events may damage or degrade the physical infrastructure under consideration, leaving it in a weakened state and more susceptible to compound or subsequent hazards.

1.1.8. Adaptation

Adaptation is “adjustment in natural or human systems in anticipation of a changing environment in a way that effectively uses beneficial opportunities or reduces negative effects.” [20]. An adaptive design is one that can “anticipate, prepare for, and adapt to changing conditions and withstand, respond to, and recover rapidly from disruptions” [24]. An example of an adaptive design is given by [22]. A rail corridor crossing a low-lying coastal area is constructed on trestles consisting of modular pre-cast piers and caps. The cap can be jacked up, and additional pier segments added to each trestle increase the overall track height if local relative sea levels are expected to change.

1.2. The Typical Role of the Geotechnical Engineer in Design

Any civil engineering project interacting with soil or rock should have some level of geotechnical engineering involvement (i.e., most civil infrastructure projects). The exact scope of work will vary from project to project as a function of locality, project owner, and reviewing agency expectations, project scale, degree of complexity, and other factors. Sec. 2.1 develops a taxonomy for classifying the type of geotechnical work based on the applicable codes and standards that govern the design. Table 2 provides a high-level overview of the typical role played by the geotechnical engineer and the engineering geologist for a given project based on various public agency geotechnical reporting guidelines (e.g. [25], [26], and many others) and the authors’ prior work experience. This summary aims to capture the *current* role of the geotechnical engineer or the engineering geologist (as commonly understood) in typical infrastructure projects. However, these roles may change as practice evolves to meet the emerging challenges of the 21st century.

Table 2. A summary of the role of the geotechnical engineer or engineering geologist in a typical infrastructure project.

Project Phase	Role of Geotechnical Engineer or Engineering Geologist
---------------	--

Exploration/planning	<ul style="list-style-type: none"> • Characterizes current surface and subsurface geology (soils, rocks, and topography) • Characterizes current and historic groundwater conditions • Characterizes the potential for geotechnical hazards¹ to impact the project • Based on the above, provide recommendations for foundation/structure types and potential mitigation strategies for geotechnical hazards
Design	<ul style="list-style-type: none"> • Determines an appropriate mathematical model for soil/rock behavior and their interaction with structures based on data from field exploration and laboratory testing • Determines stability of proposed slopes and grading • Determines geotechnical stability² of proposed project elements • Provides recommendations for site preparation and ground improvement (as necessary) to ensure the design assumptions are met
Construction	<ul style="list-style-type: none"> • Provides construction inspection services to ensure grading, foundation installation, and other geotechnical specifications are met • Provides geotechnical support for mitigation of unanticipated or changed conditions encountered during construction
Monitoring/maintenance	<ul style="list-style-type: none"> • Develops a program for ongoing measurements of settlement, groundwater levels/pore water pressures, slope movement, or other quantities of interest to verify the design is working as intended • Provides geotechnical support for proposed maintenance/rehabilitation work
<p style="text-align: center;">¹Geotechnical hazards are defined in Sec. 3</p> <p>²This typically entails analysis to ensure site soils (and ground improvements, as necessary) and proposed foundation systems can support the project features without bearing failure, excessive settlement, or sliding</p>	

1.3. Geotechnical Aspects of the Built Environment (Geostructures)

Geotechnical engineers and engineering geologists work on a highly diverse range of projects, from single-family residences to large embankment dams. This section outlines the components of typical projects that fall within the geotechnical scope of work based on various public agency geotechnical manuals (e.g., [27], [28], [29] and others) and the authors’ prior work experience. These components are also referred to as geostructures (to reflect the interdisciplinary aspects of their design).

1.3.1. Foundations

The construction of a structure such as a bridge, building, storage tank, or equipment pad characterizes most civil engineering projects. These structures need an adequate foundation system to transfer loads into the earth. Foundation systems are usually classified as shallow (e.g., spread/strip footings and reinforced concrete mats/slabs) or deep (e.g., driven piles and cast-in-place shafts). Geotechnical engineers ensure that the subsurface soil can safely support the proposed foundations that will transfer the load from the superstructure (determined in collaboration with the Structural Engineer) into the ground without failure or excessive settlement for all the load cases considered (e.g., static, hydrologic, and dynamic loads). In addition to many environmental conditions noted throughout the report, the stability

evaluation of geotechnical infrastructure also includes the effects of co-seismic hazards such as liquefaction, lateral spreading, and/or surface fault rupture as necessary. Geotechnical engineers also provide specifications for excavation, grading, and backfill/compaction (as required) for the proposed foundations.

1.3.2. Earth Retaining Structures

Many projects also involve substantial grading operations, which may require temporary or permanent earth-retaining structures when the resulting topography cannot be sloped in a stable configuration. Earth retaining structures are typically grouped into gravity (e.g., typical reinforced concrete cantilevered walls), non-gravity (e.g., sheet pile and soldier pile walls), and internally reinforced systems (e.g., mechanically stabilized earth, tie-backs, or soil nail walls). Geotechnical engineers characterize the static, hydrologic, and dynamic earth pressures imposed on these, design their foundations, and assess if the proposed anchoring/soil reinforcement can develop enough friction with the surrounding soil/rock to ensure stability when applicable. Other than the environmental conditions noted throughout the report, the analyses of earth retaining structures also include the effects of co-seismic hazards such as liquefaction, lateral spreading, and/or surface fault rupture as necessary. Geotechnical engineers also provide specifications for acceptable soil types used in backfill, underneath the foundation, and for drainage.

1.3.3. Embankment Dams and Levees

Dams are used throughout the U.S. for hydropower generation, flood control, and water storage. They can be broadly classified into concrete/masonry and embankment (soil and rockfill) dams. Levees are a specialized case of embankments primarily for temporary flood control that extend for long linear distances. In all cases, the geotechnical engineer is responsible for characterizing the foundation material and ensuring the structure can be supported safely with appropriate foundation preparation and treatment. For embankment dams and levees, geotechnical engineers also address static and dynamic slope stability under routine and extreme conditions, design seepage control measures, and provide specifications for acceptable soil/rock types used within the various zones of the constructed embankment.

1.3.4. Engineered Slopes

Site grading often requires the construction of cut and fill slopes. Additionally, many areas in the US may also have site conditions (e.g., topography, drainage, relatively weak soil/rock) that are prone to landslides or other slope instabilities. The geotechnical engineer is responsible for evaluating the stability of existing and proposed slopes under various static and seismic loading conditions. If slopes are unstable in the current or proposed configurations, the geotechnical engineer recommends appropriate stabilization measures. Geotechnical engineers also provide specifications for site grading to ensure that the proposed slopes and/or stabilization measures can be constructed safely.

1.3.5. Underground Structures

Underground (or below-grade) structures comprise a wide variety of structural components and systems, such as basement walls, culverts, tunnels, buried pipelines/utility conduits, and wet wells/lift stations, among others. These structures must resist the weight of the surrounding soil and rock, as well as the earth pressures from the surrounding ground. Geotechnical engineers characterize these vertical and lateral earth pressures and interface friction characteristics. They also provide specifications for excavation and trench backfill/compaction (as necessary) for the proposed underground construction

1.3.6. Pavements

Pavement design is an interdisciplinary effort between the geotechnical, transportation, and general civil engineers. The geotechnical engineer is typically responsible for characterizing the subgrade material and drainage characteristics and recommending preparation/stabilization as necessary. In some cases, the geotechnical engineer may also calculate required pavement section thicknesses for traffic loads (provided by a pavement or transportation engineer).

1.3.7. Construction Considerations

Building any of the above-proposed project improvements will usually involve some degree of site grading, excavation, and backfill compaction specifications. First, the role of an engineering geologist is to identify the geology aspects pertinent to the project. Subsequently, geotechnical engineers identify features of the site geology, such as hard rock, soft/weak soils, or high groundwater, that might require specialized construction equipment/techniques, temporary excavation shoring, dewatering, or subgrade stabilization to build the proposed project safely.

1.4. What Defines the State of Practice?

The “state of practice” in geotechnical engineering is somewhat nebulous. Geotechnical engineering firms vary in the localities they practice in, the scope of (and funding available for) the projects they work on, and the experience and backgrounds of their employees. Public agencies that employ geotechnical engineers have a similar diversity in their responsibilities and capabilities. From a legal perspective, the “state of practice” may be defined as: “the ordinary skill and competence exercised by members of a profession in good standing in the community at the time of the event creating the cause of action” [30]. This work summarizes the current state of practice by examining publicly available national, state, and local design standards and guidelines, assuming that these documents are, in turn, used by practicing engineers/agency reviewers as a basis for their work. The following sections briefly describe the distinction between national and state/local level guidelines.

1.4.1. National Level Guidelines

National guidelines include model building codes, design standards, design manuals, research syntheses, and other supplementary design guidelines published or adopted by federal agencies and nationwide organizations. They are usually intended to set a nationwide minimum standard for life safety design. In some cases, they may provide enough information to guide a detailed design anywhere in the nation, and in others, they may include generic guidelines intended to be amended and/or supplemented by state and local guidelines.

1.4.2. State and Local Level Guidelines

State and local guidelines typically take the form of amendments and supplements to the national documents. In some cases, state/local public agencies may also publish their own detailed design documents.

1.4.3. Practitioner and Reviewer Understanding

Once a project's design is completed, it must (usually) be submitted to the appropriate agencies for review and approval. Depending on the applicable codes and laws, this may be, for example, a Department of Buildings, Department of Transportation, Department of Public Works, or Division of Dam Safety. For higher-profile projects, there may also be a peer review component. At this stage, the relevant national and state/local standards and guidelines are distilled into their final forms: how practitioners interpret their instructions and what the reviewers are willing to accept.

1.5. Challenges in Defining the State of Practice

A primary challenge in defining the state of practice in geotechnical engineering is associated with insufficient details provided by published laws, codes, and technical guideline documents. As will be discussed in Section 3, available material, in some cases, may be prescriptive and provide detailed procedures and requirements for modeling and analysis related to safety and/or performance. On the other hand, more frequently, there are implicit guidelines that specify the design should be performed (by a qualified engineer) without providing commentary as to how it should be performed. This makes the state of practice nearly impossible to determine from publicly available information. While there are undoubtedly published research and textbook materials regarding the topics in the more implicit or indirect cases, it is not clear which references (if any) local practitioners and reviewers follow.

A secondary challenge in defining the state of practice is the sheer volume of potential publications that may define it. Even at a national level, different laws, codes, and design standards will apply depending on whether one is working on a building, bridge, dam, underground structure, or one of the many other types of geotechnical projects. Next, each state will adopt and amend these national design standards. Individual municipalities (e.g., cities and counties) may also provide further amendments to the state-level guidelines. As a further complication, the detail and breadth of the state/local amendments can vary

considerably and may be difficult to predict before reviewing the documents. The result is that it is not feasible to perform an exhaustive review of all the potentially relevant material within this project's scope.

Finally, locating the documents (in some cases) is a non-trivial task. The process is typically straightforward for national-level publications or state building codes. However, the building codes do not typically give detailed guidelines on geotechnical design criteria or methods. This is often instead found in documents like state/local geotechnical hazard mitigation guidelines, geotechnical report review criteria, or geotechnical manuals published by various public agencies. However, the exact agency by which these are published often varies from state to state and agency to agency within a single state, and often, the process for requesting/acquiring the documents can be opaque to those without practical experience in the jurisdiction.

1.6. What is Not Discussed in This Report (Project Scope and Limitations)

This report focuses primarily on geotechnical design guidelines and how considerations for future hazards are (or are not) incorporated therein. It does not address aspects of civil engineering design that are not considered the responsibility of the geotechnical engineer (even though they may be relevant to engineering practices relevant to addressing future hazards and conditions). Typically, these would fall under the purview of a structural or hydraulic engineer:

- Methods for developing wind, rain, and ice loadings
- Methods for developing design flow depths, velocities, and inundation limits for flood/tsunami loadings and scour analysis
- Methods for developing design water levels for flood control structures or dams
- Structural analysis of reinforced concrete, steel, wood, or other materials

However, it should be noted that design for these hydraulic/structural aspects still requires geotechnical expertise (i.e., determining the amount of scour to be combined with seismic loadings) and often involves iterative collaboration between different engineering specialties. NIST SP-1190 [23] describes some commonly used hazard levels for buildings and facilities. These are grouped into routine, design, and extreme event criteria, including precipitation, wind, seismic, flood, fire, and blast threats.

Additionally, the geotechnical design procedures will not be discussed in detail to keep this report a reasonable length. Instead, the report focuses on aspects of the design procedure involving geotechnical hazards potentially affected by future hazards and conditions. For example, when discussing foundation design, a technical description of methods presented in guideline documents for evaluating bearing capacity and settlement will not be given. Instead, how the design guidelines address future groundwater levels, moisture-sensitive soil behavior, and scour/erosion will be emphasized.

Finally, this report constitutes a review, not an evaluation or interpretation, of the geotechnical engineering procedures that comprise the state of practice. Many (if not all) of the geotechnical hazards discussed are areas of ongoing recent research, and there is often reasonable disagreement among practitioners, researchers, and agencies on the appropriate method(s) of analysis, design, and mitigation. This work is intended as a factual reporting of the preferred/required engineering procedures of the respective documents, not as an opinion on whether said procedures are the most appropriate for the problem at hand or incorporate the most current research. Additionally, it should not be read as a description of how practitioners may interpret said guidelines. This will be addressed in future research efforts by surveys and workshop discussions with practitioners.

1.7. Potential Interactions Between Future Hazards/Conditions and Geotechnical Design (Scope of Review)

The long-term patterns of local and regional extreme weather, temperatures, sea levels, and surface and groundwater storage are areas of active, ongoing research, which this work does not recapitulate. Documents such as [6] provide a comprehensive review of the current state of science. Importantly, these documents also describe the various sources of uncertainty in future projections resulting from inherent variability in the climate system, uncertainty in the exact response of system components (e.g., land, air, water, and ice) to future warming, and uncertainty regarding future trends in greenhouse gas emissions resulting from social, economic, and administrative factors.

For this section, potential changing future hazards and conditions are distilled into four main categories:

- Rising near-surface and subsurface temperatures (including heatwaves)
- More severe extreme weather (e.g., intense precipitation/wind, droughts, and El Niño/La Niña wet/dry cycles)
- Sea level changes (including saltwater intrusion)
- Groundwater depletion

Table 3 presents some key findings regarding past and future trends.

Table 3. A summary of current and future trends in hazards and conditions.

Category	Key Findings
Temperature	<ul style="list-style-type: none"> • Yearly average temperatures have risen between 1986-2016 relative to a 1901-1960 baseline, as has the frequency of heat waves. • Both yearly average temperatures and temperature extremes are projected to rise over the next century. Temperature extremes are projected¹ to rise more than yearly average temperatures.

<p>Extreme weather - Precipitation</p>	<ul style="list-style-type: none"> • The western and southern states have seen a decrease in annual precipitation relative to a 1901 baseline. The northern and midwestern states have seen an increase in annual precipitation. • High-intensity precipitation events including atmospheric rivers, tropical cyclones, and their coincident flood risks are projected to increase in magnitude and frequency².
<p>Sea level changes</p>	<ul style="list-style-type: none"> • Global mean sea levels have risen appreciably relative to a 1900 baseline and are projected to continue rising over the next century. • Sea level changes along most of the Gulf and eastern US coast is expected to be greater than the global average. • Sea level changes will likely exacerbate coastal flooding.
<p>Groundwater depletion</p>	<ul style="list-style-type: none"> • Large aquifers, primarily in the south, southwest, and western United States, have suffered substantial depletion relative to a 1900 baseline. The rate of depletion has increased since 1950 and accelerated in recent decades [31]. • Forecasted reductions in snowpack and snowmelt may lead to drought in the western U.S., resulting in a potential increase in demand for groundwater and accelerated rates of depletion.
<p>¹The magnitude of forecasted changes in mean trends and variability in the discussed hazards will depend on how global atmospheric greenhouse gas emission patterns (often referred to as Representative Concentration Pathways) evolve.</p> <p>²There are somewhat complicated regional differences in forecasted changes to extreme weather patterns.</p>	

Below, some potential ways these future hazards interact with aspects of geotechnical engineering design are described. The exact interaction sections are further detailed in Sec. 3.

1.7.1. Changes in Loads or Hazards (Including Compound Scenarios)

The primary effects of future conditions on typical civil engineering design will likely manifest as increases in the magnitudes and frequencies of wind, flood, and precipitation loads. There are ways that such climatic hazards directly affect geotechnical design. For example, elevated groundwater or surface water levels from extreme precipitation, flooding, and/or sea level changes will increase the total unit weight of soil as well as lateral earth pressures acting on slopes, retaining structures, or underground structures that need to be accounted for in the design. Further, elevated groundwater can increase the likelihood of internal erosion near the surface or within the subsurface. Additional lateral wind loads also add to the demand on the foundation system that must be considered when assessing sliding and overturning potential.

The risk posed by various geohazards can also be affected by long-term changes in weather conditions. For example, rising groundwater levels in coastal regions from sea level can lead to an increased potential for static and seismic soil liquefaction. Similarly, an increase in wildfire/intense precipitation sequences can lead to an increased potential for debris flow and slope instability. These hazards can also be compounded – an increase in the magnitude and frequency of tropical storms/flooding can lead to a higher probability that these events may overlap with the occurrence of a non-climatic hazard like an earthquake.

1.7.2. Changes in Soil Hydro-Thermo-Mechanical Properties

Soil behavior is fundamentally a result of the complicated interactions between its three constituent phases – solid mineral particles, pore liquids (typically water), and pore gases (typically air). These interactions are highly sensitive to environmental extremes such as prolonged drought, heavy precipitation, or melting of permafrost, which may be increasing in frequency and intensity. These extremes affect both shallow unsteady (<2 m) and deeper steady soil layers through soil-atmospheric interactions, changing the groundwater table (up to 2 m [32]) and causing heat and moisture fluxes that alter the soil's mechanical properties under static or dynamic loading (thermo-hydro-mechanical properties and behavior).

One of the most salient examples of these interactions is frost-susceptible soils and the melting of permafrost (e.g., in the Arctic), where freeze/thaw cycles of the pore water can lead to dramatic volume changes and strength/stiffness losses. Additionally, variable saturation from long-term changes in near-surface temperatures and relative humidity can result in prolonged drought, shrinkage (desiccation cracking), and swelling of soils, which can substantially alter the strength and stiffness of earthen infrastructure under subsequent events (e.g., precipitation).

1.7.3. Performance and Vulnerability of Geotechnical Structures

Flowing surface water from flooding and extreme weather can lead to increased scour and erosion of soils supporting foundations and slopes, increasing the vulnerability of geotechnical structures. At a larger scale, long-term groundwater depletion (perhaps in response to drought, which is in turn accelerated by an increase in air temperature, a decrease in relative humidity, and by groundwater pumping) can lead to subsidence and subsequent structure damage from settlement. Drought can also induce surface cracking, which can damage retaining structures, slopes, and embankments during subsequent events like precipitation. Variable saturation and flow from long-term changes in precipitation patterns and sea level changes can increase corrosion rates of buried structures and foundations (compared to permanently dry or wet cases), particularly if the sea level change leads to saltwater intrusion.

Table 4 contains several steps in a geotechnical design and the questions a practicing engineer might ask at each step to illustrate how such interactions might be incorporated into routine practice.

Table 4. A conceptual description of how future hazards and conditions could be incorporated into a geotechnical engineering workflow.

<p>Step 1: Provide recommendations to the project team regarding the appropriate foundation system and/or hazard mitigation strategy and design process</p>	<ul style="list-style-type: none"> • How is the viability of alternative solutions affected by future conditions? • How to consider the impacts of alternative solutions in the decision process on performance?
<p>Step 2: Determine problem geometry (ground elevations, soil layer elevations/stratigraphy, groundwater elevations)</p>	<ul style="list-style-type: none"> • What ground surface elevation/topography should be assumed, considering future scour and erosion? • What groundwater elevation should be assumed considering sea level changes, changes in precipitation, and changes in regional groundwater demand/pumping? • How may these factors affect substructures' performance and stability?
<p>Step 3: Employ an analytical solution (if existing) or a computational model (often in the form of finite element analyses and soil constitutive models), followed by assigning engineering soil properties</p>	<ul style="list-style-type: none"> • What is the depth of the zone where soils are expected to undergo substantial changes in saturation and/or temperature over the structure's design life? What degree of saturation should be assumed for each load case within this zone? • How is the effect of variable saturation and/or temperature captured in the assumed soil and fluid mechanical properties or constitutive models? • Are laboratory tests for determining soil's mechanical properties reflective of future weather conditions and thermo-hydraulic loading?
<p>Step 4: Apply appropriate load and resistance¹ factors and assess if the current configuration is acceptable for the case</p>	<ul style="list-style-type: none"> • Does the calibration procedure used to determine the load/resistance factors adequately capture extreme weather effects expected in the region? • Do extreme event load cases appropriately combine climatic and other hazards?
<p>¹Load resistance factor design (LRFD) terminology is used here, but the same principle applies to allowable stress designs (ASD) and associated factors of safety, as well as performance-based design processes.</p>	

1.7.4. Summary Tables

Table 5 summarizes some potential interactions between geohazards and provides relevant climate metrics based on [33]. Climate metrics are quantitative measures that serve to link outputs of climate models with relevant aspects of geotechnical/structural analyses. Examples of recent case histories that showcase the potential deleterious effects of these interactions are also provided. Table 6 summarizes some relevant vulnerabilities to specific hazards for each geotechnical class.

Table 5. A summary of geohazards, their stressors, relevant climate metrics, and illustrative case studies (as applicable).

Geohazard	Stressors	Example Relevant Climate Metrics ¹	Illustrative Case Studies
Expansive/collapsible soils	<ul style="list-style-type: none"> • Increasing magnitude and frequency of wet/dry precipitation cycles and subsequent increase in swelling/shrinking of near-surface soils • Increased surface air temperatures and subsequent reduction in near-surface soil water content 	<ul style="list-style-type: none"> • Average annual atmospheric temperature • Maximum and minimum day/nighttime temperatures • Cumulative days above average temperature • Drought severity indices (e.g., [34]) 	<ul style="list-style-type: none"> • Foundation Damage, Texas, USA [35]
Permafrost and frost-susceptible soils	<ul style="list-style-type: none"> • Increased surface air temperatures and subsequent thawing of frozen ground • Greater variability of winter temperatures and a subsequent increase in the number of freeze-thaw cycles 	<ul style="list-style-type: none"> • Average annual temperature • Maximum and minimum day/nighttime temperatures • Cumulative days above average temperature • Air freezing index (cumulative degree days below 0° C) 	<ul style="list-style-type: none"> • 2018 Earthquake, Anchorage, Alaska, USA [36]
Desiccation cracking	<ul style="list-style-type: none"> • Lowered groundwater levels from droughts and subsequent reduction in near-surface soil water content • Increased surface air temperatures and subsequent reduction in near-surface soil water content 	<ul style="list-style-type: none"> • Average annual temperature • Maximum and minimum day/nighttime temperatures • Cumulative days above average temperature 	
Corrosion	<ul style="list-style-type: none"> • Saltwater intrusion from relative sea level changes and/or groundwater depletion and subsequent increased soil salinity • Increased exposure to surface saltwater from relative sea level changes 	<ul style="list-style-type: none"> • Local relative sea levels • Local and regional groundwater levels 	<ul style="list-style-type: none"> • 2022 Fern Hollow Bridge Collapse, Pittsburgh, Pennsylvania, USA [37]
Seismic soil liquefaction	<ul style="list-style-type: none"> • Shallower groundwater conditions from relative sea level changes and subsequent increased likelihood of surface manifestations/structural 	<ul style="list-style-type: none"> • Relative sea levels • Local and regional groundwater levels 	<ul style="list-style-type: none"> • 2002 Earthquake, Denali, Alaska, USA [38] • 2018 Earthquake, Anchorage, Alaska, USA [36]

	damage if an earthquake occurs		<ul style="list-style-type: none"> • 2020 Earthquake, Ponce, Puerto Rico [39] • 2023 Earthquake, Kahramanmaras, Turkey [40]
Scour and erosion	<ul style="list-style-type: none"> • Relative sea level changes and increased coastal erosion rates • More frequent and intense precipitation events/flooding and subsequent increased scour/erosion rates 	<ul style="list-style-type: none"> • Peak or extreme quantiles of streamflow • Precipitation intensity-duration-frequency curves • Annual number of flood/precipitation events • Flood event return periods • Local relative sea level 	<ul style="list-style-type: none"> • 2013 Flood, Manitou Springs, CO, USA [41] • 2014 Landslide, Oso, Washington, USA [42] • 2022 Flood, Gardiner, Montana, USA [43]
Tsunami	<ul style="list-style-type: none"> • Relative sea level changes and subsequent higher runup elevations if a tsunamigenic earthquake occurs 	<ul style="list-style-type: none"> • Local relative sea level 	<ul style="list-style-type: none"> • 2009 Tsunami, American Samoa [44] • 2018 Tsunami, Sulawesi, Indonesia [45]4/14/2026 2:21:00 PM
Groundwater/seepage	<ul style="list-style-type: none"> • More frequent and intense precipitation events/flooding and subsequent shallower groundwater levels and increased seepage volumes • Lowered groundwater levels from droughts and associated subsidence 	<ul style="list-style-type: none"> • Precipitation intensity-duration-frequency curves • Annual number of precipitation events • Local and regional groundwater levels 	<ul style="list-style-type: none"> • 2014 Flood, Christchurch, New Zealand [46] • 2018 Flood, Kerala, India [47] • 2015 Flood, Tbilisi, Georgia [48] • 2015 Flood, Central Texas, USA [49] • 2020 Flood, Midland County, MI, USA [50]
¹ The metrics provided are intended as illustrative suggestions and should be modified for project-specific constraints			

Table 6. A matrix of vulnerabilities to geohazards for each geostructure type.

Geohazard/ Geostructure	Foundations	Earth Retaining Structures	Slopes and Embankments	Underground Structures
Expansive/ collapsible soils	<ul style="list-style-type: none"> • Total/differential settlement from collapsing soils • Uplift/heave forces on foundations from swelling soils 	<ul style="list-style-type: none"> • Total/differential settlement from collapsing soils • Lateral swelling forces on foundations from expansive soils 	<ul style="list-style-type: none"> • Decrease in soil strength from swell cycles 	<ul style="list-style-type: none"> • Total/differential settlement from collapsing soils • Uplift/heave forces on underground structures from swelling soils
Permafrost and frost- susceptible soils	<ul style="list-style-type: none"> • Total/differential settlement from thawing soil 	<ul style="list-style-type: none"> • Total/differential settlement from thawing soil 	<ul style="list-style-type: none"> • Decrease in soil strength from freeze-thaw cycles • Increased seepage and 	<ul style="list-style-type: none"> • Total/differential settlement from thawing soil

	<ul style="list-style-type: none"> • Loss in bearing capacity from thawing soil • Uplift/heave forces on foundations from freezing soils 	<ul style="list-style-type: none"> • Loss in bearing capacity from thawing soil • Lateral heave forces on walls 	<ul style="list-style-type: none"> • reduction in soil strength from thawing soil 	<ul style="list-style-type: none"> • Loss in bearing capacity from thawing soil • Uplift/heave forces on underground structures from freezing soils
Desiccation cracking	<ul style="list-style-type: none"> • Preferential seepage pathways and softening of foundation soils 	<ul style="list-style-type: none"> • Preferential seepage pathways and increased hydrostatic loading from backfill • Increased lateral earth pressures from tension cracking in cohesive backfill 	<ul style="list-style-type: none"> • Preferential seepage pathways from tension cracking • Loss of soil strength in the tension cracking zone 	<ul style="list-style-type: none"> • Preferential seepage pathways and softening of foundation soils
Corrosion	<ul style="list-style-type: none"> • Loss of structural capacity from the degradation of steel/concrete 	<ul style="list-style-type: none"> • Loss of structural capacity from the degradation of steel/concrete 	<ul style="list-style-type: none"> • Degradation structural stabilization (e.g., anchors and tiebacks) 	<ul style="list-style-type: none"> • Loss of structural capacity from the degradation of steel/concrete
Seismic soil liquefaction	<ul style="list-style-type: none"> • Total/differential settlement from liquefied soil • Drastic loss in bearing capacity from liquefied soil • Lateral spreading and associated loading on foundations 	<ul style="list-style-type: none"> • Total/differential settlement from liquefied soil • Drastic loss in bearing capacity from liquefied soil • Lateral spreading and associated increased lateral forces on walls 	<ul style="list-style-type: none"> • Drastic loss of soil strength from liquefaction 	<ul style="list-style-type: none"> • Total/differential settlement from liquefied soil • Drastic loss in bearing capacity from liquefied soil • Lateral spreading and associated loading on foundations • Uplift of buoyant underground structures
Scour and erosion	<ul style="list-style-type: none"> • Undermining and loss of foundation support 	<ul style="list-style-type: none"> • Undermining and loss of foundation support 	<ul style="list-style-type: none"> • Undermining and loss of toe material • Overtopping of embankment dams/levees and loss of supporting material 	<ul style="list-style-type: none"> • Loss of soil cover
Tsunami	<ul style="list-style-type: none"> • Hydrodynamic forces on structural elements transferred to the foundation • Softening of foundation soils from seepage resulting from inundation 	<ul style="list-style-type: none"> • Hydrodynamic forces on wall elements • Softening of foundation soils from seepage resulting from inundation 	<ul style="list-style-type: none"> • Undermining and loss of toe support • Overtopping of embankment dams/levees and loss of supporting material 	<ul style="list-style-type: none"> • Loss of soil cover

	<ul style="list-style-type: none"> • Undermining and loss of foundation support 	<ul style="list-style-type: none"> • Undermining and loss of foundation support 		
Groundwater /seepage	<ul style="list-style-type: none"> • Softening of foundation soils from seepage • Reduction in bearing capacity from rising groundwater levels • Total/differential settlement from the lowering of the groundwater table 	<ul style="list-style-type: none"> • Hydrostatic and seepage forces from saturated backfill • Softening of foundation soils from seepage 	<ul style="list-style-type: none"> • Greater driving and lower resisting forces from the combined effects of higher groundwater levels and seepage • Internal erosion and piping failures 	<ul style="list-style-type: none"> • Softening of foundation soils from seepage • Reduction in bearing capacity from rising groundwater levels • Uplift of buoyant underground structures

2. Methods

This chapter describes the guiding principles behind this literature review, the criteria for deciding which documents to include, and the process for extracting relevant information from the documents considered.

2.1. Literature Review Guiding Principles and Procedures

A primary objective of this work is to establish a baseline for the current state of geotechnical engineering practice and codes/regulations regarding the impacts of future hazards and conditions. Another objective of this report is to provide a launch point for new research efforts on specific identified gaps and future needs. However, as detailed in Sec. 1.5, an exhaustive literature review that encompasses all (or even a large majority of) local geotechnical building code sections, design standards, or technical guideline documents across the US is impractical. Instead, this work aims to provide a representative overview of the current state of practice nationwide and in a few example states. The intent is that the types of work and documents covered provide enough of a foundation that if specific details of practice in an area not covered are desired, the process for doing so should be relatively straightforward. As stated previously, this review provides a factual summary of what is stated in the documents, not a description of local practitioner interpretation and/or application.

The process of this literature review follows the general code development process in the US – national level standards detail minimum design standards, which are then adopted/amended by state and local level agencies. First, typical geotechnical engineering work is classified into distinct categories based on the unique characteristics of exploration/testing, analysis, and design performed. This work ultimately covers three categories: buildings, transportation structures, and hydraulic geostructures. These categories also include site grading, embankments, cut/fill slopes, and earth-retaining components that support the respective structures (for clarity, the specific geostructures covered by each document are provided). For each category, a respective national standard associated with each is identified and summarized in Table 7. In some cases, these design standards have associated technical guidelines, such as the Federal Highway Administration (FHWA) Geotechnical and Hydraulic Engineering Circulars (GEC/HEC), which the review also includes whenever practicable.

Table 7. The established categories for geotechnical work and their associated national standards.

Category	Example projects	National Standard(s)	Short Description
Buildings	Structures intended for human occupancy Relevant geostructures: foundations, earth retaining structures, engineered slopes,	<ul style="list-style-type: none"> • International Building Code (IBC) [51] • American Society of Civil Engineers (ASCE) 7-22: Minimum Design Loads and Associated Criteria for Buildings and Other Structures [52] 	<ul style="list-style-type: none"> • Gives minimum standards for life safety for most buildings and structures intended for human occupancy¹ • Covers the engineering basis and methodology for developing loads required to be designed for in the building code. It also gives hazard levels, targeted reliabilities, and

	underground structures		intended performance of structures designed to the standard
Transportation structures	Bridges, roads/highways, and associated infrastructure Relevant geostructures: foundations, earth retaining structures, engineered slopes, underground structures, pavements	<ul style="list-style-type: none"> American Association of State Highway Transportation Officials (AASHTO) Load Resistance Factor Design (LRFD) Bridge Design Specifications (BDS), 9th Edition [27] 	<ul style="list-style-type: none"> Covers engineering basis and methodology for the design and evaluation of highway bridges and ancillary structures (new and retrofitted)
Hydraulic geostructures	Embankment dams and levees	<ul style="list-style-type: none"> United States Bureau of Reclamation (USBR) Design Standards, Design Standards No. 13 (DS 13), Embankment Dams [29]² 	<ul style="list-style-type: none"> Covers engineering basis and methodology for design and construction of earthfill and rockfill dams designed for/by the Bureau of Reclamation for storage, diversion, or detention of water associated with federal dams
<p>¹Most detached one- and two-story single-family residences are covered by the International Residential Code. However, their geotechnical aspects of this document are often similar to the International Building Code</p> <p>²Some chapters of DS13 have been updated more recently.</p>			

While these categories and design standards cannot cover every possible scope of work, they should cover enough of day-to-day geotechnical practice to give a representative sampling of the state of practice. When reviewing the national level documents, the primary focus is on:

- Identifying sections that are specifically relevant to geotechnical exploration, analysis and design, and construction;
- Summarizing explicit (prescriptive) guidelines provided for the incorporation of future extreme weather effects and considerations into geotechnical exploration, analysis and design, and construction;
- Describing cases where direct guidelines are not explicitly provided, but considerations of future hazards and conditions are indirectly implied.

This report also reviews additional national-level guidelines to supplement the previously mentioned documents. These include:

- United States Department of Defense Unified Facilities Criteria (UFC) 3-220 manuals, formerly the Naval Facilities Engineering Command Design Manual (DM) 7.1 and 7.2 ([53], [54]).

- United States Army Corps of Engineers (USACE) engineering manuals (EMs), engineering circulars (ECs), engineering regulations (ERs), and/or engineering and construction bulletins (ECBs) ([55], [56], [57], [57], [58], [59], [60])
- American Lifelines Alliance (ALA) pipeline design guidelines ([61], [62], [63])

These are reviewed to identify notable modifications or deviations from other national-level documents (rather than giving a comprehensive summary) to avoid repeating material. Note that, as a result, some geotechnical hazards may not contain a section for additional national-level guidelines – this should be understood as these documents did not provide substantially different guidelines than the material already covered.

Next, relevant state and local-level geotechnical building code sections, design standards, or technical guidelines that modify and/or supplement the previously described national-level document are identified. The review focuses on a few selected states for geographical/administrative diversity and susceptibility to weather extremes to give a representative sampling of the current state of practice, guidelines, and regulations. The states selected are (in alphabetical order): Alaska, California, Florida, Illinois, and Texas. Table 8 summarizes some of the key state and local guidelines associated with the national level documents described above (not covering hydraulic structures).

Table 8. State and local guideline documents associated with geotechnical work categories (not including hydraulic structures).

Category	State/Local Guideline Document(s)	Short Description
Buildings	<ul style="list-style-type: none"> • Building code amendments • Hazard mitigation guidelines • Geotechnical/soils report review criteria 	<ul style="list-style-type: none"> • Modify parts of the national code to be more applicable to locally relevant weather and geology (e.g., permafrost in Alaska or hurricanes/flooding in the Gulf Coast) • Provide recommended procedures for identification and mitigation of state and municipality-specific hazards (e.g., seismic hazard in California or sinkholes/karst in Florida) • Provides consultants with information to aid in report preparation and ensure their submittals meet reviewer expectations for content and documentation of exploration/analyses performed
Transportation structures	<ul style="list-style-type: none"> • State Department of Transportation (DOT) amendments to AASHTO BDS • State DOT geotechnical manuals 	<ul style="list-style-type: none"> • Documents state DOT bridge and other structures design policy, amending or providing additional guidelines for methods in the AASHTO BDS • Documents state DOT geotechnical exploration, analysis, and design policy typically addressing at least: bridge/structure foundations, retaining walls, engineered slopes/embankments, and mitigation of locally relevant geotechnical hazards

For state and local level documents, the report focuses on identifying notable modifications to or deviations from the national level guidelines rather than giving a comprehensive summary to avoid repeating material. It again focuses on how considerations of future hazards are (or aren't) incorporated into geotechnical exploration, analysis and design, and construction.

After reviewing the relevant national, state, and local level documents, the report focuses on identifying potential gaps in the state of practice. The identified gaps by the authors focus on where a design procedure or explicit guideline is missing or the one(s) supplied are insufficiently detailed. Finally, potential research needs arising from these gaps in the state of practice are described and some ongoing work in the identified areas is summarized. It is acknowledged that this document is not a comprehensive state-of-the-art review of the impacts of future hazards on geotechnical design. Instead, general lessons from recent research are highlighted in each section in relation to where there is a need for clear guidelines in codes and standards that address geotechnical hazards and infrastructure.

3. Geotechnical Hazards

3.1. How to Use This Report

The following section describes the guidelines provided by the national and state/local level documents detailed in Sec. 2.1 for selected geotechnical hazards. For each section, the report focuses on the potential interactions between routine design-level and future geotechnical hazards and conditions, assuming that the reader is somewhat familiar with the basic mechanics of each hazard. We refer others to the FHWA GEC 5 “Geotechnical Site Characterization”, as do many introductory geotechnical engineering textbooks (e.g. [64]). For each geotechnical hazard, an overview is first given on how it might be affected by future hazards/conditions. Then, a summary of the relevant design guidelines is provided, separated into national and state/local components. If this summary does not describe aspects of geotechnical design that may interact with future hazards/conditions, it should be understood that the corresponding document does not provide such guidelines within the scope of this review.

For the state and local documents, this report focuses on highlighting the differences between them and the national-level documents. As a result, not every state will appear in each section if it does not provide substantial amendments to applicable national guidelines. If an aspect of the design is covered in the review at the national level but not at the state level, it should be interpreted as “the state/municipality defers to the national guidelines” rather than “the state/municipality provides no guidelines”. Finally, the report identifies some potential gaps in the available guidelines and current/future research efforts that can potentially fill said gaps.

As a general note, official code documents use language very intentionally -- “should”, “shall”, “recommend”, and “may” all have precise meanings in those codes. For readability, this report may not always directly replicate the language in documents as it is intended to be a general summary of the design guidelines provided, not an official interpretation or companion/commentary to the standards and codes. The original document language must be followed in design. Furthermore, terms such as “reasonable worst case”, “appropriate”, or “sufficient” may appear in this report without corresponding definitions. This usage is deliberate and is intended to reflect a similar lack of clear definitions in the original documents.

3.2. Expansive/Collapsible Soils

3.2.1. Overview and Hazard Considerations

Expansive and collapsible soils are primarily characterized by their potential for volume change from variable saturation. Expansive soils (primarily clays) will swell when saturated, and collapsible soils may experience a drastic volume decrease. This makes them susceptible to any hazard that alters the extent/behavior of the active zone where seasonal moisture changes occur. Some examples of these include drought cycles, extreme weather events that lead to intense precipitation or changes in relative humidity, groundwater depletion, and sea level changes. Drought cycles and rising near-surface/subsurface temperatures can pose a particular

concern, as the magnitude of volume change is largest when the initial water content is lowest [26].

3.2.2. Design Guidelines

3.2.2.1. IBC and ASCE 7

Section 1803 of the International Building Code (IBC [51]) requires that a geotechnical investigation, when required by the building official, be performed “in accordance with generally accepted engineering practice”. This investigation is expected to include the identification of “questionable soils [whose] strength or compressibility is in doubt”, expansive soils, if the existing (as opposed to future or historic) groundwater table is within 5 feet of the lowest finished floor elevation, and total/differential settlement (among others). Further studies should be made as necessary, including the evaluation of the effects of moisture variation on bearing capacity (perhaps including the effects of collapsible soils). Mitigation of these effects, along with special design/construction provisions, are expected to be addressed in the foundation design criteria of the geotechnical report.

The IBC provides two alternatives to identify potentially expansive soils. The first method is based on index properties (from tests performed according to their respective American Society for Testing and Materials [ASTM] standards):

1. Plasticity index (PI) of 15 or greater
2. More than 10 % of the soil particles pass a No. 200 sieve
3. More than 10 % of the soil particles are less than 5 micrometers in size

These criteria identify soils containing enough clay minerals with swell potential to pose an issue. Alternatively, identification can be based on expansion index testing (ASTM D4829). The expansion index test consists of preparing a compacted soil specimen (of the soil under consideration) to approximately 50% saturation, inundating it with water, and measuring its total height change under a vertical stress of approximately 7 kPa. The expansion index is then based on the final percent change in height after a 24-hour period, with values greater than 20% considered expansive by the IBC. Expansion index test results are allowed to supersede the index properties-based classification.

When expansive soils are present, they must be “removed to a depth sufficient to ensure a constant moisture content in the remaining soil” or stabilized by chemical treatment, dewatering, or “equivalent” techniques. The GeoTechTools database, developed by the ASCE Geo-Institute provides additional guidelines on type selection and design of relevant ground improvement strategies [65]. If expansive soils are not removed or stabilized, the IBC (sections 1807 and 1808) and ASCE 7 (section 3.2) [52] require that structural design account for the potential for swelling resulting in uplift loads on foundation elements (including deep foundations) and increased lateral earth pressures on retaining structures. The ASCE 7 commentary (C3.2) states that expansive soils should not be used as backfill behind retaining walls, while only high plasticity clays are explicitly forbidden by the IBC.

While collapsible soils are not directly addressed in the investigation and reporting requirements of the IBC, practicing engineers may consider their effects in the requirements for questionable soils, settlement estimates, and in evaluating the effects of moisture. ASCE 7 only directly mentions collapsible soils in section 20.3 to note that their presence may necessitate a seismic site class F and more detailed development of design ground motions.

3.2.2.2. AASHTO BDS and FHWA GEC 5

Section 10.4 of the American Association of State Highway and Transportation Officials LRFD Bridge Design Specifications (AASHTO BDS [27]) requires that the geotechnical exploration program be “sufficient” to characterize engineering properties of the soil/rock present, problematic subsurface conditions, including swelling/collapsible soils, groundwater conditions, and provide enough information to evaluate foundation stability and settlement with regards to the above. The BDS provides a correlation with the liquid limit and plastic limit of clay soils that can be used to establish swelling volume change potential (later, the commentary also says that expansion index testing may be used). Like the building code, the BDS requires that foundation design at the strength limit state consider the potential impact of expansive/collapsible soils, requiring that they “must be accounted for in design or removed/remediated before construction.” The FHWA provides additional guidelines on type selection and design of relevant ground improvement strategies in GEC 13 [66]. Additionally, the design of deep foundations requires that piles must extend into “moisture-stable” soils to provide uplift resistance and that “anticipated” changes in groundwater be considered in available soil resistance. Fig. 1 shows a conceptual depiction of the various components of loads and geotechnical resistances for static, swelling, and settling soils. In addition to these factors, soil collapse could also apply down drag onto the pile.

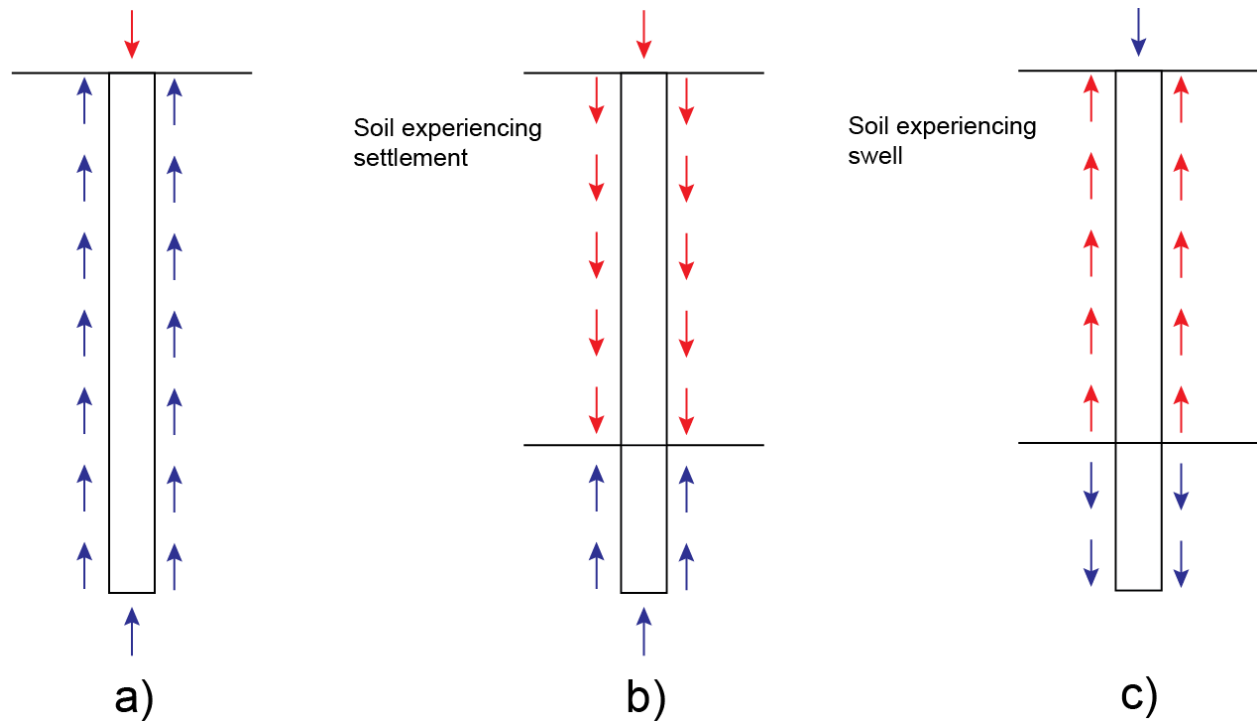


Fig. 1. An illustration of the components of loads (red) and resisting forces (blue) on a generic deep foundation element for a) typical soil conditions, b) settlement from applied stresses, lowering of the groundwater table, or soil liquefaction, c) expansion/swell.

When determining axial resistance of drilled shafts, engineers are instructed to neglect the upper 5 feet of soil to account for possible disturbance, and that this depth may need to be increased when expansive clay is present. The BDS does not directly address collapsible soils beyond the general remove or remediate requirement; however, all the design considerations for foundation capacity and settlement would need to incorporate their effects. The FHWA GEC 5 [26] provides a variety of laboratory and field test methods to aid in characterizing expansive/collapsible soils. To determine the depth of the active zone (where seasonal moisture change occurs), it is suggested to measure the water content of samples taken during the wet and dry seasons. The manual presents several classification methods based on index properties (including the ones in the BDS and IBC). The free swell test is also suggested as an additional indirect characterization. In this test, a small amount of oven-dried soil is placed in a container ring and inundated with water, and the total volume change is measured over the course of 24 hours. For direct characterization, the manual recommends a one-dimensional swell test (ASTM D4546). In this, a specimen (either an undisturbed field sample or a reconstituted specimen) is loaded under a range of vertical pressures, inundated with water, and the volume change is measured. This produces a plot of vertical stress versus wetting-induced strain, which can be directly used for design purposes. The same test can also be used to characterize collapse potential. Typical presentations of the results of laboratory swell and collapse tests are shown in Fig. 2.

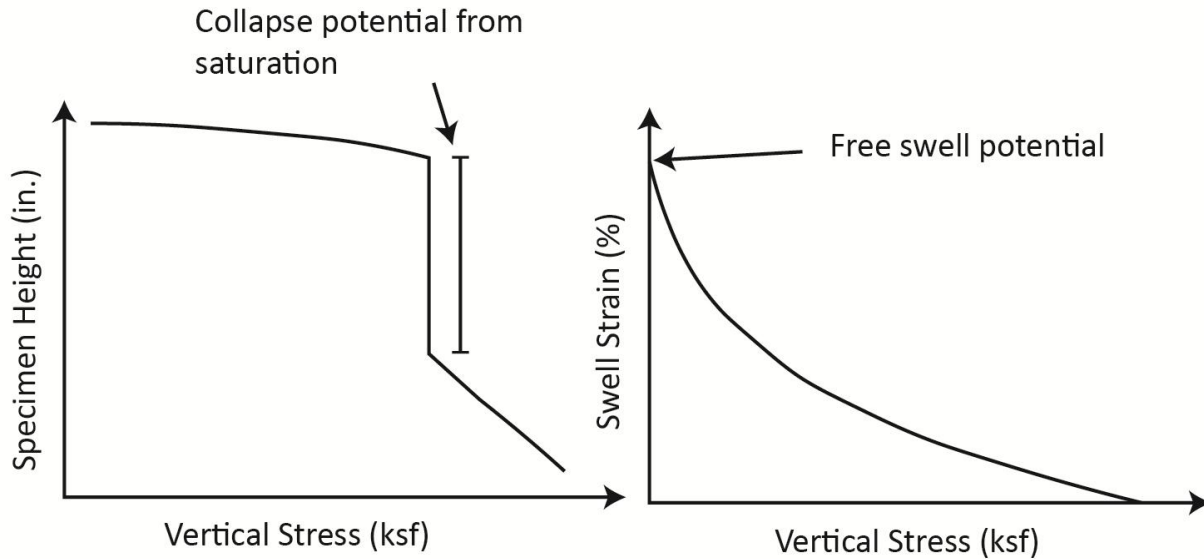


Fig. 2. Typical results of one-dimensional laboratory collapse (left) and swell tests (right). Reproduced from [26].

The manual notes that remolded/disturbed clays, perhaps from construction compaction, may exhibit higher swell behavior than in their natural state and that secondary swell for soils with long-term access to water may also be an issue.

3.2.2.3. USBR DS 13

The USBR Embankment Dam Design Standards [29] recognize that potentially collapsible materials can be a key failure mode for dams' foundation and embankments. Besides potentially leading to immediate failure, settlement from inundation can cause transverse cracking of the embankment and concentration of internal erosion. The manual notes that soils that do not collapse on first wetting can still do so as the embankment is constructed or upon first reservoir filling. The USBR's approach assumes that foundation materials will (eventually) become fully saturated during dam operation and thus must be evaluated for collapse potential using laboratory tests like the ones previously described. The tests are typically performed on undisturbed samples at their natural water content or at a saturated state. If collapse potential is indicated, removal of any problematic material is preferred. If removal is not possible (or uneconomical), preloading and prewetting are recommended, and verification that the total consolidation is achieved during construction.

The topic of expansive soils within embankments is not discussed extensively in DS13, though these soils can lead to notable problems jeopardizing the integrity of dams and embankments. There are some considerations for the shear strength of clays subject to repeated wet/dry cycles that are discussed in Sec. 3.8.

3.2.2.4. Additional National Level Guidelines

The UFC DM 7.1 [54] provides the IBC criteria to classify if soils are expansive or not. It also provides several correlations with index properties for swell and collapse potential from [64] and standard ASTM swell/collapse potential tests for quantitative estimates of volume change potential. The DM notes that factors affecting the degree of saturation and expansion potential include climate history, but does not provide quantitative guidelines to address these. It also suggests that the active zone where expansion potential is a concern is approximately 3 to 15 feet thick, depending on the climate. Finally, it notes that best maintenance practices limit exposure to the atmosphere and changes in water content, which involves collecting and diverting rain/surface water during wet periods and limiting evaporation during dry periods.

The UFC DM 7.2 [53] states that issues for foundations resulting from expansive soils occur over the range of seasonal moisture content and that shallow foundations should be placed below zones of volume change from seasonal moisture fluctuations. For collapsible soils, the DM, similarly placing shallow foundations should be placed below zones of collapse susceptible material. Deep foundations should be designed accounting for uplift loads on deep foundation elements.

The USACE EM 1110-2-1913 recommends against the use of silt as a primary construction material for levees because of its collapse potential [59]. EM 1110-2-2300 identifies expansive and/or collapsible foundation soils as a potentially problematic condition requiring specific geotechnical investigation

3.2.2.5. Notable State and Local Amendments

California. The California Geological Survey (CGS) Note Sheet 48 [67] specifically identifies collapsible soils as a geologic hazard to be addressed in geohazard reports as necessary.

The California Department of Transportation (Caltrans) amendments to the AASHTO BDS [68] do not allow spread footings where collapsible or expansive soils are present. The geotechnical manual also notes that inundation settlement can be a concern for deep compacted fills. The recommended procedure uses 1-d collapse tests to estimate settlements, considering the potential for “future wetting” and provide mitigation (e.g., higher compaction wet of optimum, prewetting and surcharging, or removal and replacement). The geotechnical manual recommends adding loads if deep foundations are used where collapsible soils are present.

Texas. The Texas Department of Transportation (TXDOT) Geotechnical Manual [69] uses a larger disregard depth for a deep foundation design of 10 feet when over water. Additionally, the manual recommends considering the potential for swelling or heave resulting from overburden removal if “considerable” excavation is planned. The effects of swell in deep foundation design are accounted for by applying a uniform swell pressure from 1-d swell tests to the surface area of deep foundation elements within the active zone, which can be estimated as 10 to 20 feet (depending on the engineer’s judgment).

Illinois. The City of Chicago’s Building Code [70] slightly modifies the IBC criteria for foundations on expansive soils, allowing that removal is not required if the applied pressure from

structures/fill exceeds the swell pressure of the soil. The Illinois Department of Transportation (IDOT) Geotechnical manual [71] states that potentially collapsible soils are usually “acceptably densified by “good construction practices”.

3.2.3. Gaps in Practice and Standards

Methods for identifying potentially expansive soils and characterizing their swell potential (viz., collapsible soils and their settlement potential) are well established. While one-dimensional swell/collapse tests have their limitations, for example, an inability to fully replicate in-situ stress states and water availability [26], they provide a readily available method for quantitatively characterizing the volume change potential from sudden saturation for expansive/collapsible soils. The calculated vertical strain can then be directly used in foundation design or to show unacceptable conditions that require mitigation. However, there are some gaps in the design guidelines provided, which are highlighted in the list below:

- The Building Code, while requiring evaluation of potentially expansive/collapsible soils in geotechnical investigation and reporting, provides very little explicit guideline or quantitative design procedures for doing so (beyond criteria for determining if expansive soils are present).
- Very limited general guidelines are provided on how to determine the extent of the current active zone, usually based on water content profiles developed from samples collected during the wet and dry seasons. However, this implicitly assumes that the climate conditions at the time of geotechnical exploration are representative of future conditions.
- Almost no guideline is provided on determining the extent of the future active zone, with or without considering future hazards.
- Sometimes, a more conservative approach is adopted specifically for dam structures and embankments, such as assuming that saturation of potentially collapsible soils will always occur, and to test/design accordingly. While this level of conservatism may be appropriate without an established method to forecast future saturation, it may be uneconomical, particularly when it results in large-scale removal or stabilization.
- Existing test methods that standardize specimen preparation or test undisturbed samples at their natural water contents may or may not represent future worst or expected conditions, considering future hazards, given that initial water content and relative density affect swell/collapse potential. Additionally, typical test methods do not incorporate multiple shrink/swell cycles.
- Guideline for addressing the effects of expansive/collapsible soils is typically limited to standard ground improvement strategies and does not specifically address future hazards/conditions or incorporate adaptive designs.

3.3. Temperature Effects (Permafrost, Frost Susceptible Soils, and Desiccation)

3.3.1. Overview and Future Hazard Considerations

Frozen soil is stronger and stiffer, has lower hydraulic conductivity, and higher volume than its unfrozen counterpart. When thawing occurs, substantial strength/stiffness loss, settlement, and seepage will occur, all of which can cause appreciable damage to the built environment. Permafrost thawing can also increase erosion potential and vulnerability to flooding. These interactions are consequential enough that they have been formally dubbed “usteq”, defined as “a catastrophic form of permafrost thaw collapse that occurs when frozen ground disintegrates under the compounding influences of thawing permafrost, flooding, and erosion.” [72]. Frost susceptibility exhibits similar phenomena – ice lenses that occur during freezing temperatures can cause differential heaving, which can create differential settlement when they thaw. The primary difference between the two is their susceptibility to different aspects of extreme weather. Permafrost is particularly vulnerable to warming near surface/subsurface temperatures, and frost susceptibility (because of its inherently seasonal nature) is more affected by increased variability in precipitation and extreme temperatures.

Desiccation of clays results from shrinkage – the mechanical opposite of swelling. The volume decreases from the lowering of water content (under the temperature range and relative humidity expected during droughts), which can lead to tensile stresses and subsequent cracking. This will lead to localized weakness in these zones but is of particular concern for slope and embankment stability, where desiccation cracks can create routes for water infiltration (which will increase seepage and destabilizing forces).

In addition to frost and desiccation, wildfires can also impact soil properties and behavior. Heat from wildfires can potentially permeate soils and affect buried infrastructure (such as pipelines). The extent of heat permeation, insulating effects of soil, and the subsequent impacts on soil properties and performance of buried infrastructure are topics of ongoing research (e.g., [73], [74]).

3.3.2. Design Guidelines

3.3.2.1. IBC and ASCE 7

Section 1809 of the IBC [51] requires that footings must be located below the local frost line unless they are specifically constructed with frost-resistant techniques or bear on rock. The IBC further requires that footings shall not bear on frozen soil unless the “frozen condition is of a permanent character”. The IBC does not specify how to determine the local frost line, but this value is usually given in local amendments (e.g. [75] or [70]). ASCE 7 [52] does not address frozen ground conditions; however, design procedures for frost-protected shallow foundations are given in ASCE 32 [76].

3.3.2.2. AASHTO BDS and FHWA GEC 5

Like the IBC, section 10.6 of the AASHTO BDS [27] requires that spread footings be located below the depth of frost potential based on local or regional data. The commentary also notes that the use of frozen material in fill underneath foundations can lead to insufficient bearing capacity and/or large settlements. Section 11 gives a similar requirement for minimum embedment retaining walls and requires that if soils used for wall backfill may expand from freezing, the wall be designed for increased lateral pressures.

The FHWA GEC 5 [26] notes that permafrost primarily occurs in Alaska but may also be present at high elevations in the mountain west. The manual primarily focuses on various geotechnical site exploration methods for characterizing the presence and extent of frozen ground. The first indication of permafrost is the presence of specific geomorphic features, which can usually be established during site reconnaissance. Traditional drilling and sampling methods can also be used, but may require a dense spacing to accurately characterize the site because of the inherent variability of frozen soils. There is also the potential for samples to thaw during extraction and preparation, potentially leading to unrepresentative laboratory test results. Geophysical methods can provide a good complement to traditional drilling and sampling. The manual suggests installing and monitoring thermocouples or thermistors at various depths to characterize the subsurface temperature regime and its potential variation over time, which is a key component of engineering analyses for frozen ground.

Neither the AASHTO BDS nor the FHWA GEC 5 directly addresses desiccation cracking. However, this is not entirely unexpected because material specifications, compaction requirements, and moisture conditioning for general fill and structural backfill are intended to ensure moisture stability (e.g., [77]). However, GEC 5 does provide test methods for determining the shrinkage limit (the water content at which further reduction in water content does not lead to volume change), which could potentially be used to qualitatively evaluate the potential for desiccation cracking via the shrinkage index (liquid limit minus the shrinkage limit).

3.3.2.3. USBR DS 13

Cracking of embankments is a key design concern because cracks can create preferential seepage pathways, accelerate internal erosion, and potentially lead to partial or total failure of the dam. Most of the USBR design and construction practices to limit potential cracking and mitigate its effects are described in the sections on scour/erosion and seepage [29]. Here, the Bureau's guideline that directly addresses cracking from desiccation is described.

The potential for desiccation cracking is directly related to the dam's cross-section type, shown in Fig. 3.

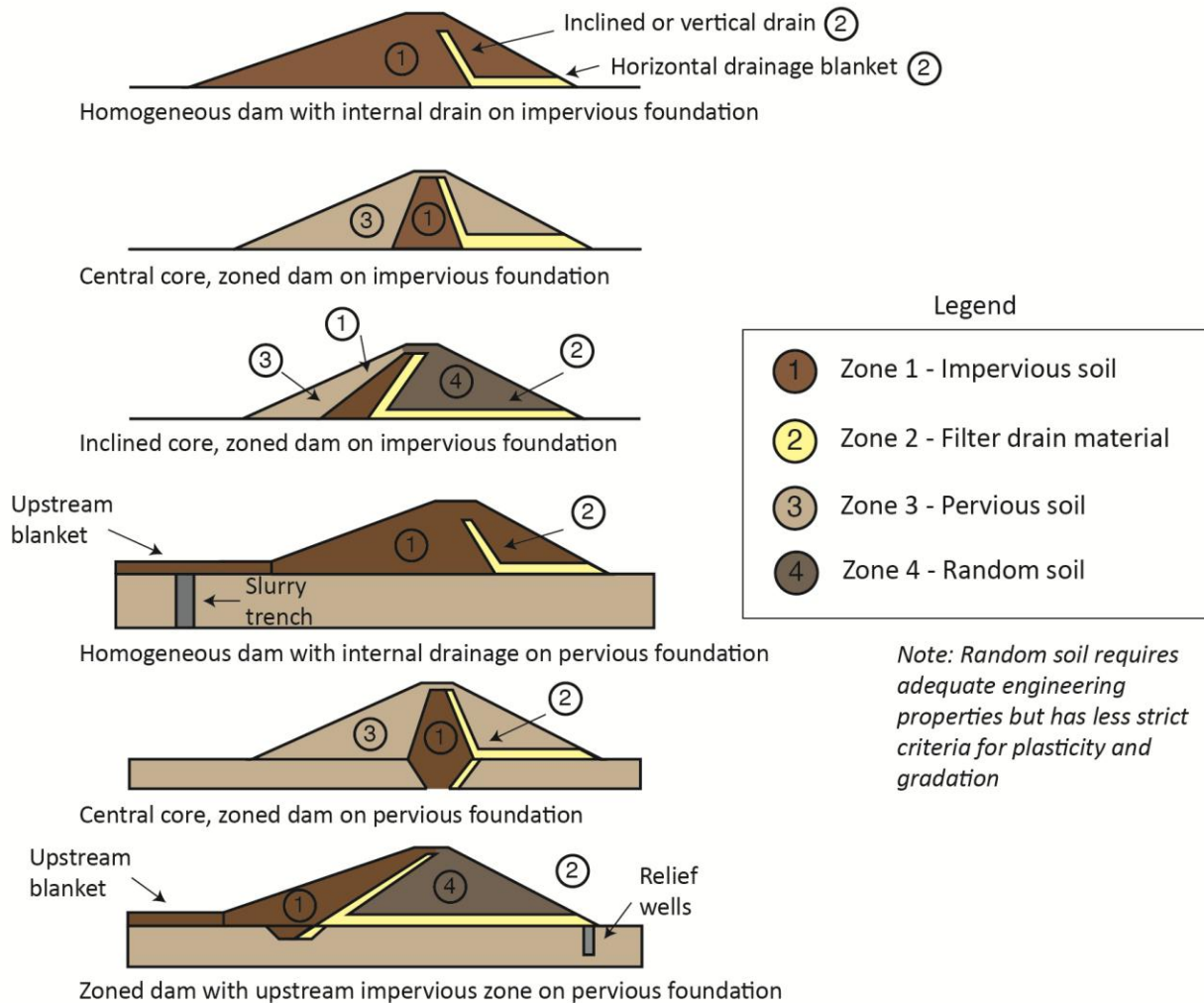


Fig. 3. Typical earthfill dam cross sections. Reproduced from [29].

Homogeneous sections of impervious material (i.e., clays) will necessarily be the most susceptible to desiccation cracking. Current USBR practice is not to construct a homogenous section if materials are subject to “moderate to severe” desiccation cracking. In zoned dams, construction detailing usually limits the height of the impervious core material to several feet below the crest, provides asphalt or concrete surfacing to ensure moisture and temperature stability of the most exposed portion of the core, and places material close to its optimum water content. When performing slope stability analyses, the manual recommends including potential tension cracks from desiccation in the model when the condition is possible. The standard practice is to ignore shear strength along these and consider the potential for cracks filling with water. The manual suggests estimating the maximum depth of cracking from a simple infinite slope formula not to exceed half the total slope height. The USBR design manual recognizes seepage through the upper portion of the embankment along desiccation cracks as a critical situation when performing seepage analyses. It also recognizes that desiccation cracks are worse in dry periods and typically occur above normal reservoir water surface, potentially causing large problems if water levels rise to a new high and seepage occurs along cracks.

However, the manual also states that “a method for evaluation or analysis of desiccation cracking is not obvious”. As detailed in other sections, the Bureau’s practice is always to provide filters and seepage control methods to intercept potential unanticipated flow (from desiccation cracks or other factors) rather than try to determine if it will occur or not based on a quantitative analysis. Typical impervious material construction specifications also require placement within -2% to +2% of optimum water content for dams less than 50 feet tall and within 0 to +2% for dams greater than 50 feet tall which will theoretically limit the potential for desiccation cracking. It is noted that the previously described USBR design guideline applies to new construction of federal dams. Many design procedures and construction details are based on lessons learned from historic failures. There are potentially existing dams built prior to these engineering advances that may have greater vulnerability to desiccation cracking.

3.3.2.4. Additional National Level Guidelines

The UFC DM 7.1 [54] provides the following design criteria for frost susceptibility, reproduced in Table 9.

Table 9. Frost susceptibility criteria for various soil types. Reproduced from [54].

Frost Group	Frost Susceptibility	Soil Type	Percent Finer than 0.02 mm	Typical USCS Classification
Not frost-susceptible (NFS)	Negligible to low	Gravels, crushed stone and rock	0-1.5	GW, GP
		Sands	0-3	SW, SP
Possibly frost-susceptible (PFS)	Possibly	Gravels, crushed stone and rock	1.5-3	GW, GP
		Sands	3-10	SW, SP
S1	Very low to medium	Gravelly soils	3-6	GW, GP, GW-GM, GP-GM
S2	Very low to medium	Sandy soils	3-6	SW, SP, SW-SM, SP-SM
F1	Very low to high	Gravelly soils	6-10	GM, GW-GM, GP-GM
F2	Medium to high	Gravelly soils	10-20	GM, GM-GC, GW-GM, GP-GM
		Sands	6-15	SM, SW-SM, SP-SM
F3	Medium to very high	Gravelly soils	>20	GM, GC
		Sands except very fine silty sands	>15	SM, SC
	Low	Clays, PI >12		CL, CH
F4	Low to very high	All silts		ML, MH
		Very fine silty sands	>15	SM
	Low to high	Clays, PI <12		CL, CL-ML
	Very low to very high	Varved clays and other fine-grained banded sediments		CL and ML; CL, ML, and SM; CL, CH, and ML; CL, CH, ML and SM.

The manual notes that loss of support from thaw can be even more detrimental than the effects of frost heave alone. The UFC DM 7.2 [53] requires that shallow foundations be placed below the depth of frost penetration. The manual suggests using data from NOAA [78] to determine local depths. These are based on 100-year return period air freezing index values [79].

UFC 3-130-3 [80] provides a comprehensive design procedure for foundations (shallow and deep) for arctic and subarctic foundations subject to freezing and thawing conditions. The following paragraphs provide an overview of specific aspects of these procedures that interact with future hazards and conditions. Further details regarding the specifics of the geotechnical and structural design procedures can be found in the original document.

A key consideration for such design is the depth of seasonal frost penetration and ground temperature fluctuations over the design life of the project. The primary recommended method of analysis to determine these is numerical modeling of the heat transfer process between soil, water, and air. The manual notes that this is a complicated topic requiring expertise in geotechnical site condition evaluation, thermodynamic principles, and climate science. Further background can be found in specific cold-region engineering textbooks such as [81] or [82].

Two key components of a thermal numerical analysis that require specific geotechnical input are material properties and geometry. Specifically, the geometry of the problem requires establishing the initial spatial distribution of frozen and unfrozen ground/water via a thorough geotechnical site exploration. The manual provides recommendations for establishing the geotechnical properties required as inputs, reproduced in Table 10.

Table 10. Some common geotechnical inputs to thermal modeling. Reproduced from [80].

Material Property	Typical Source ¹
Unit weight	Measured in soil samples obtained during site investigation, following ASTM D7263
Moisture content	Measured in soil samples obtained during site investigation, following ASTM D2216
Heat capacity (frozen/unfrozen)	Typically estimated from empirical relationships in [81]
Thermal conductivity (frozen/unfrozen)	Measured in soil samples obtained during site investigation following ASTM D5334 or estimated using empirical relationships in [81]
Unfrozen water content	Estimated from empirical relationships in [81]
¹ Empirical correlations will depend on soil unit weight, water/ice content, temperature, and salinity	

A third key component of thermal numerical modeling that requires less geotechnical input (but is no less critical to an accurate model) is boundary conditions. These represent assumptions about heat transfer between the ground surface and air and between the soil/water and any subsurface heating/cooling systems. The former must take into account projected future conditions (e.g., air temperature and precipitation) over the design life. The manual suggests using the Scenarios Network for Alaska + Arctic Planning Tool [83] to obtain

simplified projections of future climate data (e.g., air temperature or precipitation) when the project scope or resources do not allow for a full analysis of global climate models. Finally, the manual recommends direct measurements of ground temperature profiles for calibrating these numerical models.

The manual recommends designing shallow foundations based on expected deepest thaw and warmest ground temperatures (per the thermal modeling approach described above) using typical bearing capacity and settlement analyses that use soil properties consistent with this worst-case thermal regime. Thaw strain and creep potential can be estimated using modified oedometer tests on initially frozen samples with a thawing load step. The manual notes that there is no universal standard for measuring the potential for frost heave, but similar approaches involving a freezing step could be applied. When necessary, some suggested adaptation and ground improvement strategies for shallow foundations include:

- Bearing on a ventilated or refrigerated granular pad
- Maintaining thawed conditions by ground insulation and/or heating
- Maintaining frozen conditions by ground insulation and/or cooling
- Structural Isolation from frost heave forces or design for additional uplift forces
- Providing protective drainage away from the structure and foundation

Similar to shallow foundations, the manual recommends designing deep foundations based on worst case thermal conditions from numerical modeling. However, specific to deep foundations are considerations for interface friction between the foundation element and the surrounding ground. The manual suggests many considerations for both installation and operation, including the use of special slurries to construct drilled shafts and/or incorporation of ground cooling/insulation into the piles themselves to maintain the integrity of this adfreeze bond. Additionally, deep foundations must resist downdrag loads from thawing overlying soil (using similar thaw strain potential estimates.)

3.3.2.5. Notable State and Local Amendments

Alaska. The City of Anchorage includes several permafrost considerations in its amendments to the IBC [75]. These require that:

- The design of lateral soil pressure must incorporate the effects of seasonal frost penetration,
- Subsurface investigations evaluate if permafrost exists at the site within areas mapped as having a high potential for isolated permafrost conditions
- The geotechnical report should address mitigation of the effects of seasonal thawing/permafrost

The City of Fairbanks amendments [84] also require that footings are supported by unfrozen and non-frost-susceptible soil.

The Alaska Department of Transportation (AKDOT) Bridges and Structures Manual [85] states that the geotechnical investigation should identify the presence of permafrost. In permafrost areas, it is recommended that:

- Deep foundations should be isolated from movements in the active layer (to prevent uplift or draft forces)
- Seismic design should consider both thawed and frozen conditions

The manual suggests that frozen soil properties for layers with high moisture contents or below the water table can be used when analyzing dynamic load cases in areas where temperatures could remain below freezing continuously for more than two weeks. The AKDOT Geotechnical Manual [86] recommends acquiring climate data for the geotechnical report, including the range of temperatures in the vicinity of the project, seasonal extremes, design freezing/thawing indices, and the date of “general” onset of freezing temperatures.

The report “Roads and Airfields Constructed on Permafrost, A Synthesis of Practice,” was prepared for the AKDOT by the University of Alaska Institute of Northern Engineering [87] provides a recent summary of best practices (not necessarily detailed design procedures) for transportation infrastructure in permafrost regions. Although it does not constitute official design guidelines (it even states that “current knowledge does not lend itself to a manual or guideline”), some of its main conclusions are summarized here. Permafrost in Alaska is broadly categorized as continuous, discontinuous, and sporadic. Fig. 4 shows typical geomorphic characteristics associated with each.

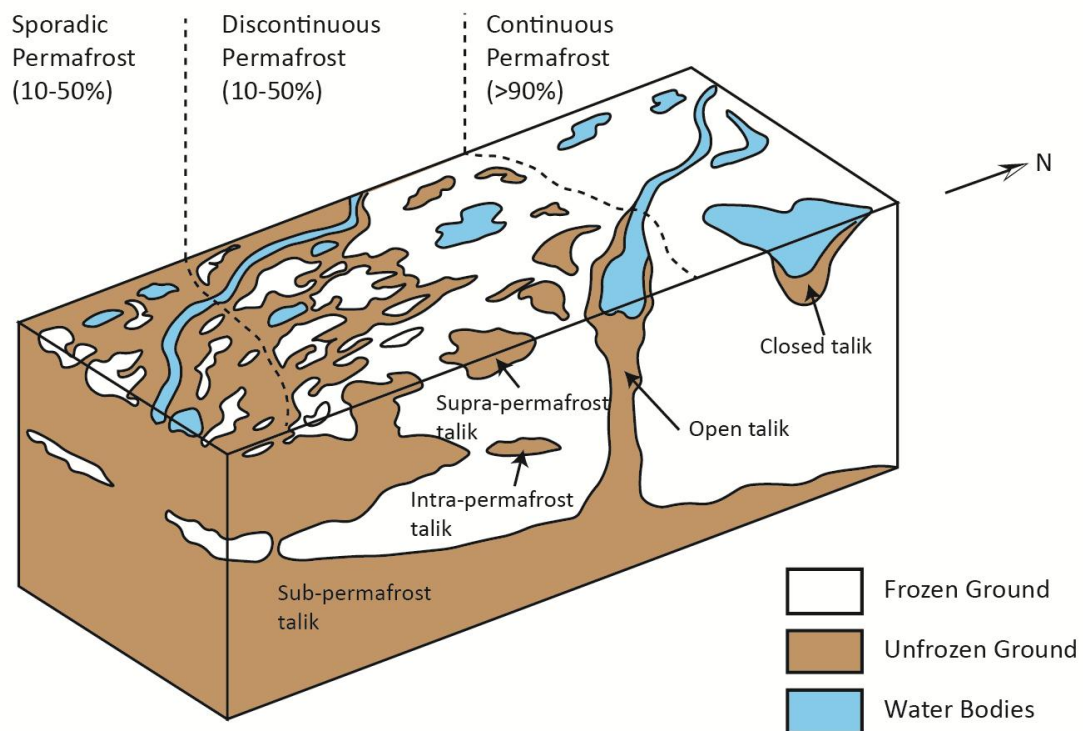


Fig. 4. Typical geomorphic features of sporadic, discontinuous, and continuous permafrost areas. Reproduced from [87].

Regarding design, the primary tasks are to characterize the extent of the active layer and the depths/consequences of seasonal or future thawing. The former can be established from equations that combine a thawing index derived from published climatic data (e.g., the Arctic-ERDS published by the University of Alaska Fairbanks [88]) and site soil properties. However, evaluation of thaw potential requires thermal modeling (which can range from simple analytical solutions to sophisticated three-dimensional finite element models). Groundwater/air flow can substantially affect permafrost behavior/thawing, and the report suggests engineers must use judgment on whether or not to incorporate it into the model. If fluid flow is incorporated, then additional properties such as permeability (which will be substantially different between frozen and thawed soil) are needed. Regardless of the solution method employed, modeling will require thermal properties of the soil and ice/water, appropriate boundary conditions, and an initial temperature distribution. Thermal properties of soil are typically established from correlations with index properties (soil classification, moisture content, mineralogy, and porosity). The initial temperature distribution can usually be measured during site investigations. Appropriate boundary conditions at the ground surface are the most complicated, particularly if one wants to incorporate future climate forecasts. With results from thermal modeling, the engineer can estimate the extent of the active zone and potential thaw settlement, usually by simple phase relationships or laboratory thaw strain tests.

The report summarizes five typical engineering strategies for dealing with permafrost:

- Avoiding construction in areas of permafrost (the preferred strategy of AKDOT)
- Keeping permafrost frozen
- Intentionally thawing permafrost during construction
- Removing and replacing frozen soil
- Accepting the consequences of thaw

However, the report notes that methods that preserve existing permafrost (insulation, heat drains, air convection, thermosyphons) should be carefully evaluated to ensure they can keep the permafrost frozen over the lifetime of the structure, particularly considering the effects of future weather conditions.

California. Blake et al. [89] recommends that tension cracks should be considered in stability analyses, filled with water “in some cases” – based on observation conditions preceding failure in similar geologic conditions. They recommend that the crack should be located on the slope such that it produces the lowest factor of safety.

Illinois. The IDOT Geotechnical Manual [71] provides the following criteria (reproduced in Table 11) for soil frost heave susceptibility:

Table 11. Qualitative frost susceptibility categories used by the IDOT.

Frost Susceptibility	Type of Soil	Percent finer than 0.02 mm
Negligible to Low	Gravelly Soils	3 - 10

Low to Medium	Gravelly Soils	10 - 20
	Sands	3 - 15
High	Gravelly Soils	>20
	Sands, except very fine silty sands	>15
	Clays, PI > 12	–
Very High	All silts	–
	Very fine silty sands	> 15
	Clays, PI < 12	–

For frost heave to affect pavements/structures, the capillary rise height of moisture above the groundwater table must be within the depth of frost penetration. The manual states reliable data is not available and gives general maximum depth ranges for various geographic areas of the state. The manual notes that even lower susceptibility soils can experience frost heaving without proper drainage. Common treatments are to lower the groundwater table with drainage, remove/replace susceptible soils, or raise the grade of the road.

3.3.3. Gaps in Practice and Standards

Much like expansive/collapsible soils, many accepted laboratory and field test methods exist for determining the thermal properties and thaw strain potential for frozen soils. However, there are also similar concerns regarding the representativeness of future conditions, particularly concerning future conditions. On the other hand, much less quantitative guideline is provided for predicting desiccation cracking and its effects. The following list describes some specific gaps identified in the state of practice:

- The building code and related guidelines do not give specific quantitative design procedures as to how to evaluate the permanent or transient nature of frozen ground.
- Heat transfer modeling, such as described in the practice synthesis report, can be used in conjunction with geotechnical data and laboratory testing to estimate the behavior of frozen ground and its potential for thaw. These models require careful consideration of boundary conditions, which can potentially be derived from future projected conditions. However, there is currently no uniformly accepted procedure for doing so (though the references suggest several possibilities).
- The spatial variability in ice content can present challenges for site characterization and modeling, particularly for distributed infrastructure.
- In some cases, a simple homogenous soil model can estimate the depth of tension cracking in earthen infrastructure. However, it does not incorporate climatic variables, and its applicability is unclear for heterogeneous slopes.

- Predicting the location and magnitude of cracking is done qualitatively. Instead of a detailed analysis, usually, some degree of cracking is expected to occur and is addressed with construction detailing and various protective measures to preemptively intercept any adverse seepage that may occur.
- However, many of these best practices are primarily based on experience with failures and near-failures. It is unclear if the current best practices guideline/construction detailing will remain sufficient for future hazards and conditions.
- No guideline is provided regarding long-term desiccation risks linked to projected droughts.
- Monitoring solutions provided for desiccation cracking, such as monitoring surface cracking in embankments, do not incorporate future hazards and conditions.

3.4. Corrosion

3.4.1. Overview and Hazard Considerations

Corrosion refers to the degradation of a metal by electrochemical processes. Most corrosion in civil engineering applications occurs because of contact with soluble salts found in soil, water, or air. From a geotechnical perspective, this typically manifests as deterioration and capacity loss of steel foundation elements (including rebars in reinforced concrete). Sulfate attack, or deterioration of cement via a chemical reaction with sulfates in soil/water, can similarly cause deterioration of concrete structures and is usually included in a geotechnical corrosivity evaluation. A great deal of research and design guideline has been published for consideration of corrosion in structural design, material specification, and construction detailing (e.g. [90] or [91]), which is not recapitulated here. Instead, this work focuses on how the corrosion potential of soil and groundwater is incorporated into the geotechnical design guideline.

The rate and amount of corrosion are primarily influenced by the moisture content of the soil (or exposure to surface water), the presence of soluble salts, soil/water pH, and oxygen concentration. Corrosion is thus primarily affected by climate conditions that result in more variable saturation, such as changes in precipitation patterns and wet/dry cycles. Corrosion is also doubly affected by sea level changes and saltwater intrusion, which will increase both moisture content and soluble salts.

3.4.2. Design Guideline

3.4.2.1. IBC and ASCE 7

Section 1810 of the IBC [51] requires that deep foundations are protected from “deleterious” effects from soils and changing water levels using “adequate materials, methods, or processes”. Additionally, many of the specifications for metallic structural components require corrosion-

resistant materials. Reinforced concrete has a minimum clear concrete cover over rebar to mitigate the effects of corrosion and sulfate attack.

3.4.2.2. AASHTO BDS and FHWA GEC 5

The AASHTO BDS [27], as a general design objective, has several provisions for the protection of structural materials against corrosion and degradation, including coatings, sacrificial area, concrete cover, and cathodic protection, with special consideration given to materials in contact with water and/or soil. The design of earth-retaining structures with metallic elements (e.g., sheet piling, anchored walls, or mechanically stabilized earth walls) and deep foundations requires consideration of corrosion and deterioration. In particular, deep foundation design requires addressing the potential for corrosion of steel, sulfate/chloride attack of concrete, and decay of timber piles (from wetting/drying cycles or insects). The specifications give the following general criteria (reproduced in Table 12) for conditions that indicate a corrosive environment:

Table 12. Criteria used in the AASHTO BDS to establish if a site is considered corrosive.

Soil	Water
Resistivity < 2,000 ohm-cm	Chloride > 500 ppm
pH < 5.5 or 5.5 < pH < 8.5 in organic soils	Sulfate > 500 ppm
Sulfate > 1000 ppm	pH < 5.5
	Wet/dry cycles

The document also gives specific criteria for when certain deleterious environments, such as industrial wastes, landfills, the presence of insects/marine borers, or highly alkaline soils. Details on specific design methods for coatings, sacrificial area, galvanic protection, etc., are given in the BDS chapters on structural design. The manual recommends the following corrosion rates (reproduced in Table 13) to estimate section losses and design sacrificial steel amounts:

Table 13. Corrosion rate assumptions used in design per the AASHTO BDS.

Environment	Corrosion Rate
Soil embedded zone	0.001 in/yr
Fill or disturbed natural soils	0.0015 in/yr
Marine atmospheric zone	0.002 in/yr
Marine immersion zone	0.004 in/yr
Marine splash zone	0.006 in/yr

The recommended test methods for determining soil/water pH, soluble salt content, and resistivity are discussed in the FHWA GEC 5, following. If industrial waste is present, a “full chemical analysis” of groundwater and soil is recommended.

The FHWA GEC 5 [26] notes that the highest corrosion rates are observed above the groundwater table and within the zone of moisture fluctuation (where both water and oxygen are present in abundance). It also notes that corrosion potential can be established by testing soil and groundwater properties, but the resources or scope of most projects may not allow for direct measurement of corrosion rates. A common test is electrical resistivity, which can be measured in the laboratory on remolded specimens in a soil box, or directly measured in the field (though one is naturally limited to near-surface soils).

The two primary standards, AASHTO T288 and ASTM G187, have one notable difference – the ASTM standard testing is done on a single saturated specimen, while the AASHTO standard testing is done over a range of water contents, and the minimum resistivity value (highest corrosion potential) is reported. Tests for pH and soluble salts (sulfates and chlorides) are typically also performed. The combined results of all three are usually integrated into a qualitative rating of corrosion potential. Many versions of these exist, one of which is shown in Table 14.

Table 14. An example qualitative soil corrosivity ranking system. Reproduced from [26].

Soil Parameter	Value	Score
Resistivity (Ω -cm)	< 500	10
	500 to 999	8
	1000 to 1999	5
	2000 to 4999	2
	5000 to 10000	1
	> 10000	0
pH	2 to 4.5	6
	5 to 6	3
	7 to 9	6
	10.5 to 12	6
Moisture	Tidal or Salt Water	10
	Poor Drainage - Always Wet	2
	Fair Drainage - Moist	1
	Good Drainage – Usually Dry	0

Total Corrosivity Score	Soil Corrosion Potential
0 to 2	Unlikely
3 to 4	Slight
5 to 6	Mild
7 to 8	Moderate
9 to 13	Aggressive
14 to 20	Severe

The FHWA report “Best Practices for Corrosion Control and Mitigation” [92] provides references to additional design methodologies for corrosion mitigation options beyond sacrificial steel area (e.g., corrosion-resistant alloys, coatings, or cathodic protection systems).

3.4.2.3. Additional National Level Guidelines

The UFC DM 7.2 [53] notes that corrosion rate is difficult to forecast because of spatial and temporal variability in localized electrochemical conditions. It requires that the concrete for drilled shafts must be “durable” in the groundwater chemistry of placed environment. It provides references to the previously reviewed FHWA and AASHTO documents for determining corrosion potential and deterioration rates. The National Academies Committee on the Corrosion of Buried Steel at New and In-Service Infrastructure recently published a comprehensive review report [93]. While not directly referenced by design guideline documents reviewed in this work, the report provides additional technical discussions and recommendations for practice.

3.4.2.4. Notable State and Local Amendments

California. The Caltrans Geotechnical Manual [94] uses a slightly different definition of a corrosive environment, where sites are initially screened based on resistivity and tested for soluble salts if the resistivity values are low enough. The department considers sites corrosive if chlorides exceed 500 ppm, sulfates exceed 1500 ppm, or the pH is less than 5.5. The manual also mentions that the corrosion evaluation should consider the effects of seasonal and long-term changes to groundwater conditions including sea level changes. The Caltrans Corrosion Guidelines [95] use the AASHTO BDS suggested corrosion rates.

Florida. The Florida Department of Transportation (FDOT) Structures Manual [96] provides the following table (reproduced in Table 15) to evaluate site corrosivity potential.

Table 15. Site corrosivity classification criteria per the FDOT Structures Manual [96].

Classification	Condition	Units	Steel	Steel	Concrete	Concrete
			Water	Soil	Water	Soil
Extremely aggressive (if any of these conditions exist)	pH		< 6.0		< 5.0	
	Chlorides	ppm	> 2000		> 2000	
	Sulfates	ppm	N.A.		> 1500	> 2000
	Resistivity	Ohm-cm	< 1000		< 500	
Slightly aggressive (if all of these conditions exist)	pH		> 7.0		> 6.0	
	Chlorides	ppm	< 500		< 500	
	Sulfates	ppm	N.A.		< 150	< 1000
	Resistivity	Ohm-cm	> 5000		> 3000	
Moderately aggressive	This classification must be used at all sites not meeting requirements for either slightly aggressive or extremely aggressive environments.					

The guidelines suggest the following minimum sacrificial steel amounts and usage limitations (reproduced in Table 16) based on site corrosivity evaluations

Table 16. Corrosion design requirements per the FDOT Structures Manual [96].

Steel Component	Embedment	Minimum Required Sacrificial Thickness (inches) and Usage Limitations Based on Substructure Classification and Pile/Wall Anchor Bar Location			
		Slightly aggressive	Moderately aggressive	Extremely aggressive	Extremely aggressive
		Land and/or water	Land and/or water	Land	Water
Pipe and H-Piles	Completely buried	0.075	0.15	0.225	Use internally redundant pipe piles only
Pipe and H-Piles	Partially buried	0.09	0.18	0.27	N/A
		0.15	0.30	Use internally redundant pipe piles	
Anchored or cantilever sheet piles	All	0.045	0.09	0.135	
Wall anchor bars	All	0.09	0.18	0.27	

Illinois. The City of Chicago’s Building Code [70] requires that the geotechnical report address the corrosivity of on-site soils. They recommend that the pile design ignores 0.8 mm of steel to account for potential losses unless geotechnical testing can establish if the soil is non-corrosive. “Natural” soils below the water table are assumed to be non-corrosive.

Texas. The TXDOT Geotechnical Manual [69] requires that steel piling be coated (to mitigate corrosion) to at least 15 feet below the maximum scour depth. The Hydraulic Design Manual [97] recommends that saltwater intrusion should be considered in the corrosion evaluation.

3.4.3. Gaps in Practice and Standards

Geotechnical corrosion evaluation is a relatively standardized procedure. Most exploration programs will perform some form of resistivity and soluble salt testing on samples obtained. However, there are some gaps in the design guidelines highlighted in the list below:

- Several documents recommend incorporating sea level changes and saltwater intrusion in site corrosivity evaluations. However, they do not provide explicit guidelines on how to do so.
- The current standard test suite (resistivity and soluble salts) does not directly measure corrosion rates and instead is typically correlated to qualitative rankings of corrosion potential. It is unclear if the current corrosion rates assumed for various environments will accurately reflect future deterioration trends, considering climatic variability

- Even when corrosion rates can be directly measured or extrapolated from historical data, there remains difficulty in incorporating future climate conditions into the model.
- Guidelines for addressing the effects of corrosive soils are typically limited to standard corrosion mitigation strategies and do not specifically address future hazards and conditions or incorporate adaptive designs. In particular, methods to evaluate soil chemistry alterations from sea level changes and saltwater intrusion are lacking.

3.5. Seismic Soil Liquefaction

3.5.1. Overview and Hazard Considerations

Like many of the other geotechnical hazards discussed in this report, seismic liquefaction is an extremely active ongoing research field. The recently published “State of the Art and Practice in the Assessment of Earthquake-Induced Soil Liquefaction and Its Consequences” [98] gives an overview of the assessment of seismic soil liquefaction triggering/manifestation and its consequences. This review focuses on aspects of the recommended design that specifically address climatic considerations rather than giving a complete technical overview of the phenomenon.

Seismic soil liquefaction is typically defined as when a relatively loose, saturated, “susceptible” soil (commonly taken as sands, low plasticity silts, and in some cases, gravels) experiences drastic strength and stiffness loss in response to pore pressure generation from earthquake loading. This can lead to settlement and bearing capacity failure of foundations, flotation of buried infrastructure, lateral spreading of the ground, and slope failures. Rising or more variable groundwater levels from changes in precipitation patterns and/or sea level changes that cause previously unsaturated, susceptible material to become saturated is the predominant mechanism by which the risk posed by liquefaction may be affected by future extreme weather. A second-order interaction can result from the current reliance on field cone penetration tests (CPT) or standard penetration test (SPT) data to evaluate soil “resistance” or, even more fundamentally, “susceptibility” to liquefaction. The measured values, which will, to some degree, be dependent on the saturation/effective stresses at the time of testing, may not necessarily be representative of future conditions.

3.5.2. Design Guidelines

3.5.2.1. IBC and ASCE 7

Section 1803 of the IBC [51] requires that the geotechnical investigation evaluate the potential for liquefaction and its consequences, including:

- Strength loss
- Total/differential settlement
- Lateral movement and increase in lateral loads

- Downdrag on deep foundations
- Buoyancy of underground structures

The geotechnical report is expected to include measures for mitigating the above effects. ASCE 7 [52] provides further guidelines primarily focused on establishing the level of ground motion for design. The commentary recommends typical semi-empirical design approaches, summarized in a 2009 National Earthquake Hazards Reduction Program resource paper for liquefaction evaluation [99]. The GeoTechTools database, developed by the ASCE Geo-Institute provides additional guidelines on type selection and design of relevant ground improvement strategies [65].

3.5.2.2. AASHTO BDS and FHWA GEC 5

Section 10.5 of the AASHTO BDS [27] requires conducting a liquefaction assessment for high seismicity areas and other zones when conditions warrant) if the “anticipated” groundwater level is within 50 feet or lower of the existing or final ground and if low plasticity silts and sands exist within the upper 75 feet with low enough penetration resistance (indicated by various field tests) that liquefaction may be possible. The manual recommends using typical simplified method design procedures (e.g., [100], [101], or [102]) up to 75 feet deep and more sophisticated numerical models below those depths. The steps for applying these models are summarized in the FHWA GEC 5 [26]. The specifications recommend designing for a “seasonally averaged groundwater elevation”. The specifications require designing for:

- Strength loss in liquefied layers
- Liquefaction-induced settlement, including downdrag loads on deep foundations
- Lateral spreading and slope stability

They suggest any combination of removal and replacement, densification, reinforcement, altering the soil composition, and enhanced drainage for mitigation but note that enhanced drainage alone is not considered adequately reliable. The FHWA provides additional guidelines on type selection and design of relevant ground improvement strategies in GEC 13 [66].

3.5.2.3. USBR DS13

Per the USBR DS13 [29], all dams are designed for seismic loading (including liquefaction) to avoid the catastrophic release of the impounded reservoir. The general procedure followed by the Bureau to quantify the risk a structure poses to human life by assessing the probability and consequences of dam failure from seismic loading. Specific failure modes considered include liquefaction and erosion through cracks following an earthquake (which may develop from liquefaction-related settlement or lateral spreading). Both have caused failure or near failure in the past.

Liquefaction analyses per the standards are typically done using the standard semi-empirical approaches with SPT or CPT field test data mentioned previously. However, they note that the case histories, which are almost exclusively from free-field, level ground sites, used to develop

the above models may not necessarily apply to embankment dam foundations. Usually, analyses are performed with groundwater conditions “that would exist just prior to the earthquake”, typically assumed as steady-state seepage with the reservoir at the top of active pool.

The suggested standard practice is to treat or remove all potentially liquefiable material for new dams rather than rely on analysis showing liquefaction consequences are acceptable.

3.5.2.4. Notable State and Local Amendments

California. The California Building Code [103] requires geotechnical reports to evaluate liquefaction and any mitigation measures required for stabilization. These are often required to be addressed in a specific geohazard report, which has more detailed requirements described in the CGS Note 48 [67] which provides a review checklist. To aid practitioners, the CGS publishes mapped zones of required investigation for liquefaction (and other seismic hazards), an example is shown in Fig. 5.

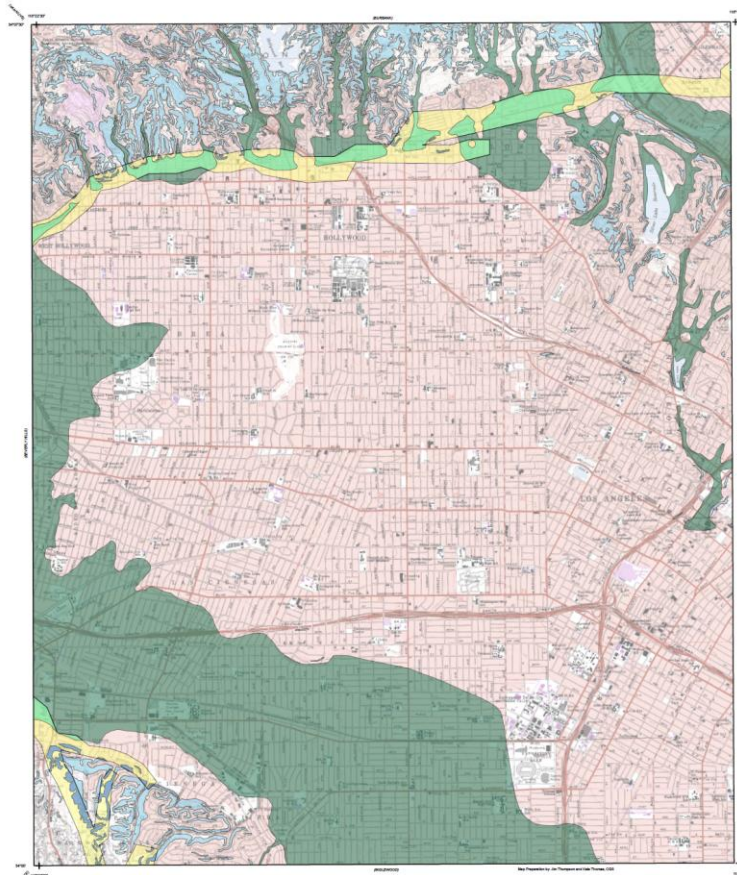


Fig. 5. An example CGS Seismic Hazard Zone map published. Liquefaction hazard zones are shown in green, landslide hazard zones are shown in blue, and surface fault rupture zones are shown in yellow. Reproduced from [104].

These zones are compiled from regional geologic information and limited site investigation data. They serve to delineate where more detailed investigation and reporting are necessary rather than giving a definitive estimate of the hazard. Not all of California is mapped, though usually, coastal, high-population regions are. Engineers are still required to address hazards even if the project is not located in a mapped zone. Further details regarding liquefaction evaluation procedures are provided in the CGS Special Publication 117A, “Guidelines for Evaluating and Mitigating Seismic Hazards in California” [28]. Sites are screened based on the existence of “potentially liquefiable” soils present that are saturated, were saturated previously, or might become saturated given “foreseeable changes” in groundwater conditions. The document requires that historically high groundwater levels be used for liquefaction analysis unless “other information indicates a higher or lower level is appropriate”. It is recommended that these be determined by reviewing available well logs, monitoring wells, and borehole data, and site-specific investigations. Often, engineers use values from the CGS Seismic Hazard Zonation reports, which map interpolated values on a 7.5’ scale based on publicly available data. The recommended analysis methods follow commonly used semi-empirical CPT or SPT-based procedures, like other documents reviewed. Foundations and structures are required to be designed for the usual consequences of liquefaction discussed previously, including settlement, reduction in bearing capacity, downdrag, and slope stability/lateral spreading. The guidelines suggest three general mitigation approaches:

- Avoid the hazard
- Reduce the hazard to an “acceptable” level
- Accommodate the hazard by strengthening the structure

They also note that only removing all potentially unstable soils or permanently lowering the groundwater table can eliminate liquefaction hazards. Some suggested mitigation strategies include over-excavation and recompaction, ground improvement (dynamic compaction, stone columns, soil mixing, etc.), or deep foundations.

The Caltrans Geotechnical Manual Liquefaction Module [105] gives similar design guidelines to the documents previously discussed. It recommends using the measured “stabilized” groundwater table for analysis and only using a higher elevation if there is “clear evidence” of seasonal or long-term fluctuations. The manual states that liquefaction should not be combined with other extreme events or conditions.

3.5.3. Gaps in Practice and Standards

As described at the beginning of this section, liquefaction evaluation is a very active area of research and a variety of tools ranging from simplified, case-history-based semi-empirical procedures to dynamic, nonlinear finite element analyses. As a result, the gaps in the state of practice are more related to limited guidelines for selecting model inputs rather than the limitations of the modeling approaches themselves. Furthermore:

- Liquefaction hazard analysis per ASCE 7 uses a 2475-year return period earthquake peak ground acceleration (i.e., a 2% probability of exceedance in a 50-year period). Other

documents have similar provisions in their seismic design criteria. However, it is unclear if combining these ground motion levels with standard semi-empirical liquefaction susceptibility, triggering, and manifestation models and site conditions consistent with various recommended groundwater levels, typically “seasonally averaged”, “historic high”, or “stabilized”, will result in the expected level of structural reliability.

- In areas with rising groundwater levels due to sea level changes or precipitation patterns, relying on historic groundwater records may produce a non-conservative assessment of the liquefaction risk and mitigation design. Conversely, in areas with groundwater depletion, maximum historical levels could produce an overly conservative design. The reviewed documents include clauses that would allow for forecasting future groundwater levels for design rather than relying on past data, but they do not provide a quantitative procedure for doing so.
- Typically, semi-empirical liquefaction design procedures rely on field test data, typically from SPT or CPT (or both). A potential issue arises when these are performed in unsaturated material, which would then be assumed to become saturated for design conditions. Theoretically, unsaturated soils are stronger and stiffer than saturated soils, and the field values would need to be adjusted downwards for design. On the other hand, the semi-empirical design procedures already involve effective stress normalization, which could make this concern a non-issue. However, the documents reviewed do not comment on this potential concern.
- When considering future conditions, seasonal variability, and limited site measurements, groundwater levels have considerable uncertainty. Applying typical volumetric settlement models (e.g., [106]) can sometimes result in large changes in estimated settlement from fluctuations in the groundwater table when relatively loose, susceptible layers exist within the zone of potential variation. Aside from the optional reliability-based criteria in ASCE 7, the documents reviewed do not provide a quantitative procedure for incorporating this potential source of uncertainty into the design.
- There are no guidelines (or prior studies) on the combined influence of temperature variations and rainfall on liquefaction susceptibility and consequence. There are also no guidelines or prior research studies on the simultaneous influence of dynamic excitation, partial saturation, and temperature on the response of freezing-thawing soils and their potential for liquefaction during seismic loading.
- Guidelines for addressing the effects of seismic soil liquefaction are typically limited to standard ground improvement strategies and do not specifically address future hazards/conditions or incorporate adaptive designs

3.6. Scour and Erosion

3.6.1. Overview and Hazard Considerations

Scour and erosion can result in undermining of shallow foundations, loss of lateral support for deep foundations, and loss of material providing stabilizing forces for slopes. Both scour and internal erosion are recognized as critical design cases that have contributed to notable past failures in bridges and embankment dams by their respective design standards. The AASHTO BDS succinctly summarizes scour and erosion as “complex physical processes involving the actions and interactions of water and soil during the occurrence of predicted ... flows.” This review focuses on the geotechnical inputs to the overall scour design procedure rather than trying to give a technical overview of the hydraulic/hydrologic components. For interested readers, the FHWA Hydraulic Engineering Circular (HEC) series (e.g., [107]) provides a good practical introduction to these topics. This review primarily discusses riverine scour (as that is the primary focus of most transportation design standards), but does briefly discuss lacustrine and tidal scour as appropriate. Scour and erosion, given their dependence on water flow, will naturally be heavily affected by future hazards that alter flood magnitudes and frequency, such as extreme precipitation patterns and sea level changes.

3.6.2. Design Guidelines

3.6.2.1. IBC and ASCE 7

The IBC [51] only discusses erosion in several sections, but in limited ways. The provisions of section 1807.7 require that structures be set back a minimum distance from slopes to provide protection from slope failures and erosion. The code minimums are shown in the section discussing slope stability as a geotechnical hazard; however, alternative setbacks may be established per the recommendations of the geotechnical report, which should consider the “erosion characteristics” of the slope material. There is also a general requirement that if shallow soils are moving or unstable, foundations must be embedded to a “sufficient depth”, which could be read to include erosion potential. The provisions of section 1804.5 prohibit grading/filling in flood hazard areas unless the fill can be placed to minimize erosion during flooding (or wave action). Appendix G on flood-resistant construction (which is not a mandatory part of the code unless specifically adopted) requires consideration of the potential for erosion damage when deciding to approve permits for construction in flood hazard areas. Finally, Appendix J on grading requires that cut and fill slopes constructed with facing details to minimize erosion, which are allowed to be “effective planting”.

Section 5.3 of ASCE 7 [52] requires that the calculation of loads in flood hazard areas include the potential effects of scour or erosion. The accompanying commentary notes that a common mitigation technique is to increase foundation embedment depth and site buildings away from receding shorelines. The document also discusses erosion/scour in the context of tsunami design, which is discussed in the corresponding geotechnical hazard section.

3.6.2.2. AASHTO BDS and FHWA HEC 18

Per FHWA HEC 18 [107], total scour for a given flood scenario consists of three components:

- Long-term degradation scour caused by sediment transport over relatively large areas
- Contraction scour caused by material removal resulting in increased flow near the bridge/structure due to the narrowing of the channel
- Local scour caused by acceleration of flow and vortices around structure elements such as piers, abutments, or embankments

Other sources of scour may include the flow concentration near a bend or lateral migration of the streambed. The general design procedures involve calculating flow velocities for a design flood (or floods) and then determining the local and contraction scour. The inputs required from the geotechnical engineer typically are:

- Subsurface profile within bridge reach
- Type of streambed material (soil or rock)
- Classification, index properties, and particle size distribution for soils
- Rock mass rating (used to compute erodibility index)

Some general erodibility/scour rate relationships are shown in Fig. 6 and Fig. 7.

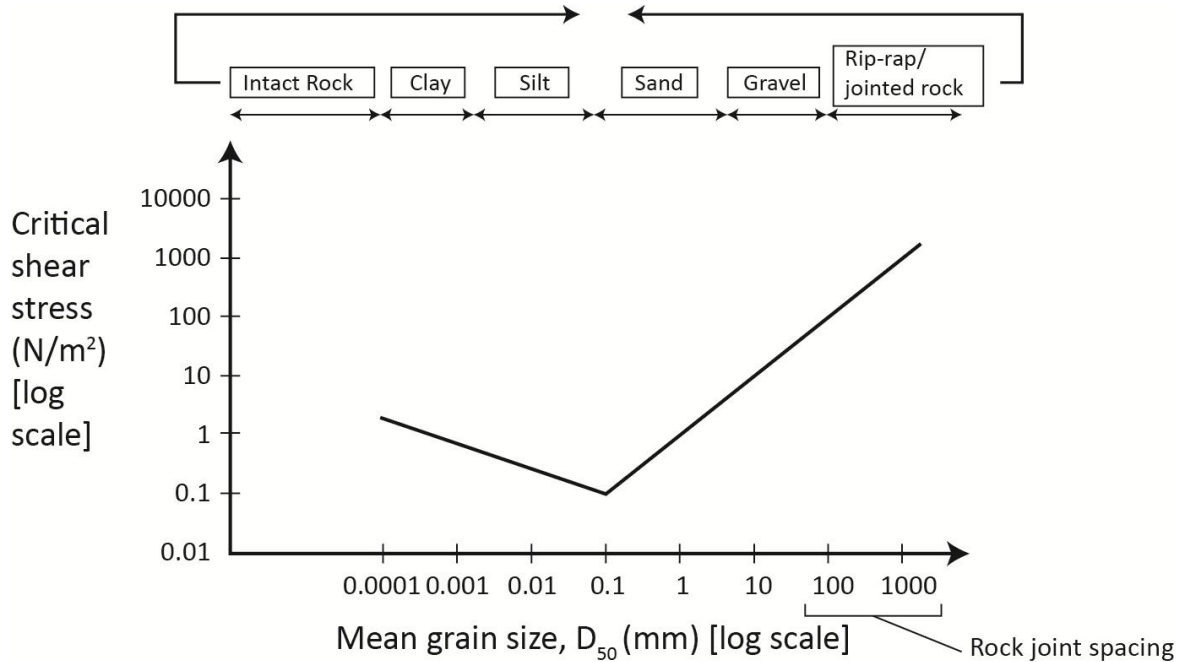


Fig. 6. A design chart used in scour calculations illustrates the required geotechnical inputs of soil classification and median grain size. Reproduced from [107].

The design equations for scour depth in soil are separated into cohesionless soils, cohesive soils, and rock. This distinction is made primarily based on the mechanics of scour for each: in cohesionless soils, scour is assumed to occur instantaneously once a threshold velocity for particle transport is exceeded, whereas cohesive soils exhibit a much slower rate of erosion increase beyond an initiation threshold and may be treated in a time-dependent manner. The manual also gives methods for estimating the long-term degradation scour from stream gauge data or sediment transport modeling.

The FHWA GEC 5 [26] describes methods for identifying dispersive clays, which have a chemical structure that causes repulsive forces between the clay particles to exceed the attractive forces when exposed to water, causing a very high susceptibility to erosion and piping. These are commonly assessed through the pinhole dispersion test (ASTM D4647) in which water is allowed to flow through a small hole in a prepared specimen and the soil is classified as dispersive, moderately to slightly dispersive, or nondispersive based on the flow rate, turbidity of discharge, and enlargement of the hole in the soil sample after a given elapsed time.

The design equations for scour depth in rock are separated into the predominant mode of scour: erosion by steady abrasion of the rock and erosion by removal and transport of rock blocks. Abrasion-related scour is computed using results from a modified slake durability test, which involves tumbling submerged rock specimens and measuring their mass loss. The second mode of scour is computed using the erodibility index and hydraulic inputs. The erodibility index for rock depends on the intact rock strength, the rock quality designation (related to the fracture spacing obtained from core samples), and the properties of joints/discontinuities. Section 2.6 of the AASHTO BDS requires the assumption of removal of material above the total scour line when evaluating foundation stability (there are some nuances with specific

components of a few load combinations). However, the flood used to

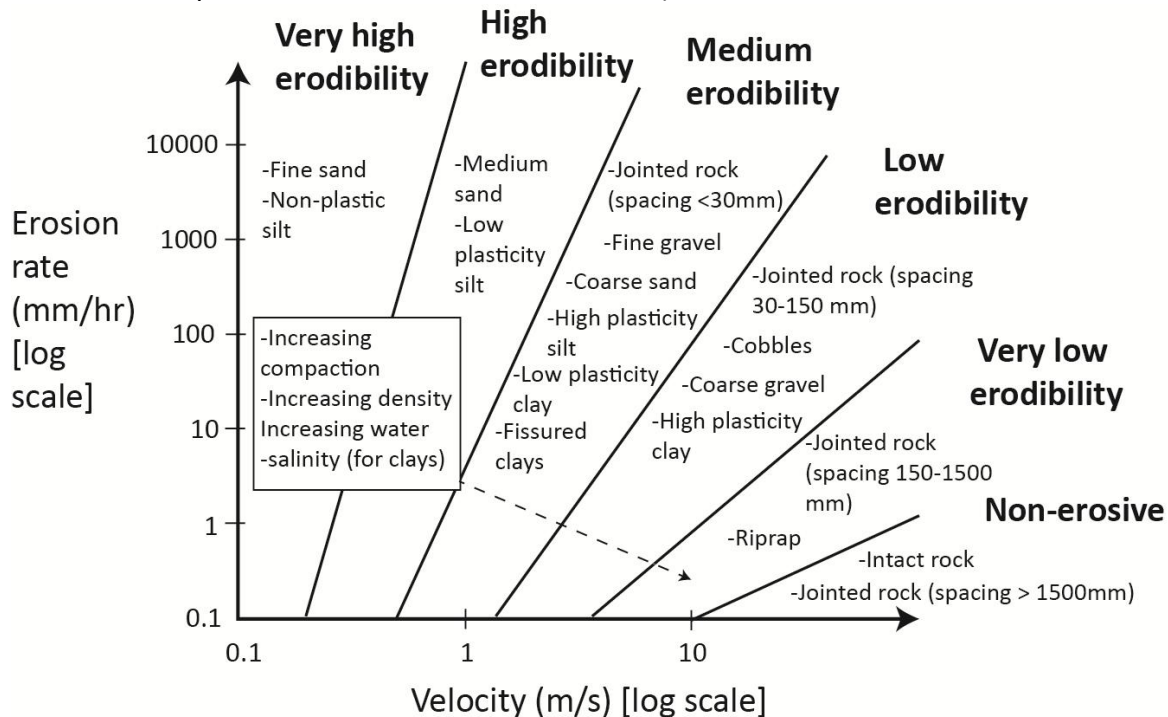


Fig. 7. A design chart used in erosion rate calculations illustrates the required geotechnical input of soil classification. Reproduced from [107].

determine this depth depends on the limit state under consideration. The strength and service limit states use a design flood “storm surge, tide, or mixed population flood ... [that is] the more severe of the 100-year events or an overtopping flood of lesser recurrence interval.” The extreme event limit state uses a check flood consisting of a “flood storm surge, tide, or mixed population flood not to exceed the 500-year event or from an overtopping flood of lesser recurrence interval.” A result of these requirements is that shallow foundations must be located to bear below a maximum anticipated depth of scour, erosion, or undermining, which results in the use of deep foundations at bridges. Deep foundation elements are then designed to have sufficient lateral and axial capacity with the appropriate total scour depth of soil removed.

Appropriate adaptation/mitigation measures can limit the potential for future erosion/undermining of geotechnical structures and foundations. HEC-23 [108] provides a comprehensive overview of the available scour countermeasures, considerations for which one(s) are appropriate for a given project, and their design procedures. This section reviews the relevant geotechnical inputs and associated considerations for future conditions/hazards presented in said manual.

Riprap, typically consisting of a layer of material placed on a vulnerable surface and potentially grouted, is a very common armoring solution. Typical design equations found in HEC-23 provide a required median stone size, gradation, and layer thickness/number based on hydraulic inputs (e.g., flow velocity, depth, or Froude number). The design equations are modified if overtopping flow is expected. Sometimes the design may also require a geosynthetic or granular filter to

prevent erosion of the underlayer through the gaps in the riprap. Other forms of flexible revetments, such as rock-and-wire mattresses, gabion mattresses, or articulating concrete blocks, are designed similarly, i.e., balancing the hydraulic and other destabilizing forces against the weight/interlocking resistance of the revetment material. Design of rigid revetments, such as grout-filled mats, often requires that the hydraulic flow stresses do not exceed the interface friction between the vulnerable soil and the countermeasure structure. Complete slope protection by concrete facing can also be provided, but the manual notes that toe erosion can be a concern.

Countermeasures can also include nature-based solutions, which are referred to as biotechnical countermeasures in HEC-23. These involve using vegetation to protect exposed slopes from scour and erosion, often in combination with some structural scaffolding. The manual notes, however, that quantitative design procedures for these countermeasures not available – specifically, how much slope reinforcement can be provided for various types of vegetation. The manual also notes that these solutions often require some form of traditional toe stabilization and that if “failure of the countermeasure could lead to failure of a bridge or highway structure, the only acceptable solution in the immediate vicinity of a structure is a traditional, ‘hard’ engineering approach.”

Infrastructure in the coastal environment is highly vulnerable to the effects of future hazards such as relative sea level changes and increased flooding. HEC-25 [109] provides a comprehensive discussion of the methods of analysis, mitigation/adaptation solutions available to engineers. Geotechnical input is required for coastal engineering approaches and the design of protective geostructures.

Coastal engineering is a specialty area of civil engineering that combines aspects of fluid dynamics, climate science, and geomorphology. A brief overview of the field’s technical background is provided in HEC-25. Models to estimate coastal erosion rates and spatial distribution can range from simple empirical equations based on past data to complex coupled sediment transport-hydrodynamic numerical models. It should be noted that currently, geotechnical input to these models is typically limited to a soil classification and particle size distribution of the material to be studied. These numerical models can also allow for forecasting future trends based on projected future conditions. The results of the previously mentioned methods of analysis can be used to study and design nature-based solutions (e.g., constructed beaches, dunes, or coastal marshes – sometimes referred to as “living shorelines”). Webb et al. [110] gives an in-depth discussion of nature-based solutions to improve the resilience of coastal infrastructure.

Geostructures (i.e., engineering-based solutions) are an alternative to nature-based solutions to protect infrastructure in the coastal environment, although often the two can be combined. The first category of these includes seawalls and bulkheads, which are designed using standard procedures for earth retaining structures discussed in this document and the provided references. However, a key consideration for geotechnical design specifically for the coastal environment is the design water level. This is typically developed based on hydrodynamic models/equations for wave height/runup or flood levels for a design level storm (here, design level refers to the appropriate return period specified by the corresponding engineering

manual). These water levels are intrinsically linked to the design still water levels, which can be modified to account for the effects of relative sea level changes using the methods given in HEC-25.

The second category of these includes revetments, breakwaters, groins, and jetties constructed from riprap. The geotechnical design of these is conceptually similar to scour armoring methods discussed earlier in this section and requires analysis of the stability of the slope and developing specifications for the riprap. These riprap specifications typically take the form of the median stone weight, thickness of the armor layer, the number of stones per layer, and, if an underlayer is required (and its gradation). These riprap properties are developed based on design wave heights and energies, which are developed based on hydraulic/hydrodynamic models and modified for relative sea level changes using the procedures of HEC-25.

3.6.2.3. USBR DS 13

Internal erosion, where seepage causes transport and loss of embankment or foundation material, has caused several notable past dam failures and is a major design consideration in the USBR Design Standards [29]. Homogeneous cross sections, as shown previously in Fig. 3, are not recommended for use if materials are erodible or dispersive (the manual says always to perform laboratory testing to assess this). Dispersive materials are permitted in the impervious zone of a zoned embankment as long as a protective filter is provided to limit fines migration. A basic design requirement of all dams is that seepage is controlled to prevent erosion and piping of embankment or foundation material. An additional design requirement is that embankment surfaces are also protected from wave action and surficial erosion. Erosion is primarily a concern in sand/gravel foundations or dispersive clays. Usually, some foundation treatment, typically in the form of removal and replacement of erodible material, would be provided.

The design procedure for assessing erosion/piping of embankment or foundation material begins with a seepage analysis using a flow net, which may be accomplished by hand or numerically, and is described in greater detail in Sec. 3.9. The primary output of the analyses are estimated hydraulic heads (which can be converted to pore water pressures) and groundwater flow paths throughout the embankment and its foundation for a given set of boundary conditions. An example is shown in Fig. 8.

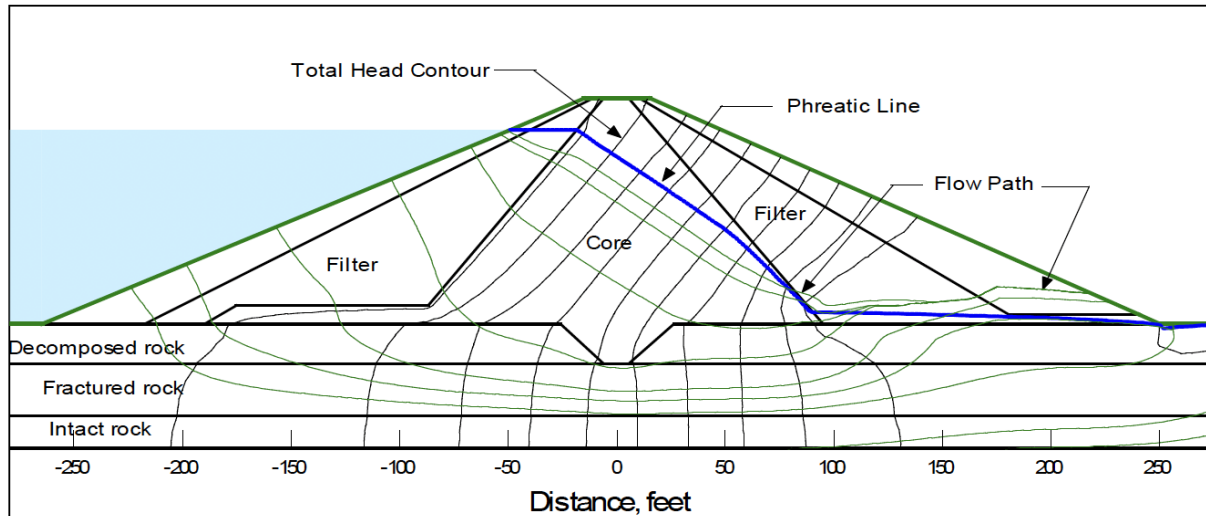


Fig. 8. Example results from a numerical seepage analysis. Reproduced from [29].

High hydraulic gradients at the embankment's free face or toe can cause liquefied (sometimes called quick or static liquefaction) conditions in sands or other low-plasticity soils if the critical gradient is exceeded. This leads to loss of shear resistance, internal erosion that proceeds backward from the point of initiation (also referred to as piping), and potential catastrophic failure of the embankment. The critical gradient highly depends on fines content and plasticity and ranges from 1 for clean sands to 10 or 100 for clays. Relatively large factors of safety, defined as the ratio of critical gradient to calculated gradient from seepage analysis, are used because of the catastrophic nature of potential failures. Uplift or blowout of cohesive material at the toe is another design concern and is assessed by comparing the estimated pore pressure at the base of the confining layer to the overburden pressure. Calculations for heterogeneous toe conditions are more complicated and are not provided in the manual.

The manual notes that even if safety factors for quick conditions or uplift are relatively low, internal erosion is not necessarily expected to initiate or progress, as this will be affected by other factors along the seepage path.

High internal gradients, characterized by primarily horizontal flow instead of the vertical flow at exits, pose a similar internal erosion hazard. These can cause failure as a result of backward erosion from an exit point piping through the embankment and subsequent dam breach, oversteepening and sloughing of downstream slope and subsequent progressive failure, or internal migration of soil leading to void formation and collapse. However, there is uncertainty regarding what internal gradients, in what soils, and in what embankment configurations are capable of initiating and sustaining internal erosion. Furthermore, internal gradients are expected to vary along the seepage path substantially. The manual notes that estimated or measured internal gradients can often exceed minimum gradient criteria from some prior research. The manual states that there are no widely accepted means to estimate a factor of safety against internal erosion (i.e., piping).

USBR practice is to handle design for embankment internal erosion potential through a risk-based approach incorporating subjective and quantitative assessments. Internal seepage is

primarily controlled by requiring filters on all new dams or modifications to existing dams, and by providing impermeable cores, cutoff trenches, internal drains, and foundation grouting to reduce horizontal flow velocities. A general requirement is that seepage through erodible soils does not have an open or unfiltered exit. Special caution is advised for critical situations, including seepage through:

- Low plasticity soils
- Dispersive clays or other highly erodible materials
- Portions of the upper embankment with cracking from desiccation or other causes

The manual notes that controlled compaction and water conditioning during the construction of embankment material create soils that are relatively “resistant” to internal erosion. The resistance to internal erosion decreases if transverse cracks form, as discussed in prior sections of this report. In some cases, quantitative tests such as the Hole Erosion and Jet Erosion Test can give simplified indices for general estimates of initiation potential and time of failure from internal erosion. The accuracy of such tests is unknown since the construction method of embankment dams proceeds by compacting relatively thin layers (e.g. 12-inches thick), which results in a significant and relatively unquantifiable difference in soil permeability in the vertical and horizontal directions. The manual suggests that dispersive soils can be identified using typical methods such as the pinhole test discussed earlier.

The manual provides design suggestions for various seepage mitigation measures, including internal filters, toe drains, grout curtains, relief wells, cutoff walls, etc. In particular, filter materials are constrained within certain material gradations and tested for material retention (i.e., erosion potential) and self-healing. Self-healing is the tendency of a material to collapse cracks within the filter rather than allow them to propagate, which is an important feature for preventing through cracking and uncontrolled seepage from conditions causing embankment deformation, such as from a seismic event or other causes leading to settlement of slope deformation. The tests necessarily rely on a simplified laboratory representation of filter configuration.

Instrumentation, monitoring of discharge flows, and regular observations are recommended to assess the efficacy of seepage control measures and potentially detect initiation/progression of failure modes. Modern designs require that a temporary emergency drawdown of the reservoir pool be possible. The manual also describes various other emergency procedures (such as injection grouting) that can be deployed in response.

3.6.2.4. Additional National Level Guidelines

The UFC DM 7.2 [53] requires placing shallow foundations below maximum scour depths. It provides the following qualitative criteria for assessing the erosion potential of fill materials, reproduced in Table 17.

Table 17. Qualitative erosion resistance categories for various soil types. Reproduced from [53].

Erosion Resistance Category	Applicable Soil Types
-----------------------------	-----------------------

1 (best)	CL, CH, and well-graded SC with $PI > 15$, any compaction level
2	Well-graded with clay binder, $15 > PI > 6$, any compaction level
	Well-graded, coarse-grained, $PI < 6$, well compacted
3 (worst)	Well-graded, coarse-grained, $PI < 6$, poorly compacted
	Very uniform, fine sands, $PI < 6$, any compaction level
	Gap-graded soils, any compaction level

The manual also provides quantitative scour assessment methods based on median particle size versus critical flow velocity for coarse-grained material and erosion rate versus flow velocity for fine-grained material and rock. These are the same as the FHWA HEC relationships described previously. The manual also notes that “modeling the fluid dynamics and erosion mechanics of scour problems is a complex undertaking that is typically undertaken by specialists”.

3.6.2.5. Notable State and Local Amendments

California. The CGS Special Publication 117A [28] notes that the long-term stability of cut/fill slopes requires ongoing erosion control and repair of surficial erosional failures before they progress. The document recommends assessing surficial stability to capture the effects of local erosional failure during intense rainfall or irrigation using an infinite slope model, assuming groundwater at a minimum assumed depth of 1m. It is also noted that the California Department of Water Resources Division of Safety of Dams is world-renowned for dam safety, and most practitioners in the US need to familiarize themselves with their processes and procedures, though they do not provide a separate manual or standard.

The Caltrans Amendments to the AASHTO BDS [68] require that spread footings and deep foundations with pile caps are located below the depths of the estimated scour, as shown in Fig. 9. The document states that even lower elevations should be considered if piles may become exposed and damaged by corrosion or erosion from exposure to water.

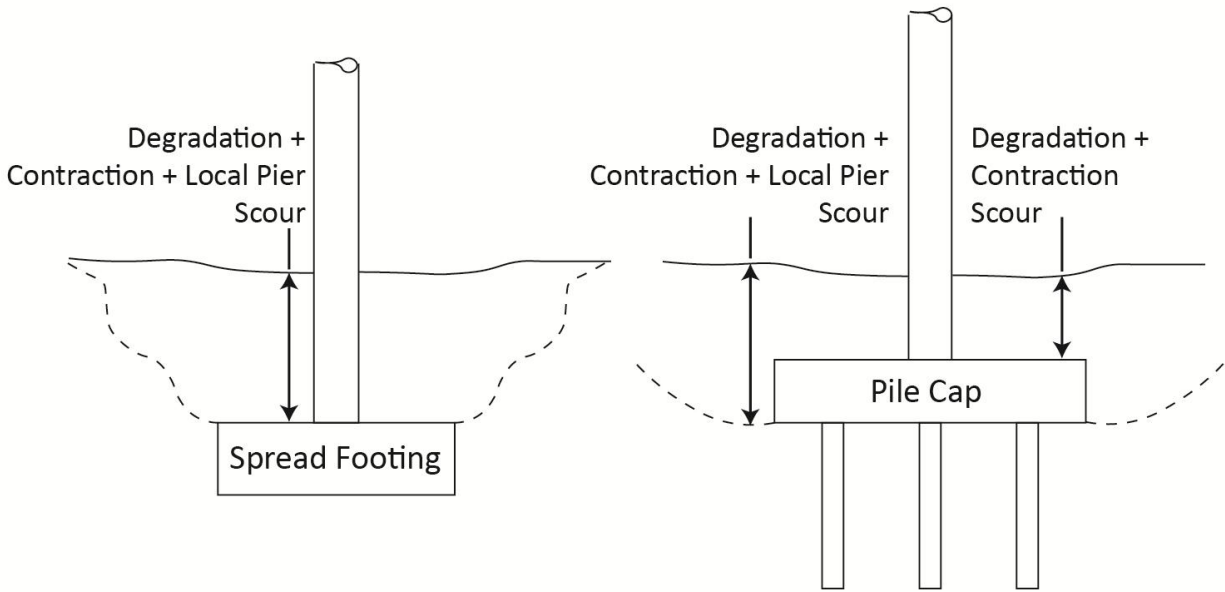


Fig. 9. An illustration of how Caltrans combines scour estimates for foundation design. Reproduced from [68].

The amendments require similar consideration of scour in assessing geotechnical lateral and axial capacity of foundation elements, but use slightly different required amounts, reproduced in Table 18.

Table 18. Caltrans scour design criteria.

Limit State	Type of Soil	Degradation/Aggradation	Contraction Scour	Local Scour
Strength	Minimum	0%	0%	0%
	Maximum	100%	100%	50%
Service	Minimum	0%	0%	0%
	Maximum	100%	100%	100%
Extreme Event I	Minimum	0%	0%	0%
	Maximum	100%	100%	0%

The Caltrans amendments allow for exceptions to the required scour combinations to be made if “site conditions dictate otherwise.”

Florida. The FDOT Bridge Scour Manual [111] provides alternative equations for local scour that could be used instead of the ones from HEC-18. These include a gradation parameter (ratio of 16th and 84th percentile particle sizes), which better captures the behavior of sands with fines. The manual also suggests that scour equations could incorporate washload or concentration of

suspended fines, but further research is needed to quantify their effects. The FDOT also has two custom test setups (Rotating Erosion Test Apparatus and Sediment Erosion Rate Flume) that can be used to test site soils and rock to develop erosion rate versus flow-induced shear stress relationships for use with the SRICOS analysis procedure.

Illinois. The IDOT Bridge Manual [112] provides similar design guidelines to the AASHTO BDS. IDOT uses a 200-yr flood for the Extreme Event II limit state (instead of the AASHTO standard with a 500-yr flood). Otherwise, IDOT scour procedures follow HEC-18, which only requires geotechnical inputs for soil classification and grain size distribution. The manual notes that HEC-18 equations give potentially overly conservative estimates for cohesive soils. Engineers potentially can perform lab erosion rate testing on undisturbed samples for use with the Scour Rate in Cohesive Soils (SRICOS) [113] procedure. Otherwise, the geotechnical engineer should recommend reduction factors for estimated scour amounts in cohesive soils, as shown in Table 19.

Table 19. Recommended scour reduction factors from the IDOT Bridge Manual.

Soil/Rock Type	Scour Reduction
Non-weathered limestone or dolomite	100%
Shale or sandstone	90%
Stiff to hard cohesive soil	50%
Soft to stiff cohesive soils	25%
Very soft to soft cohesive soils	0%

The manual notes that most sites will not easily be classified into one of the above categories and that IDOT is currently conducting research to improve its scour analysis procedures.

Texas. The TXDOT Geotechnical Manual [69] follows similar procedures to the AASHTO BDS for incorporating calculated scour depths into geotechnical foundation design. The TXDOT also has a Scour Analysis Guide and Scour Evaluation Guide [114], [115] that provide further guidelines on calculating scour depths. The procedure begins with classifying channel invert material as erodible or non-erodible based on soil or rock type reproduced in Table 20.

Table 20. TXDOT qualitative erodibility categories for various soil and rock types.

Material	Subcategory	Erodibility
Rock	Hard	Non-Erodible
	Soft	Mildly Erodible
Clay	Hard	Mildly Erodible

	Soft to medium stiff	Erodible
Sand	All	Very Erodible

The maximum scour depth for design is the depth to non-erodible layers. For sands, the guides recommend using the standard HEC 18 design equations with modifications to the hydraulic inputs if pressure scour from overtopping of the bridge is expected to occur. If the fines content exceeds 12% the guides recommend reducing the calculated local pier scour by 50% and using the FDOT methodology (described previously). The guides recommend the SRICOS method for estimating scour in clay, summarized in Fig. 10.

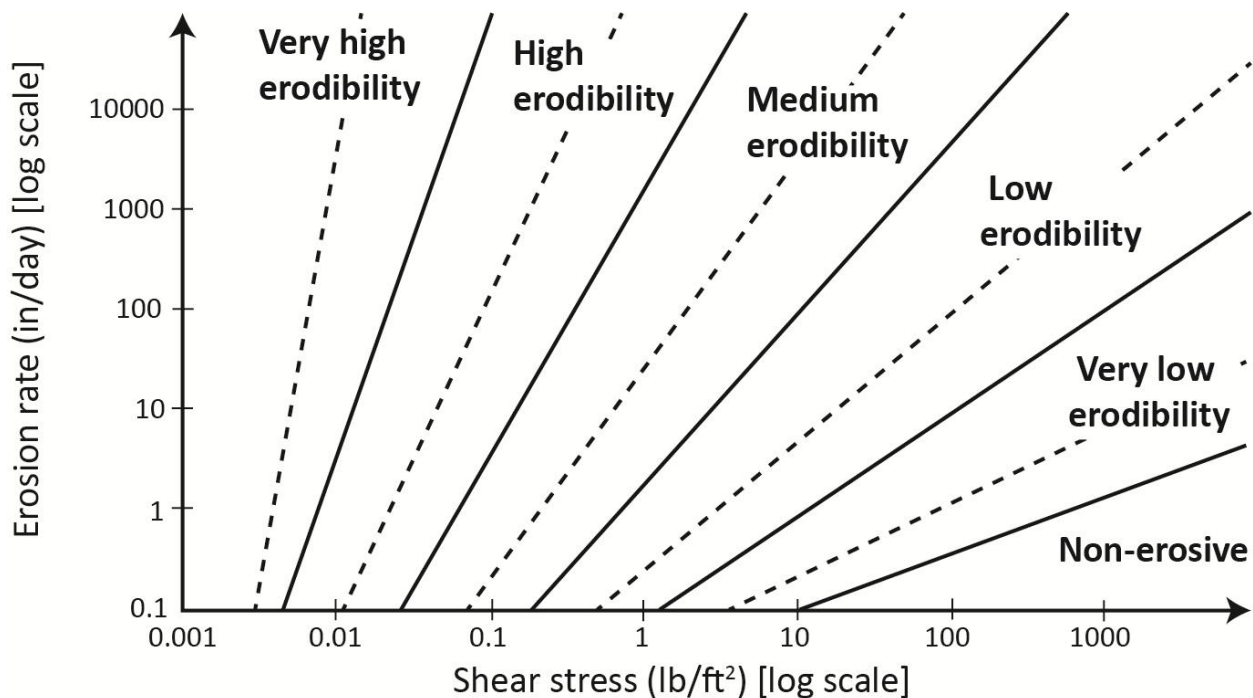


Fig. 10. The slightly modified erosion rate versus material type relationship used by TXDOT. Reproduced from [114].

The guides note that lacustrine scour is primarily driven by wind-generated waves, extreme events, or long-term drought, causing the lake to morph into a river. They recommend taking wind-driven scour equal to one pier diameter and applying it at each pier and abutment. For coastal design, the geotechnical inputs remain the same, but the hydraulic modeling becomes more complicated to account for tides, storms, waves, etc. The Hydraulic Design Manual [97] recommends including relative sea level changes in analyses for determining design wave heights and minimum road/bridge elevations. It states that shoreline change should be accounted for. The Texas Bureau of Economic Geology [116] publishes historical change rates, which are typically extrapolated into the future for routine design. More advanced designs can use sediment transport models, but may require coastal engineering expertise.

3.6.3. Gaps in Practice and Standards

In general, guidelines for protecting buildings and other structures that fall under the scope of the building code from the effects of scour and erosion are limited and do not include quantitative procedures. However, for transportation structures (i.e., bridges crossing waterways), design procedures explicitly incorporate scour estimates for various flooding return periods into the limit states considered. Additionally, flood protection structures have a great deal of attention paid to internal erosion as a potential failure mechanism.

- Methods for predicting riverine scour are widely used to design transportation structures. However, there is concern regarding their accuracy. In particular, research is ongoing to improve predictions in sands with appreciable fines, cohesive soils, and rock.
- Methods for predicting coastal/lacustrine scour or erosion from flooding outside of the streambed typically use the same design equations as riverine scour and only modify the hydraulic analyses used to develop design flow velocities.
- Predictive equations for scour typically use relatively simple geotechnical inputs (e.g., median particle size and soil classification) and are typically developed for uniform soil conditions. It remains to be seen if models that incorporate more detailed geotechnical inputs and/or soil layering could produce more realistic predictions.
- Embankment dam design procedures include the usage of seepage control measures such as protective filters, construction detailing, and quality control to pre-emptively mitigate potential internal erosion concerns. However, quantitative prediction of erosion rates for a given hydraulic scenario remains a reasonably difficult task, particularly considering the potential for soil variability.
- Guidelines for addressing the effects of scour and erosion are typically limited to foundation embedment depth or standard armoring strategies and do not specifically address future hazards/conditions or incorporate adaptive designs.

3.7. Tsunami

3.7.1. Overview and Hazard Considerations

Tsunamis are large, powerful waves typically generated from permanent vertical seafloor displacement during earthquakes and are primarily a threat to the West Coast and Pacific Islands in the US. Tsunamis can also result from landslides that displace a large volume of water. Tsunami-resistant design requires expertise in seismology, fluid dynamics, and fluid-soil-structure interaction and is an active research area. An overview of recent developments can be found in the ASCE 7 commentary for Chapter 6. The incoming and receding water flow during a tsunami event can lead to large hydrodynamic loads on the structure, debris impacts, and scour/softening of soil supporting foundations. From a hazard perspective, the tsunami hazard will be predominantly influenced by sea level changes.

3.7.2. Design Guidelines

3.7.2.1. IBC and ASCE 7

Section 1615 of the IBC [51] requires designing for tsunami loads (when applicable) per ASCE 7 [52] methodology (Chapter 6). Exactly what structures are explicitly designed for tsunami loads is somewhat complicated. For example, including design requirements for one and two-family dwellings and low-rise buildings of light-frame construction may be cost-prohibitive, as ASCE 7 cites no precedent for historical survivability. However, state or local governments can designate a threshold height to apply tsunami design requirements to tsunami risk category 2 structures.

Tsunami Guidelines always apply to risk category 3 and 4 structures, typically those whose failure could pose a substantial risk to human life, substantial economic impact, or disruption of emergency services. The exact definitions of tsunami risk categories are slightly different from the general risk categories in ASCE 7 and allow for some discretion on the part of local governments. The basics of tsunami design begin with determining runup elevations, as shown in Fig. 11.

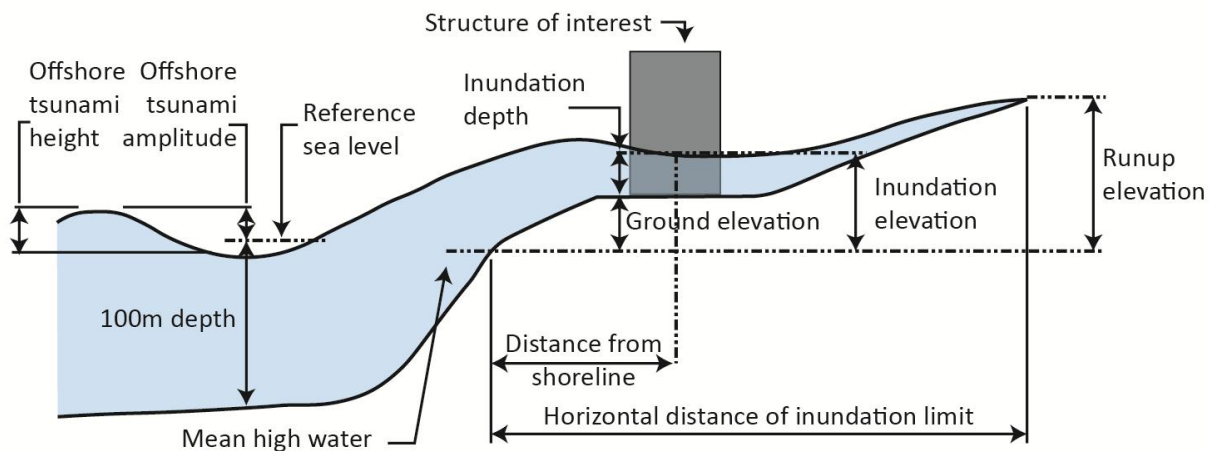


Fig. 11. A schematic showing definitions of various terms used in ASCE 7-22 tsunami design procedures. Reproduced from [52].

A digital database companion to ASCE 7 provides mapped values for design zones, runup elevations, and inundation depths. These values are from tectonic origin, and landslide generated tsunami is not considered in these guidelines. The standards require adding the effects of potential relative sea level changes to runup elevations. At a minimum, the commentary suggests extrapolating the historical average rate over a 50-year minimum design life. The hydraulic analyses for determining flow velocities from mapped parameters are fairly involved but typically only require surficial soil classification as a geotechnical input.

Changes in site surface and soil properties (e.g., from liquefaction) from the preceding earthquake are used as initial conditions for subsequent tsunami analysis. The standards require that the foundation design consider the potential for soil strength loss using the calculated flow velocities and other hydraulic inputs. The suggested procedure for determining this involves the soil unit weight, coefficient of consolidation, and drainage factor (which is given as a function of soil type). The design also needs to consider the cumulative effects of erosion and scour. ASCE 7 provides similar scour evaluation procedures to those discussed in Sec. 3.6 of this report.

3.7.2.2. USBR DS 13

While embankment dams are usually not located close enough to the coast to be threatened by oceanic tsunamis, earthquake-generated waves from vertical fault movement within the reservoir or water displaced by landslides can present an overtopping risk. These types of waves are identified as a seiche. A landslide-generated seiche may be earthquake-induced or caused by any other mechanism generating large slope displacements into the reservoir. The USBR Design Standards [29] provide references to several technical publications for predicting wave runup in these cases. For seismic-induced seiches, probabilistic seismic hazard analyses may be utilized to estimate overtopping risk. For non-seismic induced landslides, a risk analysis should be performed to assess the probability of the landslides and the potential for overtopping the dam by a seiche. Including a large freeboard to protect against seiche overtopping, though sometimes uneconomical, should be assessed using risk-based methods to determine the safety of the dam. This practice is consistent with all other hazards (i.e., if the probability is high, then consideration to remove the dam, lower the reservoir elevation, or other mitigation strategies should be made).

3.7.2.3. Notable State and Local Amendments

California. The CGS Note Sheet 48 [67] refers to ASCE 7 design procedures for tsunami evaluation. The Caltrans Seismic Design Criteria [117] include tsunami as a design seismic hazard to be considered. A memo to designers [118] provides some criteria for when explicit design is required for new bridges and widenings when:

- Bridge elevation is less than 40 feet above mean sea level
- Project is within half a mile of the Pacific Ocean
- Location is considered “vulnerable” to tsunami hazards based on maps/data published by other sources (such as the California Geological Survey)
- Bridge is identified as part of a local emergency response plan that must remain functional

3.7.3. Gaps in Practice and Standards

Comprehensive tsunami-resistant design is a relatively recent addition to ASCE 7 (and, by extension, the building code) and incorporates the results of highly active areas of geotechnical earthquake engineering research. As a result, at least for the scope of this review, the gaps in practice are relatively minimal:

- Fairly minimal guidelines provided regarding how much relative sea level changes should be added to mapped runoff elevations.
- The recommended equations for predicting scour and erosion produced by tsunami-driven flows are similar to or directly taken from the guidelines previously reviewed for more general scour and erosion. Similar commentary regarding geotechnical inputs would apply to the tsunami case.
- The determination of initial (post-earthquake, pre-tsunami) site geometry and soil properties can be somewhat subjective.
- The pre-earthquake water level used in estimating reservoir wave heights is unclear.

3.8. Slope Stability

3.8.1. Overview and Hazard Considerations

Slope failures are broadly classified according to their predominant geologic material and mode of movement, shown in Fig. 12.

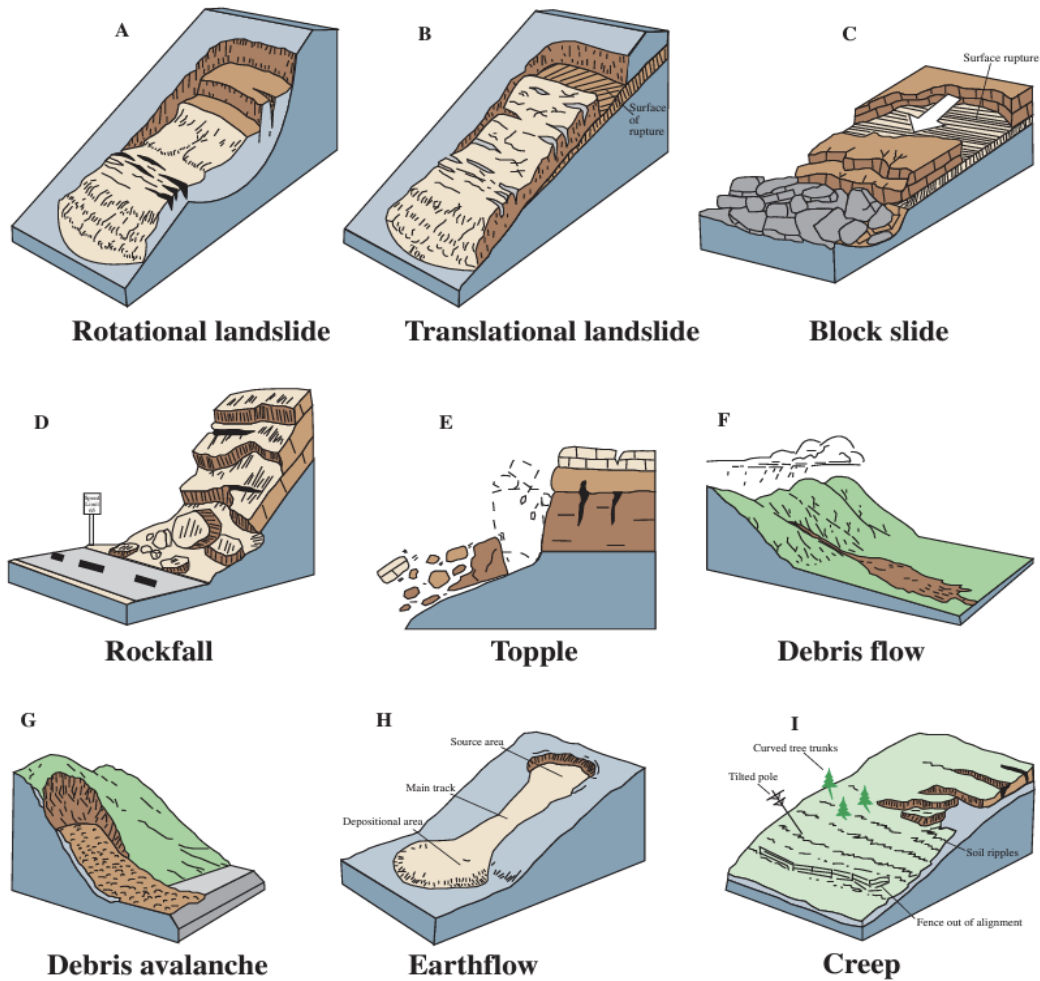


Fig. 12. Illustrations of various slope failure types. Reproduced from [119].

These include several common modalities such as landslides, rockfalls, and debris flows. Slope stability is widely recognized as an important geotechnical hazard and is the subject of many textbooks, research articles, agency guidelines, and lessons learned from practical experience. Given the vast amount of technical material, this report focuses on aspects of site exploration, laboratory/field testing, design, and mitigation strategies for slope stability that specifically address climatic considerations. Interested readers may wish to consult other documents, such as the Transportation Research Board's Special Report 247 [120] or Duncan et al. [121], to get a complete technical overview of the topic.

It has long been recognized that climatic factors, including intense rainfall, rapid snowmelt, and other causes of changing groundwater levels, are a major trigger of landslides [120]. Elevated groundwater levels and seepage within a slope will cause excess pore pressures and decrease soil strength. Saturation will also lead to higher total unit weights and destabilizing forces. Intense precipitation can also lead to surficial erosion and/or undermining of slopes, further reducing available resisting forces. Wildfires, which are becoming more common in the west

and across the country, can leave hydrophobic conditions in surficial soils and reduce vegetation cover, leading to an increased potential for erosion and debris flow [122]. All the above factors will be heavily influenced by potential future changes in precipitation/storm magnitude.

3.8.2. Design Guidelines

3.8.2.1. IBC and ASCE 7

Section 1803 of the IBC [51] requires that a geotechnical investigation, when required by the building official, be performed “in accordance with generally accepted engineering practice”. This investigation is supposed to include an evaluation of slope stability “as necessary” or when located in a Seismic Design Category C or higher. The geotechnical report is expected to provide alternative minimum setbacks/clearances for structures relative to adjacent slopes, as shown in Fig. 13.

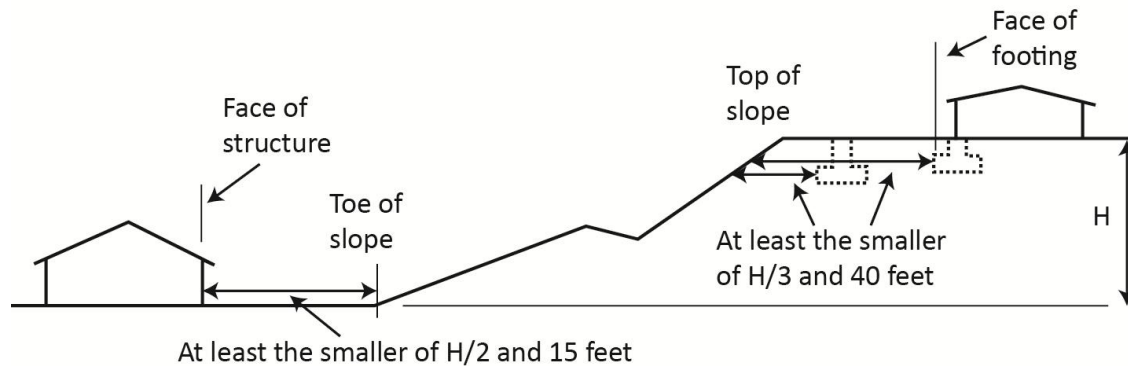


Fig. 13. Required minimum setbacks from the top and toe of slopes. Reproduced from [51].

The IBC requires that foundations on or adjacent to slopes are sufficiently embedded to support the overlying structure without excessive settlement and located below any soils of “a shifting or moving character”. Appendix J of the IBC provides additional criteria for site grading and slopes. While its criteria are not “mandatory unless specifically referenced in the adopting ordinance,” many municipalities adopt it or provide similar guidelines. In general, a supporting geotechnical report is required for any proposed slopes steeper than 2H:1V (there are limited exceptions for slopes that do not support structures). Cut and fill slopes are expected to have erosion control and drainage such as swales, ditches, or interceptor drains. Erosion control is permitted to consist of “effective planting”. The IBC does not explicitly address debris flow.

ASCE 7 [52] does not address slope stability. However, it includes the effects of “moving surcharges” in its lateral soil loads (section 3.2), which would apply to structural elements designed to resist landslide forces. Additionally, in Seismic Design Categories D and higher, geotechnical investigations are expected to address seismically induced permanent ground displacement, i.e., seismic slope stability. ASCE 7 does not explicitly address debris flow.

3.8.2.2. AASHTO BDS and FHWA GEC 5

Section 10.4 of the AASHTO BDS [27] requires that a geotechnical exploration program be “sufficient” to characterize engineering properties of the soil/rock present and problematic subsurface conditions, including unstable slopes and groundwater conditions, and provide enough information to evaluate foundation stability and settlement with regards to the above. The stability of retained fills and earthen slopes (with or without a foundation element) is evaluated at the strength and extreme event limit states using load and resistance factors based on whether the foundation elements serve to stabilize or destabilize the slope. It should be noted that higher region-specific values can be used based on substantial statistical data and engineering experience. Conversely, smaller values should be used if site variability is unusually high, assumptions for design result in increased uncertainty, or if there is a potential for soil/rock disturbance.

The BDS recommends selecting geotechnical design parameters (e.g., total unit weights and shear strength) that are “appropriate” for the limit state considered and mathematical model used for soil behavior. This typically requires selecting undrained and drained strength parameters based on the loading rates relative to how quickly excess pore pressures dissipate for the soil types considered. For cohesive soils, the BDS emphasizes undrained triaxial tests for undrained strengths and drained (or undrained with pore pressure measurements) triaxial tests for drained strengths. It also recommends choosing between peak, fully softened, or residual strengths based on expected deformation levels. For granular soils, the BDS emphasizes correlations with SPT or CPT field values to establish a drained friction angle. Laboratory tests can also be performed on reconstituted specimens if a design compaction and water content are specified. Undrained shear strength of granular materials is typically only a concern for seismic loading and is discussed in the section on soil liquefaction.

The BDS does not specifically address how to select groundwater levels for slope stability evaluation beyond some limited commentary regarding understanding the “history” of groundwater levels at the site. General guidelines for incorporating groundwater conditions into design procedures (that may not necessarily be specific to slopes) are discussed in Sec. 3.9.

The FHWA GEC 5 [26], in its section on site characterization for landslide and rockfall hazards, recognizes the need for the geotechnical exploration program to adequately evaluate the potential for slope instabilities to impact the proposed project, particularly if there is evidence of current or prior failures. It recommends that groundwater conditions should be “well understood”, particularly with regards to spatial and temporal variability. The manual also recognizes the potential for intense precipitation to trigger failures. The manual provides similar guidelines for rockfall and does not discuss debris flow.

3.8.2.3. USBR DS 13

The USBR Embankment Dam Design Standards [29] devote substantial attention to slope stability evaluations. This review focuses on the general aspects of the design and construction procedures. Aspects relating to instability from specific geotechnical hazards such as seepage, erosion, and collapsible soils will be covered in their respective sections.

Modern embankment dams are typically constructed with one of the typical cross sections shown in Fig. 3.

The practice requires that embankment dams maintain stability and limit deformations to acceptable levels under all loads from construction, operation, and earthquakes. Current practice consists of adopting slopes and design configurations of existing successful dams, analyzed under site-specific conditions for material type and loading. This is supplemented by careful control of embankment material quality and placement. The Bureau has a philosophy that radical innovations are avoided and fundamental changes in design procedures are adopted gradually through practical experience. Table 21 shows the load cases considered for static stability analyses.

Table 21. Embankment dam stability load cases and associated minimum factors of safety. Reproduced from [29].

Loading Condition	Shear Strength Parameters	Pore Pressure Characteristics	Minimum Factor of Safety
End of construction	Effective	Generation of excess pore pressures in embankment and foundation materials with laboratory determination of pore pressure and monitoring during construction	1.3
		Generation of excess pore pressures in embankment and foundation materials, and no field monitoring during construction, and no laboratory determination	1.4
		Generation of excess pore pressures in embankment, only with or without field monitoring during construction, and no laboratory determination	1.3
	Undrained strength		1.3
Steady-state seepage	Effective or undrained	Steady-state seepage under active conservation pool	1.5
Operational conditions	Effective or undrained	Steady-state seepage under maximum reservoir level (during a probable maximum flood)	1.2
		Rapid drawdown from the normal water surface	1.3

		to the inactive water surface	
		Rapid drawdown from maximum water surface to active water surface (following a probable maximum flood)	1.2
Other	Effective or undrained	Drawdown at maximum outlet capacity (Inoperable internal drainage; unusual drawdown)	1.2
		Construction modifications (applies only to temporary excavation slopes and the resulting overall embankment stability during construction)	1.3

The minimum factor of safety criteria is developed based on the “estimated reliability” of soil properties and pore pressure prediction, “probable” quality of construction control, experience, duration of loads, and assumed planned monitoring during operation. Designers are allowed to design for higher or lower than minimums provided with appropriate justification. The pore pressures used in each load case are established through numerical modeling with upstream water surface boundary conditions corresponding to Fig. 14.

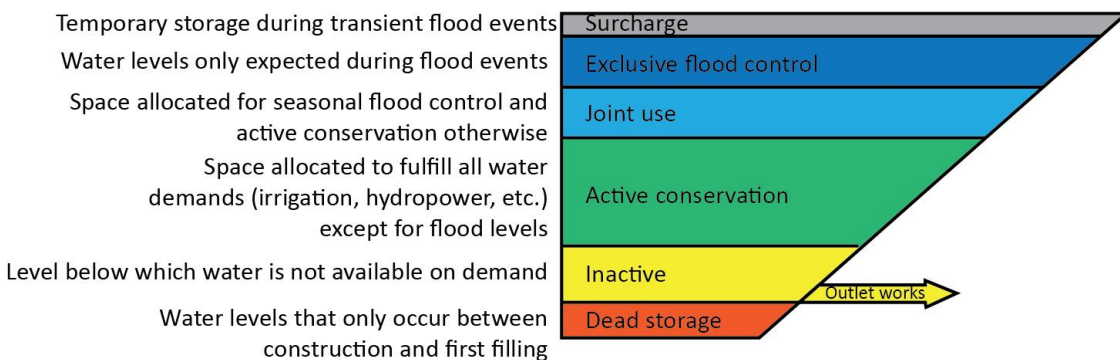


Fig. 14. Definitions of various reservoir storage levels. Modified from [29].

The exact design values of these water levels are typically established by the project team based on hydrologic/hydraulic modeling and forecasted future water demand. Steady-state seepage is typically assumed to develop even if the worst-case water levels are reached only temporarily. This is considered a conservative assumption, and in some cases, a time-dependent analysis based on a design hydrograph is performed.

The manual gives similar guidelines to the BDS and other documents for the determination of soil shear strength parameters. In particular, it recommends that undrained tests be conducted at anticipated placement densities and water contents and that laboratory tests should be performed at 100% saturation (or as close as practical). It also mentions that certain fat clays may have strength reduced to a fully softened state from repeated wet/dry cycles.

The manual's preferred method for slope stability analysis is limit equilibrium, but allows for finite element or finite difference methods to be used at the discretion of the designer. The manual does not explicitly discourage these more advanced numerical methods, but notes that the results should be compared to limit equilibrium analyses. The potential for progressive failure should be considered, and the manual suggests using higher factors of safety or residual strengths when this is possible.

The manual also gives methods for estimating vertical deformations of embankment dams during "normal" operations. The recommended degree of analysis performed depends on project needs. It is often considered sufficient to use simple and conservative rules/deformation patterns developed from past observations.

In some cases, more advanced analysis may be warranted, such as where the following exist: soft and extensive soil foundations, homogenous clay embankments with high moisture content, unique geometries, or highly variable foundation material. The manual primarily focuses on limiting potential deformations by removing or improving weak materials and providing defensive measures (filters) to intercept seepage, should cracking occur. The manual suggests that deformations should be monitored during operation, but it is often difficult to determine an "unacceptable" level.

The manual notes that the performance of embankment dams in earthquakes has been acceptable when liquefiable materials are not present, and construction is of high quality. The Bureau's practice is to not require seismic deformation analysis if the following criteria are met:

1. Liquefiable/sensitive foundation materials are not present;
2. The dam is "well built", with a minimum relative density greater than 75% or at least 95% of maximum laboratory dry density;
3. The up/downstream slopes are shallower than 2.5:1 (H:V), and the phreatic line is "well below" the downstream face;
4. The peak horizontal acceleration at the embankment base is less than 0.35g;
5. That static factor of safety for all potential failure surfaces that could result in loss of crest elevation is greater than 1.5, assuming pore pressure conditions "reasonably" expected to be present immediately prior to the earthquake;
6. Minimum freeboard with active or joint use water levels (see Fig. 14) is greater than 3-5% of the total embankment height and 3 feet;
7. There are no appurtenant features that could be harmed by small embankment movements and lead to internal erosion (or other failure modes);

If only requirement number 4 is not met, then typical simplified seismic slope displacement procedures (e.g. [123]) are applied considering soil strengths appropriate for rapid loading and steady-state seepage from active pool levels. The manual notes that numerical analyses “should not be considered definitive predictions” but can be beneficial for showing “general trends” in seismic deformation. Specifically, it states that three-dimensional numerical dynamic analyses can provide insight for potential locations where tensile/shear strains could lead to cracking (though it does not provide explicit threshold values). However, like many other aspects of USBR practice, the potential for cracking and internal erosion in new dams is pre-emptively addressed by freeboard requirements, internal filters, and construction quality control

3.8.2.4. Additional National Level Guidelines

The UFC DM 7.2 [53] requires considering changes in the properties of soils that are susceptible to softening from weather or climatic effects.

USACE does not set nominal top-of-levee (TOL) elevations based on a single design event. Instead, alternative levee configurations are evaluated in a risk-based framework that considers variability and uncertainty in both future rainfall and its associated flood water levels and levee soil properties (e.g., classification, strength parameters, and hydraulic conductivity) throughout the reach [59]. Further details regarding the specifics of the procedure can be found in ER-1105-2-101 [56] and EM 1110-2-1619 [60]. Considerations for incorporating potentially changing future conditions are provided in EC 1100-1-113 [58]. The primary geotechnical input to these studies is a performance curve defining the probability of failure for various water levels under steady state seepage conditions. EM 1110-2-1913 [59] also provides minimum factor of safety criteria for deterministic seepage and stability analyses, the trial sections for each reach (a length of levee with similar geometry and embankment/foundation soils).

USACE also provides design guidelines related to improving the resilience of levees and other structures to future hazards in several documents. EP 1100-2-1 [57] discusses potential physical threats posed by sea level change to coastal structures, including foundation scour, increased risk of overtopping, increased design wave heights (and associated loads), and greater inland penetration of coastal storm surge. The document notes that both rising and falling sea levels can potentially be a concern. Rising sea levels can lead to higher design wave heights and risk of overtopping, but falling sea levels can result in a greater potential for structure foundation undermining or toe erosion of slopes/levees. It also states that geotechnical investigations are required to adequately characterize subsurface material that may become exposed or relied on for structural support in the event that sea level change directly or indirectly alters the topography. ECB 2019-8 [55] states that the geotechnical engineer is expected to estimate settlement to aid the design team in establishing the required top of levee elevation. This settlement includes the effects of shrinkage, cracking, and geologic subsidence. The design is also required to account for the increased risk of flooding from regional settlement or sea level changes.

3.8.2.5. Notable State and Local Amendments

Alaska. The City of Anchorage Amendments to the IBC [75] requires that slopes supporting or adjacent to a foundation require a geotechnical investigation and stability assessment. The document states that the analysis should consider “geotechnical conditions”, “erosion characteristics” of soil, and the potential for liquefaction. The targeted minimum factors of safety from limit equilibrium analyses are 1.5 for static/post-earthquake conditions and 1.1 for seismic conditions (or acceptable deformations from typical simplified seismic slope displacement procedures). The document additionally states that “it may be necessary to extend the geotechnical investigation beyond the immediate site boundaries to evaluate the applicable hazard.”

California. The California Building Code [103] requires geotechnical reports to evaluate slope stability (static and seismic), and any mitigation measures required for stabilization. These may be required to be addressed in a specific geohazard report, which has more detailed requirements than a general geotechnical report. Geohazard reports must meet the criteria of the CGS Note 48 [67] which provides a review checklist. Specifically, regarding slope stability, the documents require consideration of the potential for on-site and off-site landslides to impact structure and that analyses include “appropriate” groundwater conditions. Further detail is provided in the CGS Special Publication 117A “Guidelines for Evaluating and Mitigating Seismic Hazards in California” [28] and “Recommended Procedures for Implementation of DMG Special Publication 117: Guidelines for Analyzing and Mitigating Landslide Hazards in California” [89] are discussed next.

Special Publication 117A and its related publications are fairly involved documents that describe the geotechnical engineering basis for identifying the potential for, quantifying the impact of, and designing mitigation measures related to geohazards – specifically slope stability (static and seismic) and liquefaction. The focus of the documents is identifying “methods, procedures, and references currently representative of quality practice”. When evaluating slope stability and landslide hazards, the documents recommend an initial screening that includes the following potentially relevant factors:

- Presence of subsurface water or sources of concentrated water infiltration on slopes
- Susceptible or vulnerable landforms, including steep slopes and cliffs/banks being undercut by wave/stream action
- Potential for changes in subsurface hydrology from proposed development (e.g., irrigation, concentrated runoff from new impervious areas)

In geotechnical exploration, the documents recommend that the location and extent of groundwater should be identified. This may include long-term monitoring to capture periodic/seasonal changes and delayed pore pressure response resulting from slow infiltration of fine-grained soils. The document recommends typical laboratory tests for developing design strength parameters, like other documents reviewed. It notes that strength tests should be performed on saturated samples unless it can be “convincingly” shown that saturation is “unlikely” over the design life. The documents also discuss modifying strength properties for weathering, noting that the extent/depth of weathering depends on slope material and

conditions and can be difficult to estimate. A simplified methodology is presented based on observations and testing of material from slopes that have existed for a length of time similar to the design life of the project and have similar geologic conditions.

Slope stability analyses will be required when slopes steeper than 2H:1V are planned/present or if the screening criteria are met. The documents note that the “potential” for “significant pore water pressure” should be evaluated and included in the analysis. The maximum subsurface water level associated with “extreme winter storms” and irrigation should be used for static analysis, while seismic analysis should consider “typical” conditions and “normal” seasonal rainfall. The documents state that “analytical models together with conventional subsurface water-modeling techniques can provide reasonable estimates of future subsurface water levels.” For minimum factor of safety criteria, the documents describe “commonly accepted ... historic” values of 1.5 for long-term static stability, 1.25 for post-construction short-term conditions, and 1.1 for dynamic (seismic) loadings. They also suggest that certain cases, such as highly fractured bedrock or cemented materials, may require potentially designing for a higher factor of safety. The documents also provide some general considerations for debris flows, though they stop short of recommending a quantitative design procedure. They note that these are typically triggered by periods of intense rainfall following dry conditions. It is suggested that certain geomorphic features (e.g., ravines, swales, or hollows) that could serve as potential source material or concentrate flows be identified, and that structures be located away from their base. The documents recommend that the “expected volume of debris” should be estimated and that the speed and travel distances depend on slope, soil type, and water content. The main mitigation measures suggested are to avoid runout zones or provide barriers to deflect flow.

The Caltrans Geotechnical Manual has several modules that address slope stability concerns. Embankments less than 10 feet in height and a maximum 2H:1V slope are typically designed with minimal site investigation and based on past precedence/judgment, unless problematic foundation soils exist [124]. The manual recommends monitoring of groundwater levels in “major” fill areas to establish variation in groundwater levels. It suggests that readings taken during the wet season (typically October - April) can establish the “likely highest” groundwater levels that could affect analyses. The manual recognizes that rockfall can be triggered by environmental factors, including rain, freeze-thaw, and erosion. It states that most rockfall in California occurs from erosion of supporting material during rainstorms. A suggested approach is to obtain historic conditions data and compare it to rockfall frequency records to determine possible triggering thresholds. Caltrans uses the Rockfall Hazard Rating System [125] to qualitatively assess sites, shown in Table 22.

Table 22. An example rockfall hazard rating system with highlighted components. Reproduced from [125].

CATEGORY	3 POINTS	9 POINTS	27 POINTS	81 POINTS
SLOPE HEIGHT	25 FEET	50 FEET	75 FEET	100 FEET
DITCH EFFECTIVENESS	Good catchment	Moderate catchment	Limited catchment	No catchment
AVERAGE VEHICLE RISK	25% of the time	50% of the time	75% of the time	100% of the time

PERCENT OF DECISION SIGHT DISTANCE			Adequate sight distance, 100% of low design value	Moderate sight distance, 80% of low design value	Limited sight distance, 60% of low design value	Very limited sight distance, 40% of low design value
ROADWAY WIDTH INCLUDING PAVED SHOULDERS			44 feet	36 feet	28 feet	20 feet
GEOLOGIC CHARACTER	CASE 1	STRUCTURAL CONDITION	Discontinuous joints, favorable orientation	Discontinuous joints, random orientation	Discontinuous joints, adverse orientation	Continuous joints, adverse orientation
		ROCK FRICTION	Rough, Irregular	Undulating	Planar	Clay infilling or slickensided
	CASE 2	STRUCTURAL CONDITION	A few differential erosion features	Occasional erosion features	Many erosion features	Major erosion features
		DIFFERENCE IN EROSION RATES	Small difference	Moderate difference	Large difference	Extreme difference
VOLUME OF ROCKFALL/EVENT			1 Foot / 3 cubic yards	2 Feet / 6 cubic yards	3 Feet / 9 cubic yards	4 Feet / 12 cubic yards
CLIMATE AND PRESENCE OF WATER ON SLOPE			Low to moderate precipitation; no freezing periods; no water on slope	Moderate precipitation or short freeze periods; intermittent water on the slope	High precipitation and long freezing periods, or continual water on the slope	High precipitation and long freezing periods; continual water on the slope and long freezing periods
ROCKFALL HISTORY			Few falls	Occasional falls	Many falls	Constant falls

The landslide module emphasizes that monitoring slope movement and groundwater levels is necessary to adequately characterize a landslide, which may need to extend for months or years. It suggests that slope movements from monitoring can be correlated with rainfall and groundwater levels. Caltrans' practice is that investigation and analysis should be proportional to risks posed to public safety, potential impacts on highway operation, and the cost of mitigation strategies. As a result, in some cases, when slides have already experienced large movements, little to no geotechnical exploration is performed, and the slide is mitigated with mass grading. The preferred mitigation strategy is selected through a risk assessment, an example of which is shown in Table 23.

Table 23. An example landslide risk matrix. Reproduced from [126]

Landslide Mitigation Strategy	Score: 0 - 10		Score: 0-5				Total: (Max. 45)
	Effectiveness	Cost	Traffic Impact	Impact	Right-of-	Constructability	

					Way Impact			
Total Removal of Landslide	10	2	0	3	4	2	1	22
Partial Removal of Landslide	7	4	1	5	5	4	2	28
Stabilize Landslide with Earthen Buttress	10	2	0	3	4	2	1	22
Stabilize Landslide with Horizontal Drains	2	4	2	5	5	2	2	22
Buttress with Anchored Soldier Pile Wall	10	4	5	2	3	2	2	28
Manage Landslide through Periodic Closures and Repaving	1	10	5	5	5	5	5	36

The manual’s guidelines for slope stability analysis procedures are similar to those of other documents previously reviewed. The manual suggests a minimum static factor of safety of 1.3 and a seismic factor of safety of 1.0 to be targeted when “reasonable”. Otherwise, it suggests targeting the “highest achievable” factor of safety working within project constraints.

Illinois. The IDOT Geotechnical Manual allows some soils previously classified as unsuitable to be potentially acceptable in embankment cores if they can be protected against erosion, freeze-thaw, or shrink-swell effects. The required minimum factors of safety are:

- Embankments: 1.3 if based on laboratory tests or 1.5 if based on field tests
- Cut slopes: 1.5 if based on laboratory tests or 1.7 if based on field tests

The manual also states that the potential for cut slope failure should consider slope saturation from precipitation, seepage, or rising groundwater.

Texas. The TXDOT Geotechnical Manual requires “documented evaluation” for slopes steeper than 3H:1V. The manual suggests minimum long-term and short-term factors of safety of 1.3

for most slopes and 1.5 for slopes that support structures or critical utilities. If high plasticity clays are exposed to weathering or wet/dry cycles (e.g., cut slopes), the manual recommends using residual shear strength for long-term stability assessment

3.8.3. Gaps in Practice and Standards

Slope stability is a highly active area of geotechnical engineering research, with a great deal of attention paid to the potentially destabilizing forces of groundwater and seepage, as discussed at the beginning of this section. Many guideline documents go into detail on how to measure groundwater levels during site exploration and when to assume saturation for determining strength parameters. However, there are a few specific gaps, which are discussed in the list below:

- The choice of groundwater levels for different slope stability load cases (particularly seismic loadings) is often left to the engineer's judgment. In some documents, using levels associated with seasonal high precipitation or "reasonable worst case" is suggested. Still, a quantitative procedure for determining these or accounting for future extreme weather is not provided.
- Factors of safety are typically based on experience, and while they aim to capture potential uncertainty in analyses and consequences of failure, typically, there is no formal calibration procedure.
- Resistance factors (for LRFD design) are usually calibrated for uncertainties in soil properties but typically don't incorporate variability in groundwater conditions.
- Like factors of safety, many other aspects of slope design, construction, and monitoring rely on lessons learned from past failures or near failures. However, future conditions are unlikely to look like the past. It is unclear what current best practices are sufficient and what will require modification.
- Several documents recognize the potential for degradation of soil strength from repeated wet/dry cycles or weathering, but do not provide a detailed procedure for modifying design values besides testing samples from slopes that have experienced similar weathering (which may not be practical).
- Quantitative guidelines addressing how freeze-thaw frequency changes and wildfire impacts affect slope integrity are not provided.
- There is no guideline or prior research on the compound influence of changes in temperature, relative humidity, and moisture on slope stability and performance under both static and seismic loading conditions.
- Guidelines for addressing slope instabilities are typically limited to standard stabilization (e.g., regrading, de-watering, and/or anchoring) and do not specifically address future hazards/conditions or incorporate adaptive designs.
- Monitoring of slope movements, groundwater levels, seepage volumes, and potential trigger conditions (e.g., local weather or seismicity) is identified as an important aspect

of risk management for slope stability and embankment dams/levees in many of the documents reviewed. Many documents (e.g., [127]) provide recommendation for selecting instrumentation and developing monitoring plans. However, considerations for or modifications to monitoring plans in the context of potentially changed future conditions are not explicitly provided.

- There are almost no explicit guidelines (e.g., how to determine flow volumes/runout distances or what rainfall intensities to analyze) provided for debris flow.

3.9. Groundwater/Seepage

3.9.1. Overview and Hazard Considerations

Groundwater is not typically considered a geotechnical hazard in the same sense that many of the other topics discussed in this report are. It is usually covered in the subsurface conditions (e.g. [26], [25]) of a geotechnical report, and engineers primarily focus on describing where/if groundwater was encountered during site exploration (or ongoing monitoring) and the potential impacts it may have on design and construction. However, the presence (or absence) of groundwater and the presence (or absence) of seepage forces can dramatically affect the bearing capacity/settlement potential of foundations, earth/water pressure loads on structures, and susceptibility to many geohazards (e.g., static or seismic liquefaction). In this context, the presence of groundwater and/or seepage can be a source of danger, especially if the groundwater or seepage conditions that may transpire were not addressed during project design. This section will primarily focus on general aspects of groundwater/seepage as a geotechnical hazard. Groundwater, as it relates to specific other geotechnical hazards (e.g., slope stability, volume change potential, or liquefaction), will be covered in its respective sections.

As mentioned previously, the presence of groundwater will decrease soil effective stress, leading to a reduction in strength and stiffness and an increase in soil weight. Elevated groundwater from intense precipitation, flooding, or sea level changes can thus lead to lower bearing capacity of soils and increased earth/water pressure loads. Seepage (flowing groundwater) will further increase pore water pressures and can lead to internal erosion and, at its most extreme, static liquefaction. On the other hand, unsaturated soils are appreciably stronger and stiffer than in their saturated state. While this may seem more of a benefit than a hazard, it becomes problematic when field/laboratory tests are done on unsaturated material (perhaps in an area undergoing drought), giving apparently higher values for design, which may be potentially unconservative in the face of changed and increased groundwater conditions, possibly driven by future storms or flooding. Groundwater depletion can also lead to subsidence as pore water is removed and the soil consolidates, which can occur naturally as environmental changes take place or be driven by anthropologic groundwater pumping. Hence, groundwater and seepage will be sensitive to future extreme precipitation patterns, wet/dry cycles, and regional water demand.

3.9.2. Design Guidelines

3.9.2.1. IBC and ASCE 7

Section 1803 of the IBC [51] requires that a geotechnical investigation, when required by the building official, be performed “in accordance with generally accepted engineering practice”. The investigation is supposed to include determining if the *existing* groundwater table is within 5 feet of the lowest finished floor elevation. Further studies should be made “as necessary” that include evaluating the effects of moisture variation on bearing capacity. The IBC also requires designing for lateral soil loads plus full hydrostatic pressure unless wall drains are provided. Additionally, the IBC requires waterproofing or damp proofing of walls/foundations unless a groundwater control system is provided.

ASCE 7 [52] includes groundwater pressure in its load combinations (section 3.2) and, like the building code, requires designing for full hydrostatic pressures below the groundwater table. However, it also provides a method for separating the H load (lateral pressure from soil, groundwater, and/or bulk materials) into separate contributions from soil and groundwater. This requires determining the maximum groundwater level with a given annual probability of exceedance limits (for different risk categories). However, the commentary notes that site-specific information is required to establish hazard-consistent groundwater levels, and obtaining enough of this data can be difficult. It also suggests one can always conservatively set the groundwater level to the ground surface in a non-flood zone and cautions that the groundwater levels may or may not correlate with flood elevations depending on soil permeability and flood duration.

Subsidence is not mentioned directly in ASCE 7 or the IBC. However, its effects would probably fall under the general design requirement to limit differential settlement to acceptable levels.

3.9.2.2. AASHTO BDS and FHWA GEC 5

Section 10.4 of the AASHTO BDS [27] requires that a geotechnical exploration program be “sufficient” to characterize groundwater conditions and provide enough information to evaluate foundation stability and settlement with regard to the above. Like the building code, the BDS requires that earth retaining structures be designed for full hydrostatic pressure below the groundwater table if drainage cannot be provided (though drainage is always recommended when possible). They also recommend that seepage effects on pore water pressures should be included if there are imbalanced water levels on either side of the wall. For seismic design, the commentary recommends that it is more appropriate to use a “time-averaged mean” level rather than a maximum groundwater elevation.

The BDS includes several provisions for incorporating groundwater conditions into foundation design at the various limit states. Shallow foundations are required to be designed for the “highest anticipated” groundwater table, potentially including the effects of seepage forces and uplift. The BDS recommends considering “anticipated changes” in the groundwater level and subsequent effects on geotechnical resistance for deep foundation elements. It also requires

evaluating the potential for down drag loads to develop from the “substantial lowering” of the groundwater table.

The FHWA GEC 5 [26] discusses the potential for time-varying conditions to exist at a project site, and that the predominant cause of change in soil properties is groundwater/variable saturation. The manual suggests a general rule that unsaturated soils are stronger, stiffer, and less permeable than saturated soils, but the exact magnitudes of these depend on matric suction, which is transient and difficult to predict. As such, practice relies on design parameters and pore water pressures established assuming saturated conditions or enforcing saturation in laboratory testing. However, it recommends that field tests and undrained strength tests (in which saturation cannot be controlled) should be interpreted with respect to conditions that existed at the time of sampling, and that the direction of change can be either conservative or nonconservative. It notes that changes in ground conditions over time can be difficult to address, though this can be “reasonably addressed” by monitoring groundwater conditions over time. The chapter on groundwater primarily focuses on the different methods for measuring hydraulic properties (e.g., pumping tests, borehole slug tests, or laboratory permeability tests) and the different technologies available for monitoring groundwater at a site (typically some variant of piezometers/monitoring wells). The manual emphasizes the need to understand time-varying groundwater conditions and cites primary factors that drive this: seasonal variations in precipitation, groundwater pumping in the vicinity, and changes in nearby surface water levels. In the section on developing design parameter values, the manual recommends using “worst case groundwater conditions that may rationally arise during construction or operation”. These are unlikely to be the same as those during site exploration. The manual suggests establishing these “rational worst case” conditions using a combination of site observations, modeling, and (considerable) engineering judgment.

3.9.2.3. USBR DS 13

Seepage is a major design consideration for embankment dams. Many historical embankment failures have resulted from seepage-induced internal erosion and excessive pore water pressures. Pore pressures and seepage forces (which are influenced by spatial changes in total and pressure heads) are typically evaluated through flow nets below the embankment dams. The USBR Design Standards [29] require that seepage must be controlled to prevent excessive uplift pressures, softening of embankment/foundation material, and erosion of embankment/foundation materials. As a general protective measure, nearly all embankment dams are constructed as one of the cross sections shown in Fig. 3.

The design procedures assume that seepage takes place in water-retaining embankments. Theoretically, homogeneous fills do not necessarily have homogeneous seepage flow. Seepage can progress unexpectedly along cracks or horizontal stratifications (typically resulting from compaction). As previously mentioned, compacted embankments can have a difference in horizontal and vertical hydraulic conductivity. As a result, current USBR practice is to always incorporate seepage control measures, even in homogenous sections, unless very specific exceptions are met. Additional foundation treatments, such as grouting, cutoff walls, or relief wells, are sometimes provided to remove preferential flow paths and limit under seepage.

A primary seepage control measure used within the embankment is protective filters, an example of which is shown in Fig. 15.

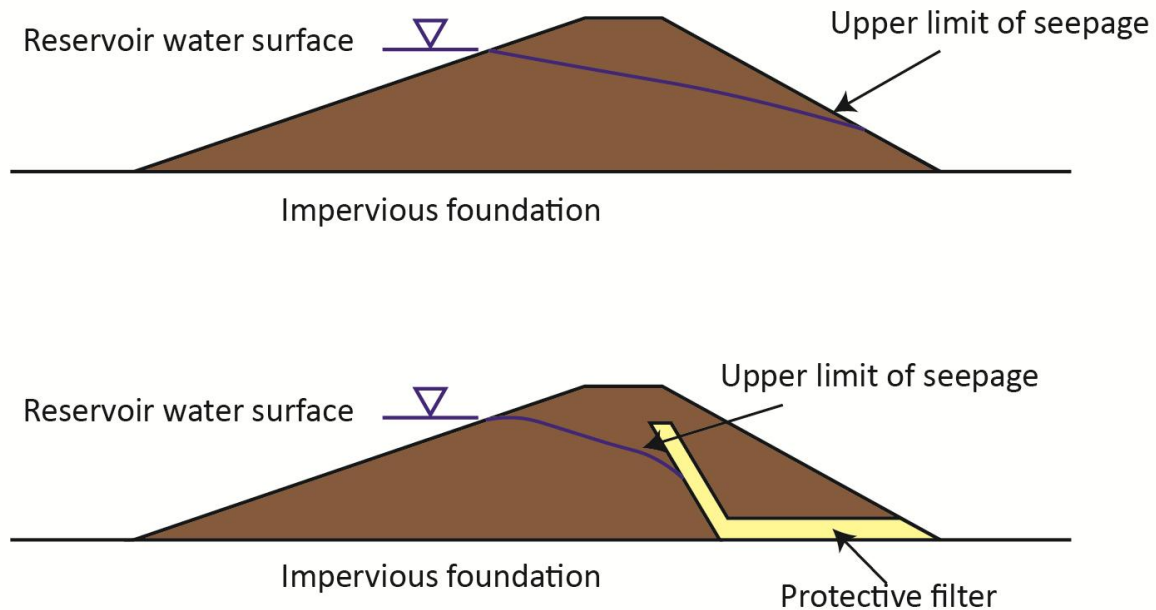


Fig. 15. An illustration of how protective filters/drains intercept through seepage in embankment dams. Reproduced from [29]. The upper limit of seepage is defined as the upper limit of the potential future phreatic surface when seepage reaches a steady state.

Filters are provided to intercept seepage through embankments and help prevent internal erosion. They are also provided as a hedge against cracking (both unanticipated and from seismic deformation), material variability and particle mobility (specifically with regard to permeability and erodibility), and uncertainties in hydraulic analyses. The USBR design philosophy assumes explicitly that some form of cracking or other defect will be present. Filter materials are specifically tested for their ability to heal or prevent the progression of internal cracking. From experience with past failures, recommendations are provided to ensure:

- Filters can be constructed to the required dimensions
- Filter material can be placed uniformly and continuously
- Filters are protected against contamination by fines
- Filter material can be compacted and moisture conditioned to specifications
- Adequate monitoring and quality control are provided during construction

The required filter thickness is determined from seepage analysis, and to maintain the operation of the filter after seismic deformation (conservatism is recommended in both calculations).

The USBR design guidelines require performing numerical analyses to provide pore pressure for slope stability analyses (discussed in section 3.8.2.3) and to evaluate seepage quantities and related failure modes. The manual emphasizes that: “predicting seepage behavior is difficult due to the many variables involved... and the often complex nature of foundation conditions.” The results are considered as being able to estimate the order of magnitude of seepage behavior.

Seepage analyses require establishing the hydraulic properties of the embankment/foundation soils and setting appropriate boundary conditions. Permeability is estimated using published data, laboratory testing, and field testing. Permeability has an extremely wide range of values across soil/rock types. The manual recommends focusing on embankment and foundation soils with the highest permeability, which will control seepage results. It may also be variable in the horizontal and vertical directions within rocks, native soils, and constructed embankments. The manual emphasizes that anisotropy (different permeabilities in horizontal and vertical directions) is very important for design, particularly when horizontal flow is faster (e.g., from compaction in an embankment, or stratification of natural soil deposits). Modeling seepage in partially saturated soils requires establishing a relationship between the fully saturated and partially saturated permeability as a function of soil porosity and water content. This is often considered more time-consuming to establish from laboratory testing, and engineers will often use default curves for transient analyses. The manual recommends that boundary conditions (e.g., reservoir water levels) should correspond to various stability analysis cases. Steady-state conditions are usually assumed even if seasonal fluctuations are expected, though transient analyses may be performed for floods or other short-lived loadings.

The USBR also recommends ongoing monitoring of seepage. The suggested methods include piezometers and other instrumentation (e.g., observation wells) to measure pore pressures within the embankment, measurement of seepage quantities, and measuring turbidity (suspended sediment) to identify if internal erosion of material is occurring. The manual also suggests complementing these quantitative measurements with routine visual evaluations and inspections performed by a qualified professional. The manual suggests that the frequency of these inspections should be tied to the risk posed by a dam failure. The data collected in these monitoring efforts is used to update the likelihoods of potential failure modes in the Bureau’s risk analysis process.

3.9.2.4. Additional National Level Guidelines

The UFC DM 7.2 [53] notes that proper drainage, both surface and subsurface, is a critical (but sometimes overlooked) aspect of fill construction. Deep fills should be designed for some degree of future wetting from climate and human activity. The manual recommends flow nets or numerical seepage analyses to calculate increases in lateral loads on earth retaining structures from imbalanced water levels (e.g., for waterfront structures or during rainfall), but does not specify how to develop hydraulic boundary conditions for these analyses. It also requires that foundations resist “maximum probable” uplift pressures and that design should consider for this load case should consider movements from groundwater fluctuations. For deep foundations, downdrag analyses are required for lowering of the groundwater table, but

the manual notes “these sources of loading are ... difficult to predict”. Finally, UFC 3-220-1 [128] requires providing the “stabilized” groundwater elevation with anticipated local variability and its causes (e.g., seasonal, tidal, etc.). Its design load criteria also include assessing potential future flooding or dewatering.

3.9.2.5. Notable State and Local Amendments

Alaska. The City of Anchorage Amendments to the IBC [75] require that the subsurface investigation identify the location and elevation of water.

California. The Caltrans Geotechnical Manual module on groundwater [129] notes that seasonal variations in precipitation, pumping from nearby wells, and surface water level (e.g., streams and tides) fluctuations can result in complex groundwater behavior. The recommended extent and detail of groundwater characterization depend on project constraints -- some projects may require continuous or multiple measurements. The manual suggests that ground evaluations should include reviewing historical data, data from nearby sources, and physical measurements taken during site investigation. The information collected should be “sufficient” to support design.

Florida. The FDOT Soils and Foundations Handbook [130] recommends summarizing seasonal high and low groundwater levels in geotechnical reporting.

Illinois. The IDOT Geotechnical Manual [71] states that long-term groundwater monitoring (via piezometers or wells) may be required for some structure designs, but the 24-hour delayed reading is often required. Geotechnical reports summarize climatic conditions for (at least) the previous three months and discuss the “fluctuation effects of the departure from normal precipitation on observed groundwater conditions”. The geotechnical engineer is also required to classify drainage characteristics for pavement design, reproduced in Table 24.

Table 24. Qualitative pavement drainage categories used by the IDOT

Drainage Classification	Characteristics
Good	<ul style="list-style-type: none"> • Water table (temporary or permanent) is low enough that the subgrade will not become saturated • Surface water can be removed quickly enough to prevent subgrade saturation • No other conditions that will produce saturation/instability of the subgrade
Fair	<ul style="list-style-type: none"> • Possibility of a high temporary water table • Possibility of slow surface drainage • No other conditions that will produce saturation/instability of the subgrade
Poor	<ul style="list-style-type: none"> • Possibility of a high temporary water table • Possibility of slow surface drainage

	<ul style="list-style-type: none"> • Possibility that saturation will produce instability of the subgrade
Very Poor	<ul style="list-style-type: none"> • Permanent water table is high • Surface drainage is slow • Saturation will produce instability of the subgrade

Texas. The TXDOT Geotechnical Manual [69] states that groundwater should be measured during/after drilling. Site conditions and design criteria may require establishing a “long-term” or “steady-state” groundwater condition by piezometer monitoring.

3.9.3. Gaps in Practice and Standards

Measurement of groundwater (and, to some degree, investigation of historical conditions) is considered a key component of a geotechnical exploration and reporting document in nearly every document reviewed. The documents devote much attention to the various methods of field measurement/monitoring techniques or data sources available to engineers. However, they spend considerably less time giving explicit instructions on how to use the data collected in design:

- Standards rarely say precisely how to select a groundwater level, often leaving it to the engineer's judgment based on data collected during site investigations. Two primary exceptions exist: the (optional) risk-based design level from ASCE 7 and the requirement to design for “highest anticipated” conditions for shallow foundations per the AASHTO BDS. A secondary exception is specific instructions provided for liquefaction assessment, which are discussed in the corresponding section of this report.
- However, the optional risk-based ASCE 7 design levels do not have a corresponding quantitative procedure for determining them. The commentary notes that they can be difficult to establish in practice.
- Additionally, it is not clear what “highest anticipated” means or if it can be established purely based on levels measured during site exploration and limited historical data. A methodology for incorporating future conditions into its estimation is not provided.
- Engineers are frequently instructed to “understand” or “consider” the potential for variations in groundwater levels, both seasonal and long-term. However, quantitative procedures for forecasting said changes are not provided. Further, it is unrealistic to have sufficient data to predict long-term changes when performing an investigation that is limited to a specific short-term project (commonly, geotechnical investigations may be limited to days, weeks, or months, and sometimes years, which are normally insufficient for defining long-term changes in groundwater and seepage).
- Detailed seepage analyses are performed routinely, particularly for dam safety evaluations. However, these analyses will (at a minimum) only be as good as the water surface elevations provided by the hydraulics analysis. Which, in turn, will be dependent on future conditions.

- There are general code requirements that locally, foundations provide adequate bearing capacity, while minimizing settlement and differential settlement. These considerations influence the design of the foundation that transfers the load from the structure, fill, or adjacent structures to the subsurface locally. However, none of the reviewed documents guides how to estimate regional subsidence trends and incorporate them into the design.
- Monitoring pore pressures and seepage quantities is identified as an important aspect of risk management for embankment dams/levees. Many documents (e.g., [127]) provide recommendation for selecting instrumentation and developing monitoring plans. However, considerations for or modifications to monitoring plans in the context of potentially changed future conditions are not explicitly provided.

3.10. Research Progress and Needs

This section highlights some recent research that discusses the incorporation of future hazards into analysis and design procedures for the geotechnical hazards addressed in this report.

Effects of seasonal variations in temperature and moisture (without relative humidity, 'RH') have been studied on swelling and shrinkage of expansive clays (e.g., [131]; [132]). These studies showed irreversible changes in swelling/shrinking and fabric ([133]; [134]; [135]). Several researchers have also considered the influence of temperature, specimen dimension, initial water content, RH, and wet-dry (W-D) cycles on crack formation ([136]; [137]; [138]; [139] among others) as well as post-crack soil properties. However, all of these studies were confined to specimen sizes less than 50 cm at 1 g, and boundary conditions that do not represent field conditions. The prior numerical and theoretical models of desiccation cracking (e.g., [137]) are also limited due to uncertain or non-existent boundary-value physical models for their validation.

The impact of medium- and long-term seasonal W-D cycles has been studied on the performance of clay slopes in the centrifuge (e.g., [140]; [141]). These tests did not include simultaneous thermal, moisture, and RH variations that characterize drought-rainfall cycles in the field, nor desiccation behavior. These gaps, in particular, the lack of sufficient studies at the boundary-value scale, have led to unclear guidelines related to these changes in thermo-hydro-mechanical soil properties and desiccation cracking under environmental extremes.

As emphasized by the 2021 Arctic Report Card [142], both recorded historical observations and future projections consistently highlight the widespread permafrost degradation. Soil properties undergo fundamental changes, as soils in previously permafrost regions become particularly vulnerable to mass movements stemming from the release of unfrozen groundwater, weak soils, and erosion [143]. These considerations elevate the urgency of an enhanced understanding of the evolving soil behavior in Alaska with thawing permafrost.

While the physics of freeze-thaw in soils is well-understood, important gaps remain. For example, soil and permafrost properties are key modeling parameters that exert a strong control on the distribution of ice and water as well as the movement of water and heat [144]. Yet, current knowledge is limited on how these hydro-mechanical properties of thawing

permafrost evolve under warming conditions. Additionally, there is limited understanding of how water and heat move in variably saturated soils during freeze-thaw under non-equilibrium conditions ([145]; [146]). Prior element-level tests (e.g., cyclic triaxial tests) on frozen granular soil samples have shown an increase in shear modulus with decreasing temperature (e.g. [147]). The damping ratio, however, decreased at lower temperatures. Soil shear stiffness can change by a factor of two during the freeze-thaw process, altering seismic site response ([148]; [149]). Larger-scale centrifuge tests involving frozen soils have only investigated heave or settlement caused by freeze or thaw ([150]; [151]), without consideration of other subsequent loading scenarios. Such considerations in experimental, analytical, and numerical models can help develop clear guidelines for practice, related to the static and dynamic response of freezing-thawing soils.

Finally, while slope stability analysis methods for rotational/translational landslides are routinely performed in practice, some gaps remain for assessing the potential for and consequences of debris flow. Simulation tools (e.g., [152] or [153]) are available for engineers; however, research is needed to develop clear guidelines on appropriate rainfall intensities to consider (particularly with regards to future conditions) and how the soil properties used should be modified for post-fire conditions.

4. Conclusions

4.1. Evaluation of the State of Practice

After completing this review of national and state/local level geotechnical guideline documents, several key conclusions are apparent. First, existing standards have three levels of coverage for potential effects of future hazards: explicit (prescriptive) requirements, implicit (indirect) requirements, or no requirements. However, a complete lack of requirements is rarer than explicit or implicit requirements. Because geotechnical design almost always requires at least some consideration of groundwater levels (and potential variation thereof), nearly all geotechnical design procedures reviewed explicit or implicit requirements. The cases with a lack of requirements apply primarily to geohazards such as debris flow, where the hazard is both physically complex and geographically limited. This results in designs based on qualitative risk management and local expertise, as opposed to manualized approaches. Therefore, the discussion provided in this report is limited to the state of geotechnical practice explicit and implicit requirement categories in this section, with some representative examples.

4.1.1. Explicit Requirements

In this report, we define explicit requirements as (reasonably) unambiguous statements that specifically identify expected consequences of future hazards as a required aspect of a detailed and quantitative geotechnical design procedure. Such requirements are rarely present in the national and local recommendations. In fact, among the documents reviewed, the number of instances of occurrence is few enough to fit into the following list and is limited to aspects of sea level changes:

- ASCE 7 requires adding relative sea level changes to tsunami design heights.
- Various DOT highway and hydraulic design manuals incorporate relative sea level changes in aspects of coastal highway design, such as road/bridge geometries or coastal erosion mitigation measures;
- Corrosion design manuals often suggest considering the effects of sea level changes and saltwater intrusion;
- DoD foundation design criteria in arctic and subarctic conditions require thermal modeling using projected future conditions as boundary conditions to evaluate temperature profiles and maximum thaw depths over a project's design life;
- Army Corps flood risk assessment and levee design guidelines incorporate relative sea level changes and study-specific forecasts of future meteorological conditions.

There is no explicit statement, guideline, or procedure regarding how other aspects of future hazards impact soil properties or the geotechnical design approach (e.g., changes in temperature, relative humidity, permafrost, or flood/wind patterns and their relation with soil properties and performance). However, even though the existing criteria specifically aim to incorporate sea level changes into geotechnical building code-based design, they do not

provide a detailed procedure. For example, there are multiple sources for global and United States-specific sea level estimates (e.g., [154] or [155]). These estimates may require downscaling to obtain local estimates and must be combined with other sources of vertical land movement (e.g., uplift or subsidence from tectonic activity, groundwater use, or other sources) to obtain estimates of relative sea level change at the site/area of interest. Additionally, the relative sea level change estimates are tied to different modeled assumptions regarding forecasted socioeconomic growth and greenhouse gas emission patterns. Even for a straightforward design procedure where a single point estimate of relative sea level change is used, the design team currently must decide which one of these scenarios is “appropriate” for the project. It should be noted that although DoD and USACE design methodologies do include various projected future conditions, such procedures are not yet required in more common building code-based geotechnical design. It is also acknowledged that this is still an active research area, and codes and standards often substantially lag research and early application.

4.1.2. Implicit Requirements

Although instructions that specifically mention aspects of future hazards, like sea level changes or changing precipitation patterns, are typically lacking, many existing design requirements and design guidelines still require consideration of geotechnical factors such as groundwater table/soil saturation, seepage, and hydraulic boundary conditions, or erosion/scour potential. All of these will be affected by hazards such as extreme precipitation, flooding, or sea level changes to some degree. As a result, many of the documents could potentially be read to have an implicit requirement to incorporate the effects of future hazards. For example, in a region with sea level changes and/or more frequent extreme precipitation events, the “highest anticipated”, “seasonally averaged”, or “associated with winter storms” groundwater elevations would need to incorporate future projected conditions. Indeed, any design philosophy that instructs engineers to understand how uncertainty and variability in subsurface conditions (e.g., [26]) can affect design and construction, and could be broadly construed to include a requirement to consider future hazard effects, given that these will primarily manifest in uncertainty and variability in (future) subsurface conditions.

However, the challenge with this interpretation is that it entirely relies on the engineer’s reading of the code language and access to or knowledge of other models. How practitioners or reviewing agencies interpret these statements will ultimately dictate if and how these potential future hazards design requirements translate to the state of practice. Given that the vast majority of design procedures fall into this implicit requirement category, there is a clear need to better understand the day-to-day interpretation and application of these aspects of the recommendations. This is likely a highly non-trivial task – all the challenges related to the amount of and variation between guideline documents will also apply to consulting firms and public agency engineering departments, because of differing capability, access, and experience. A major challenge will be to not only understand what is being practiced on the highest profile and largest budget projects, but also on more routine work. These efforts will likely involve some combination of surveys, workshops, and practitioner interviews – all of which can be guided based on the information contained in this report.

A secondary case of implicit requirements appears where the geotechnical design relies on the inputs of hydrologic/hydraulic analysis, such as seepage modeling or scour/erosion evaluation. When these inputs have been modified to account for future hazards (such as precipitation or flooding magnitude and frequency), the geotechnical design will implicitly incorporate these considerations for future hazards and conditions. However, this requires the procedures for estimating these non-geotechnical components to adopt modifications that explicitly incorporate future hazard effects. While a comprehensive summary of the state of practice in this area is not within the scope of the report, an overview is provided by the ASCE 7-22 commentary, which states, “the hazards in this Standard do not reflect potential effects of climate variability and climate change.”

4.2. Ambiguity in Design Guidelines

The documents reviewed include an implicit or explicit requirement to incorporate aspects of future hazards into the design for geotechnical infrastructure or hazards. Yet, these documents seldom provide procedures for doing so. Additionally, the level of detail given in design procedures varies substantially from document to document. The design guidelines are classified into six categories to summarize this variation in Table 25.

Table 25. Categories for the level of detail provided in design guidelines for various geohazards.

Category	Symbol
Does not directly mention the geo-hazard	i
Requires that the hazard is identified and addressed in geotechnical exploration and design but only in limited design scenarios such as only for tsunami-resistant design. Does not provide a specific design procedure.	ii
Requires that the hazard is identified and addressed in geotechnical exploration and design. Does not provide a specific design procedure.	iii
Requires that the hazard is identified and addressed in geotechnical exploration and design. Qualitatively addresses the hazard by construction detailing and quality control, but not through explicit design processes.	iv
Requires that the hazard is identified and addressed in geotechnical exploration and design. Provides at least one detailed evaluation/design procedure.	v
Requires that the hazard is identified and addressed in geotechnical exploration and design. Provides at least one detailed evaluation/design procedure that specifically incorporates effects of future hazards.	vi

The assigned categorizations are provided in Table 26. Care should be taken not to overinterpret these categorizations. In many cases, when a document does not devote substantial discussion to a certain geotechnical hazard, it is because the hazard is often not a major threat to the type of structure the document covers.

Table 26. Level of detail in design procedures provided by the national standards reviewed, with consideration for the incorporation of future hazard effects.

Geohazard/Document	IBC/ASCE 7	USBR DS13	AASHTO BDS/FHWA GEC 5
Expansive soils	v	iv	v
Collapsible soils	iii	v	iii
Permafrost	iii	i	iii
Desiccation cracking	i	iv	i
Corrosion	iii	i	v
Liquefaction	v	v	v
Scour and erosion	ii	iv	v
Tsunami	vi	iv	i
Landslides	iii	v	v
Rockfall	i	i	iii
Debris flow	i	i	i

The level of variation in guidelines identified in Table 26 can result in ambiguity, nonuniformity, and uncertainty in practice. In particular, this can occur when the agency having jurisdiction over a given project provides fewer design guidelines relative to those provided for other types of projects. It is not immediately obvious which design recommendations from other documents can be adopted (with or without modification) while remaining in compliance with the design philosophy of the governing code, standard, or guideline.

4.3. Future Needs in Research and Regulations

Even when consideration of future hazards is (explicitly or implicitly) required, accompanying quantitative analysis procedures are often lacking or not present. This presents a unique opportunity to develop forward-thinking design procedures together with empirical, analytical, and numerical models, building off rapidly advancing work by climate and geotechnical/geomechanics experts, which is described in the following subsections.

4.3.1. Inputs to Geotechnical Models

In many cases, the future effects of long-term weather patterns on surface and subsurface water conditions from changes in extreme temperatures, flooding, and sea level changes come in the form of input parameters to the assumed geotechnical model. For example:

- Seepage modeling requires assumptions for hydraulic boundary conditions (e.g., upstream and downstream water levels or infiltration rates from precipitation);
- Slope stability analysis requires assumptions for pore pressure conditions, which are typically derived from seepage modeling;
- Foundation capacity and settlement evaluation requires assumptions for a groundwater level;
- Scour/erosion analyses require flow velocities derived from a hydraulic/hydrologic analysis;
- Thermal analyses, such as modeling thaw of permafrost or evaporation/desiccation, require specifying a temperature at the ground surface (which may potentially vary with time and depth).

Ultimately, this becomes a question of forecasting: “over the structure’s intended operational period: what will groundwater conditions look like? How will groundwater respond to rising sea levels or changing weather patterns? What will the flood flows look like? How much soil might be lost to erosion?” Developing rigorous answers to these questions is beyond the capacity and training of the geotechnical engineer alone and requires expert input from diverse fields like city planning, climate science/metrology, hydrologic/hydraulic engineering, coastal engineering, and hydrogeology. For the cases of loads transmitted to foundations, hydraulic boundary conditions (i.e., reservoir levels) on dams and hydraulic structures, or design flood flows, these inputs are traditionally provided to the geotechnical engineer by other members of the project team. The FHWA HEC 19 [156] summarizes recent research and provides readily applicable design methods for incorporating projected future conditions into various hydrologic and hydraulic calculation procedures routinely performed for transportation projects. The manual describes how tools such as the Coupled Model Intercomparison Project (CMIP) tool [157] and NOAA Atlas 14 (with modifications for nonstationary) [158] can be used to develop project-specific estimates of meteorology under future changed conditions. The Army Corps of Engineers also provides several documents for incorporating estimates of sea level change and future weather into project design (e.g., [57], [58], [159], [160]). Similarly, for evaluating the thaw potential of permafrost, the University of Alaska Institute of Northern Engineering [87] practice synthesis and Department of Defense UFC [80] describe how projected future conditions can be downscaled and incorporated as boundary conditions in thermal models.

However, in other cases, such as selecting design groundwater levels (for various load cases), the responsibility falls to the geotechnical engineer. In this situation, things become more complicated when one needs to forecast future conditions rather than rely on historical data or limited observations during site exploration (neither of which may be representative of future conditions). Groundwater levels at a given site will result from the complicated interactions

between regional groundwater flow, regional aquifer depletion and recharge, local infiltration from precipitation, pumping or injection in the site vicinity, and surface water levels. Additionally, changes in the saturation degree above the water table are highly influenced by fluctuations in temperature and relative humidity. Further, the exact time-dependent response of a site to changes in the above factors will depend on the soil/rock present, their hydraulic properties, and demands.

Rigorous quantification of the magnitude and potential time-dependent nature of these fluctuations requires substantial data, even if only a simple extrapolation of a historical trend is desired. For comparison, the NOAA methodology to estimate historical sea level trends in tidal gauge data required a minimum of 30 years of monthly average measurements [161]. One may not want to merely extrapolate from historical trends but also include climatic forecasts from other sources. Geotechnical engineers often lack the data, knowledge, and resources to develop such models for groundwater projections. In summary, a process is needed for integrating limited site observations with numerical groundwater models incorporating boundary conditions derived from projected future conditions, accounting for underlying uncertainties.

In summary, several research needs are identified as priorities:

- Usable forecast models (tied to various RCP's) of conditions such as temperature, precipitation, relative humidity, and sea level changes need to be downscaled to a site-specific level for code implementation. A clear and consistent methodology for integrating these forecasts with the philosophy and methodology of geotechnical code-based design procedures is needed. In particular, the process should address the various uncertainties in projected future conditions related to inherent system variability, uncertainty regarding how components of the models (e.g., land, air, and water) respond to changes in conditions, and uncertainty regarding future trends in greenhouse gas emissions as a result of economic, social, and administrative factors.
- When the design aspect is related to but not directly predicted by these models (e.g., groundwater level or degree of saturation), a methodology for developing these forecasts that is consistent with the intent of the codes and current state of science is needed.
- These procedures should be code-compatible in the sense that they can be readily understood and put into use by practitioners and reviewers, ideally in a probabilistic framework.

4.3.2. Fundamental Characterization of Soil Hydro-Thermo-Mechanical Behavior

Standard field and laboratory tests are quite capable of quantitatively characterizing soil strength parameters, the potential for volume change (e.g., expansion, collapse, or settlement), and qualitative corrosion to the degree necessary for the relatively simple constitutive models used in routine practice. However, these tests are usually performed on undisturbed samples at their natural water contents or specimens reconstituted to a standardized density and moisture content (typically a state consistent with design material specifications). For several

geotechnical hazards, it is already known that the initial state of the soil (density, saturation, etc.) will affect the magnitude of its potential for volume change, strength loss, or other deleterious effects. In other cases, it is not fully known to what extent multiple wetting/drying cycles and/or large temperature fluctuations will affect soil strength and stiffness, as well as its constitutive behavior under static or cyclic loading. These gaps in the characterization of properties and the response of variably saturated soils persist under compound and cascading or -seismic hazards.

It is not obvious that in-situ or standard remolded states, or in cases where the saturation degree is a question, full saturation will always represent a worst-case scenario, particularly with future hazards potentially altering the extent of the active zone and/or subsurface temperature profiles. In some cases, it may be possible to prepare specimens to a reasonable worst-case condition (however defined), considering future hazards and conditions. However, this could potentially result in very conservative designs with potentially unknown probability levels. Alternatively, the soil behavior over various states could be characterized for use in a performance-based engineering procedure that incorporates uncertainty in future saturation regimes. However, to use this in design would require forecasting the magnitude and frequency of wetting/drying cycles and those expected to occur over the design life, which is not currently given in any of the documents reviewed. In summary, several research needs are identified as priorities:

- A better ability to quantify the effects of wet/dry cycles and temperature variations on soil constitutive response is needed. This will likely require insights from laboratory element-level tests and boundary value level centrifuge/field studies in which realistic conditions can be simulated. These results can be used to calibrate coupled thermo-hydro-mechanical numerical models for forward analyses.
- However, performing fully coupled thermo-hydro-mechanical analyses is not practical for most design cases. Instead, numerical parametric studies with a variety of realistic soil profiles and conditions can be performed and analyzed to develop simplified models that are more suitable for routine design. These models should be code-compatible in the sense that they can be readily understood and used by practitioners and reviewers, ideally in a probabilistic framework.
- The above research paths should distinguish between single and compound hazards.

4.3.3. Performance-Based Predictive Models

Most extreme event design procedures currently require developing a design event with a specified return period, such as a 500-yr flood or a 2,475-yr earthquake ground motion. This will, theoretically, result in a structure that can safely (i.e., with an appropriate factor of safety or target reliability) support the factored loads from the hazard level plus the dead/live loads that might be present. However, this single case analysis can pose difficulties when one wants to incorporate a material degradation process, such as erosion/scour or corrosion of structural materials, or analyze compound hazards like subsidence, flood, or fire following or prior to even a minor earthquake event. Because degradation is inherently cumulative, the question of “how

much material degradation should be combined with extreme event loadings?” can only be answered by “when will the extreme event occur?”. In a similar vein, analyzing compound hazards requires asking “what is the probability of compound and cascading events, with characterization of different compound hazard intensities, sequence, and time gaps,” and “what condition will the structure be in when this happens?”. Even for static load cases, many of the challenges associated with integrating future hazards in geotechnical design procedures can also be phrased as questions of uncertainty: “What is a ‘reasonable’ groundwater level to design for?”, “Should I combine desiccation cracking with high water levels in a seepage analysis?”.

‘Consequences’ may be fair, but then things are a little mixed up. Essentially, there is a structural analysis incorporating all the things stated up to that point. From the structural analyses, there are damage measures, which are essentially fragility functions. These are a type of consequence. Then there is the loss calculation based on the damage, which includes the downtime, cost, and casualties (casualties include injuries and death - injuries are not included if you only refer to the 3-D’s).

Performance-based engineering (PBE) (e.g. [162], [163]) rigorously quantifies the uncertainty in loads, material properties, structural response, and consequences of said structural response. These consequences are often defined in terms of damage measures (fragilities), which are then used to estimate loss, including dollars, downtime, and casualties [164]. The final product of this analysis is a probabilistic characterization of the potential losses for a given (geo)structural configuration and operational life, which can help analyze the suitability of the current design. More recent efforts toward transformative resilience concepts also consider social vulnerability and consequences in terms of community well-being. These considerations include broader aspects of infrastructure, including all types of buildings, utilities, and transportation, as opposed to the individual project-based (e.g., building or embankment) or component-based (e.g., foundation) concepts addressed in most of this report.

The method for obtaining an answer to the previously posed questions, from a PBE perspective, is straightforward. For example, consider the question of a “reasonable” groundwater level for a foundation bearing capacity analysis. Instead of selecting a single value, the analyst characterizes the uncertainty in future groundwater or saturation conditions at the site and combines that with the estimated uncertainties in the loads or conditions transmitted to the foundations and properties of soil within the zone of influence. However, as discussed in the previous section, the uncertainty in the groundwater levels or saturation degree above the water table is not so simple. In particular, it is highly unlikely that the levels at any given time are independently lognormally distributed with a given mean and standard deviation (a common assumption for loads/resistances), particularly when incorporating long-term weather patterns. The mathematics of this procedure can be quite complex. If loads or material properties are non-stationary in time, many standard assumptions of probabilistic hazard analyses are violated. Additional work is needed to properly account for non-stationarity, to make the procedure applicable for use in geotechnical practice. Unfortunately, this condition is most likely to occur when considering the effects of long-term weather patterns (e.g., increasing groundwater levels from sea level changes). Further, additional uncertainties

associated with soil's hydro-thermo-mechanical properties under those loading conditions affect the reliability of such performance predictions.

In summary, the following is identified as a research priority in this area:

- The principles of PBE can be used to develop complex, time-dependent models to analyze the lifetime reliability of structures considering nonstationary future conditions and their effects on geotechnical properties, material degradation, multi-hazard loadings, and explicit consideration of associated damage and recovery processes.
- Developing and combining the components of these analyses often requires substantial quantitative expertise, data (e.g., field, experimental, and numerical), and time. This often makes them less likely to perform routine design at this time. However, long-term research can be guided in this direction.
- Short-term research could focus on evaluating when (and where) existing code-based design procedures result in adequate or inadequate design for the geotechnical hazards discussed in this report in the context of future hazards and conditions. When existing procedures produce potentially inadequate results, research can suggest modifications (possibly in the form of recalibrated load/resistance factors).

4.3.4. Integration of Adaptation Measures into Code-Based Design

Adaption is defined by [20] as “adjustment in natural or human systems in anticipation of a changing environment in a way that effectively uses beneficial opportunities or reduces negative effects”. Various documents provide a comprehensive discussion of the topic and suggested design procedures. A nonexhaustive list includes:

- FHWA Vulnerability Assessment and Adaptation Framework [24]
- ASCE Manual of Practice 140: “Climate-Resilient Infrastructure: Adaptive Design and Risk Management” [22]
- USACE EP 1100-2-1 [57]

A general framework typically involves (modified from [24]):

1. Define study scope and objectives
2. Obtain asset (e.g., geotechnical and structural) data for vulnerability assessment
3. Obtain future condition data for vulnerability assessment
4. Assess asset vulnerability
5. Identify, analyze, and prioritize adaptation options
6. Incorporate study results in decision-making

In short, adaptation can be conceptualized as a paradigm shift to an “active” management strategy in contrast to the more “passive” typical building code-based design. The goal of an adaptive analysis is to take the most recent data, evaluate the future performance of the geostructure under projected conditions, and develop adaptation measures if such evaluations

reveal that future conditions represent a worst-case scenario (the original design using traditional approaches and unchanged conditions is usually taken as a “base” case). These adaptation measures are then compared in a cost-benefit framework that considers their anticipated performance under future conditions and potential need for further engineering interventions. Importantly, these adaptation analyses are re-evaluated over the geostructure’s lifetime as data and conditions evolve.

Adaptation is also inherently linked to monitoring, which is required to use engineering properties representative of current conditions in geostructure vulnerability assessments. Similarly, many documents also describe best practices in this area (e.g., [165]). However, these documents tend to focus on the technical details of the various types of site instrumentation and their suitability for anticipated subsurface conditions. They often leave the management plan (i.e., what engineering decisions are made with the data collected) to the discretion of the engineer.

In summary, the following is identified as a research priority in this area:

- Adaptation design methodologies are available. However, they have not yet been incorporated into national, state, and local building codes or standards. A key research need is to identify how to incorporate these methodologies into (and potentially required by) building and transportation codes.
- Similarly, technical documentation of available geotechnical field instrumentation and their strengths and weaknesses for different tasks is present in guidelines. A limitation is the lack of explicit and prescriptive methods for designing monitoring plans, guidelines on when monitoring is required for a given structure, and building code requirements for monitoring. A research need is to develop explicit methodologies for developing monitoring and management plans for geostructure that can be systematized and incorporated into codes and design standards. Critically, these methodologies should actively incorporate future hazards and conditions as a primary consideration.

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