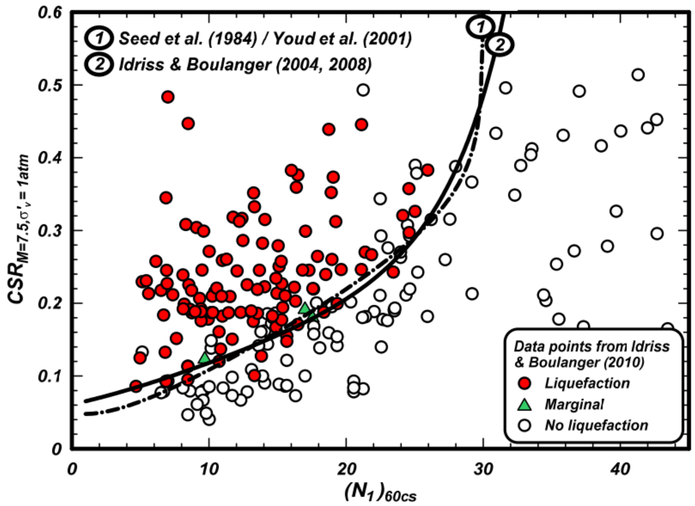

Technical Note 5 - Simplified Seismic Analysis Procedure for Montana Dams

Prepared for:

Montana Department of Natural Resources and Conservation

Water Resources Division

Water Operations Bureau

Dam Safety Program

November 30, 2020

Prepared by:

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Cover figure: Updated database plotted with Idriss and Boulanger 2004 and 2008 SPT-liquefaction triggering correlations (From Boulanger and Idriss, 2014)

1. Executive Summary

The Montana Department of Natural Resources and Conservation (DNRC) has developed this Simplified Seismic Analysis Procedure to help engineers in the state of Montana conduct evaluations for dams under DNRC's jurisdiction. This procedure is an update of a previous simplified procedure developed by the DNRC in 2006 (as described in Lemieux and Grant, 2006). Because of the general low level of seismic hazard for most of the state and the limited financial means for dam owners, the DNRC has not generally required sophisticated and expensive seismic evaluations for most of the dams in the state. Instead, the simplified procedure allows an engineer to more quickly decide if additional investigations and higher levels of analysis are warranted, or alternatively if remedial measures or other risk reduction measures should be taken. This report provides updated notes and guidance to help the engineer with each step within the Simplified Seismic Analysis Procedure.

This DNRC Simplified Seismic Analysis Procedure is intended to be applied for seismic shaking of embankment dams that retain water. It is not intended to address potential fault movements within the dam's foundation or abutments, nor is it to be used to evaluate tailings dams or similar structures.

2. Introduction

2.1. Background

The purpose of this document is to provide engineers in Montana with updated guidelines to the 2006 Simplified Seismic Analysis Procedure to evaluate the seismic stability of embankment dams in Montana. The simplified procedure was originally published at the 2006 Association of Dam Safety Officials (ASDSO) conference by DNRC engineers Michele Lemieux and Brian Grant (Lemieux and Grant, 2006).

The following DNRC documents have been used to develop these guidelines along with other external technical references, which are listed in the References section of this document:

- Montana DNRC Simplified Seismic Analysis Guidelines (Lemieux and Grant, 2006) – referred to as 'previous guidelines.'
- Independent External Review of the Montana DNRC Simplified Seismic Analysis Procedure (HDR, 2016).
- Assessment of DNRC Probabilistic Ground Shaking Maps and Use of ShakeMap in Montana (HDR, 2019).

2.2. Report Outline

The contents of this report are organized as follows:

Section 1: Executive Summary.

Section 2: Introduction and Background.

Section 3: Overview and scope of technical note, need for the simplified seismic analysis, and seismic guidelines from other agencies.

Section 4: Seismic Analysis Procedure.

Section 5: Closing Comments

Section 6: References

Appendix A: Glossary of seismic analysis terms

Appendix B: Simplified seismic analysis flow charts

3. Overview and Scope of Technical Note

3.1. Overview and Need for Simplified Seismic Analysis

The Montana DNRC regulates the safety of non-federal dams in the state of Montana. The majority of DNRC's regulated dams in Montana are small dams that are less than 30-feet tall (DNRC, 2018). The majority of non-federal dams in Montana are owned by ranchers, canal companies, and small communities that have small operating budgets and limited funds for the evaluation, repair, or improvement of their dams. For most of the dams, the downstream areas below the dams are generally rural with low populations at risk. In addition, the majority of the state, largely the eastern two-thirds, is considered to have relatively low seismicity. For example, the United States Geological Survey predicts that half of the state would have a Peak Ground Acceleration (PGA) on exposed rock surfaces of less than 0.1g for a 2,500-year return period, and only about 25 percent of the state would exceed 0.2g for the same return period.

As a result of the low seismic risk and owner limited financial means, the Montana DNRC has not required sophisticated and expensive seismic evaluations for most of the dams in the state until the need is justified. Under these conditions, DNRC developed a three-step simplified procedure for the evaluation of seismic stability of earth dams within its jurisdiction. The three steps in the simplified procedure are as follows:

Step 1: Estimate the seismic hazard potential at each dam site

Step 2: Conduct a simplified analysis of seismic stability

Step 3: Reality Check: consider repair/mitigation versus additional exploration/analysis

Key features of this simplified seismic procedure are:

1. In an environment where performing detailed, sophisticated seismic stability analyses for all high hazard dams is not feasible due to financial constraints, the completion of a simplified screening analysis allows the dam owner to make informed decisions, whether to complete additional analysis or to address issues with rehabilitation.
2. The simplified procedure has been considered to be relatively conservative, simple to use, and employs state-of-practice or state-of-art level information and correlations, as suitable with respect to the seismic stability of dams in Montana.
3. The simplified guidelines have served as a risk analysis tool which is to be applied as a framework for the overall seismic evaluation leading to final decisions. The application of risk assessment procedures in this qualitative and quantitative manner has helped improve the consistency of decisions on the seismic safety of dams.

3.2. Basic Parameters for Performing Scope of Work and Developing Recommendations

The recommendations developed for this report are based on the original DNRC simplified seismic evaluation procedure, recent manuals published by the United States Army Corps of Engineers, United States Bureau of Reclamation, Federal Energy Regulatory Commission, and recent research and reports published by the University of California, Berkeley, University of California, Davis, University of Washington, and other universities as appropriate.

4. Seismic Analysis Procedure

The overall seismic analysis procedure can be divided into six major steps:

Step 1 - Determination of Earthquake Loading

Step 2 - Determination of Dam and Foundation Characteristics and Initial Assessments

Step 3 - Determination of Potential for Significant Loss of Dam/Foundation Soil Strengths through Liquefaction or Cyclic Softening and Determine Residual and Remolded Strengths.

Step 4 - Calculation of Post-earthquake Slope Stability Factors of Safety

Step 5 - Calculation of Earthquake-induced Deformations and Settlements

Step 6 - Assessment of Stability and Deformations

The Simplified Seismic Analysis Procedure (Procedure) is illustrated schematically in Figure 4-1. Each step in the procedure consists of sequential subtasks often followed by a screening question that needs to be completed by the engineer to proceed through the simplified seismic analysis. Subtasks are identified as A through K and screening questions are numbered 1 through 5. Flowcharts for the individual subtasks and screening steps are presented in Appendix B. This chapter describes the six major steps with the associated subtasks and screening questions following the flow chart in Figure 4-1.

To effectively use these guidelines, it is recommended that the flowcharts in Appendix B be printed and reviewed along with the text in proceeding chapters to aid understanding.

4.1. Step 1 - Determination of Earthquake Loading

Step 1 of the Procedure includes two subtasks and a screening question that aims to determine the required level of seismic hazard in terms of return period to be considered (determined in Subtask A) and then the corresponding PGA for the dam at that return period (determined in Subtask B). The determined PGA from Subtask B is then screened to a minimum level of shaking, below which no detrimental effects on embankment dams are expected (Screening Question 1).

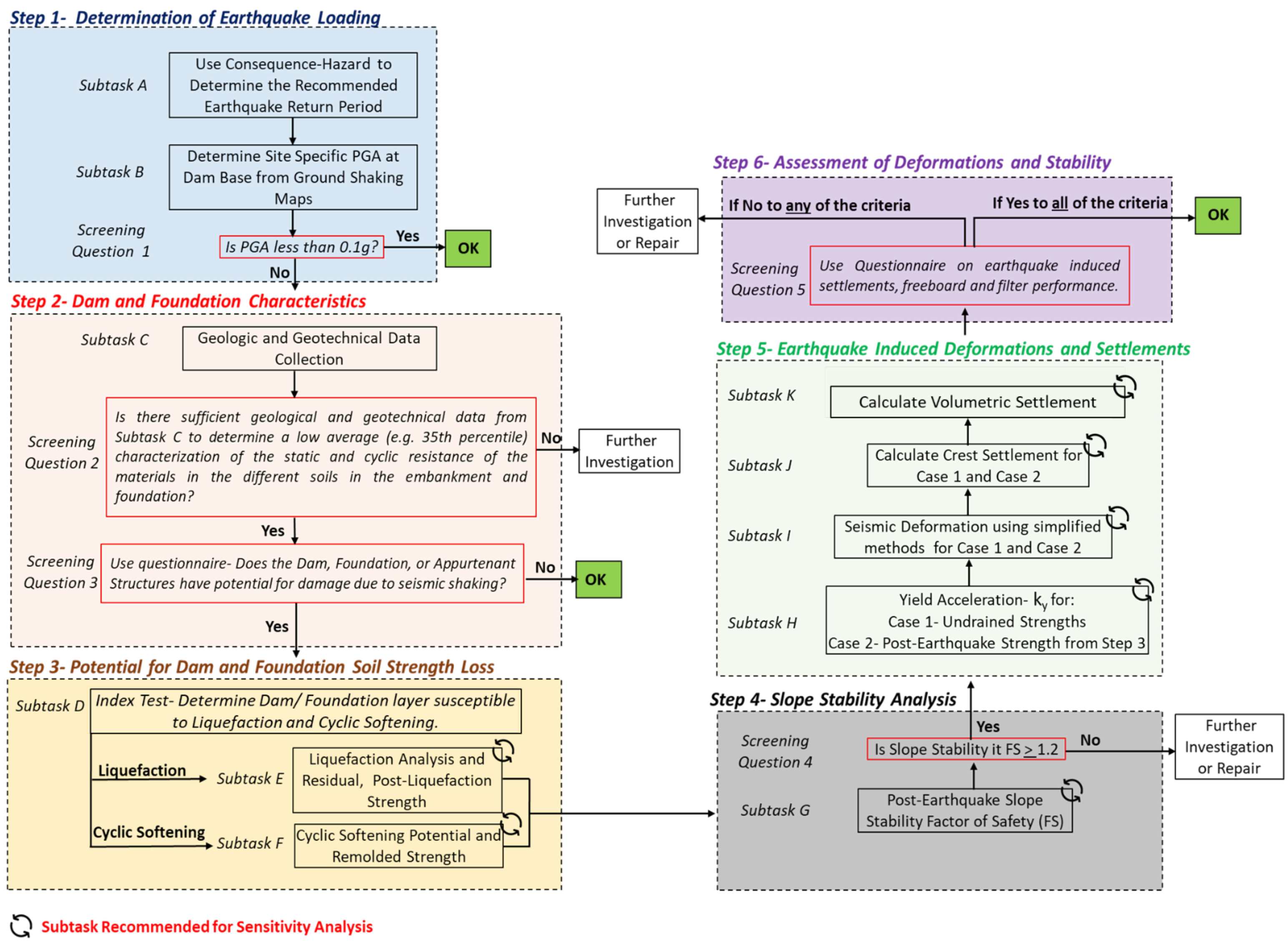


Figure 4-1: Simplified Seismic Analysis

Subtask A

Earthquake Loading or Return Period

Subtask A (Figure B-2 of Appendix B) of the Procedure aims to determine the appropriate earthquake return period and seismic shaking level (PGA) for an embankment dam that reflects the potential consequence level associated with seismic damage. The appropriate earthquake return period for the dam is selected based on: embankment height, reservoir volume, and downstream population at risk using the Consequence-Hazard Matrix presented in Figure 4-2.

Hazard Classification	Height of Dam		
	<50 feet	50 – 100 feet	>100 feet or Reservoir Volume > 5,000 acre-feet
High Hazard	5,000-year	5,000-year	10,000-year
Significant Hazard	2,500-year	5,000-year	5,000-year
Low Hazard	1,000-year	2,500-year	5,000-year

Figure 4-2: Consequence-Hazard Matrix

This risk matrix was developed for the specific purposes of this guidance document and uses the three level hazard classification (High, Significant, and Low) based on potential for loss of life downstream. The definition of these hazard classifications are as follows:

High Hazard

Dams assigned the high hazard potential classification are those where failure or mis-operation may cause loss of life.

Significant Hazard

Dams where failure or mis-operation may not result in loss of life, but can cause economic loss, environmental damage, disruption of lifeline facilities, or other concerns. Such dams are typically located in predominantly rural or agricultural areas but could be located in areas with population and significant infrastructure.

Low Hazard

Dams where failure or mis-operation may not result in loss of life but cause minor economic or environmental impacts. Losses are principally limited to the owner's property.

The recommended matrix is based on hazard classification and height of the dam. As an example, for using this matrix, a dam with Significant Hazard classification and height 70 feet (51-100 feet) would be associated with a recommended earthquake return period of 5,000 years (approximately 1% probability of exceedance in 50-years). PGA is estimated for the current site conditions and not post-seismic conditions. Note that the 10,000-year return period recommended for High Hazard dams greater than 100 feet in higher, or reservoir volume greater than 5,000 acre-feet, is intended to correspond to conditions where there are high populations that may be at risk.

Subtask B

Peak Ground Acceleration

Two sources for estimating Peak Ground Acceleration (PGA) for dams in Montana are:

1. Montana State Probabilistic Seismic Hazard Analysis (PSHA) Study (Wong et al., 2005), and
2. 2014 USGS Seismic Hazard Study

These sources were compared and discussed in detail in the Assessment of DNRC Probabilistic Ground Shaking Maps and Use of ShakeMap in Montana report by HDR (2019). The Wong et al. (2005) Montana and the USGS 2014 studies have advantages and disadvantages. HDR (2019) recommended that engineers employ both studies in developing seismic hazards for Montana dams, and treat them with equal weight.

The general approach using the two methods would be as follows:

Method 1

Montana State PSHA Study (Wong et al., 2005)

1a. If the National Earthquake Hazards Reduction Program (NEHRP) V_{S30} Site Class at the dam matches with the regional geological ground surface assignment in Wong et al, 2005, use the 2005 estimates for PGA for ground surface.

1b. If NEHRP V_{S30} Site Class at the dam does not match, use the 2005 estimates for Soft Rock (V_{S30} = 760 m/sec), and then apply appropriate amplification factors to modify ground motion, as described in Wong et al., 2005

Note: The 10,000-year return period motion can only be determined using a site specific Probabilistic Seismic Hazard Analysis. For a PGA estimate the ratio can be determined as:

$$Factor\ of\ Increase = \frac{5{,}000\ year\ soft\ rock\ PGA\ from\ \text{Wong et al. (2005)}}{2{,}500\ year\ soft\ rock\ PGA\ from\ \text{Wong et al. (2005)}}$$

Multiply the *Factor of Increase* with the 2,500-year PGA estimate from Wong et al. (2005) to arrive at an equivalent 5,000-year soft rock PGA.

Apply this factor of increase twice to arrive at relatively high PGA value to proceed with the simplified analysis for 10,000-year return period. Please note, this method is only an approximation to arrive at a higher ground motion and not intended to reflect the seismologic effects and uncertainty associated with such long return period ground motions.

$$Relative\ 10{,}000\ year\ PGA = 2{,}500\ year\ PGA\ x\ (Factor\ of\ Increase)^2$$

Method 2

USGS, 2014 with additional site class estimates from Shumway et al., 2018.

For 2,500-year estimate:

> **2a.** Estimate the dam site location (coordinates) and use the online USGS Unified Hazard Tool if the dam is founded on soft rock (V_{S30} = 760 m/sec calculator). If the dam is not founded on soft rock, use appropriate Seismic Hazard maps from the USGS 2018 Data Release for Additional Period and Site Class sites from Shumway et al., 2018.

For 5,000-year and 10,000-year estimates:

> **2b.** For a soft rock site, estimate a Factor of Increase in the ground motion estimate (PGA) moving from 2% to 1% exceedance. This can be done by using the Wong et al., 2005 study. For a PGA estimate the ratio can be determined as:

$$Factor\ of\ Increase = \frac{5{,}000\ year\ soft\ rock\ PGA\ from\ \text{Wong et al. (2005)}}{2{,}500\ year\ soft\ rock\ PGA\ from\ \text{Wong et al. (2005)}}$$

Multiply the *Factor of Increase* with the 2,500-year PGA estimate from USGS 2014 to arrive at an equivalent 5,000-year soft rock PGA.

Apply this *Factor of Increase* twice to arrive at relatively high PGA value to proceed with the simplified analysis for 10,000-year return period.

$$\text{Relative 10,000 year PGA} = \text{2,500 year PGA x (Factor of Increase)}^2$$

> **2c.** For all site classes other than soft rock, multiply the estimate above with amplification factors provided in Shumway et al., 2018 to arrive at the ground motion estimate for the required site class.

For 2,500-year and 5,000-year estimates finally, use the average of the two ground motion estimates from **Method 1a OR 1b** and **Method 2a OR 2b OR 2c** to obtain the estimated PGA at the dam site.

For 1,000-year estimate, estimate the dam site location (coordinates) and use only the online USGS Unified Hazard Tool as discussed in **Method 2a.**

The predominant earthquake magnitude, M_w for the site that will be required in later steps of the Procedure can be determined using the deaggregation results of the PSHA calculation shown on the USGS Website.

Screening Question 1

If the estimated PGA at the site of the dam is less than 0.1 g, the dam is considered to be seismically stable and no further evaluations are recommended.

Screening Question 1: Is PGA at the dam site less than 0.1g?

Answer: *Yes* - Dam is considered seismically stable.

No - The analysis must proceed to Step 2 in the simplified analysis procedure.

Support for this screening step is provided by the following studies regarding peak ground acceleration and damage:

1. Seed et al. (1978), Seed (1981) and Seed (1983) documented that hydraulic fill dams in southern California had withstood earthquakes with estimated peak ground accelerations of 0.2g or higher without appreciable damage. These studies went on to conclude that many hydraulic fill dams have performed well when they are built with reasonable slopes on good foundations and can apparently survive earthquake motions up to 0.2g from magnitude 6½ earthquakes with no detrimental effects.
2. The studies by Swaisgood (2003) and Swaisgood (2014) indicate that dams experience generally no damage for earthquakes with peak accelerations of 0.1g or less (see Figure 4-3).

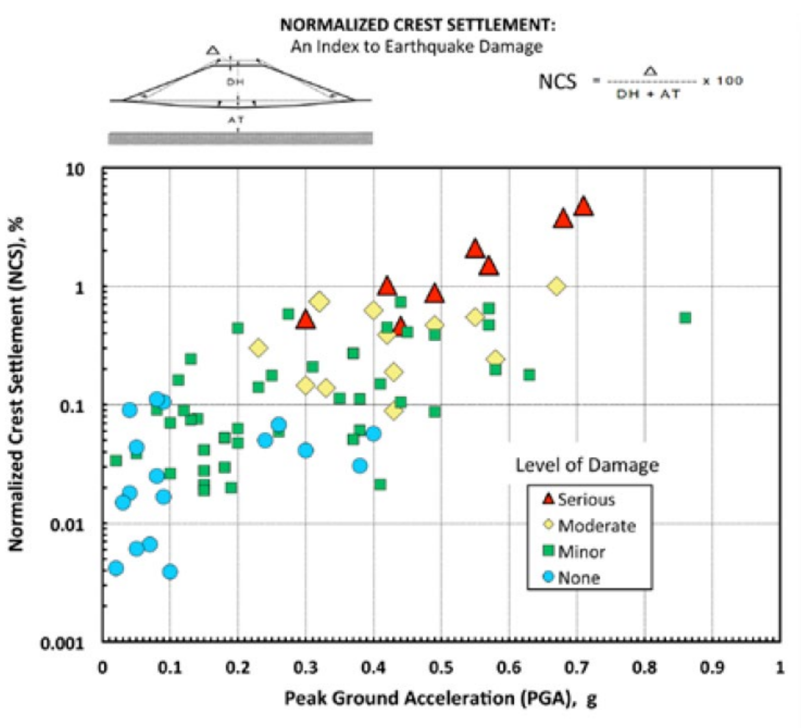
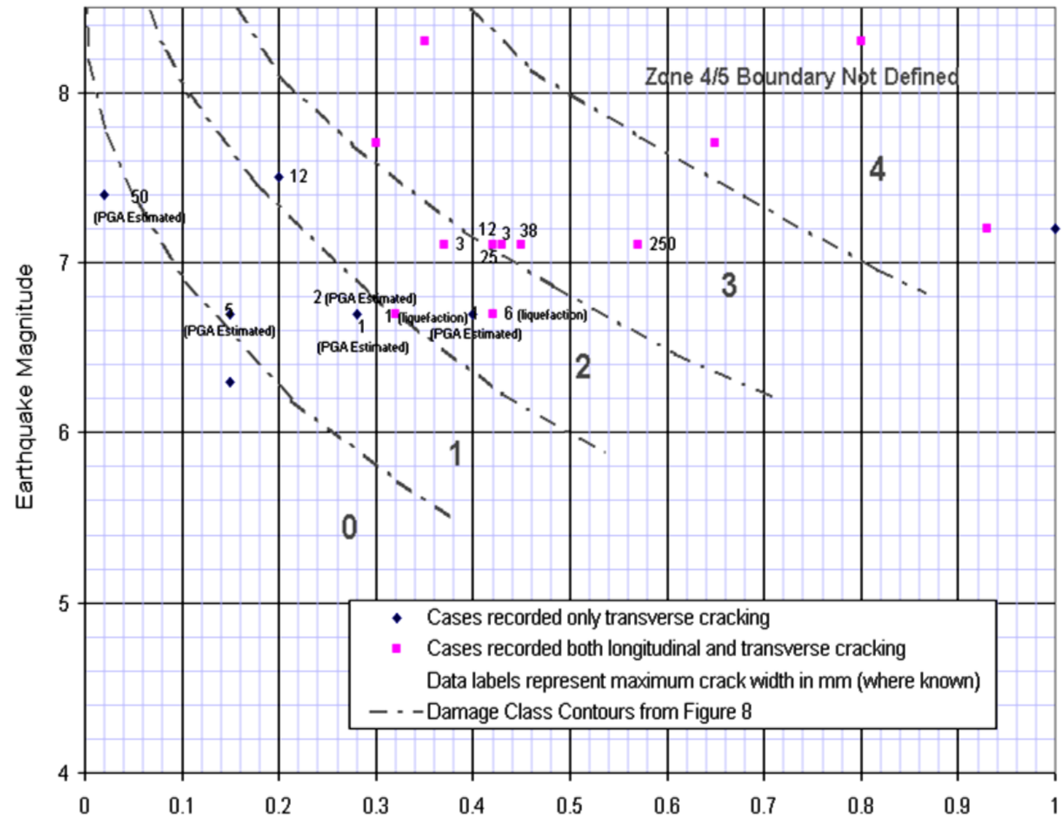


Figure 4-3: Settlements and Damage for Embankment Dams during Earthquake-excluding liquefaction settlement (Swaisgood, 2014)

3. The studies by Pells and Fell (2003) also indicate either slight or no damage (Damage Category 0) for earth dams which sustained foundation PGA of 0.1g or less for earthquakes ranging up to Magnitude 7 (see Figure 4-4).



Damage Class		Maximum Longitudinal Crack Width [1] (mm)	Maximum Relative Crest Settlement [2] (percent)
Number	Description		
0	No or Slight	< 10	< 0.03
1	Minor	10 to 30	0.03 to 0.2
2	Moderate	30 to 80	0.2 to 0.5
3	Major	80 to 150	0.5 to 1.5
4	Severe	150 to 500	1.5 to 5
5	Collapse	> 500	> 5

Figure 4-4: Contours of damage class versus earthquake magnitude and peak ground accelerations for earth fill dams (Pells and Fell, 2003)

4.2. Step 2 - Dam and Foundation Characteristics

Step 2 includes Subtask C and two screening questions. Subtask C consists of performing a desktop level study collecting all pertinent geologic and geotechnical data to make the following decisions:

- Is there sufficient geological and geotechnical data available to move forward to the analysis stage? (Screening Question 2)
- Is there sufficient information to rule out potential strength losses during an earthquake related to liquefaction or cyclic softening? (Screening Question 3).

<u>Subtask C</u>

Desktop Level Geotechnical Data Collection

An important aspect of the Procedure is the ability to make a decision on potential for seismic damage to an embankment subject to accelerations greater than 0.1g, without performing an actual seismic analysis (Screening Question 2). A desktop level study is recommended for collection and perusal of geotechnical data pertinent to seismic performance of the embankment, foundation, and appurtenant structures (Appendix B, Figure B-3).

Information considered necessary to carry out this Subtask is listed below:

1. Regional and Site-Specific Geology:
    a. Broad-level assessment of the geology and stratigraphy at the dam site.
    b. Specifically, foundation details such as orientation of bedding planes (Dip and Dip Direction).
    c. Presence of shears and faults with the estimated slip rates for active faults, and footprint of faults on the embankment and appurtenant structures.
    d. Presence of problematic rock or soil in the area. Example of problematic foundation includes corrosive rock, exothermic rock, Karst rock, collapsible, expansive or soluble soil, and other such conditions.
2. Design and Construction History:
    a. Dam cross section details and internal zoning.
    b. Construction techniques:
        - Foundation – Foundation materials and characterization, foundation treatments, including blasting of rock foundation/abutments, removal or compaction of soil foundations, grouting techniques and results, dental concrete, shaping of foundation/abutment slopes.
        - Embankment – Material placement and compaction, average testing frequency and results, material gradations, material properties for the internal zones, borrow sources, history of the dam and how it was constructed, including what types of materials were used in the dam, and the levels of compaction.

The above pertains to both original construction and any remedial measures or improvements made to the dam since original construction.

c. Design phase seepage and stability analyses – design assumptions, material properties, analysis methods and results including calculated seepage gradients, seepage quantities, and slope stability Factors of Safety (FS). Updated re-analyses and investigations completed after original design and any analyses related to remedial measures or improvements should also be documented.

d. Filter analyses for dam and foundation materials detailing the percentages of the filter gradations that would likely result in "no erosion, some erosion, excessive erosion, and continuing erosion" based on Foster and Fell (2001) erosion boundaries.

e. Special considerations taken during design of the dam to overcome geotechnical issues. For example, special design of the core and filter material to account for fault offset that may occur within the core, filter soil foundation [example: see case history of Cedar Springs Dam, San Bernardino, California by Arnold and Kreese (2010) where a complete redesign of the dam was performed following discovery of an active fault in the footprint of the dam]

3. Penetrations:

   a. Penetrations – the design and construction detailing penetrations needs to be available and documented, if present; outlet works or utility conduits through the embankment or its foundation that might be damaged during earthquake shaking or be associated with potential failure modes associated with the performance of the dam and/or uncontrolled release of reservoir water.

4. Performance History:

   a. Dam performance over time under static or seismic conditions (e.g. seepage and settlement). Knowledge of previous seismic and hydrologic loadings on the dam and documented performance after such events.

5. Geotechnical Parameters for Foundation and Embankment:

   a. Investigation of the embankment and foundation materials by use of appropriate in situ penetration tests (e.g. Standard Penetration Test (SPT), Cone Penetration Test (CPT), and Becker Penetration Test (BPT)) together with classification tests of representative samples (i.e. gradation and Atterberg Limits).

   b. Knowledge of shear wave velocities (V_{S30}) in the dam and foundation is desirable for an accurate seismic site-class classification and should at least be estimated using correlations with penetration test results and material properties, if not actually measured.

c. Characterizing the dam and foundation materials by dividing them into discrete units or layers based on penetration resistance and soil type and assigning various engineering properties to each unit or layer. The assignment of properties should include the total range of properties measured (e.g. penetration resistance, relative compaction, fines content, plasticity values) together with median and low average (e.g. 35th percentile) values.

6. Operations and Maintenance Information
    a. Reservoir operation information – maximum normal operating reservoir elevation and maximum design limits
    b. Winter/summer reservoir operational plans
    c. Spillway or low-level outlet capacity and operating limits, together with ability to draw down the reservoir following a seismic event.
    d. Ability to quickly inspect for damage and possibly intervene after earthquake events

Screening Question 2

This step checks if there is sufficient geotechnical data to characterize the dam embankment and foundation to perform the simplified analysis, or if additional investigation is required.

Screening Question 2: Is there sufficient geological and geotechnical data from Subtask C to determine a low average (e.g. 35th percentile) characterization of the static and cyclic resistance of the materials in the different soils in the embankment and foundation?

Answer: If yes, then the analysis proceeds to Screening Question 3. If no, then further investigation of the dam is recommended to provide the information required for Step 2 and the analyses outlined in this report.

Screening Question 3

Screening Question 3 (Appendix B, Figure B-4) requires the engineer to assess the potential for the embankment, foundation, or appurtenant structures to sustain damage from seismic shaking by answering the following seven questions based on USBR (2015) and FEMA (2005) guidelines.

1. Are the materials within the dam and foundation NOT composed of any one of the following:
    i. Liquefiable soils
    ii. Sensitive clay
    iii. Clayey soils with potential for softening with cyclic loading: such as puddled clayey cores, hydraulic fills, and normally consolidated clay foundations?

2. Is the dam well-built and compacted to at least 95 percent relative compaction or relative density greater than 75 percent?
3. Are the upstream slopes 3:1 or flatter for earth dams, or 2:1 or flatter for dams with upstream rockfill shells, AND downstream slopes 2:1 or flatter, AND does the phreatic line NOT exit on the downstream face of the dam?
4. Is PGA at the dam less than or equal to 0.30g and the predominant earthquake magnitude determined from deaggregation results in Step 1 is less than or equal to 6.5 for earth dams and 7.0 for earth and rockfill dams.
5. Does the dam have static slope stability factors of safety equal to or greater than 1.5 for potential sliding surfaces that might involve loss of crest elevation?
6. For dams with heights of 100 feet or less, is the available total freeboard equal to at least 10 percent of the embankment height, but not less than 6 feet? For dams higher than 100 feet, is the available total freeboard equal to at least 10 feet?
7. Can it be documented that critical appurtenant features or penetrations that might lead to an uncontrolled release of the reservoir would not be harmed by small movements of the embankment following a seismic event? Documentation can be based on engineering judgment after detailed of review of design and as-built documents. For example, a conduit that is placed in a trench excavated into hard rock foundation and concreted will be less susceptible to damage from the embankment movement compared to a conduit which runs through the embankment fill.

If the answer to ALL of the above is **Yes**, then the dam is considered seismically stable. Otherwise, the evaluation must proceed to Step 3 in the Procedure.

4.3. Step 3 - Potential for Dam and Foundation Soil Strength Loss

Step 3 of the Procedure includes 3 subtasks and does not include a screening question. The procedure consists of identifying the potential for strength loss in either the embankment or foundation (Subtask D) and assessing the strength loss by liquefaction (Subtask E) or by cyclic softening (Subtask F). An initial set of index tests are used initially in Subtask D to determine if the potential for strength loss is controlled by coarse-grained sand-like behavior (liquefaction) or by fine-grained clay-like behavior (cyclic softening).

Based on the outcome from Subtask D for each soil layer or zone the engineer is then led to a liquefaction triggering and post-liquefaction strength analysis (Subtask E) and/or cyclic softening and remolded strength analysis (Subtask F). Once the post-earthquake strengths are determined then the engineering proceeds to Step 4.

Subtask D

Liquefaction and Cyclic Softening Susceptibility Index Test

Subtask D is an index test (Appendix B, Figure B-5) to determine if discretized soil layers in the dam and foundation are either susceptible to liquefaction or to cyclic softening. The questionnaire recommended for this screening is based on modifications to studies by Malvick et al. (2014a), Armstrong and Malvick (2014 and 2016), and Malvick et al. (2014b). These studies discuss the use of liquefaction susceptibility criteria from the point of view of the California Division of Safety of Dams (DSOD). Earlier work by Seed et al. (2003); Bray and Sancio (2006); and Boulanger and Idriss (2006) on index tests to determine liquefaction susceptibility are summarized in Figure 4-5 for reference (soils susceptible to liquefaction fall within the yellow zones, and soils not susceptible to liquefaction fall into the blue zones). Liquefaction triggering analysis is conducted for the conditions at the time of investigation, specifically to the groundwater condition and SPT N value.

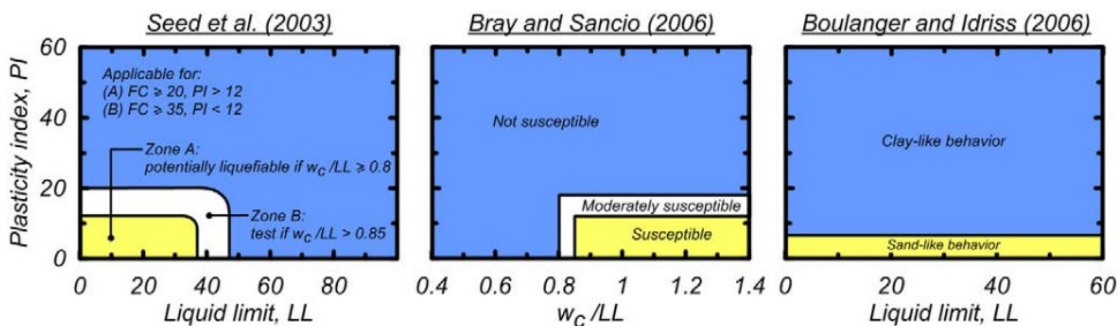


Figure 4-5: Methods used to determine liquefaction susceptibility: Seed et al. (2003); Bray and Sancio (2006); and Boulanger and Idriss (2006).

The following criteria are recommended for liquefaction and cyclic softening susceptibility:

1. Soils with a plasticity Index (PI) less than 7 and/or with a Fines Content (FC) less than 20 percent are considered to be “sand-like”, liquefiable, and recommended for liquefaction evaluations (Subtask E). The category is represented by the yellow region in Figure 4-6.

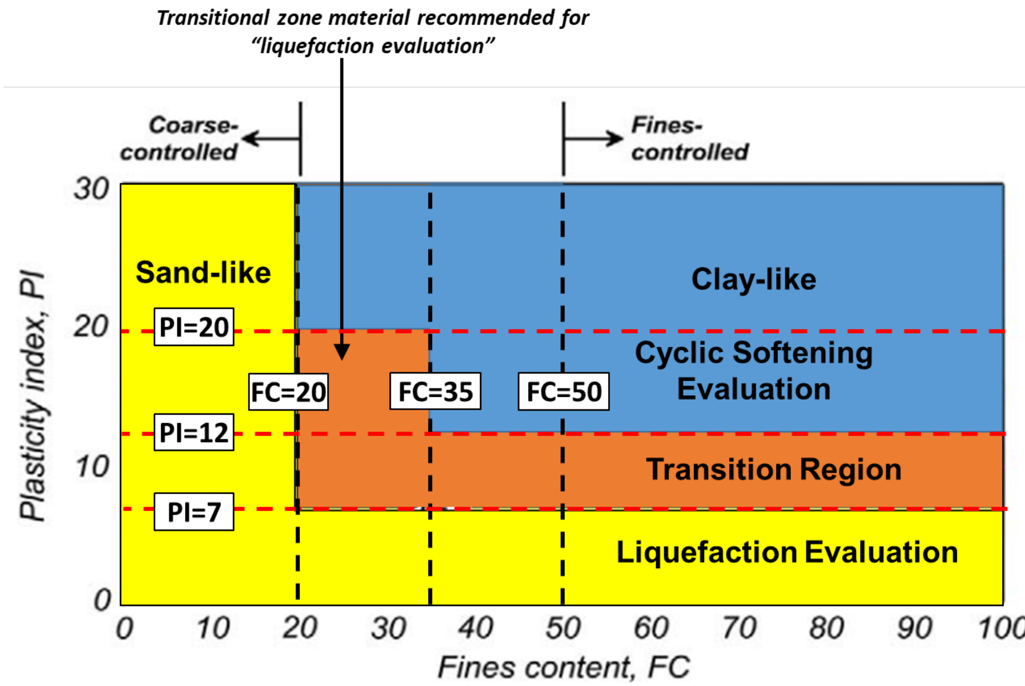


Figure 4-6: Liquefaction susceptibility chart

(Original figure from Armstrong and Malvick, 2016 modified to reflect additional recommendations made in this report)

2. Soils with FC greater than 35 percent and a PI greater than 12 and soils with FC greater than 20 percent with PI of 20 or more are considered to be "clay-like" and recommended for cyclic softening and remolded strength evaluations (Subtask F). The difference between this recommendation and the California DSOD guidance consists principally of moving the dividing lines for the transition area:

   - The dividing line between the transition zone and the clay-like zone based on fines content was reduced from 50 percent in the DSOD criteria to 35 percent. The reason for this is that soils with 35 percent FC are generally considered to have behaviors based on their matrix rather than on their particle skeleton (this is supported by the recommendations by Seed et al., 2003 and Boulanger and Idriss, 2006).

   - A transition zone was added for PI values between 7 and 12 (This was supported by considering the differences and uncertainties in the different studies such as those by Seed et al., 2003; Boulanger and Idriss, 2006; Malvick et al., 2014b; and Armstrong and Malvick, 2016).

3. The transition region is considered to be a transition between sand-like and clay-like behavior and reflects uncertainty about which behavior dominates. In the absence of any cyclic laboratory testing, soils falling in this zone were recommended by Malvick et al.

(2014b) and Armstrong and Malvick (2016) to be evaluated by both liquefaction and cyclic softening analyses, with the more conservative result to be used. However, in such comparisons, the liquefaction evaluation will likely give lower strengths. Therefore, it is probably more expedient and conservative to simply assume that soils in the transition area are potentially liquefiable, and that the evaluations for such soils proceed to Subtask E.

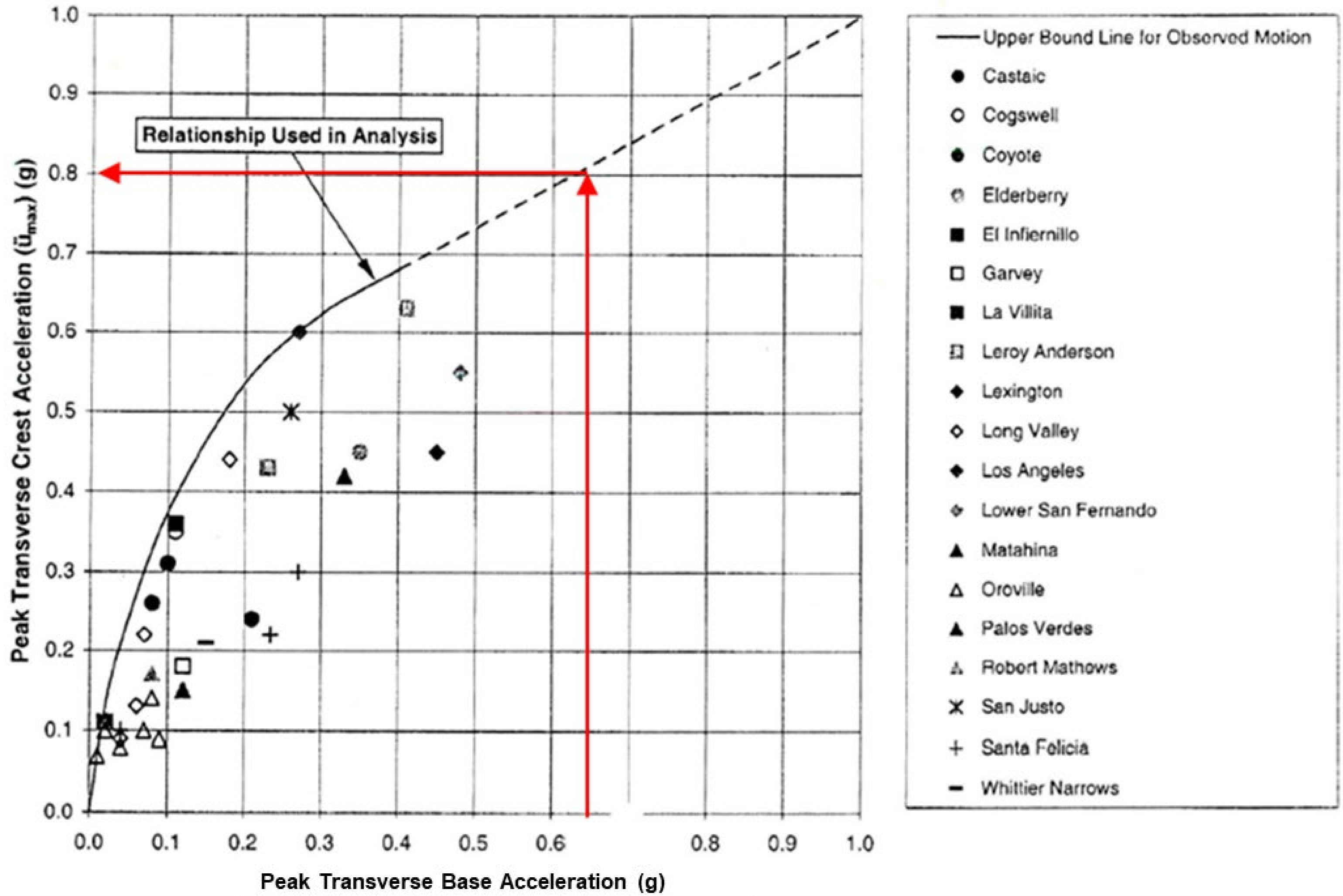
To clarify, references are made only to 'sand-like' and 'clay-like' soils. Free draining soils such as coarse to fine sand, and medium to coarse gravel can be generally understood to be sand-like, commonly located in filters, drains and sand/gravel foundation layers of some dams. Non-free draining soils can generally be understood to be clay-like, such as those commonly used in the core of some dams.

Subtask E

Liquefaction Triggering and Post Liquefaction Strength Evaluation

Subtask E of the simplified procedure (Appendix B, Figure B-6) is focused on calculating the Factor of Safety for triggering liquefaction, FS_{liq}. The Idriss and Boulanger (2008) monograph published by the Earthquake Engineering Research Institute (EERI) is used as a guidance document for this task. The basic elements of this procedure consist of the following:

1. Use the bedrock PGA estimate from Step 1 (Section 4.2)
    a. Foundation liquefaction evaluations beyond embankment toes - PGA estimate Step 1 can be used as the surface peak acceleration
    b. Embankment and foundation liquefaction evaluations beneath the embankment - use Harder, 1997 (Figure 4-7) to transform the base PGA to PGA at dam crest. This can be used as the surface peak acceleration at the dam crest. For slope areas between the toes and the crest, interpolate the surface PGA between the embankment toes and the estimated crest peak acceleration.
2. Determine the earthquake induced Cyclic Stress Ratio (CSR) at regular intervals along the embankment and foundation as described in Idriss & Boulanger (2008) adjusting for the stress reduction coefficient (r_d), and earthquake magnitude effects (MSF).
3. Determine the SPT or CPT based Cyclic Resistance Ratio (CRR) at regular depths and intervals along the embankment and foundation as described in Idriss & Boulanger (2008) applying the required correction factors- overburden correction factor, K_σ and sloping ground correction factor, K_α.
4. Calculate the Factor of Safety against liquefaction, FS_{liq}.
5. Select residual and post-liquefaction strengths for use in Subtask G.



Adopted from: "Castaic Dam Left Abutment Stability Evaluations" Harder et. al., March 1997.
Updated with Northridge Earthquake data by T. Craddock in December 1998.

Figure 4-7: Relation between Peak Transverse Crest Acceleration (U_{max}) and Peak Base Acceleration (Plot from Harder et. al., 1997)

The Idriss and Boulanger (2008) monograph includes relationships between SPT and liquefaction triggering (Figure 4-8). A comparison of SPT liquefaction triggering correlations is presented in Figure 4-9. The SPT-liquefaction triggering curve developed by Cetin et al., (2004) is included in the figure and does not correlate with the other two relationships. However, if the 50th-percentile curve from Cetin et al. (2004) studies was to be used, it would fit well with the Idriss and Boulanger (2008) curve. Therefore, the correlations and curves in the 2008 monograph by Boulanger Idriss and Idriss (2014) are recommended for use with the DNRC Simplified Seismic Stability approach.

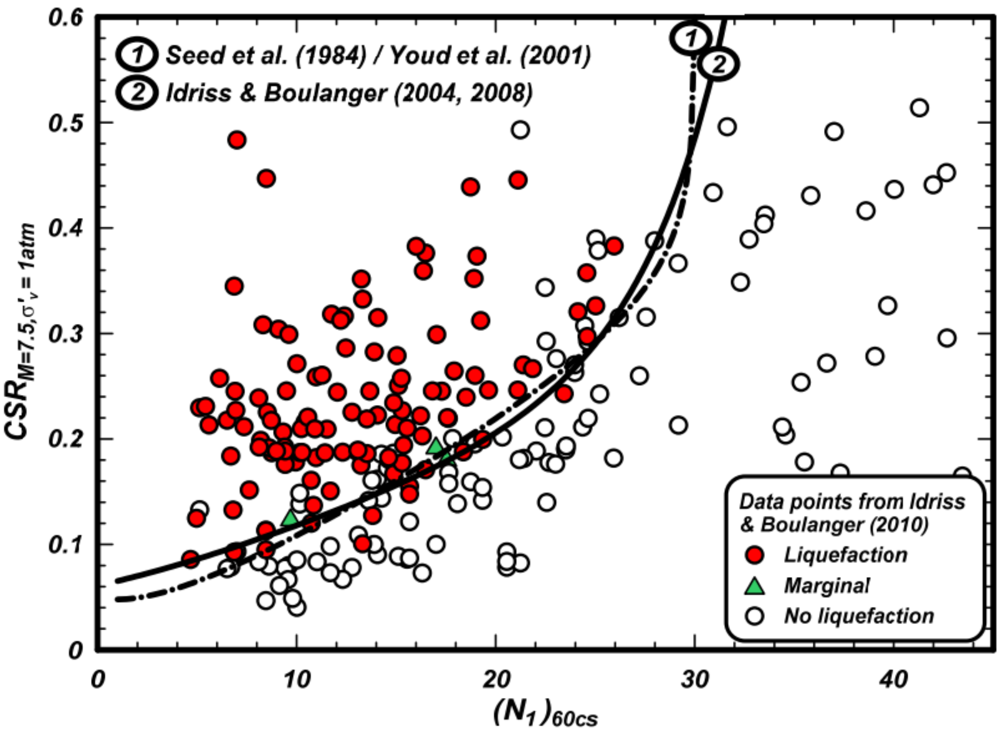


Figure 4-8: Updated database plotted with Idriss and Boulanger 2004 and 2008 SPT-liquefaction triggering correlations (From Boulanger and Idriss, 2014)

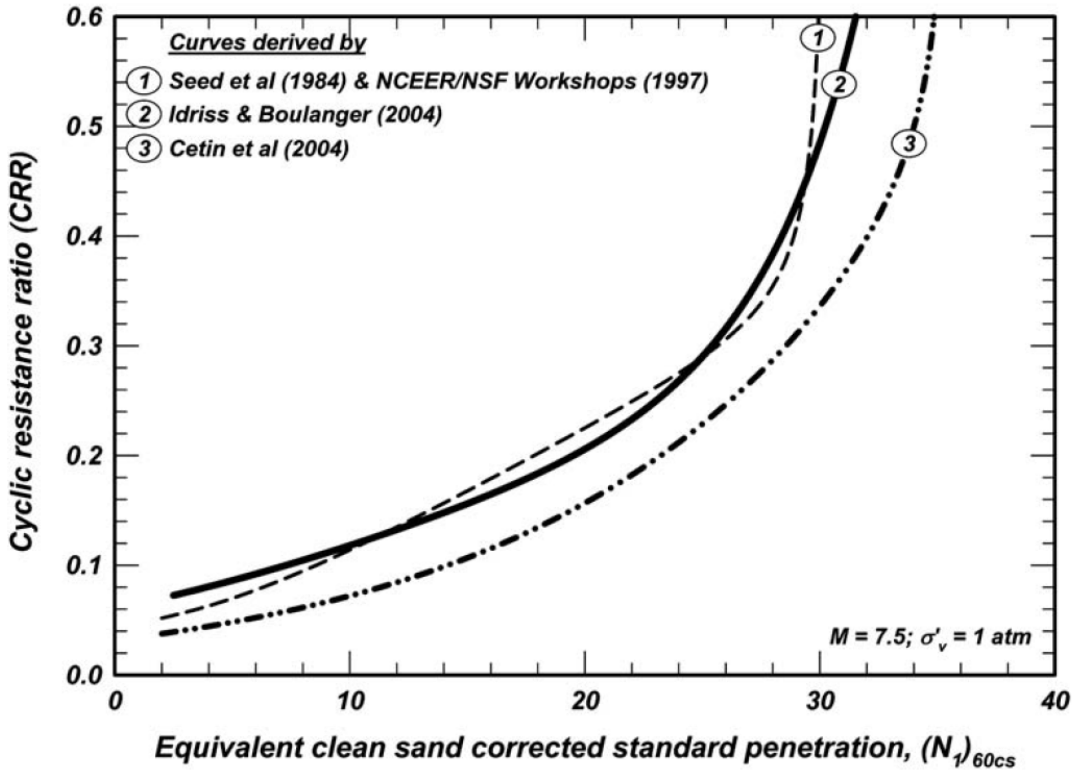


Figure 4-9: Comparison of SPT-liquefaction triggering correlations (From Idriss and Boulanger, 2012)

Selection of Residual Shear Strength

- For “sand-like” soils that have a factor of safety for triggering liquefaction, FS_{liq} less than 1.1, use the residual undrained shear strength, S_r.
- For “sand-like” soils that have a factor of safety for triggering liquefaction, FS_{liq}, of between 1.1 and 1.4, use shear strengths interpolated between the residual undrained shear strength, S_r, for FS_{liq} = 1.1 and 90 percent of the peak static drained strength of the soil for FS_{liq}= 1.4.
- For “sand-like” soils that have a factor of safety for triggering liquefaction, FS_{liq}, of 1.4 or more, use 90 percent of the peak static drained strength of the soil.

There are several correlations available between corrected SPT blowcounts, $(N_1)_{60cs\text{-}Sr}$ and the residual undrained shear strength, S_r, for use with “sand-like” soils. These have been developed from back-calculations of past liquefaction-related sliding. Some of the correlations relate corrected SPT blowcounts to the residual shear strength, S_r, (e.g. Seed and Harder, 1990; Idriss and Boulanger, 2008) while others correlate SPT blowcounts to the residual shear strength normalized by the effective overburden pressure, σ_v', as (S_r/ σ_v') (see Olson and Stark, 2002; Idriss and Boulanger, 2008).

More recently, studies by Weber (2015) and Kramer and Wang (2015) employ a hybrid approach. Table 4-1 and Table 4-2 show comparisons of the residual undrained shear strength, S_r, which would be predicted for two different SPT blowcounts [$(N_1)_{60cs\text{-}Sr}$ = 10 and 14] and three different overburden pressures (σ_v'= 2,000 psf, 4,000 psf, and 8,000 psf). For some overburden pressures, there is relatively close agreement between the different correlations (e.g. see σ_v'= 2,000 psf for $(N_1)_{60cs\text{-}Sr}$ = 10 or σ_v'= 4,000 psf for $(N_1)_{60cs}$ = 14), but for other values of overburden pressures SPT blowcounts, there can be a significant range.

It is recommended that the more recent correlations by either Weber (2015) and/or Kramer and Wang (2015) be employed. These have the advantage of more recent information and are not too different from the average of all the correlations. Further, they also employ a hybrid approach which is more promising. **Care needs to be given, however, to ensure that the residual shear strength, S_r, used does not exceed the static drained strength of the soil.**

Table 4-1: Comparison of predicted residual shear strengths, S_r, from different correlation for SPT $(N_1)_{60cs\text{-}Sr}$ = 10

Residual Shear Strength Correlation	Residual Shear Strength S_r (psf) for Different Effective Stresses		
	σ_v'= 2,000 psf	σ_v'= 4,000 psf	σ_v'= 8,000 psf
Seed and Harder (1990) S_r = 200 psf	200	200	200
Olson and Stark (2002) S_r/ σ_v'= 0.10	200	400	800
Idriss and Boulanger (2008) S_r/ σ_v'= 0.096	192	384	768
Kramer and Wang (2015)	285	425	640
Weber (2015)	240	360	480
Mean	**223**	**354**	**578**

Table 4-2: Comparison of predicted residual shear strengths, S_r, from different correlation for SPT $(N_1)_{60cs\text{-}Sr}$ = 14

Residual Shear Strength Correlation	Residual Shear Strength S_r (psf) for Different Effective Stresses		
	σ_v'= 2,000 psf	σ_v'= 4,000 psf	σ_v'= 8,000 psf
Seed and Harder (1990) S_r = 200 psf	500	500	500
Olson and Stark (2002) S_r/ σ_v'= 0.10	250	500	1,000
Idriss and Boulanger (2008) S_r/ σ_v'= 0.096	300	600	1,200
Kramer and Wang (2015)	455	665	975
Weber (2015)	460	660	1,000
Mean	**393**	**585**	**935**

Additional recommendations for selection of shear strengths are provided below:

1. For each material or layer, the representative shear strength should be based on a low average of the penetration or laboratory strengths. An appropriate low average is the 35th percentile, which represents the lower third of the strength that might be determined together with being equivalent to the median minus one-half of the standard deviation.
2. For any existing shears or landslide surfaces in the foundation, residual shear strengths for these discontinuities/features should also be used.
3. For determining shear strengths of ductile embankment and foundation materials, the peak shear strength should be no higher than that determined at axial strains of less than 10 percent for triaxial tests, or 7 percent shear strains for simple or direct shear tests. For brittle materials, a larger reduction in shear strength should be considered unless the calculated deformations are very small.
4. It should be clarified that only saturated fine-grained soils are to use the peak undrained strengths and that unsaturated fine-grained soils should use peak drained strengths.
5. Only the peak strengths are reduced by 10 percent, and that the residual undrained strength for sheared soils is the minimum strength and should not be further reduced.
6. It should be remembered that different types of laboratory tests with different boundary conditions produce different shear strengths. For example, for downward, active sliding in the embankment, a triaxial compression test more closely represents the type of sliding. For horizontal or block sliding in the foundation, the simple shear strength is more representative (see Figure 4-10). Simple shear strengths are commonly about 75-80 percent of the shear strengths determined in triaxial compression tests.

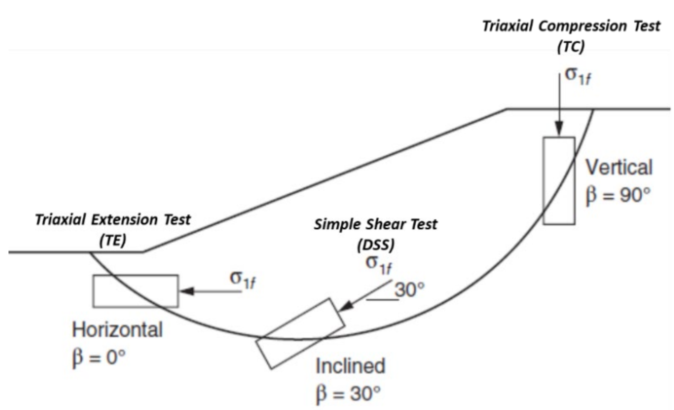


Figure 4-10: Stress orientation at failure and undrained strength anisotropy of clays
(Duncan et. al., 2014)

Subtask F

Cyclic Softening and Remolded Strength Evaluation

Subtask F of the simplified procedure (Appendix B, Figure B-7) is focused on calculating the Factor of Safety against cyclic softening, $FS_{cyclic\text{-}softening}$. As previously discussed for Subtask E, the Idriss and Boulanger (2008) monograph published by the Earthquake Engineering Research Institute (EERI) is used as a guidance document for this subtask. The basic elements of this procedure consist of the following:

1. Use the bedrock PGA estimate from Step 1and modify as described below:
    - Foundation Liquefaction – Bedrock PGA estimate Step 1 can be used.
    - Embankment Liquefaction – Use Harder, 1997 (Figure 4-7) to transform the bedrock PGA to PGA at dam crest. This can be used as the surface peak acceleration at the dam crest. For slope areas between the toes and the crest, interpolate between the surface PGA between the embankment toes and the estimated crest peak acceleration.
2. Use PGA from step above to estimate CSR and 35th percentile CRR based on undrained S_u using Idriss and Boulanger (2008) approaches.
3. Determine the earthquake induced Cyclic Stress Ratio (CSR) at regular intervals along the embankment and foundation as described in Idriss & Boulanger (2008) adjusting for the stress reduction coefficient (r_d) and appropriate Magnitude scaling factor (MSF) for clay-like soils.
4. Determine the Cyclic Resistance Ratio (CRR) based on undrained shear strength (S_u) at regular intervals along the embankment and foundation as described in Idriss & Boulanger (2008).
5. Apply K_α and K_σ to the CRR based on monograph
6. Calculate the Factor of Safety against cyclic softening, $FS_{cyclic\text{-}softening}$.
7. Determine remolded and cyclic strengths for use in Subtask G.

<u>Remolded Strength</u>

For "clay-like" soils that are triggered for cyclic softening ($FS_{cyclic\text{-}softening}$ <1) use the remolded shear strength, S_{ur}. Mitchell and Soga (2005) included ratios for remolded strength S_{ur}, in terms of peak shear strength S_u using liquidity index and vertical effective shear strength data from several clays (see Figure 4-11). This correlation is recommended for determining the remolded strength of clays in this subtask..

The present Simplified Seismic Analysis Procedure includes Figure B-8 (Appendix B) that summarizes the residual and remolded strength correlations discussed in Subtasks E and F.

Cyclic Softening Strength

Estimate areas where factor of safety against cyclic softening are less than 1, areas where fully remolded shear strengths should be assumed, and areas where static strengths (90% of peak strengths) should be used

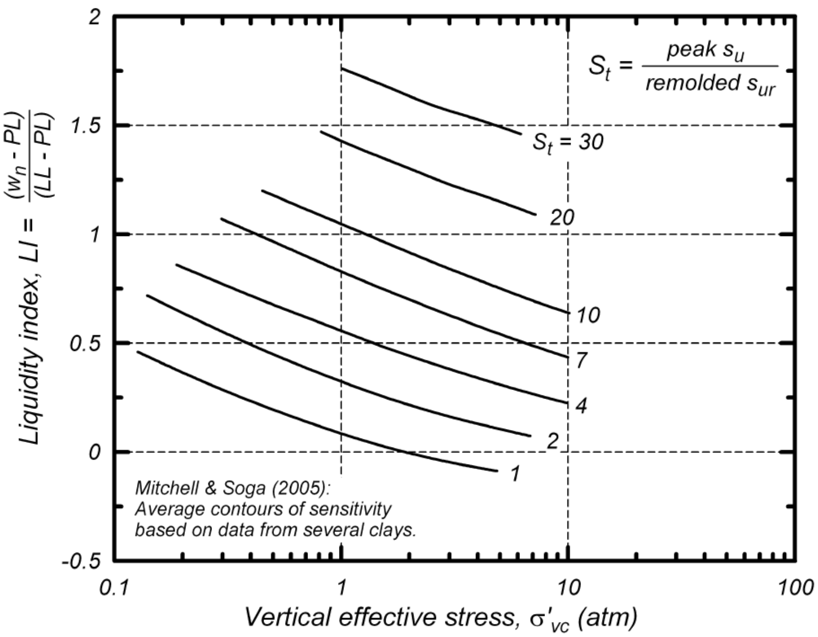


Figure 4-11: Mitchell and Soga (2005) correlations for remolded strength of clays (from Idriss and Boulanger, 2008)

4.4. Step 4 - Post-Earthquake Stability Analysis

Step 4 consists of performing a post-earthquake stability analysis by assigning the soil strengths as determined in Step 3, performing a limit equilibrium stability analysis (Subtask G) and then answering Screening Question 4.

If the result of Screening Question 4 is 'No', then the dam requires further investigation or repair and the Procedure is halted. If the result of Screening Question 4 is 'Yes', then the analysis proceeds to Step 5, to estimate the earthquake induced deformations and settlements.

Subtask G

Post-Earthquake Stability Analysis

This subtask involves performing 2-Dimensional (2D) plane-strain limit-equilibrium analyses to determine Factors of Safety (FS) for slope stability of the upstream and downstream slopes for each internal phreatic surface related to the reservoir level.

In these analyses, post-earthquake residual and/or remolded shear strengths are applied to soil layers as determined in Step 3. As discussed in Step 3, soils which do not liquefy or cyclically soften are assigned 90 percent values of their peak drained or undrained shear strengths, as discussed in Subtasks E and F. Soils with intermediate factors of safety against liquefaction triggering or cyclic softening are assigned intermediate shear strengths.

Post-earthquake slope stability analyses should be carried out for both the upstream and downstream slopes using the appropriate post-earthquake shear strengths for the various soils within the embankment.

In general, the limits of post-earthquake slope stability failure surface should intercept or be near the dam crest and, for circular failure surfaces, be concentrated through embankment zones and/or semi-continuous foundation layers which are predicted to lose substantial shear strength due to liquefaction or cyclic softening. In addition, for the case where foundation liquefaction or cyclic softening has occurred in a layer, wedge-shaped sliding surfaces or composite circle-wedge sliding surfaces should be performed locating the base of the wedge portion near the bottom of the layer with significant strength loss.

Acceptable methods for calculating post-earthquake FS employing limit-equilibrium analyses include Modified Bishop, Spenser's Method, and the Morgenstern-Price Method.

Screening Question 4

Screening Question 4 assesses the calculated post-earthquake seismic stability of the dam. If the calculated FS is > 1.2 (based on FEMA, 2005 guidelines) the dam is stable but could experience seismic deformations determined in Step 5. If the post-earthquake stability is < 1.2 the dam is potentially unstable and additional higher level analyses and/or risk reduction measures (remediation) are required.

Screening Question 4: Are the post-earthquake slope stability Factors of Safety ≥ 1.2?

Answer: *Yes* - Proceed to subsequent steps in the Procedure.

Answer: *No* - Stop the Procedure. Perform additional investigations and higher-level analyses or implement risk reduction measures such as remedial strengthening or restrictions on the reservoir level.

4.5. Step 5 - Earthquake Induced Deformations and Settlements

Step 5 consists of first calculating the pseudo-seismic acceleration for the critical failure surfaces for two cases: Case 1 Static Undrained/Drained Strengths and Case 2 Post-Earthquake strengths determined in Step 3 for the stability models developed in Step 4 (Subtask H). Once the yield accelerations are determined, the rigid block deformation, crest settlement and volumetric settlements of the dam are estimated (Subtasks I, J, and K, respectively). With the completion of this analysis the engineer proceeds to Step 6 assessing the deformation.

Subtask H

Pseudo-Static Yield Acceleration

Subtask H (Appendix B, Figure B-9) involves 2D pseudo-static limit-equilibrium analyses to calculate yield acceleration (k_y) values for use in a Newmark-style sliding block deformation analyses. Yield accelerations for critical slip-surfaces are to be calculated for the following cases:

Case 1 – Static shear strengths for all materials (90% of peak undrained/drained strengths),

Case 2 – Post-earthquake residual and remolded shear strengths as from Step 3.

The reason for having the two cases is to represent conditions that may occur at the start of and end of significant ground shaking. At the beginning of the earthquake shaking, soils in the embankment and foundation are near their peak static shear strengths and, by the end of the earthquake, some soil layers may have lost significant portions of their shear strengths. So, the inertial shaking occurs as the soils transition from near their static peak strengths to reduced strengths, even residual or fully remolded shear strengths.

The Case 1 shear strengths represent 90% of the peak drained strengths for cohesionless free-draining or unsaturated soils and 90% of the peak undrained shear strengths for saturated and unsaturated clayey soils. The Case 2 shear strengths should be those reduced shears determined from Step 3.

Yield accelerations are calculated using a series of iterative slope stability analyses with varying horizontal acceleration (k_h) values (pseudo static acceleration factors). The pseudo static coefficient (k_h) that results in a Factor of Safety (FS) equal to unity (FS=1) for a selected slip surface is defined as the yield acceleration (k_y). Stability analyses must be performed using the reservoir surface anticipated prior to the earthquake together with the seepage pore pressures and phreatic line within the dam associated with that reservoir surface. Multiple potential sliding surfaces on the upstream and downstream slopes that intercept or are near the crest should be considered as only potential sliding surfaces that would cause an uncontrolled release of the reservoir should be considered for iteration of horizontal acceleration.

At the end of these analyses, a range of k_y values calculated for each of the various upstream and downstream slope configurations and foundation conditions for the two cases.

Results from this task should only be considered as an index of the seismic resistance available in the embankment shear strength not subject to true reduction due to build-up of pore pressure from shaking. It is not possible to predict failure by pseudo static analysis, and other high level analyses are generally required to provide a more reliable evaluation of dynamic performance.

<u>Subtask I</u>

Seismic Deformation Analysis

Two simplified methods are recommended, both based on the Newmark sliding block methodology to estimate seismic deformations;

Method 1 – Modified Makdisi and Seed (1977) as modified by Harder et al (1997) and

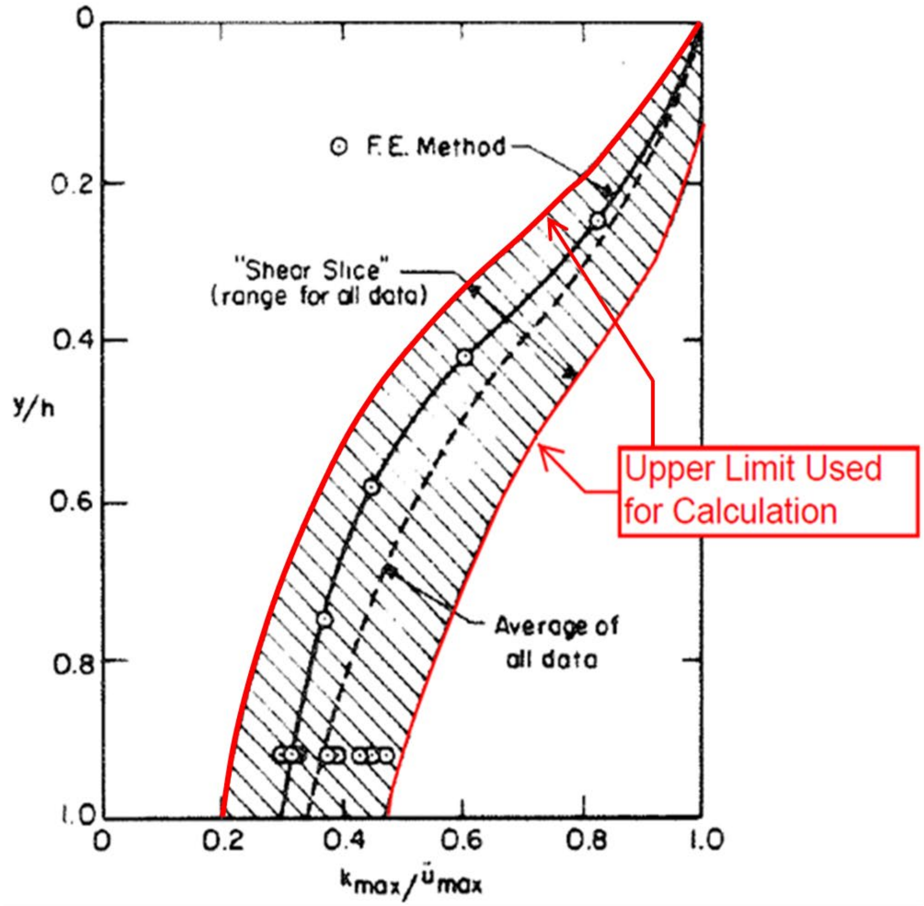
Method 2 – Bray and Travasarou (2007).

The average of the two deformation estimates is carried forward to Step 6 for the assessment of deformation. Note that since there are two sets of calculated yield accelerations (Case 1 and Case 2) from Step 5, there will also be two sets of deformation estimates. For cases other than high hazard dams, this will result in two sets of average deformation estimates carried forward into Step 6. For High Hazard and Extremely High Hazard Dams, the deformations and settlements should be based on the Case 2 estimates. The approach in this step is illustrated in Figure B-10 of Appendix B.

Method 1

This is a modified form of the Makdisi and Seed (1977) procedure that applies the peak crest acceleration developed from Harder 1997 as this relationship was developed based on actual recordings of accelerations measured at the base and crest of dams. The method is as follows:

1. Estimate the peak crest acceleration, u_{max}, based on Figure 4-7 from Harder et al (1997) that was developed based on actual recordings of accelerations measured at the base and crest of dams. The PGA at the base of the dam is used with this chart to conservatively estimate the peak crest acceleration, u_{max}.
2. Once u_{max} has been estimated, the average peak acceleration of the potential sliding surface, k_{max}, is determined using a chart developed by Makdisi and Seed (1977) based on the depth of the sliding surface from the crest relative to the total height of the dam shown in Figure 4-12a.
3. The procedures outlined above would only be used for dams with heights greater than 50 feet. For dam heights less than 50 feet, assume that k_{max} is equal to the PGA at the base of the dam.
4. Estimate the dam displacement from Figure 4-12b using k_{max}, k_y, and the predominant earthquake moment magnitude, Mw. A range of yield accelerations for different potential sliding surfaces and for Cases 1 and 2 would be generated, leading to a range of different potential displacements



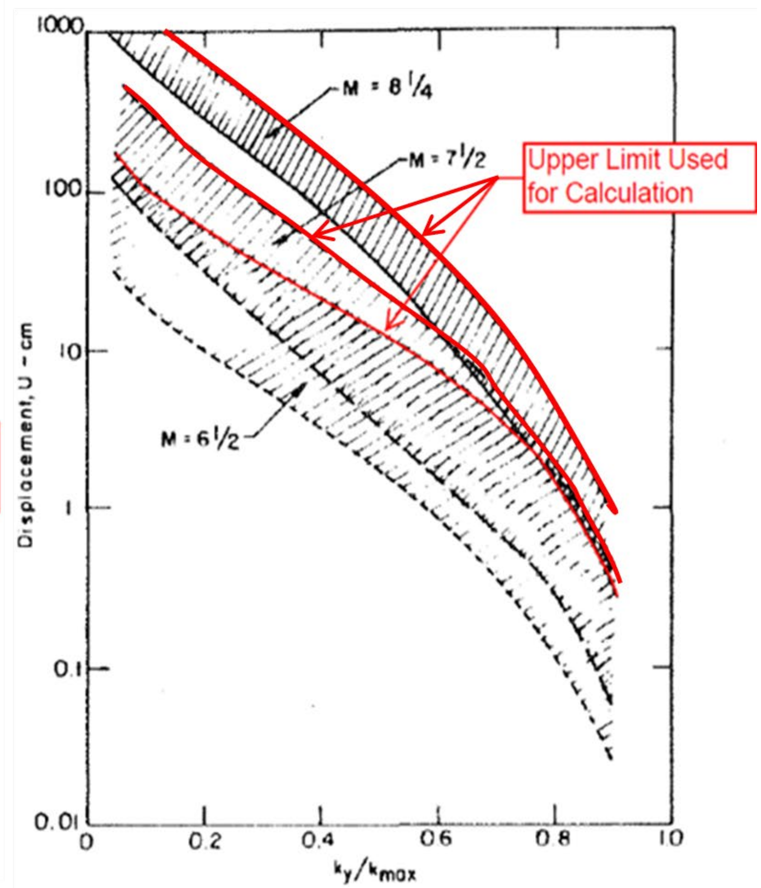


Figure 4-12: a) Variation of "Maximum Acceleration Ratio" with depth of sliding mass; b) Variation of predicted displacement (U) with K_y/K_{max} ratios (Figures from Makdisi and Seed, 1977)

Method 2

Estimate deformations using the method developed by Bray and Travasarou (2007) which uses the following parameters:

- Yield acceleration, k_y
- Estimated initial fundamental period of slide mass at its initial stiffness, T_s, estimated using the height of the potential slide mass, H, together with the average shear wave velocity, V_s, as follows:

$T_s \approx 4H/V_s$ for wide sliding masses, $T_s \approx 2.6H/Vs$ for triangular-shaped masses

- Estimated degraded fundamental period of slide mass $T_{degraded} \approx 1.5$ Ts
- Estimated spectral acceleration, S_a, at degraded fundamental period of slide mass (1.5 T_s). The S_a value is estimated using ground motion correlations such as those by Abrahamson et al. (2008).
- Predominant earthquake magnitude, M.

Subtask J

Crest settlements (S_{Crest}) associated with sliding or spreading should be estimated by using two-thirds of the calculated displacement estimates from Subtask I (i.e. apply a factor of 0.7).

It is also recommended that the calculations of total crest settlement (S_{total_1}) also be done using the correlations based on peak ground acceleration sustained at the dam site and earthquake magnitude, developed by Swaisgood (2014) as shown below in Figure 4-13.

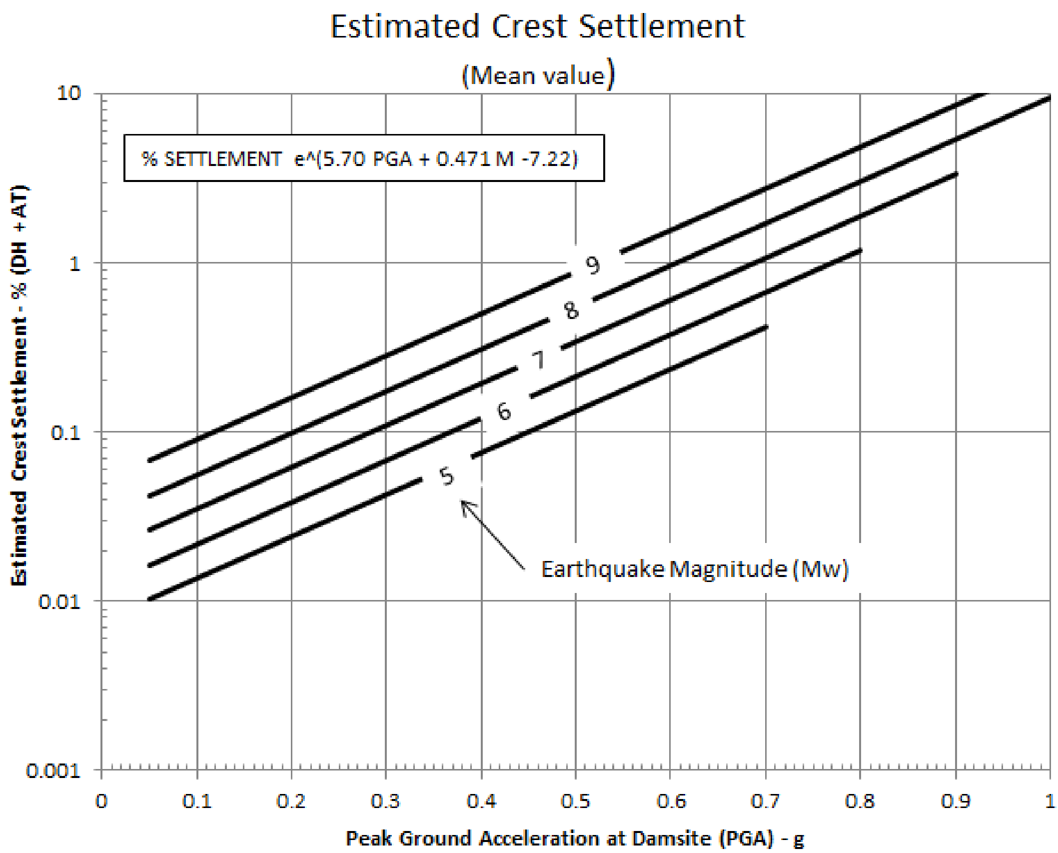


Figure 4-13: Predicted normalized crest settlements of dams (from Swaisgood, 2014)

Subtask K

Calculate volumetric settlement ($S_{Volumetric}$) associated with volumetric compression potentially induced by liquefaction and add this value to crest settlement (S_{Crest}) calculated in Subtask J, but only to settlements calculated as a fraction of the earthquake-induced settlements to get second estimate for total crest settlement (S_{total_2}). Do not add volumetric settlement to those estimated using the Swaisgood (2014) correlation.

Volumetric vertical settlements are calculated using Tokimatsu and Seed (1987) for sands, and studies by Stewart et al. (2004) for soils with significant fines contents. Subtasks J and K are summarized in Appendix B, Figure B-11. Compare S_{total_1} and S_{total_2}, and use higher of the two for Step 6.

4.6. Step 6 - Assessment of Deformations and Stability

Step 6 consists of Screening Question 5 that asks specifics about the results of Subtasks I, J and K in relation to the tolerable risk requirements for earthquake induced settlements in relation to the height of the dam, the amount of freeboard, and the widths of the transition/filter zones. The evaluation of the deformation and the ability of the internal zoning to prevent internal erosion are considered.

Screening Question 5

In order to meet tolerable risk requirements, the estimated earthquake-induced displacements must meet all the following requirements (Appendix B, Figure B-12) to be judged as seismically stable:

a. Total earthquake-induced settlement less than 0.5 – 1 percent of dam height, and less than 0.5 to 3 feet for dams higher than 100 feet in height. High Hazard Dams should have total earthquake-induced crest settlements of less than 0.5 percent.

b. Total earthquake-induced crest settlement less than one-third of total freeboard.

c. Total earthquake-induced crest settlement less than one-half of core freeboard, defined as the top elevation of the core relative to the elevation of the reservoir.

d. Total earthquake-induced displacement less than one-half of width of filter/transition zones

e. In addition, there must be a further evaluation of zoning/filter zones for:

    - The potential impact of the deformation on the ability of the materials/zoning in the dam to prevent internal erosion
    - The potential impact of the displacements of appurtenant facilities on top of, adjacent to, or buried within or beneath the dam.
    - All filter zones should be determined to have predominantly “no erosion”, or “some erosion”, with limited “excessive erosion” or “continuing erosion” gradations.

Much of the above will require good engineering judgment to fully evaluate, including where the likely deformations will be relative to the Case 1 and Case 2 estimates. If all criteria are not met, the dam will require further investigation and/or repair. In addition, sensitivity analyses should be performed as discussed in Section 4.7 to account for resiliency. Those dams not meeting the criteria set forth in Screening Question 5 will require further investigation or repair as presented in Section 4.8

In general, the estimated deformations and settlements used for these assessments should employ the average of the deformations calculated for Case 1 (pre-earthquake static shear strengths) and Case 2 (post-earthquake residual and remolded shear strengths) for Low and Significant Hazard Dams. However, for High Hazard Dams, the deformations and settlements should be based on the Case 2 estimates.

4.7. Sensitivity Analysis

It is recommended that sensitivity analyses be considered for Subtasks D through K, particularly for High Hazard Dams. To satisfy this recommendation, the analysis for each of the listed subtasks should be repeated, even if the initial analysis resulted in tolerable dam safety requirements. Sensitivity of the analysis to the following parameters can be evaluated:

- Reanalyze the dam for a PGA at the base of the dam that is 33 percent higher. For example, if the dam was initially analyzed for a PGA of 0.3g at the base of the dam, reanalyze it for a PGA of 0.4g. Alternatively, if the dam was initially evaluated for a 2,500-year earthquake, analyze it again for the 5,000-year earthquake.
- After estimating post-earthquake slope stability factors of safety and earthquake induced deformations, reanalyze with strengths reduced a further 10 to 20 percent and/or estimate deformations with a yield acceleration, (k_y) that is 20 percent lower than the initial values.

This additional step helps reduce uncertainties and assure the resiliency of the dam during a seismic event. It is intended to address the potential situation where the dam barely meets criteria and is considered to be seismically stable, but if the earthquake was slightly larger or the soils slightly weaker, then a devastating failure would result. With the simplified evaluation process in place, it should be relatively easy to conduct such sensitivity analyses and to then make decisions based on the results and resiliency considerations using sound engineering judgment.

4.8. Further Investigation or Repair

Cases identified for further investigation and repair need to be reconsidered for analysis using advanced methodologies for specific subtasks. Some of these possible methodologies are listed below:

a. Subtask B – If a dam site is identified to be in a region of high seismic activity or if known active-faults pass through the embankment, then a site specific PSHA and fault mapping can be carried out. This can be particularly useful to resolve cases where there is a large difference (greater than or equal to 0.2g) in estimated seismic hazard for a dam site from two different published sources.

b. Subtask E and F – If the analyses conducted using the Simplified Seismic Analysis Procedure had large uncertainties in material properties used for analyzing liquefaction potential and/or cyclic softening, then additional geotechnical exploration of the embankment and/or foundation may be justified if the anticipated risk level is high. This geotechnical exploration would involve drilling through the embankment/foundation to sample the material and run specialized laboratory tests (e.g. triaxial tests, cyclic direct simple shear, etc.).

c. Subtask H to K – All four subtasks can be performed in a suite of advanced dynamic analysis dynamic analysis using Finite Element Method (FEM) or Finite Difference (FD) methods using software packages such as Plaxis (FEM by Bentley) or FLAC (Fast Lagrangian Analysis of Continua, by Itasca Consulting).

   Advanced non-linear material constitutive models in these modeling tools can be used to analyze the seismic performance of the dam in two- or three-dimensions.

If additional analysis is expected to produce results that will not meet tolerable dam safety requirements, options such as just proceeding with seismic remediation and retrofit should be considered. Many such seismic remediation options are used in the current state-of-practice and the selection of the appropriate method varies with the geotechnical problem being solved. Some of these options are listed below:

- Remove and replace
- Berms or buttressing
- Soil improvement for foundation- such as stone columns, Cement Deep Soil Mixing (CDSM), permeation grouting, vibratory compaction, and others. Figure 4-14 shows some of the commonly used soil improvement methods to mitigate liquefiable soil and their applicability based on the gradation of the soil being improved (from Mitchell; 2008).
- Downstream drainage – additional blanket and chimney drains can be built on the downstream to lower the phreatic surface in the embankment. This can be done either by excavating and rebuilding the downstream slope, or as part of a buttress added to the existing downstream slope. This will help reduce the likelihood of liquefaction within the overall downstream slope by reducing the amount of saturated materials within the total slope mass.
- Seismic retrofit – for appurtenant structures like spillway structures and gates, power plant structures, and intake structure or low-level outlets. Alternatively, penetrations such as low level outlets subject to damage and consequent internal erosion distress could be filled and abandoned, or replaced.

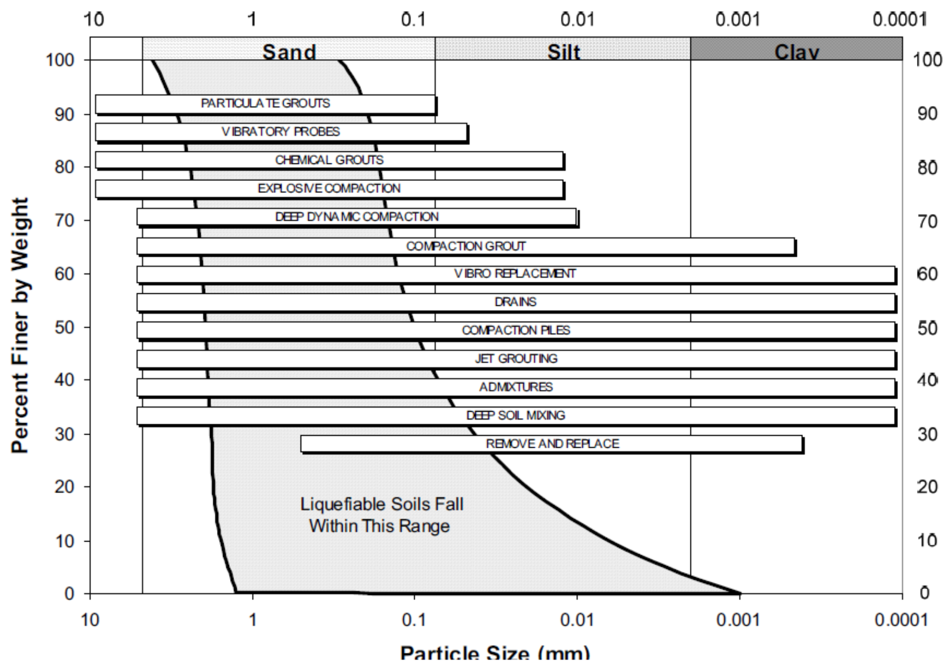


Figure 4-14: General applicability of ground improvement methods for liquefiable soils
(from Mitchell, 2008)

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5. Closing Comments

This Simplified Seismic Analysis Procedure is intended to provide a method for relatively quickly assessing whether embankment dams are potentially vulnerable to earthquake shaking and represent an unacceptable rise for uncontrolled reservoir release. The loadings and acceptability criteria are consistent with risk-based concepts in that dams with higher downstream consequences need to be evaluated for higher seismic loadings and meet higher acceptability or tolerability criteria.

The steps used in this Procedure are intended to be conservative. For dams that don't meet the criteria outlined in this guidance document, particularly for High Hazard Dams, considerable conservatism is encouraged in selecting parameters to represent seismic loading and soil strength. For such dams with relatively high seismic loadings, say PGA > 0.35g, higher level analyses and more rigorous approaches are encouraged.

Regardless of the relative hazard of the dam and its potential seismic loading, the evaluations will only be as good as the geological, geotechnical, design, and construction information that is available to characterize the materials in the dam and foundation. If such information is limited, then the analyses and evaluations will likely not be relevant. In such cases, it is strongly encouraged to obtain as much information about the dam, its materials, and its history as possible and to document this for the future.

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Appendix A – Glossary of Terms

Definitions of terms covered here are selected from the following sources:

- State of the Art and Practice in the Assessment of Earthquake-Induced Soil Liquefaction and Its Consequences (2016), The National Academies Press, Committee on State of the Art and Practice in Earthquake Induced Soil Liquefaction Assessment; Board on Earth Sciences and Resources; Division on Earth and Life Studies; National Academies of Sciences, Engineering, and Medicine
- Idriss I. M. and Boulanger, Ross W. (2008), "Soil Liquefaction during Earthquakes," Monograph MNO-12, Earthquake engineering Research Institute.
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Amplification or deamplification of seismic waves: Modification of the amplitude and phase of seismic waves caused regionally by structures in Earth's crust (e.g., amplification in basins that are filled with soft sediments) and locally by near-surface deposits.

Consequence-Hazard Matrix: A risk analysis matrix with different levels of seismic hazard, in terms of earthquake return period for earth dams of varying reservoir height corresponding to three levels of consequences, represented by the hazard classification of the dam.

The matrix consists of three columns with ranges of reservoir height (less than 50 feet, 50-100 feet, and greater than 100 feet) and three rows with levels of hazard classification (high hazard, significant hazard, and low hazard). The recommended earthquake return period for an earth dam is found at the intersection of the corresponding row and column.

Cone Penetration Test (CPT): An in-situ soil testing procedure in which a standardized rod with a conical tip is pushed into the soil at a constant rate. The resistance at the tip (qc) and along a frictional sleeve (fc) are measured continuously as the probe advances.

Cyclic Resistance Ratio (CRR): The capacity of a soil at a particular depth and state to resist liquefaction triggering. It is evaluated by processing field data from standard penetration tests (SPT), cone penetration tests (CPT), shear-wave velocity (V_s) measurements, or other tests. These measurements are usually correlated, through case histories, to the minimum cyclic stress ratios (CSRs) at which surface manifestations of liquefaction were produced. Thus, the CRR is an estimate of the value of the CSR that could trigger liquefaction in a particular soil layer and depth.

Cyclic Softening: Cyclic softening" is used to describe the response of clay-like or cohesive soils to cyclic loading that involves loss of strength - the undrained cyclic loading of a clay sample results in a progressive increase in excess pore water pressure (decreasing effective stress) to some limiting level, at which time the sample develops rapidly increasing strains with each subsequent loading cycle. The excess pore pressure ratio reaches a limiting value of about r_u=80%, such that the sample never has less than about 20% of its initial effective stress. The evaluation of cyclic strengths by laboratory testing of field samples requires recourse to fundamental procedures, with each site potentially representing unique and challenging considerations. Cyclic softening can be evaluated using estimated CSR and values of peak undrained shear strengths.

Cyclic shear strain: (τ_{cyc}): Shear strain induced in soil due to cyclic loading.

Cyclic Stress Ratio (CSR): The seismic demand induced at a particular depth in the soil, usually expressed as the average earthquake-induced cyclic shear stress (τ_{cyc}) divided by the initial vertical effective stress (σ_{vo}').

Damping: Dissipation of energy associated with deformations of a dynamically loaded material or system. The critical damping ratio, commonly shortened to damping ratio (D or β), normalizes the damping in a material or system to the damping necessary to prevent oscillatory motion in free vibration. A damping curve relates the damping ratio to the amplitude of shear strain induced in the soil.

Drained Conditions: Conditions in which the hydraulic conductivity (permeability) is so large or loading is so slow that any excess porewater pressures dissipate during shear and do not contribute to the response of the soil.

Effective Stress: The normal stress in a soil element from which any porewater pressure has been subtracted. It represents the portion of the total stress that is transmitted by contacts between grains in the soil skeleton, but it is not equal to the contact stress between individual grains.

Intensity: A qualitative measure of the severity of earthquake shaking at a particular place. Determined from observations of the earthquake's effects on humans, buildings, and the Earth's surface.

Intensity measure: A quantitative measure of ground motion characteristics, such as peak ground acceleration or Arias intensity.

Liquefaction: The phenomena of seismic generation of excess porewater pressures and consequent softening and loss of strength of saturated granular soils, typically manifested by fluid-like behavior. The material is typically sand, less commonly silt or gravel.

Magnitude (Earthquake): A quantitative measure of the relative size of an earthquake, irrespective of the observer's location based on the energy released during the earthquake. There are many different definitions of earthquake magnitude, but the moment magnitude (M_w) is commonly used for the evaluation of earthquake loadings for dams. The moment magnitude scale does not saturate and, therefore, is a better measure for larger earthquakes than other magnitude scales, which include the local, or "Richter," magnitude, ML.

Magnitude Scaling Factor (MSF): A factor that adjusts the cyclic stress ratio (CSR) for durational effects, which correlate to earthquake magnitude. The adjustment is necessary because the standard liquefaction and cyclic softening relationships are based on earthquakes of M 7.5. In general, a larger earthquake lasts longer and has more cycles of loading, while a smaller earthquake has a shorter duration and has fewer cycles.

One-dimensional site response analysis: Mathematical analysis in which the earthquake is assumed to be composed of vertically propagating shear waves and analyzed using one-dimensional approaches..

Overburden correction (K_σ): A multiplicative factor used to correct the cyclic resistance ratio of liquefiable soils for the effects of initial confining pressure different from 1 atmosphere or 100 kPa.

Peak Ground Acceleration (PGA): Peak ground acceleration (PGA) is equal to the maximum ground acceleration that has occurred or will occur during earthquake shaking at the ground surface at a particular location..

Percentile: A percentile is a point on a scale where a certain proportion of the population of a data set lies at or below. Iff the distribution of a certain parameter is known then the percentile value is a measure of confidence of that the value is within the central tendency of the range. Median is the 50th percentile of a distribution - the score at which half of the data set lies above and half of the data set lies below. To identify the median, the midpoint of the rank order distribution is used. For example, a 75th percentile means that 75% of the population have scores at or below that value. Percentiles are a type of standard score, which means that a raw score is converted into a score that has a known ("standard") meaning. Usually the standard is a comparison to the distribution of scores in a population (of subjects or other objects being compared) or a comparison to a known range of values (as is the case in laboratory measures). In the case of percentiles, the known range is 0% to 100% and the midpoint (with half of the population above and half below) is 50%

Relative density (D_R): A measure of the density of a granular soil between its loosest and densest states, as determined from standardized laboratory tests or by correlations with other measurements such as penetration tests. It can be expressed in terms of minimum and maximum void ratios instead of densities. Relative density can be defined as $(e_{max} – e) / (e_{max} – e_{min})$, where e denotes void ratio.

Residual strength: Shear strength in a soil that has liquefied, and which may be undergoing shear strains orders of magnitude greater than those at which the liquefaction was triggered.

V_{S30}: This parameter is defined as the average seismic shear-wave velocity from the surface to a depth of 30 meters, has found wide-spread use as a parameter to characterize site response for simplified earthquake resistant design as implemented in building codes worldwide.

Appendix B – Simplified Seismic Analysis Flow Charts

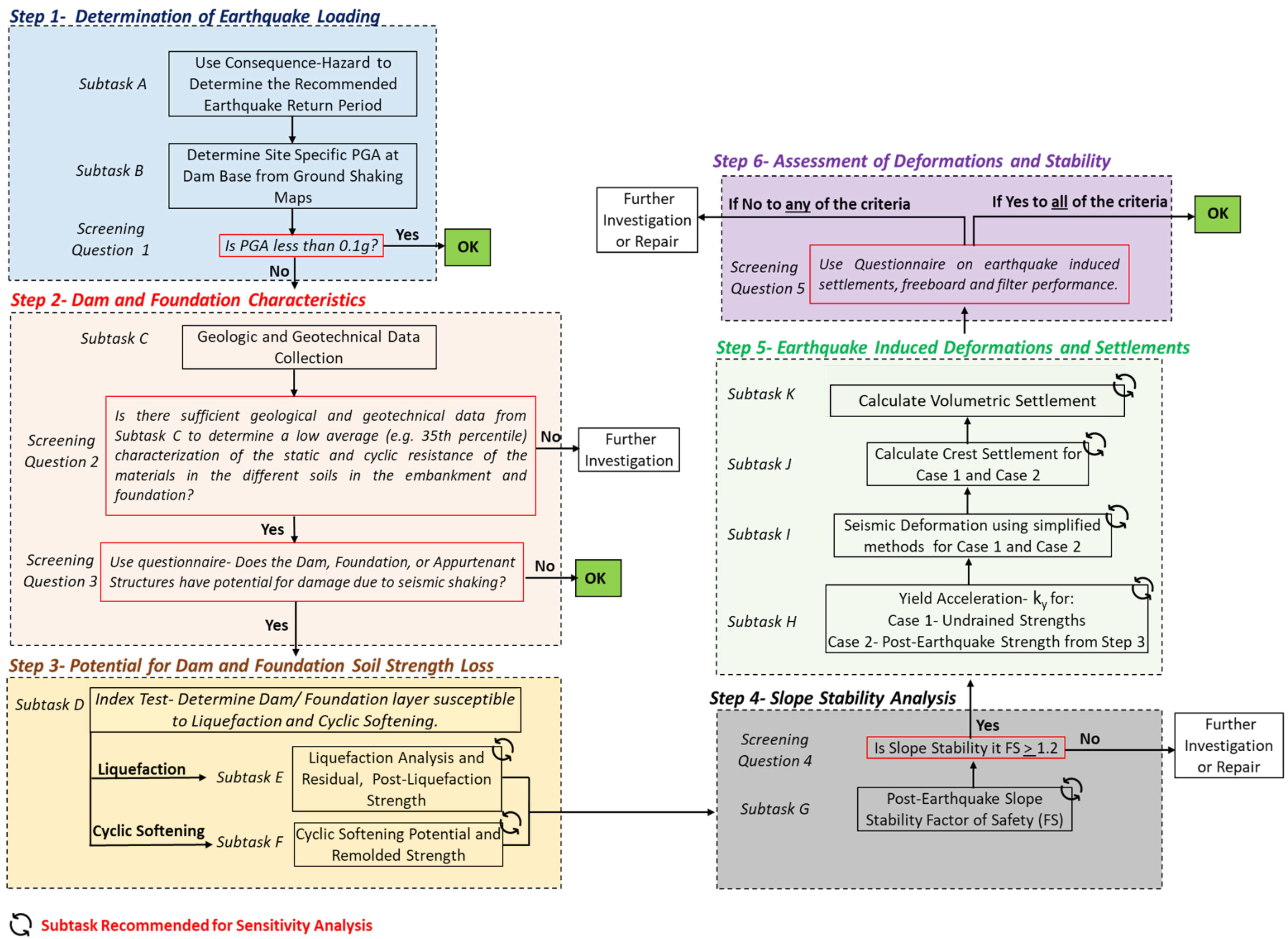


Figure B-1: Simplified Seismic Analysis Procedure

Step 1 – Determination of Earthquake Loading

Subtask A- Determine Earthquake Return Period based on Consequence-Hazard Matrix

Hazard Classification	Height of Dam		
	<50 feet	50 – 100 feet	>100 feet or Reservoir Volume > 5,000 acre-feet
High Hazard	5,000-year	5,000-year	10,000-year
Significant Hazard	2,500-year	5,000-year	5,000-year
Low Hazard	1,000-year	2,500-year	5,000-year

Subtask B- Determine PGA estimate at base of Dam

Method 1

Wong et al, 2005 Montana Seismic Hazard Study

1a. If the NEHRP V_{S30} Site Class = Soft Rock at the dam matches with the regional geological ground surface assignment in Wong et al, 2005, use the 2005 estimates for PGA and PSA for ground surface.

1b. If NEHRP V_{S30} Site Class at the dam does not match, use the 2005 estimates for Soft Rock (V_{S30} = 760 m/sec), and then apply appropriate amplification factors to modify ground motion, as described in Wong et al., 2005

Note: The 10,000-year return period motion can only be determined using a site specific Probabilistic Seismic Hazard Analysis, but as a rough starting point for a high PGA value relative to a known ground motion to proceed with the simplified analysis- determine the Factor of Increase in the ground motion estimate (PGA) moving from 2,500-year to 5,000-year motion. The Factor of Increase is determined from Wong et al., 2005.

Factor of Increase = (5,000−year PGA)/(2,500−year PGA)

Relative 10,000−year PGA = 2,500-year PGA x (Factor of Increase)^2

Method 2

USGS, 2014 with additional site class estimates from Shumway et al., 2018.

For 2,500-year estimates:

2a. Estimate the location and use the online USGS Unified Hazard Tool to if the dam is founded on Soft Rock (V_{S30} = 760 m/sec). If the dam is not founded on Soft Rock, use appropriate Seismic Hazard maps from the USGS 2018 Data Release for Additional Period and Site Class sites from Shumway et al., 2018.

For 5,000-year and 10,000-year estimates:

2b. For a Soft Rock site, estimate a Factor of Increase in the ground motion estimate (PGA or PSA) moving from 2% to 1% exceedance. This can be done by using the Wong et al., 2005 study.

Factor of Increase = (5,000−year firm rock PGA)/(2,500-year firm rock PGA)

Multiply the Factor of Increase with the 2,500-year PGA estimate from USGS 2014 to arrive at an equivalent 5,000-year soft rock PGA.

Apply this factor of increase twice to arrive at relatively high PGA value to proceed with the simplified analysis for 10,000-year return period.

Relative 10,000-year PGA = 2% in 50-year PGA x (Factor of Increase)^2

2c. For all site classes other than soft rock, multiply the estimate above with amplification factors provided in Shumway et al., 2018 to arrive at the ground motion estimate for the required site class.

Finally- Use average ground motion PGA estimate from **Method 1a OR 1b** and **Method 2a OR 2b OR 2c.**
For 1,000-year estimate, use only the online USGS Unified Hazard Tool as discussed in Method 2a.

Figure B-2: Step 1-Determination of Earthquake Loading

Step 2- Subtask C

1. **Regional and Site-Specific Geology:**
   - a. Broad-level Geology, Stratigraphy ☐
   - b. Dip and orientation of bedding planes ☐
   - c. Faults and shear zones
     - i. Slip rates for active faults, and footprint of faults ☐
   - d. Problematic rock or soil in the area ☐
2. **Design and Construction History:**
   - a. Design cross section and internal zoning ☐
   - b. Construction techniques:
     - i. Foundation
       - Foundation preparation ☐
       - Grouting ☐
     - ii. Embankment
       - Placement and compaction procedure ☐
       - Testing frequency and results, material gradations ☐
       - Construction timeline- issues during construction ☐
   - c. Filter analyses for dam and foundation materials ☐
   - d. Special geotechnical design consideration to overcome specific issues. ☐
3. **Penetrations:**
   - a. Penetrations such as Outlet Works or utility conduits that may sustain earthquake damage ☐
4. **Performance History:**
   - a. Seepage and settlement performance under static conditions (e.g. seepage and settlement). ☐
   - b. Seismic and hydrologic loading on the dam and documented performance ☐
5. **Geotechnical Parameters for Foundation and Embankment:**
   - a. Index properties for material- Gradation and Atterberg Limits ☐
   - b. SPT, CPT, BPT test data ☐
   - c. Shear wave velocities (V_{s30}) ☐
6. **Operations and Maintenance Information**
   - a. Maximum normal operating reservoir elevation and maximum design limits ☐
   - b. Winter/ summer reservoir operational plans ☐
   - c. Spillway or low-level outlet capacity and operating limits ☐
   - d. Ability to quickly inspect for damage and possibly intervene after earthquake events ☐

Figure B-3: Step 2-Subtask C

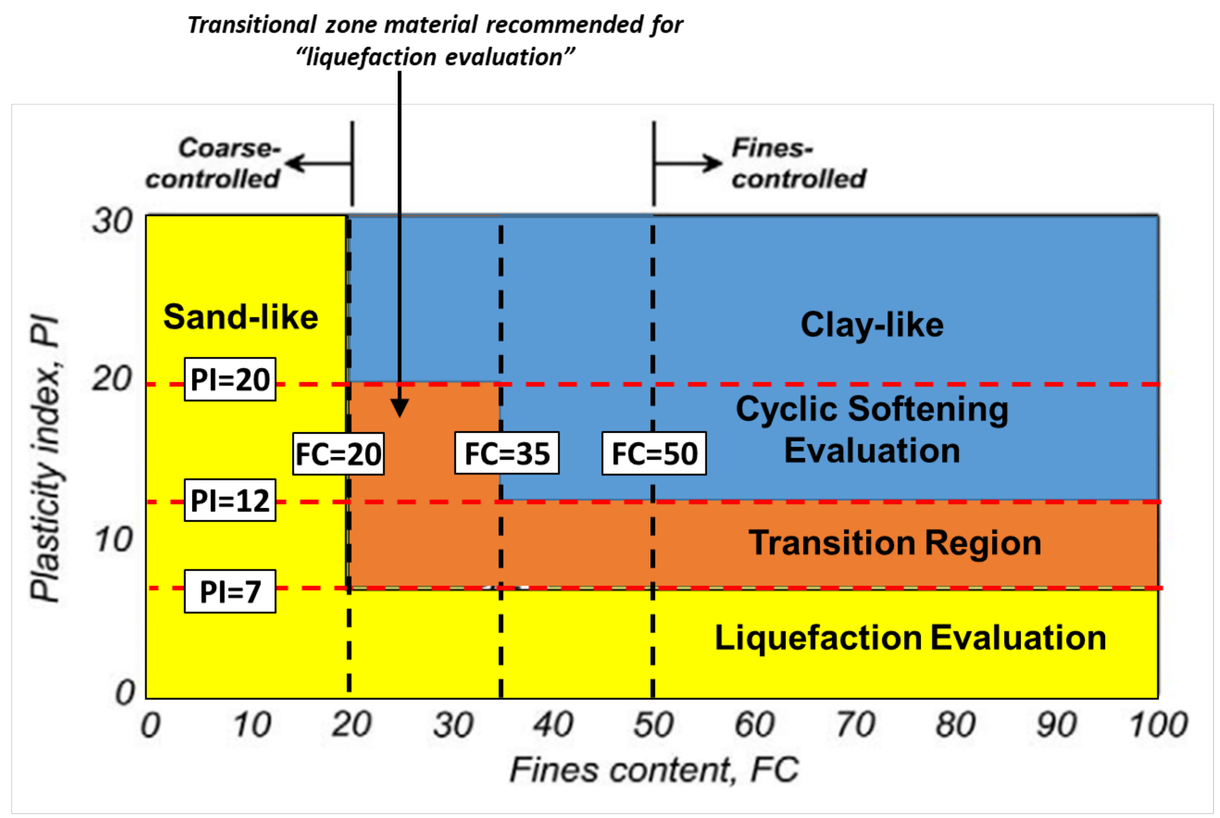
Step 2- Screening Question 3

1. Materials within the dam and foundation are NOT composed of any one of the following types ☐
    i. Liquefiable soils
    ii. Sensitive clay
    iii. Clayey soils with potential for softening with cyclic loading: including the puddled clayey cores, hydraulic fills, and normally consolidated clay foundations.
2. Is the dam well-built and compacted to at least 95 percent relative compaction or relative density greater than 75 percent? ☐
3. Are the upstream slopes 3:1 or flatter for earth dams, or 2:1 or flatter for dams with upstream rockfill shells, AND downstream slopes 2:1 or flatter, AND does the phreatic line NOT exit on the downstream face of the dam? ☐
4. Is PGA at the dam is less than or equal to 0.30g and the predominant earthquake magnitude is less than or equal to M = 6.5 for earth dams and M = 7.0 for earth and rockfill dams. ☐
5. Does the dam have static slope stability factors of safety equal to or greater than 1.5 for potential sliding surfaces that might involve loss of crest elevation? ☐
6. For dams with heights of 100 feet or less, is the available total freeboard equal to at least 10 percent of the embankment height, but not less than 6 feet? For dams higher than 100 feet, is the available total freeboard equal to at least 10 feet? ☐
7. Can it be documented that critical appurtenant features or penetrations that might lead to an uncontrolled release of the reservoir would not be harmed by small movements of the embankment following a seismic event? Documentation can be based on engineering judgment after detailed of review of design and as-built documents ☐

Figure B-4: Step 2-Screening Question 3

Step 3- Subtask D

Liquefaction and Cyclic Softening Susceptibility Index Test



Soils in yellow AND orange region are considered potentially susceptible to liquefaction. Proceed to ***Subtask E***

Soils in the blue region are considered potentially susceptible to cyclic softening. Proceed to ***Subtask F.***

Figure B-5: Step 3- Subtask D

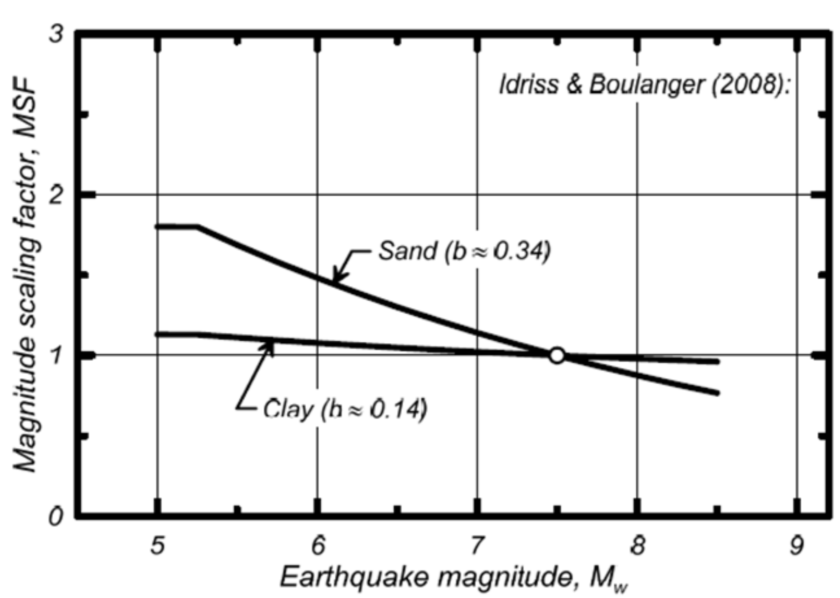
Step 3- Subtask E

Liquefaction Triggering and Post-Liquefaction Strengths

Use Bedrock PGA determined in Step 1 as ground motion (a_{max}/g) at the base of the dam

Foundation Liquefaction
Bedrock PGA estimate from Step 1 can be used going forward

Embankment Liquefaction
Use Harder, 1997 to transform the Bedrock PGA to PGA at Dam Crest



Cyclic Stress Ratio (CSR)
Determine the earthquake induced CSR at regular intervals along the embankment and foundation as described in Idriss & Boulanger (2008) adjusting for the stress reduction coefficient (r_d), earthquake magnitude effects (MSF) and overburden effects.

$$\mathrm{CSR}_{M=7.5,\sigma'_{vc}=1} = 0.65\frac{a_{\max}}{g}\frac{\sigma_{vc}}{\sigma'_{vc}}r_d\frac{1}{\mathrm{MSF}}$$

Cyclic Resistance Ratio (CRR)
Determine CRR based on SPT or CPT at regular intervals along the embankment and foundation as described in Idriss & Boulanger (2008) applying correction factors: Overburden correction K_σ and Sloping Ground correction K_α

Factor of Safety
Calculate the Factor of Safety against liquefaction- FS_{liq} or $FS_{liquefaction}$

$$FS_{liq} = \frac{\mathrm{CRR}_{M=7.5,\sigma'_{vc}=1}}{\mathrm{CSR}_{M=7.5,\sigma'_{vc}=1}}$$

Figure B-6: Step 3- Subtask E

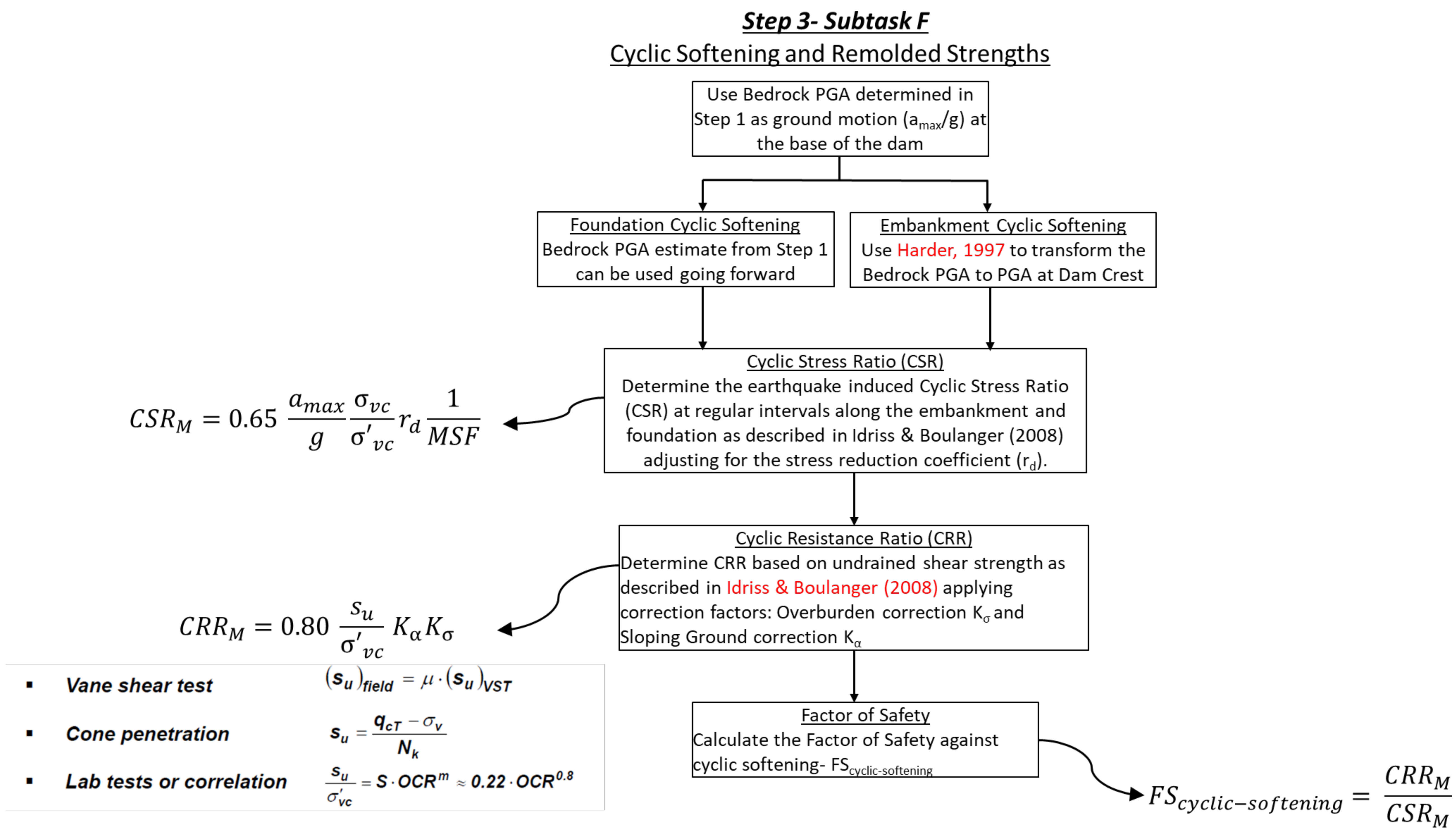


Figure B-7: Step 3- Subtask F

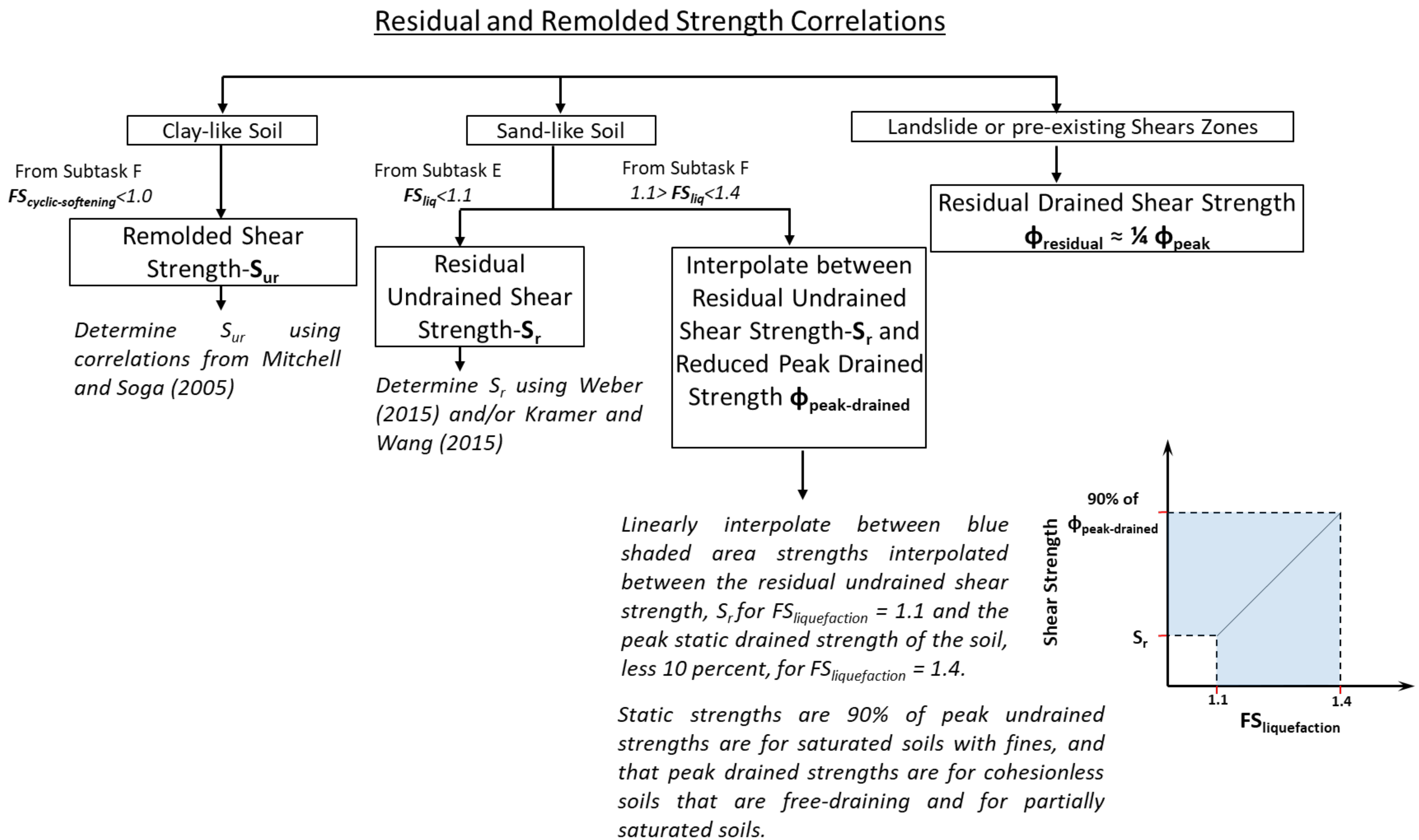


Figure B-8: -Summary of Strength Selection

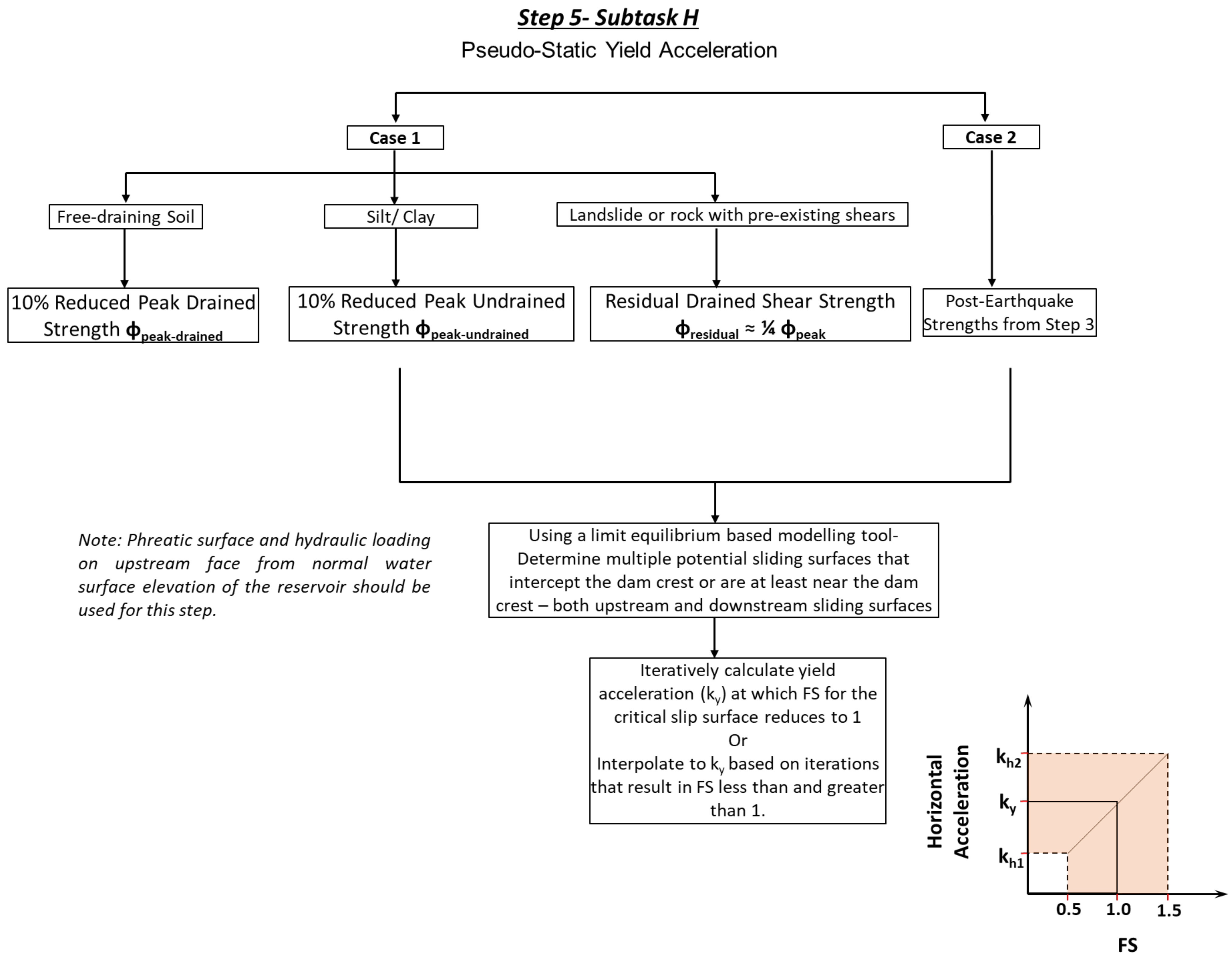
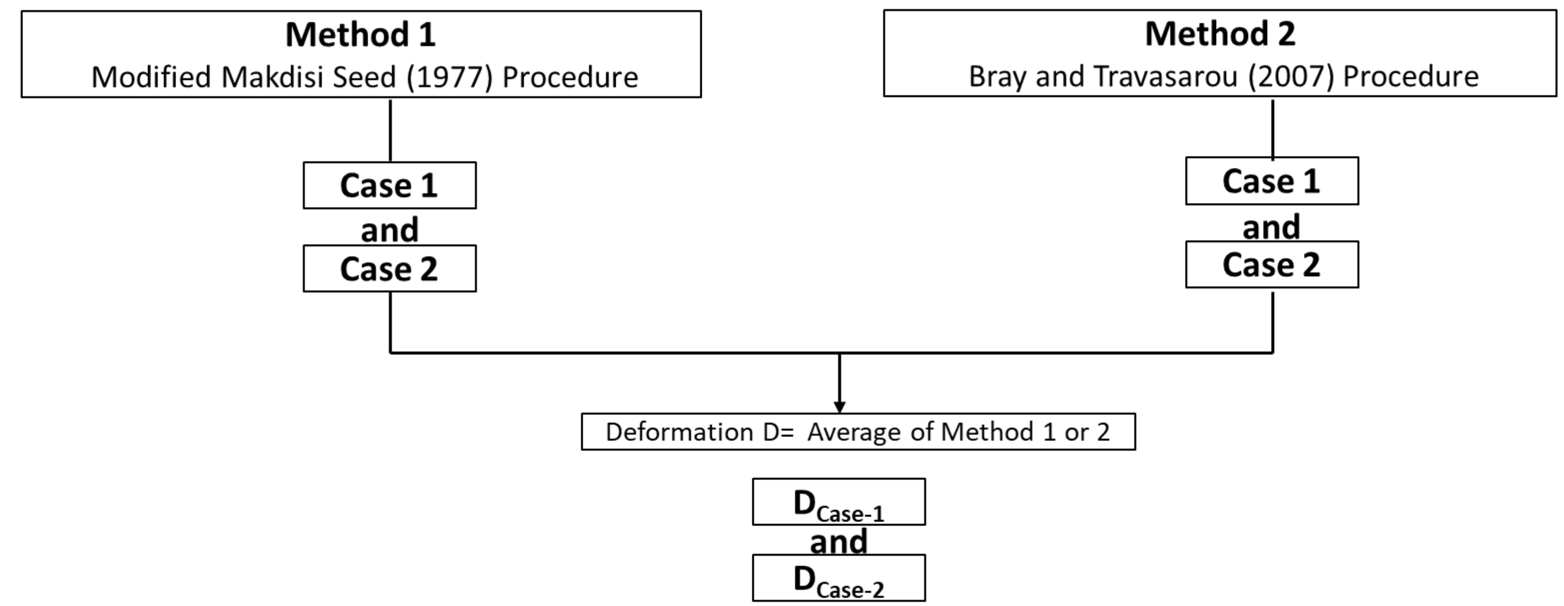


Figure B-9: Step 5- Subtask H

Step 5- Subtask I

Seismic Deformation Analysis



Case 1 - *Static shear strengths for all materials (90% of peak undrained/drained strengths)*

Case 2 - *Post-earthquake residual and remolded shear strengths as from Step 3.*

Figure B-10: Step 5- Subtask I

Step 5- Subtask J

Crest Settlement

Crest Settlement (S_{Crest}) associated with sliding or spreading = 0.7 x Newmark-based Deformation (determined in Subtask I)

↓

Calculate total crest settlement (S_{total_1}) using the Swaisgood (2014)

↓

Step 5- Subtask K

Volumetric Crest Settlement

$S_{volumetric}$

Sands- Use Tokimatsu and Seed (1987) correlations

Soils with significant fines content- Use Stewart et al. (2004)

↓

$S_{total\_2} = S_{Volumetric} + S_{Crest}$ (from Step 5)

↓

Compare S_{total_1} and S_{total_2}
Use higher of the two for Step 6

Figure B-11: Step 5- Subtask J and K

Step 6- Screening Question 5

Tolerable limits for resulting deformations are:

- Total earthquake-induced settlement less than 0.5 – 1 percent of dam height, and
  - less than 0.5 to 3 feet for dams higher than 100 feet in height.
  - High Hazard Dams should have total earthquake-induced crest settlements of less than 0.5 percent..
- Total earthquake-induced crest settlement less than one-third of total freeboard.
- Total earthquake-induced crest settlement less than one-half of core freeboard, defined as the top elevation of the core relative to the elevation of the reservoir.
- Total earthquake-induced displacement less than one-half of width of filter/transition zones

In addition,

- All filter zones predominantly “no erosion”, or “some erosion”, with limited “excessive erosion” or “continuing erosion” gradations.

Figure B-12: Step 6- Screening Question 5