CHAPTER 2

IS GROUND IMPROVEMENT NECESSARY?

A number of analyses and decisions may be required to determine if ground improvement is necessary. A series of flow charts to aid in this process are listed in Table 2 and included as Figures 1 through 26. Each level of analysis, which is represented by a single chart or a series of charts, requires progressively more detailed information. Figure 1 shows the overall evaluation process necessary to assess the need for ground improvement for a facility. Figure 2 can be used for a preliminary evaluation of site conditions and design/performance requirements. If, based on the results of the preliminary evaluation, more detailed analyses are required, Figures 3 through 8 are used. These charts include evaluations for difficult soils, liquefaction potential, slope stability, bearing capacity and settlement, and seepage instability. "Difficult soils" include collapsing soils, expansive soils, sensitive clays and dispersive clays. These soil types are discussed below under the heading "Difficult Soils Evaluation." The evaluations for difficult soils, bearing capacity and settlement, and seepage instability are complete after this step.

A further level of analysis could be required for liquefaction and slope stability evaluations. These analyses are performed to estimate deformations for situations where the factor of safety is inadequate. The steps necessary for gross deformation estimates are shown in Figures 9 and 10, while the procedure for refined deformation estimates is shown in Figure 11. Methods for determination of the properties and parameters listed in Figures 2 through 11 are described in Figures 12 through 26.

Preliminary Evaluation

The preliminary evaluation (Figure 2) can be performed for new or existing facilities. For the preliminary evaluation, project performance requirements need definition and site

characterization must be completed. The project performance requirements that pertain to the potential need for ground improvement include loading conditions and allowable deformations for the facility, as well as an assessment of the impacts of natural hazards, such as floods, earthquakes or hurricanes, and the performance required during these events. For a new facility, the performance requirements should be determined during the early stages of analysis and design. For an existing facility, the performance requirements may be the result of an upgrade in the facility or deficiencies requiring remedial work to improve performance during a flood or an earthquake. In addition, re-evaluations of hazards, such as earthquake magnitude, peak flood and sustained wind velocity, often lead to increased demands on structures and facilities so that retrofitting is required.

The site characterization step includes investigations to evaluate the soil profile, ground water levels and soil properties. New projects will likely require a detailed geotechnical investigation or series of investigations to obtain the information necessary to make ground improvement decisions. Guidelines for planning these studies are presented in EM 1110-1-1804, Geotechnical Investigations. The geotechnical investigations can be performed in stages, beginning with a preliminary subsurface investigation and proceeding to more detailed investigations as more specific and detailed information is required.

At existing facilities, old records, such as geotechnical investigation reports and boring logs, may provide sufficient information to make decisions regarding the need for ground improvement. However, it is likely that supplemental information or investigations will be necessary. Additional geotechnical investigations should be performed in accordance with EM 1110-1-1804.

All available information should be used to aid in the decision-making process. Regional geologic references can be consulted for general information about the soil composition, fabric and structure. Experience with similar soils or nearby sites can be used to provide guidance regarding the performance of a soil and the need for ground improvement. Boring log data from adjacent properties can provide information about the stratigraphy

and ground water conditions in the immediate vicinity of the site. Assessment methods for design/performance requirements and subsurface conditions are presented in Figures 12 through 16.

The information on the subsurface conditions should be used in conjunction with the project performance requirements to make a series of decisions regarding the need for further analysis. Further analysis is required if there is evidence of any of the following:

1. difficult soils, such as expansive or collapsing soil and sensitive or dispersive clay;
2. potential for liquefaction;
3. potential for slope instability;
4. inadequate bearing capacity or excessive settlement; and,
5. potential for excess seepage, high uplift pressures, or erosion and piping.

The flow chart in Figure 2 requires a "Yes" or "No" answer for each of the five items listed above. If the answer to one or more of the decisions is "Yes," then an additional evaluation for each item with a "Yes" response should be performed before a decision can be made regarding the need for ground improvement (or alternative corrective action). The additional evaluations are discussed below. If the answer to every one of the five decisions is "No," then ground improvement is not required and further evaluation is not necessary.

Difficult Soils Evaluation

Difficult soils are considered to be collapsing soils (e.g. loess, mud and debris flow deposits, hydraulic fills and tailings deposits), expansive soils, sensitive clays and dispersive clays. Collapsing soil deposits have a loose, collapsible structure. When saturated and disturbed, collapsing soils can undergo large decreases in volume or liquefy with sudden loss of strength. Expansive soils can also experience extreme volume changes, but for different reasons. While the low density soil structure is the primary reason for volume

change in collapsing soils, soil composition is the usual culprit in expansive soils. Most expansive soils contain smectite clays such as montmorillonite or bentonite. In the presence of water, these clays attract free water and swell; in the absence of water, the clays release free water and shrink. A detailed discussion of expansive soils is provided in Wray (1995).

Sensitive clays lose undrained strength when remolded. Sensitivity can be formed by a variety of factors, including metastable fabric, cementation, leaching, weathering, thixotropic hardening, and formation or addition of dispersing agents. Dispersive clays are highly erodible because the clay particle associations are structurally unstable and easily dispersed. The individual particles will spontaneously detach from each other and go into suspension in quiet water.

The steps necessary for difficult soils evaluation are listed in Figures 3 and 4. Assessment methods for soil state parameters are shown in Figure 17. If difficult soils are present at a site, the need for remedial action depends on the type of facility under consideration. Dispersive clays are a threat to dams and levees because they can initiate erosion and piping through the embankment or foundation that may lead to failure. Numerous canals in the west and southwest are constructed in collapsing soils or dispersive clay. Sensitive clays can be a concern for natural slopes. Collapsing and expansive soils may be more of a concern for structures with footings that could be exposed to water. Engineering judgment is required to make the final determination as to whether improvement of difficult soils is required.

Liquefaction Evaluation

Loose, saturated sands are susceptible to liquefaction or lateral spreading if subjected to earthquake motion. The development of excess pore water pressures and the subsequent loss of soil strength associated with liquefaction can result in ground settlement, lateral

spreading, and/or loss of foundation support. The potential liquefaction hazards at a site can be evaluated by considering the following questions:

1. Is the soil susceptible to liquefaction?
2. If the soil is susceptible, will liquefaction be triggered?
3. If liquefaction is triggered, will damage occur?

Figure 2 can be used to address the first question. If the answer to the liquefaction question in Figure 2 is "No," it can be concluded that a liquefaction hazard does not exist. If the answer to the liquefaction question in Figure 2 is "Yes," Figure 5 can be used to evaluate the liquefaction potential, which will address the second question. If the factor of safety against liquefaction is above 1.5 and the anticipated settlement is less than half the allowable amount, ground improvement is not required and the liquefaction analysis is complete. If the factor of safety against liquefaction is less than one and the anticipated settlement is more than twice the allowable amount, liquefaction will likely be triggered and the anticipated deformations may be too high. Ground improvement or other mitigation methods will be required. If the results of the analysis are between these limits, gross deformation estimates, which are outlined in Figure 9, are necessary before ground improvement decisions can be made.

The gross deformation estimates involve calculations to determine a bearing capacity factor of safety and the amount of settlement and lateral deformation anticipated. If the bearing capacity safety factor is greater than 1.2, and the anticipated settlement and lateral deformation are less than half the allowable vertical and horizontal movement, respectively, ground improvement is not required and the liquefaction analysis is complete. If the bearing capacity factor of safety is less than 0.8 or the anticipated settlement or lateral deformation is more than twice the allowable vertical or horizontal movement, respectively, it is likely that liquefaction will be triggered and the anticipated deformations will probably be too high. Ground improvement or other mitigation methods will be required. For major projects, if the results are between these limits, a refined deformation estimate may be warranted before ground improvement decisions can be made. The parameter as-

sessment methods required for the gross deformation estimates are summarized in Figure 22.

The refined deformation estimates require that settlement and lateral spreading be calculated using a dynamic deformation analysis. Figure 11 is a flowchart which outlines the steps necessary for a refined deformation analysis. Assessment methods for the parameters necessary for the refined deformation estimates are shown in Figures 24 through 26. If the results of the deformation analysis indicate that the anticipated lateral deformation or settlement are more than two-thirds the allowable, ground improvement or other mitigation methods will be required. Otherwise, ground improvement is not required. The liquefaction analysis is complete after this step.

Stability Evaluation

For dams, levees and slopes, stability evaluations will usually be required. The most common method for stability evaluation is a limit equilibrium analysis. Factors which must be considered in the analysis include static loading conditions, earthquake loading, soil and rock parameters, and site conditions. Figure 6 is a flowchart which outlines the factors and parameters required to perform a limit equilibrium stability analysis. Limit equilibrium slope stability analysis are discussed in EM-1110-2-1902, Stability of Earth and Rockfill Dams. Methods for assessing the parameters necessary for slope stability analyses are discussed in that manual. Parameter assessment methods are also summarized in Figure 20.

If the site is located in a seismically active area, a pseudostatic limit equilibrium analysis is the simplest and usually the first type of analysis used to consider the effects of seismically-induced motions. In a pseudostatic analysis, the earthquake shaking is represented by horizontal and vertical inertial forces applied at the centroid of the failure mass (Kramer, 1996). These forces, called pseudostatic forces, are calculated by multiplying the weight of the failure mass by vertical and horizontal pseudostatic coefficients. The effect of the pseudostatic forces on the factor of safety is then determined in a limit equi-

librium analysis. If the analysis results in a pseudostatic factor of safety less than that required for the particular facility, which is often 1.0, the slope is considered to be unstable. The vertical inertial forces usually have a negligible effect on the calculated factor of safety and are often ignored in the analysis.

The most important factor in performing a pseudostatic analysis is selection of the appropriate pseudostatic coefficient. The selection of the coefficient should be related to the anticipated ground motion in some way, because it controls the additional force applied to the failure mass. The value selected is often significantly less than the peak acceleration for two reasons. First, the duration of the peak acceleration is usually short. Also, applying an inertial force equal to the product of the horizontal acceleration and the potential sliding mass would be appropriate only for a rigid material. Since the slope can deform under earthquake loading, the applied force will be smaller than this (Kramer, 1996).

In selecting a pseudostatic coefficient for design, Kramer (1996) recommends that the coefficient correspond to some fraction of the anticipated peak acceleration. Since the pseudostatic method was first used, many studies have been performed to evaluate appropriate values for the pseudostatic coefficient (e.g. Terzaghi, 1950, Seed, 1979a, Marcuson, 1981). Several of these studies are reviewed in Kramer (1996).

Hynes-Griffin and Franklin (1984) applied the Newmark sliding block analysis (Newmark, 1965) to over 350 accelerograms to predict permanent deformations using a yield acceleration and assuming a rigid slope material. The yield acceleration depends on the soil properties and the geometry of the slope. When the induced acceleration is greater than the yield acceleration, permanent deformation occurs along the failure plane. Hynes-Griffin and Franklin (1984) determined that "dangerously large" deformations would not develop in earth dams if the pseudostatic factors of safety is greater than 1.0 using $k_h = 0.5\ a_{max}/g$. Kramer (1996) suggests that this criterion should be appropriate for most slopes, although engineering judgment is necessary in all cases.

If the factor of safety is found to be inadequate using the pseudostatic method, a detailed deformation analysis is required. A simplified method for estimating earthquake-induced deformations for dams and embankments was developed by Makdisi and Seed (1978). The method is based on the Newmark sliding block analysis, but accounts for the dynamic behavior of the embankment rather than assuming rigid body behavior. The method makes several simplifying assumptions, including: (1) failure occurs on a well-defined slip surface, (2) the soil behaves elastically at stress levels below failure, and (3) the soil behaves plastically at stress levels above the yield stress. The earthquake-induced accelerations are represented by average time histories calculated using dynamic response analyses.

The factors and parameters required to perform gross deformation estimates by the Makdisi-Seed method are outlined in Figure 10. The earthquake parameters required for the analysis are shown in Figure 15, while the soil parameters required are shown in Figure 23. Note that the procedure was developed for dams and embankments. Therefore, if it is used for other types of slopes, the results should be interpreted with caution.

If the results of the gross deformation analysis indicate that the anticipated displacement is tolerable, ground improvement is not required and the stability analysis is complete. However, if the anticipated displacement is greater than the allowable displacement, a refined deformation analysis will be required before ground improvement decisions can be made. The procedure for performing a refined deformation analysis was discussed above under the heading "Liquefaction Evaluation."

Bearing Capacity and Settlement Evaluation

For a new structure, a bearing capacity and settlement evaluation can be performed to determine if adequate bearing capacity is available and if estimated settlements will be in the permissible range. If the results of the evaluation indicate that the bearing capacity may be too low or that excessive settlements are likely, ground improvement may be one way to

solve the problem. For the case of a lightly loaded structure placed on a cohesive "crust" over a liquefiable layer, a simplified procedure has been developed by Naesgaard et al. (1998) to determine the factor of safety against bearing failure and to estimate the deformation of the foundation after liquefaction. If the factor of safety against bearing failure is adequate and the anticipated settlements are tolerable, it may not be necessary to improve the liquefiable layer. For existing facilities, if excessive settlement has occurred or there is evidence that the bearing capacity may be inadequate, ground improvement may be a suitable remedial measure. The procedures for the bearing capacity and settlement evaluation are outlined in Figure 7. The parameter assessment methods required for the evaluation are summarized in Figure 20.

Seepage Evaluation

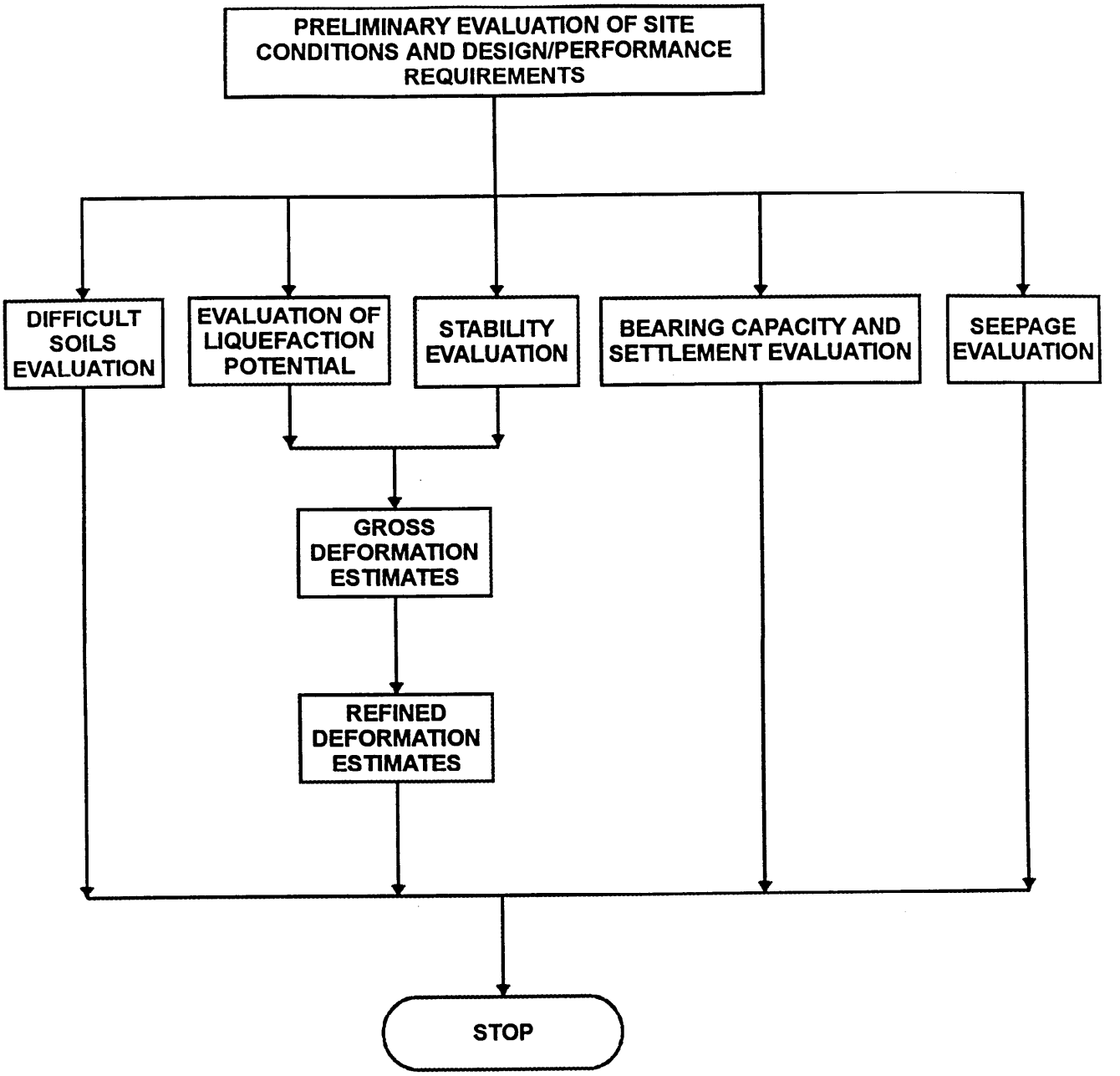
A seepage evaluation will be required for all dams and levees. Ground improvement methods may have applications if the seepage quantity or uplift pressures are too high, or if the factor of safety against erosion and piping is too low. Figure 8 is a flow chart which outlines the factors and parameters necessary to perform the seepage evaluation. Assessment methods for the factors and parameters listed in Figure 8 are summarized in Figure 21.

Table 2 - Flow Charts for Determination of the Need for Ground Improvement

Figure	Title	Page
1	Evaluation of Site Conditions and Design/Performance Requirements to Assess Need for Ground Improvement	17
2	Preliminary Evaluation of Site Conditions and Design/Performance Requirements	18
3	Difficult Soils Evaluation – Collapsing or Expansive Soils	20
4	Difficult Soils Evaluation – Sensitive or Dispersive Clay	21
5	Evaluation of Liquefaction Potential	22
6	Slope Stability Evaluation	24
7	Bearing Capacity and Settlement Evaluation	25
8	Seepage Evaluation	26
9	Evaluation of Liquefaction Potential – Gross Deformation Estimates	27
10	Slope Stability Evaluation – Gross Deformation Estimates	29
11	Refined Deformation Estimates for Liquefaction and Slope Stability Evaluations	30
12	Assessment Methods for Soil Classification and Experience Parameters for Preliminary Evaluation of Site Conditions and Design/Performance Requirements	32
13	Assessment Methods for Boundary Condition Parameters for Preliminary Evaluation of Site Conditions and Design/Performance Requirements	33
14	Assessment Methods for Loading Conditions and Settlement Parameters for Preliminary Evaluation of Site Conditions and Design/Performance Requirements	34
15	Assessment Methods for Earthquake Characteristic Parameters for Preliminary Evaluation of Site Conditions and Design/Performance Requirements	35
16	Assessment Methods for Flood Parameters for Preliminary Evaluation of Site Conditions and Design/Performance Requirements	36
17	Assessment Methods for Soil State Parameters for Difficult Soils, Slope Stability, and Seepage Evaluations	37
18	Assessment Methods for Earthquake Loading and Liquefaction Resistance Parameters for Evaluation of Liquefaction Potential	38
19	Assessment Methods for Strength Properties for Slope Stability and Bearing Capacity Evaluation	39
20	Parameter Assessment Methods for Slope Stability, Bearing Capacity and Settlement Evaluations	40
21	Parameter Assessment Methods for Seepage Evaluation	41
22	Parameter Assessment Methods for Evaluation of Liquefaction Potential - Gross Deformation Estimates	42

Table 2 (cont.) - Flow Charts for Determination of the Need for Ground Improvement

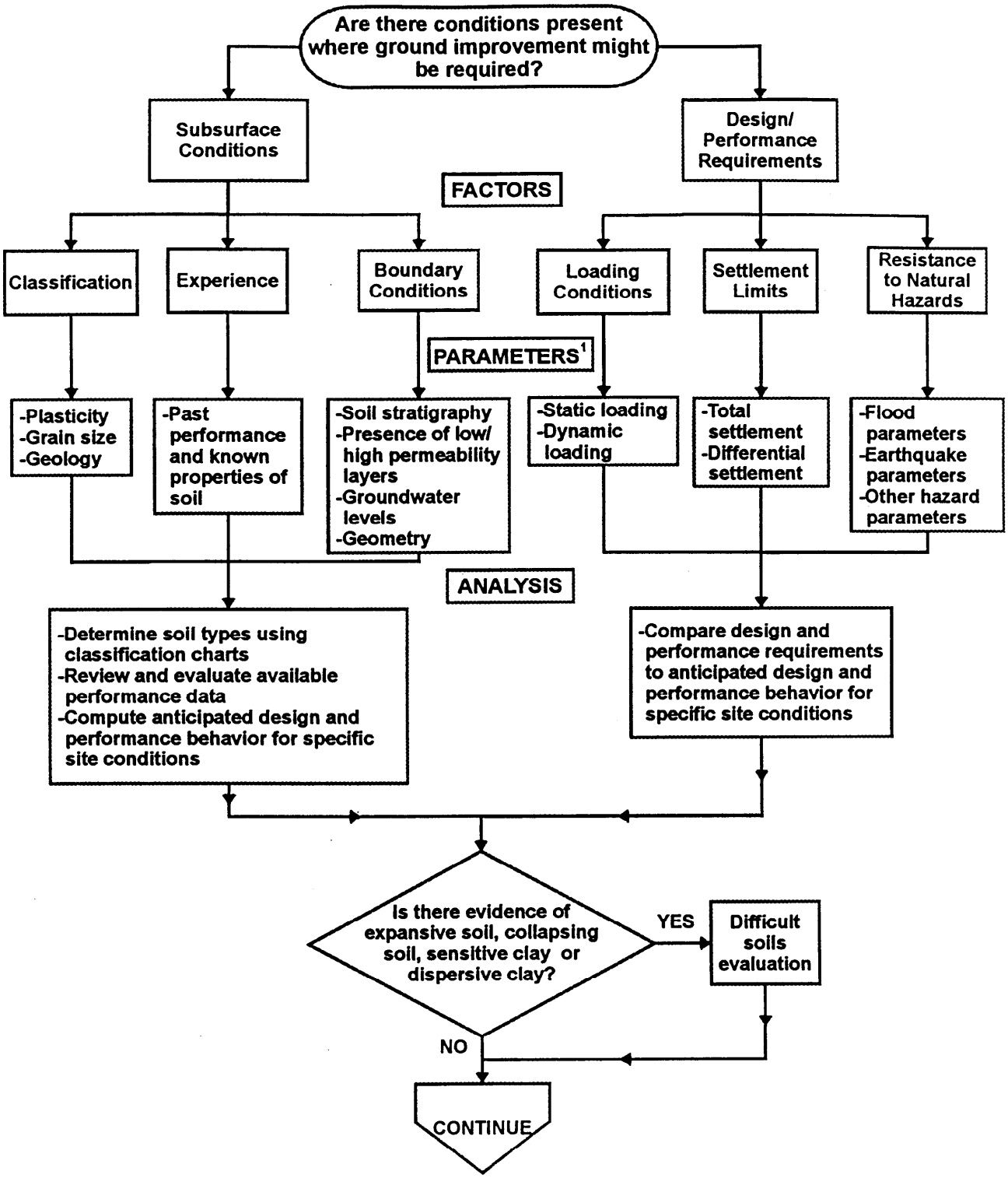
Figure	Title	Page
23	Assessment Methods for Soil Parameters for Slope Stability Evaluation – Gross Deformation Estimates	43
24	Assessment Methods for Earthquake Loading and Stress State Properties for Refined Deformation Estimates for Liquefaction and Slope Stability Evaluations	44
25	Assessment Methods for Strength Properties for Refined Deformation Estimates for Liquefaction and Slope Stability Evaluations	45
26	Assessment Methods for Stiffness and Recompression Properties for Refined Deformation Estimates for Liquefaction and Slope Stability Evaluations	46



Notes:

1. The factors, relevant parameters, analytical methods, and decisions for each step are given in Figures 2 through 11.

FIGURE 1 Evaluation of Site Conditions and Design/Performance Requirements to Assess Need for Ground Improvement



Notes:

1. Assessment methods for parameters are given in Figures 12 through 16.

FIGURE 2 Preliminary Evaluation of Site Conditions and Design/Performance Requirements

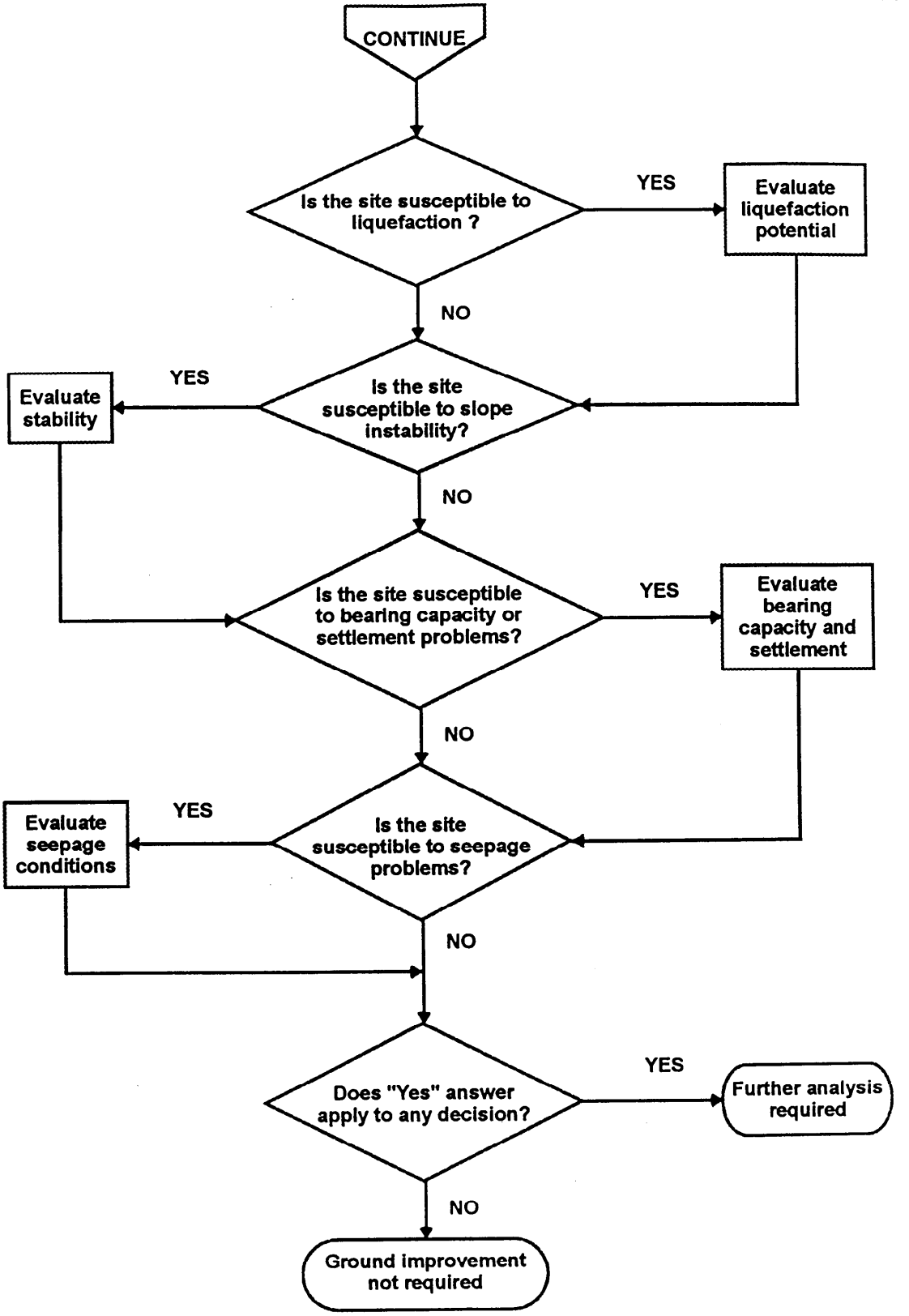
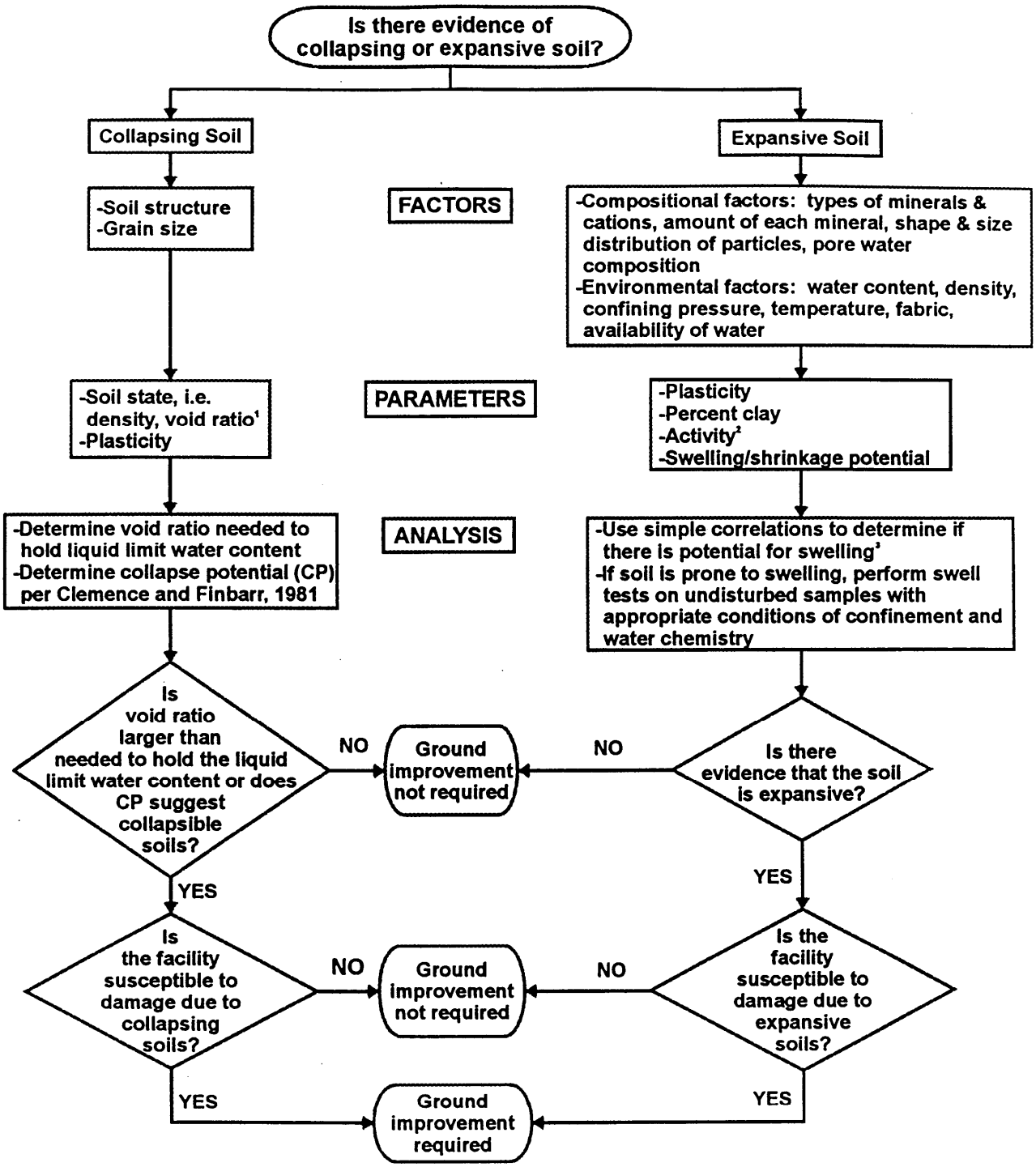
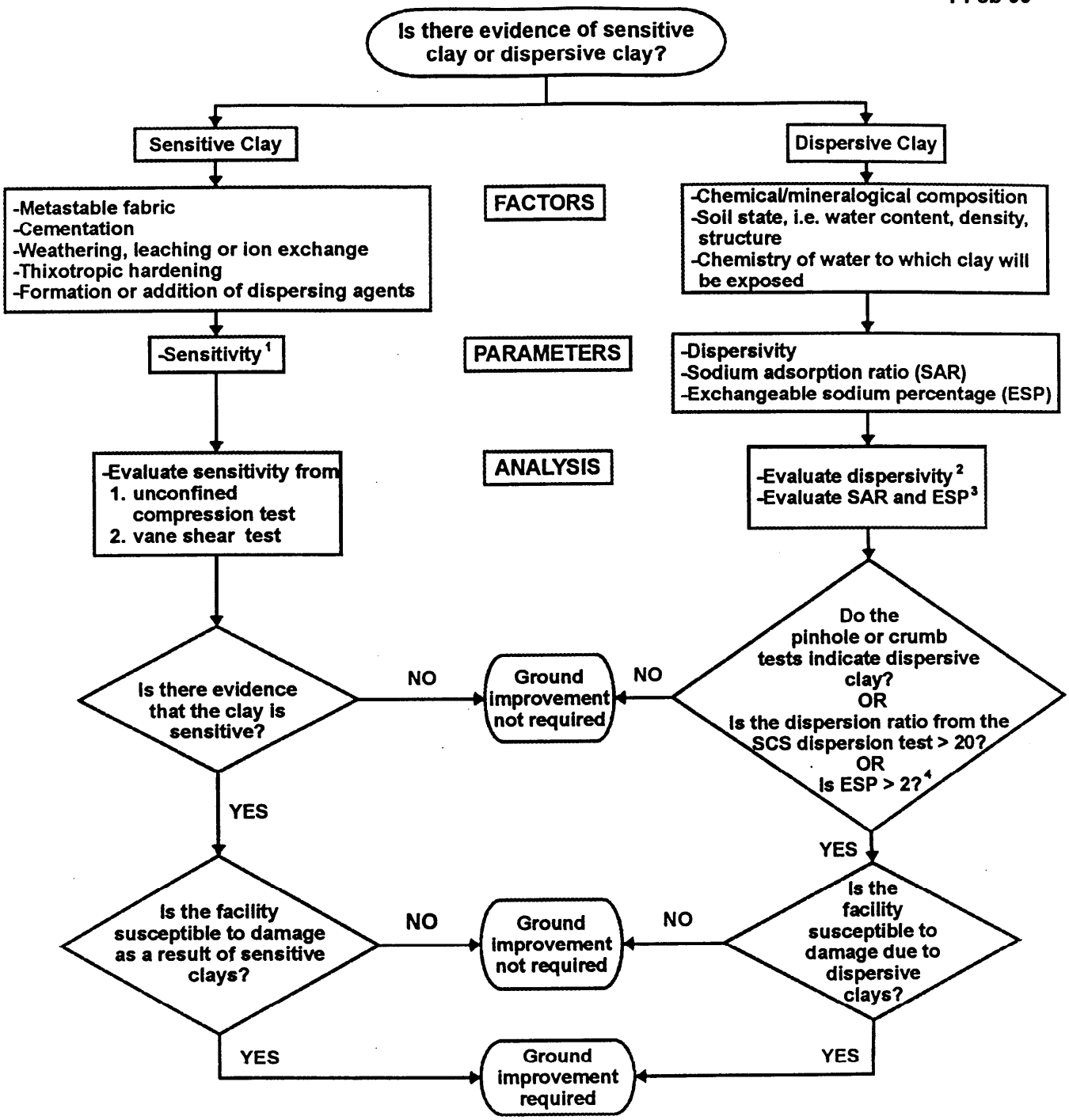


FIGURE 2 continued



Notes:
1. See Figure 17 for assessment methods for soil state parameters.
2. Activity, A = (Plasticity Index)/(Percent clay)
   Percent clay, C = Percent by weight of particles finer than 2 microns
3. Two correlations are discussed in Mitchell (1993), pp. 186-187

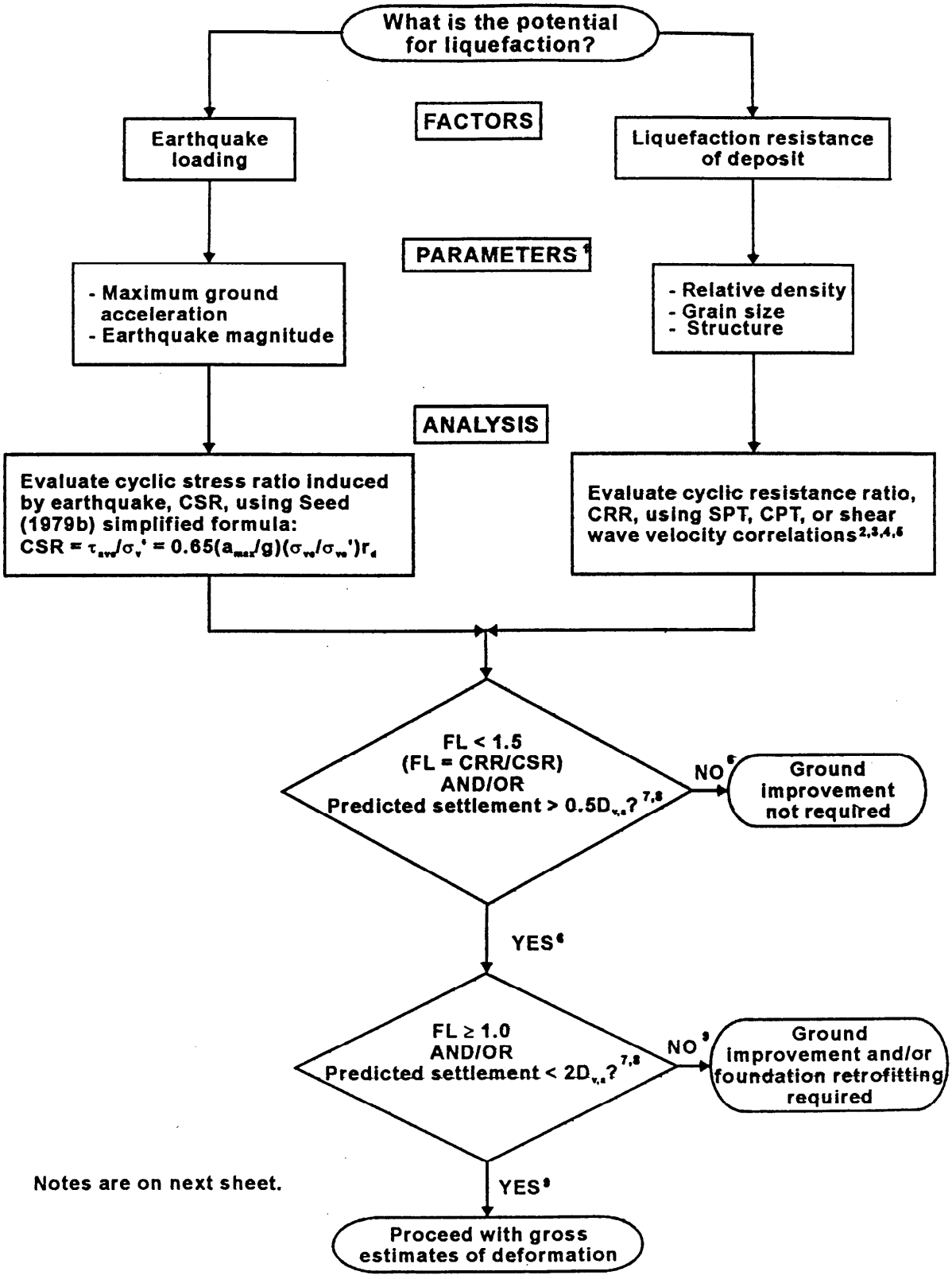
FIGURE 3 Difficult Soils Evaluation - Collapsing or Expansive Soils



Notes:

1. Sensitivity, S_t, is the ratio of peak undisturbed strength to remolded strength at the same water content.
2. Evaluate dispersivity from pinhole test (ASTM D 4647), SCS dispersion test (ASTM D 4221) or crumb test (Sherard et al., 1976). Pinhole test is considered most reliable (Mitchell, 1993).
3. Evaluate SAR by chemical analysis of pore water. Calculate ESP from SAR (Mitchell, 1993).
4. "No" response appropriate if it applies to all results from dispersivity tests. Otherwise, "Yes" response appropriate.

FIGURE 4 Difficult Soils Evaluation - Sensitive or Dispersive Clay



Notes are on next sheet.

FIGURE 5 Evaluation of Liquefaction Potential

Notes:

1. Assessment methods for parameters are given in Figure 18.

2. Evaluation of liquefaction resistance by CPT is generally preferred because penetration data is nearly continuous with depth and more reliable. Obtain SPT and CPT correlations with CRR from NCEER (1997) for clean sands.

   Correct SPT and CPT correlations with CRR per NCEER (1997) for: fines content, influence of thin soil layers, earthquake magnitudes different than M = 7.5, vertical effective confining stress using K_σ, and static horizontal shear stress using K_α.

3. Shear wave velocity can be used as a supplemental method to SPT or CPT for evaluating cyclic resistance ratio (CRR) per NCEER (1997).

4. Liquefaction resistance of gravelly soils should be evaluated per NCEER (1997). The Becker Penetration Test (BPT) may be required for soils with high content of gravel and cobbles.

5. If possible, site specific liquefaction potential curves should be developed and used when no liquefaction resistance correlations are available for the soils encountered. These curves can be developed in the laboratory for soils which can be sampled (using specialized methods if necessary and possible) using cyclic CU triaxial or cyclic simple shear tests.

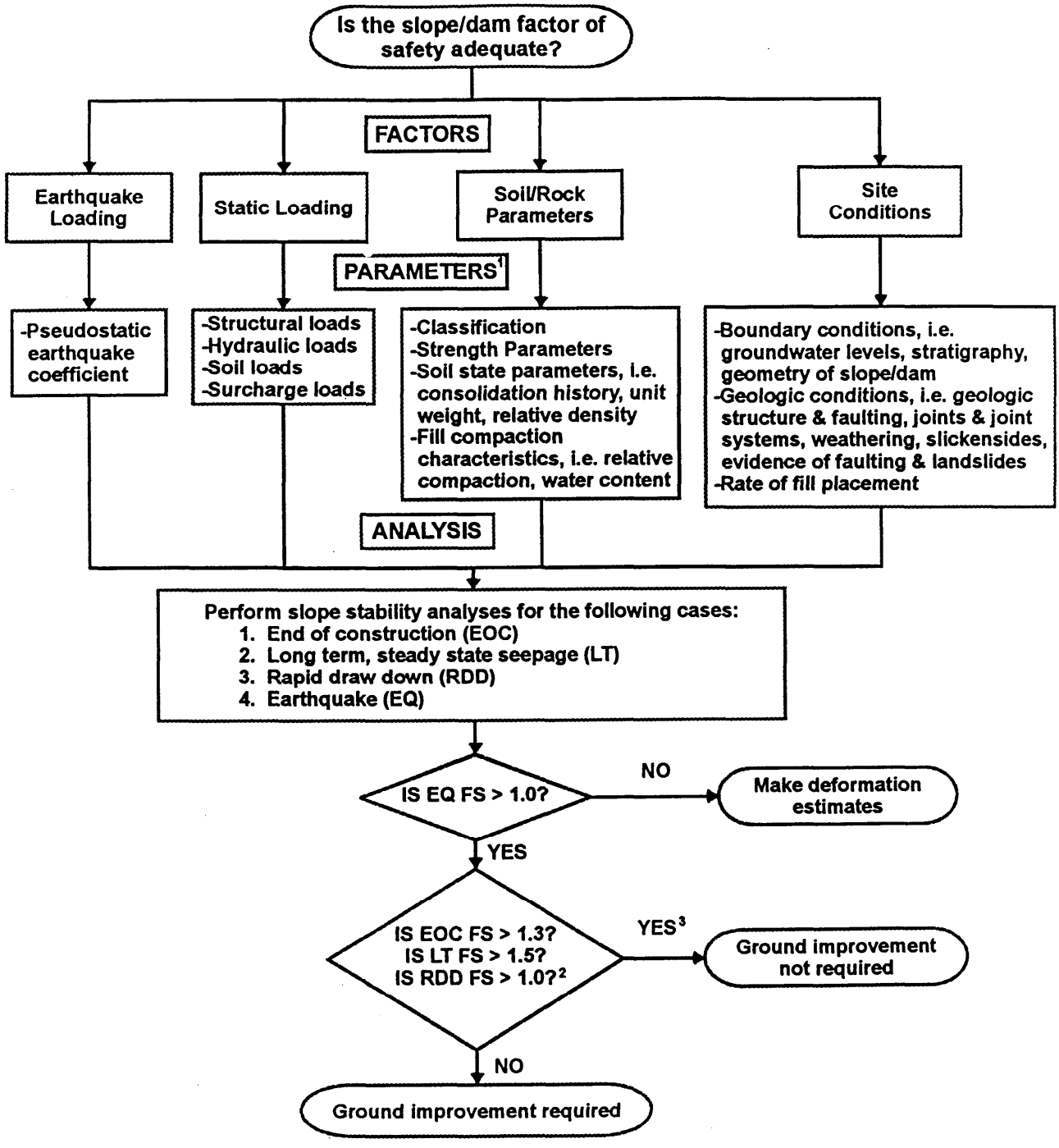
6. "No" response appropriate if it applies to both factor of safety and settlement criteria. "Yes" response appropriate if it applies to either or both criteria.

7. Deposits of cohesionless soils above groundwater (particularly those which are loose) are also susceptible to densification settlement during earthquake shaking. Estimated settlements of these deposits should be calculated using available methods (e.g. Tokimatsu and Seed, 1987) and included in settlement estimates for comparison to acceptable settlement limits.

8. $D_{v,a}$ is the allowable vertical movement (allowable settlement) of the foundation determined by the structural engineer.

9. "Yes" response appropriate if it applies to both factor of safety and settlement criteria. "No" response appropriate if it applies to either or both criteria.

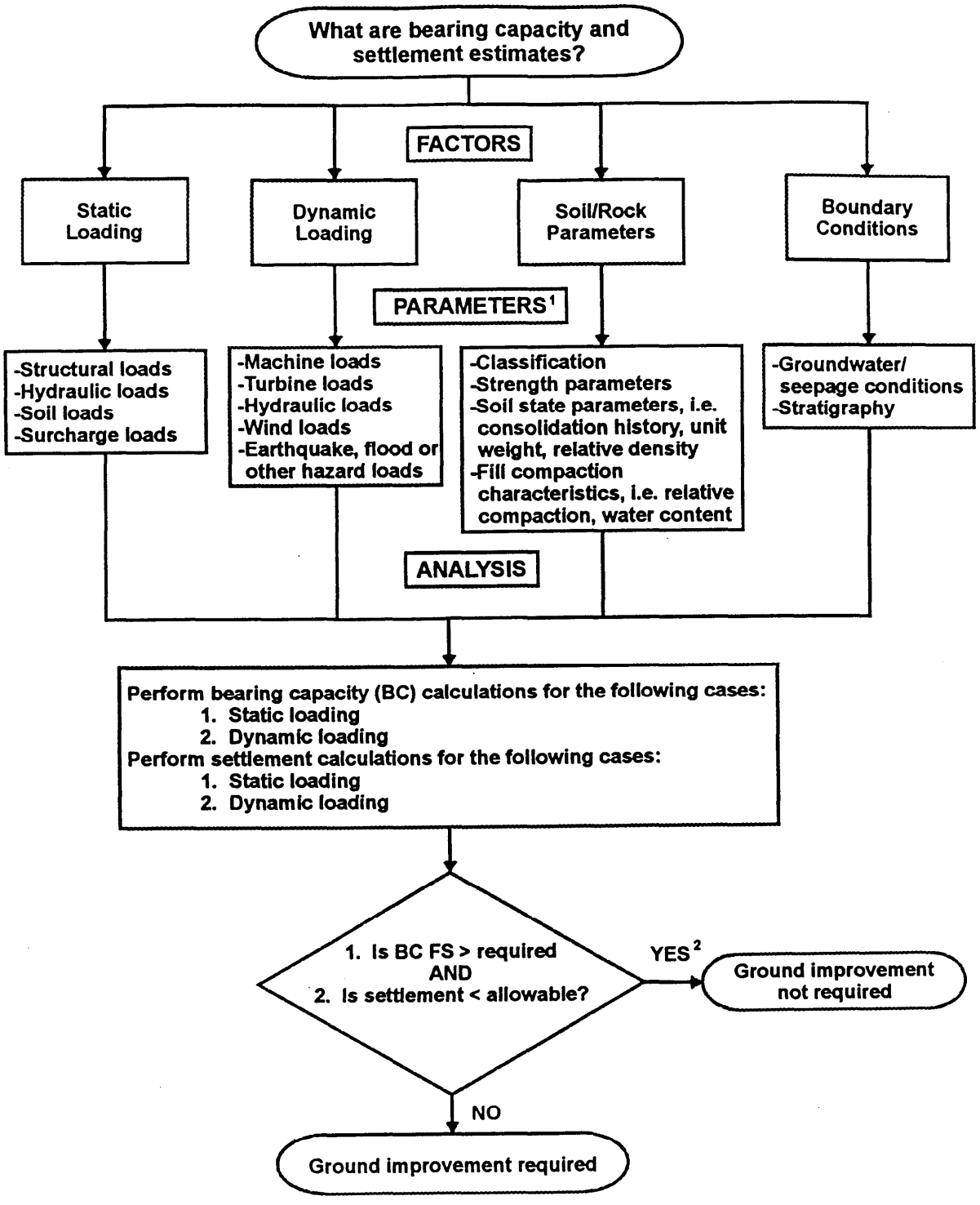
FIGURE 5 (continued)



Notes:

1. Assessment methods for parameters are given in Figures 19 and 20.
2. Based on EM-1110-2-1902 (Stability of Earth and Rockfill Dams). Criteria may be different for different projects.
3. "Yes" response appropriate if it applies to all criteria. "No" response appropriate if it applies to any criterion.

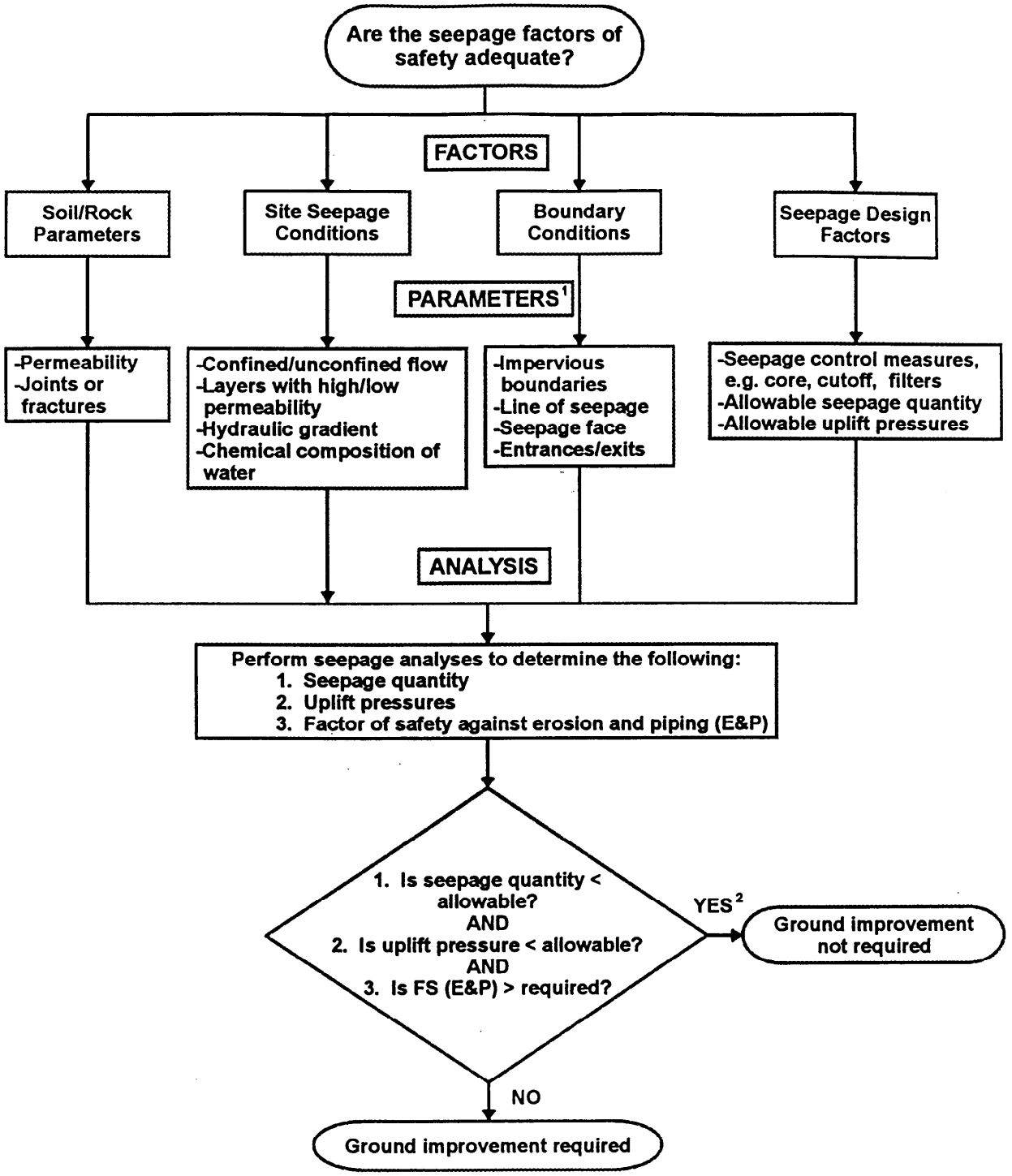
FIGURE 6 Slope Stability Evaluation



Notes:

1. Assessment methods for parameters are given in Figures 19 and 20.
2. If "Yes" answer applies to both decisions, ground improvement is not required. If "No" answer applies to either decision, ground improvement is required.

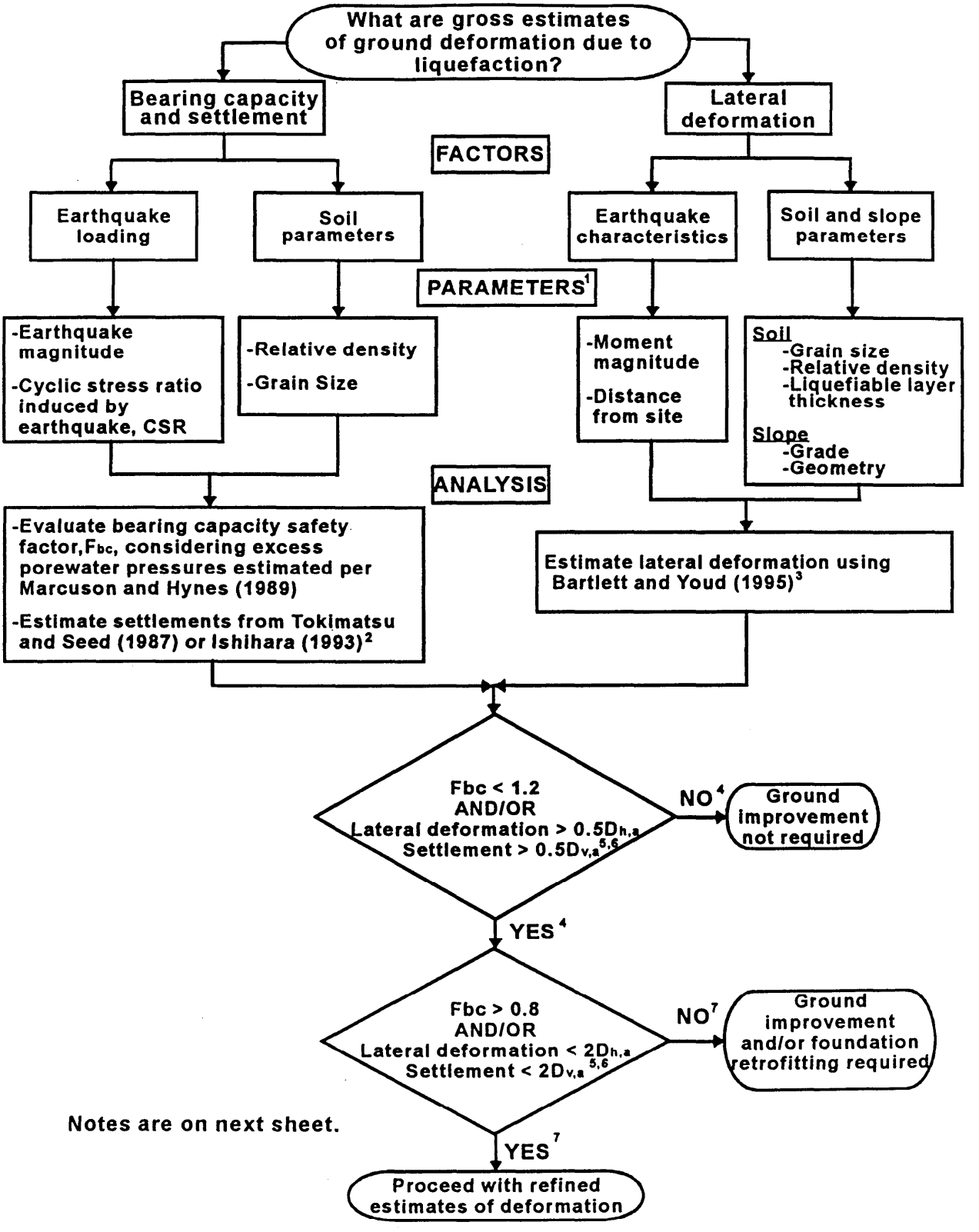
FIGURE 7 Bearing Capacity and Settlement Evaluation



Notes:

1. Assessment methods for parameters are given in Figure 21.
2. If "Yes" answer applies to all decisions, ground improvement is not required. If "No" answer applies to any decision, ground improvement or other mitigation strategy is required.

FIGURE 8 Seepage Evaluation



Notes are on next sheet.

FIGURE 9 Liquefaction Evaluation - Gross Deformation Estimates

Notes:

1. Assessment methods for parameters are given in Figure 22.

2. Tokimatsu and Seed (1987) and Ishihara (1993) procedures were developed for "clean" sands. For silty sands an equivalent "clean " sand $(N_1)_{60}$ value can be computed using the method described by NCEER (1997) for use with charts.

   For other soil types susceptible to liquefaction, settlements can be estimated using results from cyclic CU triaxial tests on "undisturbed" samples subjected to cyclic stress levels causing liquefaction. Samples are reconsolidated after liquefaction to obtain volumetric strain data. Volumetric strain is then correlated to the factor of safety against liquefaction, F_L, and the relative density/ penetration resistance of the soil.

3. For sites not satisfying seismic and site condition limits specified by Bartlett and Youd (1995), lateral deformations can be estimated using Newmark's (1965) method. Reduced shear strengths should be used along the failure surface in liquefied soil.

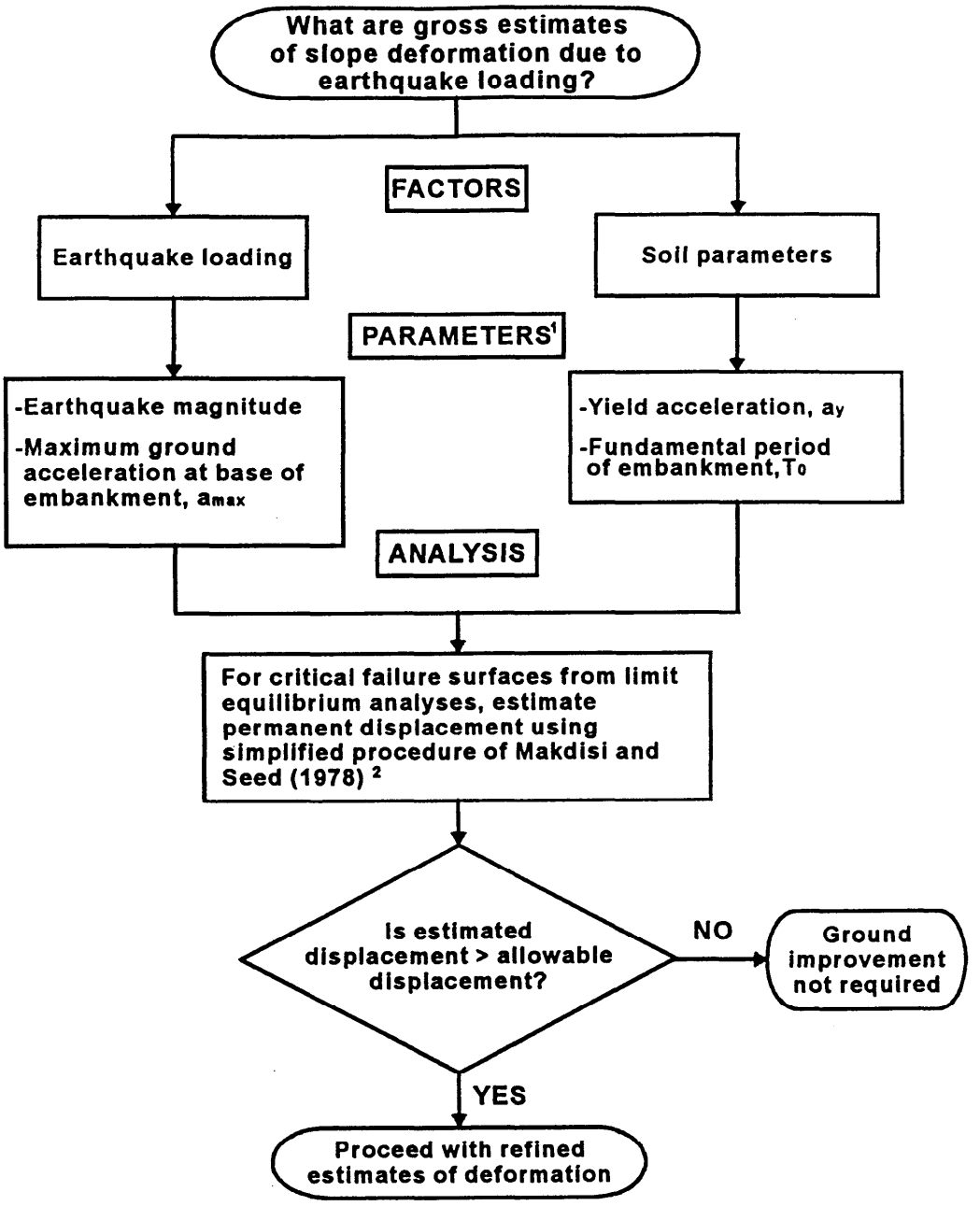
4. "No" response appropriate if it applies to both factor of safety and settlement/ lateral deformation criteria. "Yes" response appropriate if it applies to either or both criteria.

5. Estimated settlements should include densification settlements of cohesionless soils above groundwater (per Tokimatsu and Seed, 1987) and settlements due to deformations from lateral spreading and reduction in bearing capacity, as well as those from dissipation of liquefaction-induced excess porewater pressures of saturated soils.

6. $D_{v,a}$ and $D_{h,a}$ are the allowable vertical and horizontal movements, respectively, of the foundation as determined by the structural engineer.

7. "Yes" response appropriate if it applies to both factor of safety and settlement/ lateral deformation criteria. "No" response appropriate if it applies to either or both criteria.

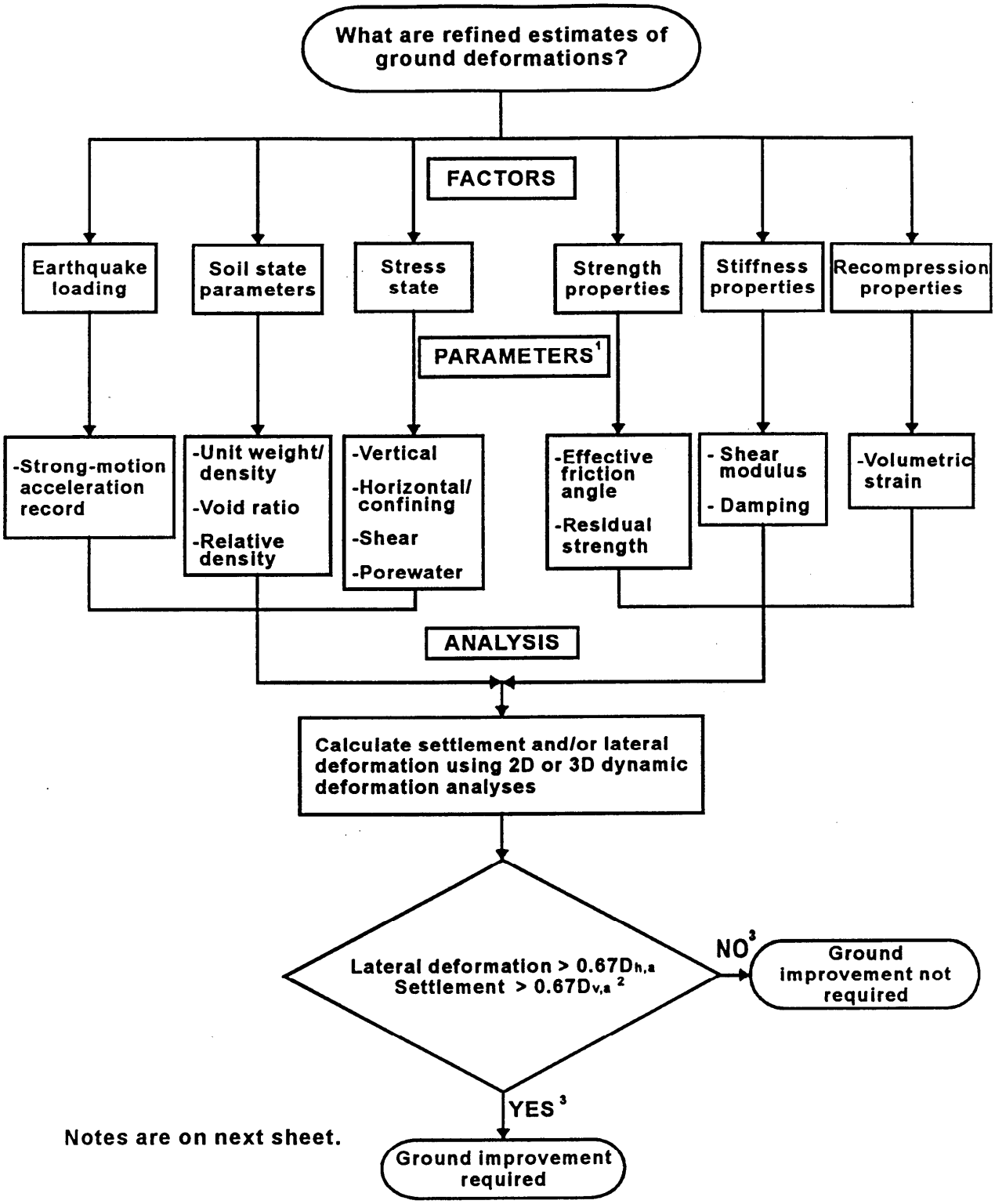
FIGURE 9 (continued)



Notes:

1. Assessment methods for earthquake parameters are given in Figure 15. Assessment methods for soil parameters are given in Figure 23.
2. This procedure was developed using the dynamic response characteristics of dams and embankments. If used for other types of slopes, the results must be used with caution.

FIGURE 10 Slope Stability Evaluation - Gross Deformation Estimates



Notes are on next sheet.

FIGURE 11 Refined Deformation Estimates for Liquefaction and Slope Stability Evaluations

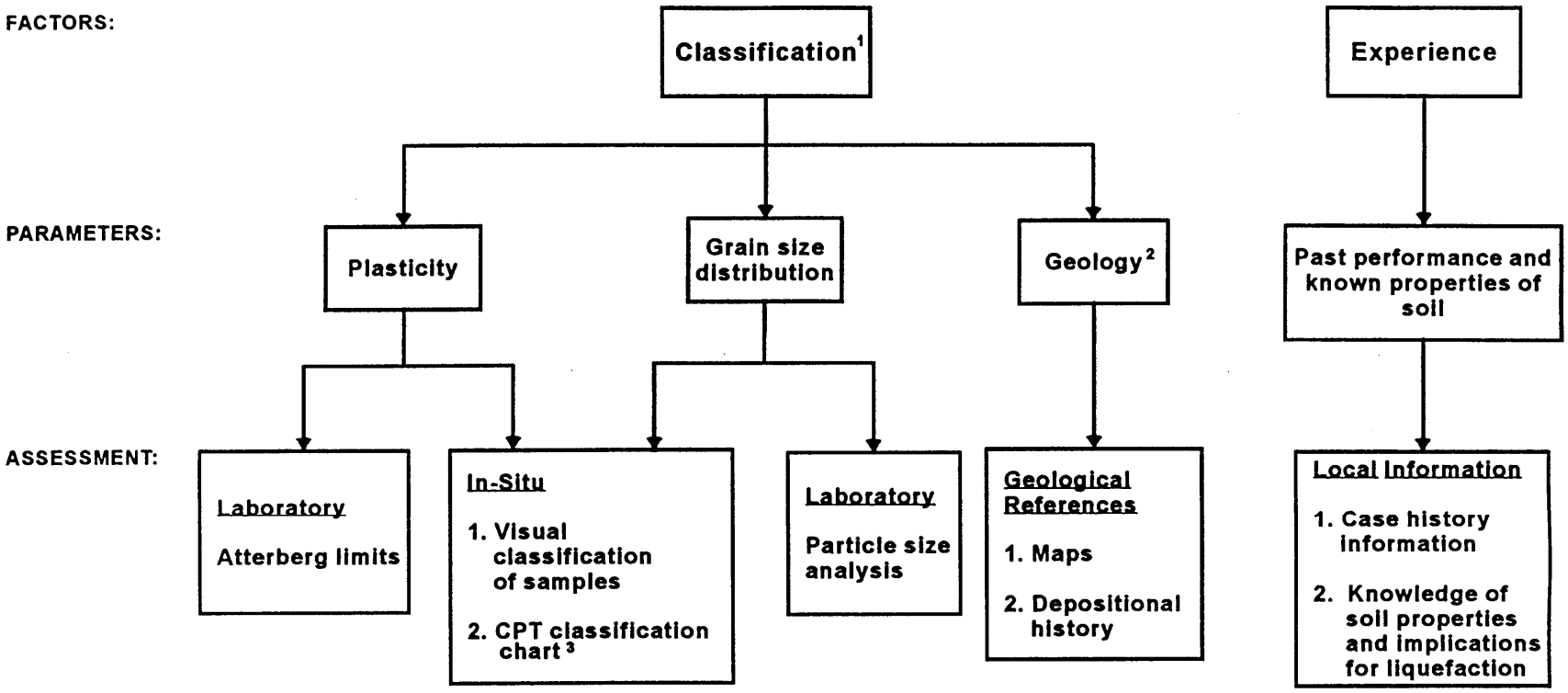
Notes:

1. Assessment methods for parameters are given in Figures 17, 19 and 24 through 26.

2. Estimated settlements should include densification settlements of cohesionless soils above groundwater (e.g. Tokimatsu and Seed, 1987) and settlement due to deformations from lateral spreading and reduction in bearing capacity, as well as those from dissipation of liquefaction-induced excess porewater pressures of saturated soils.

3. "No" response appropriate if it applies to both lateral deformation and settlement. "Yes" response appropriate if it applies to either lateral deformation or settlement, or both.

FIGURE 11 (continued)



Notes:

1. Refer to EM 1110-1-1804 (Geotechnical Investigations) and EM 1110-2-1906 (Soil Sampling) for COE procedures.
2. Geologic information provides some general insights regarding soil composition, fabric, and structure.
3. Use chart in accordance with NCEER (1997). CPT should only be used for classification after verification of its suitability by soil sampling adjacent to some CPT soundings.

FIGURE 12 Assessment Methods for Soil Classification and Experience Parameters for Preliminary Evaluation of Site Conditions and Design/Performance Requirements

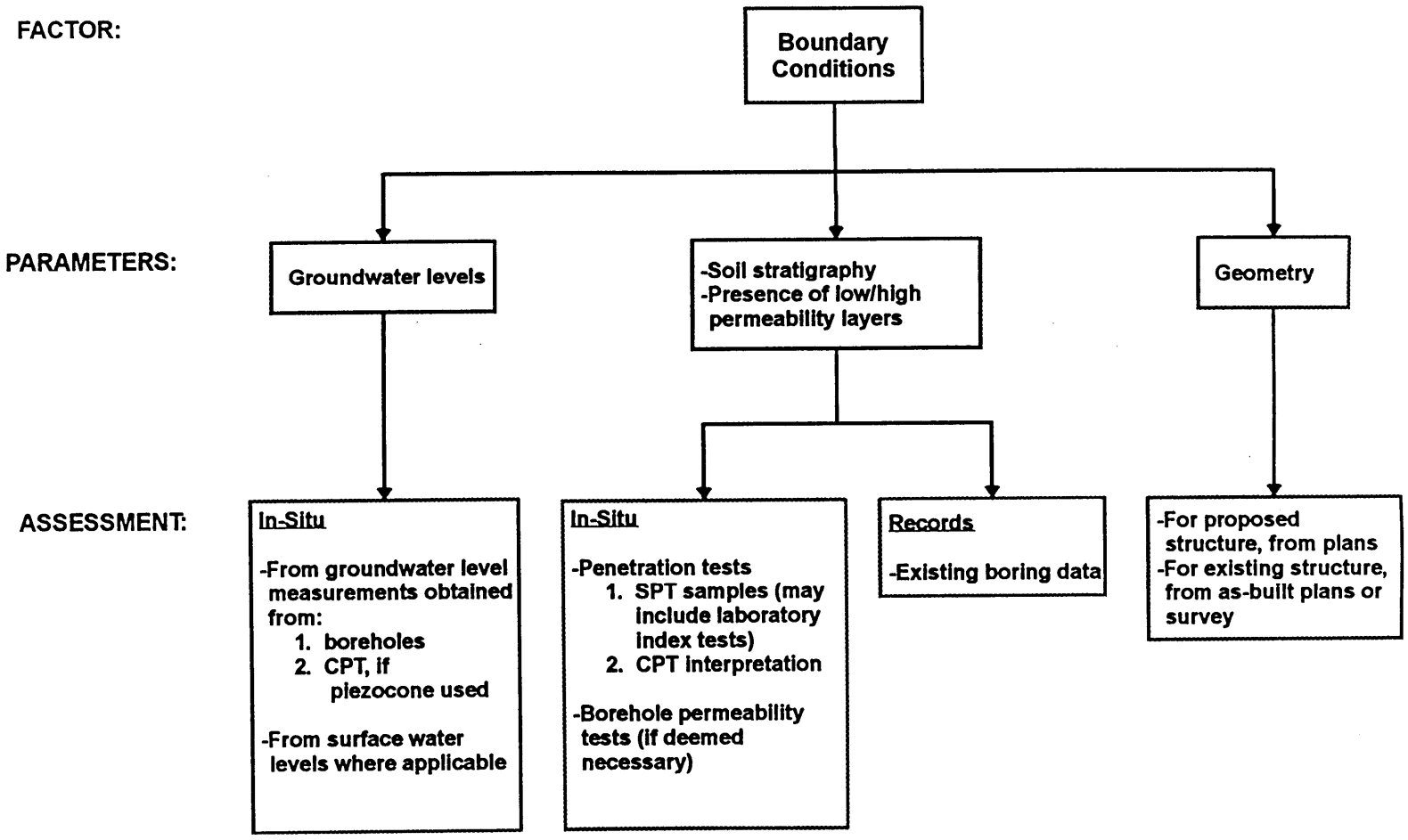


FIGURE 13 Assessment Methods for Boundary Condition Parameters for Preliminary Evaluation of Site Conditions and Design/Performance Requirements

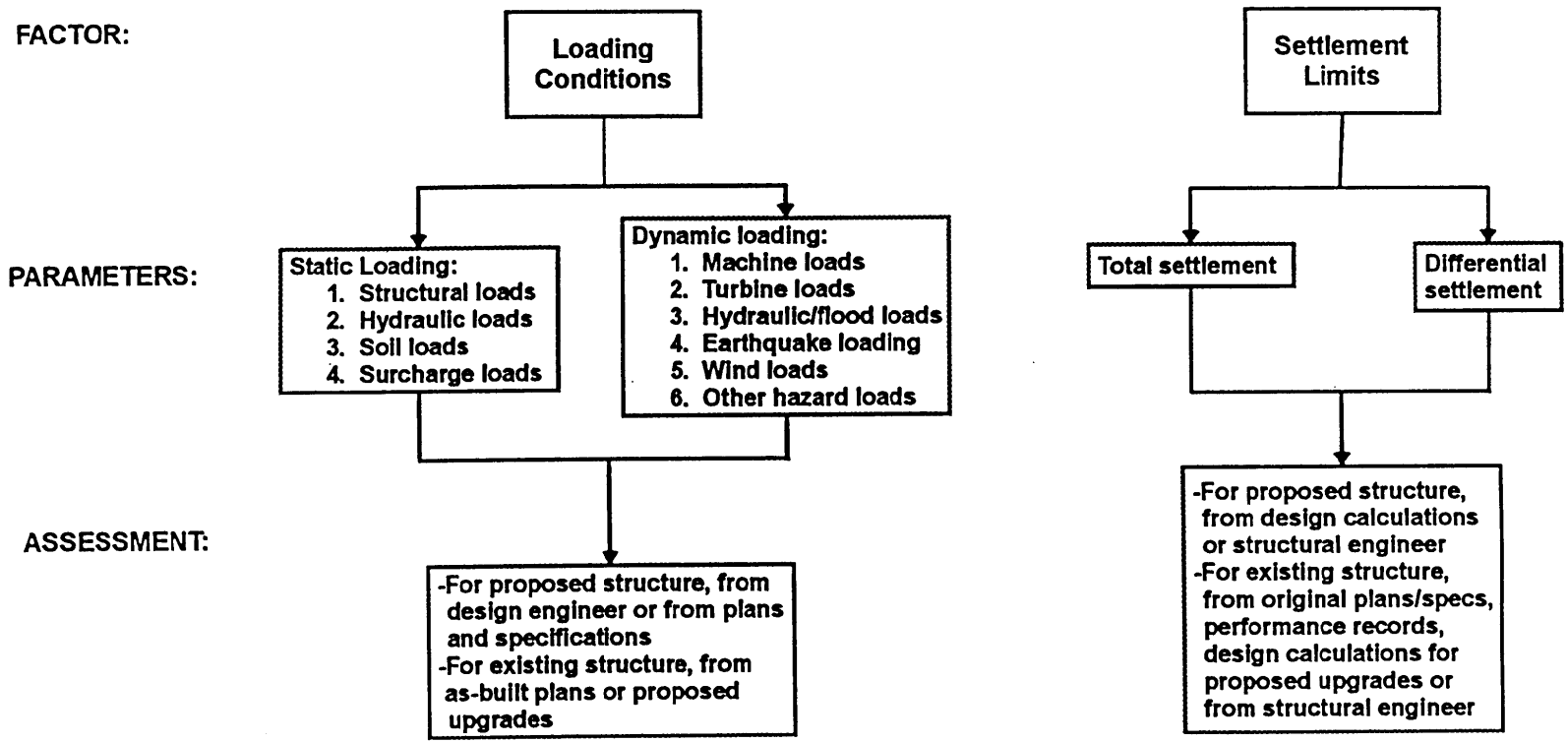
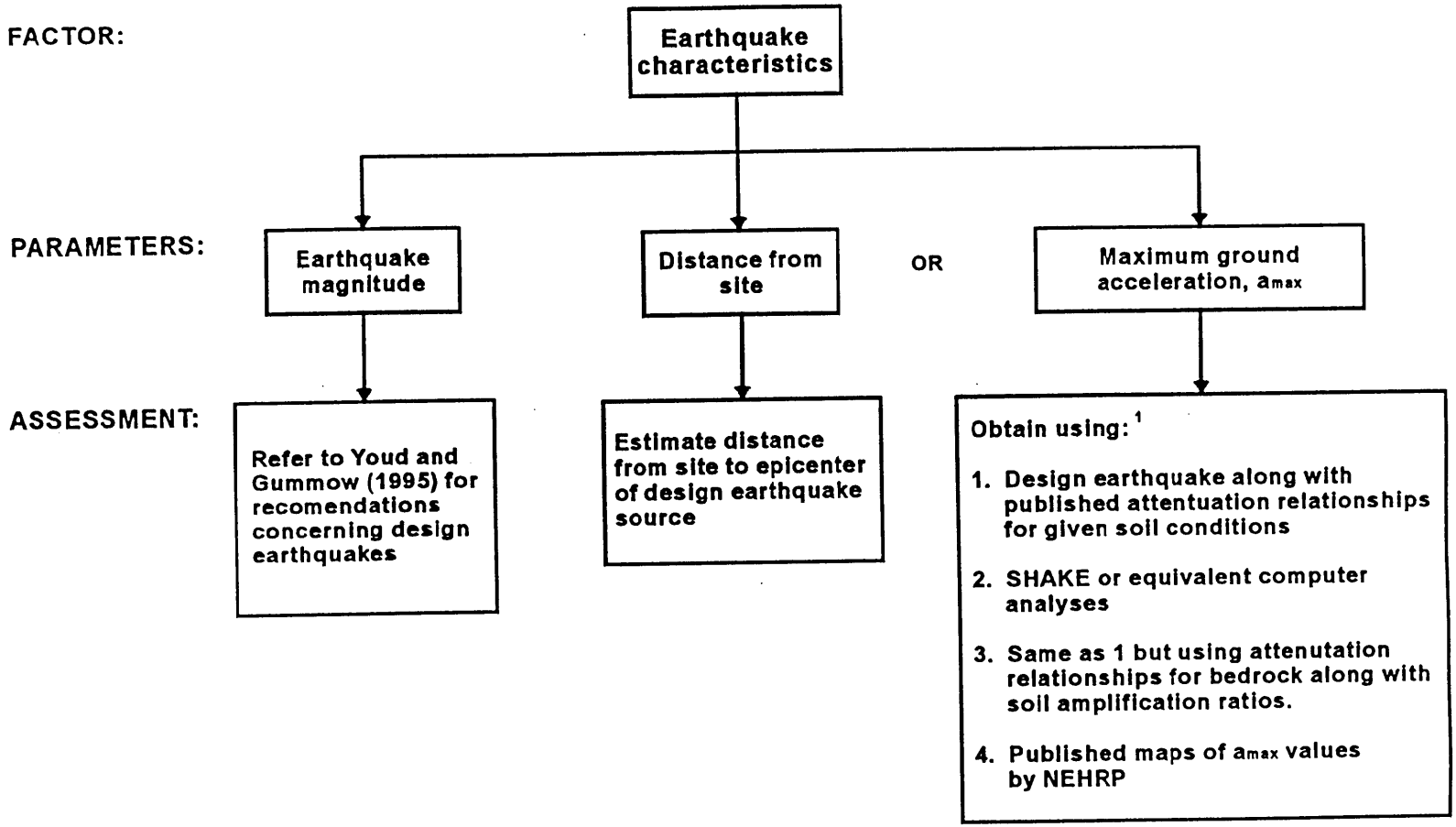


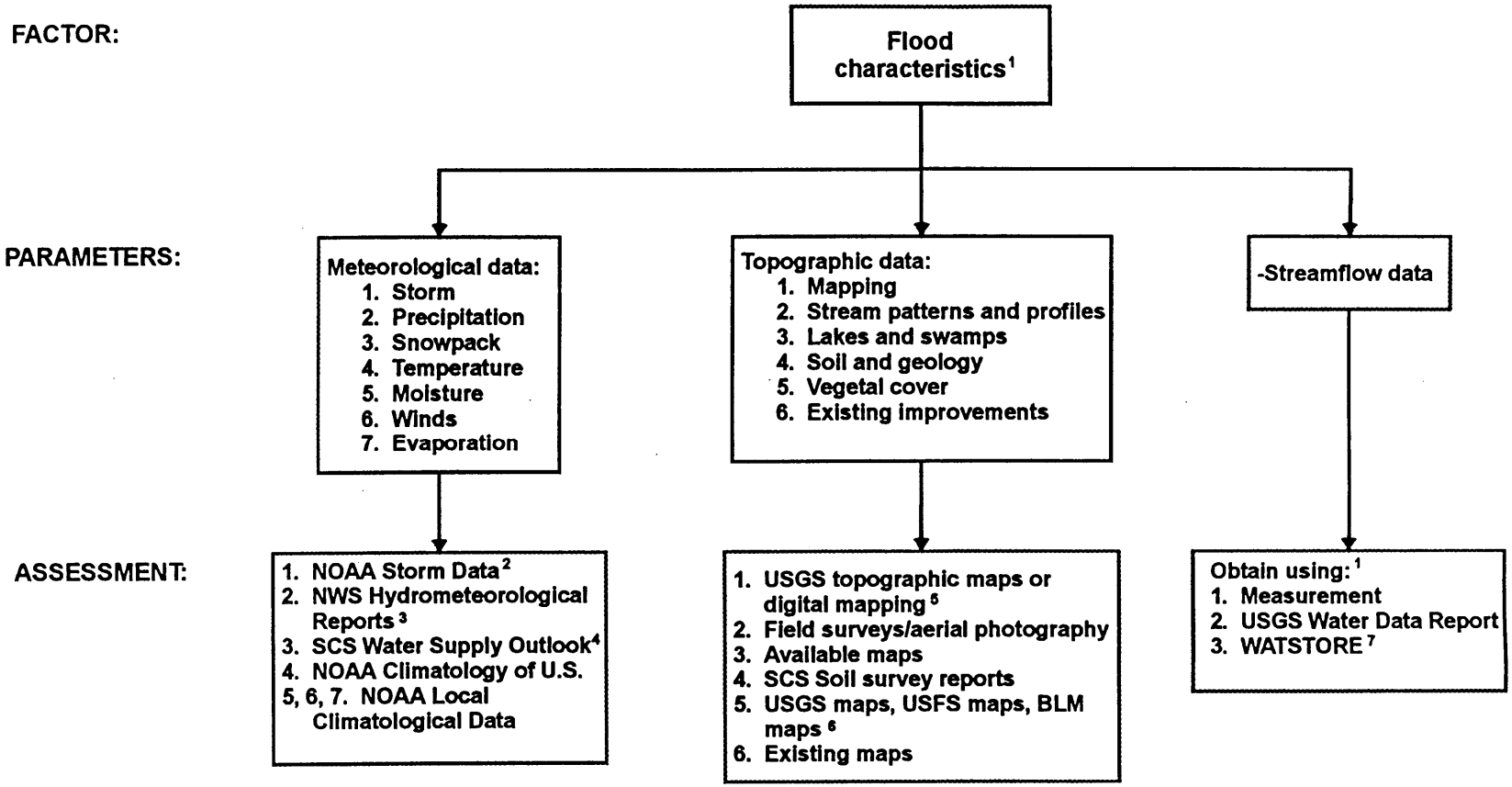
FIGURE 14 Assessment Methods for Loading Conditions and Settlement Parameters for Preliminary Evaluation of Site Conditions and Design/Performance Requirements



Notes:

1. From NCEER (1997): Option 1 is the preferred method when such relationships are available for the given soil conditions. Option 2 should be used if attenuation relationships for given soil conditions (Option 1) are not available. Option 3 is least desirable because of magnitude and potential frequency dependancy of amplification. Option 4 is not discussed.

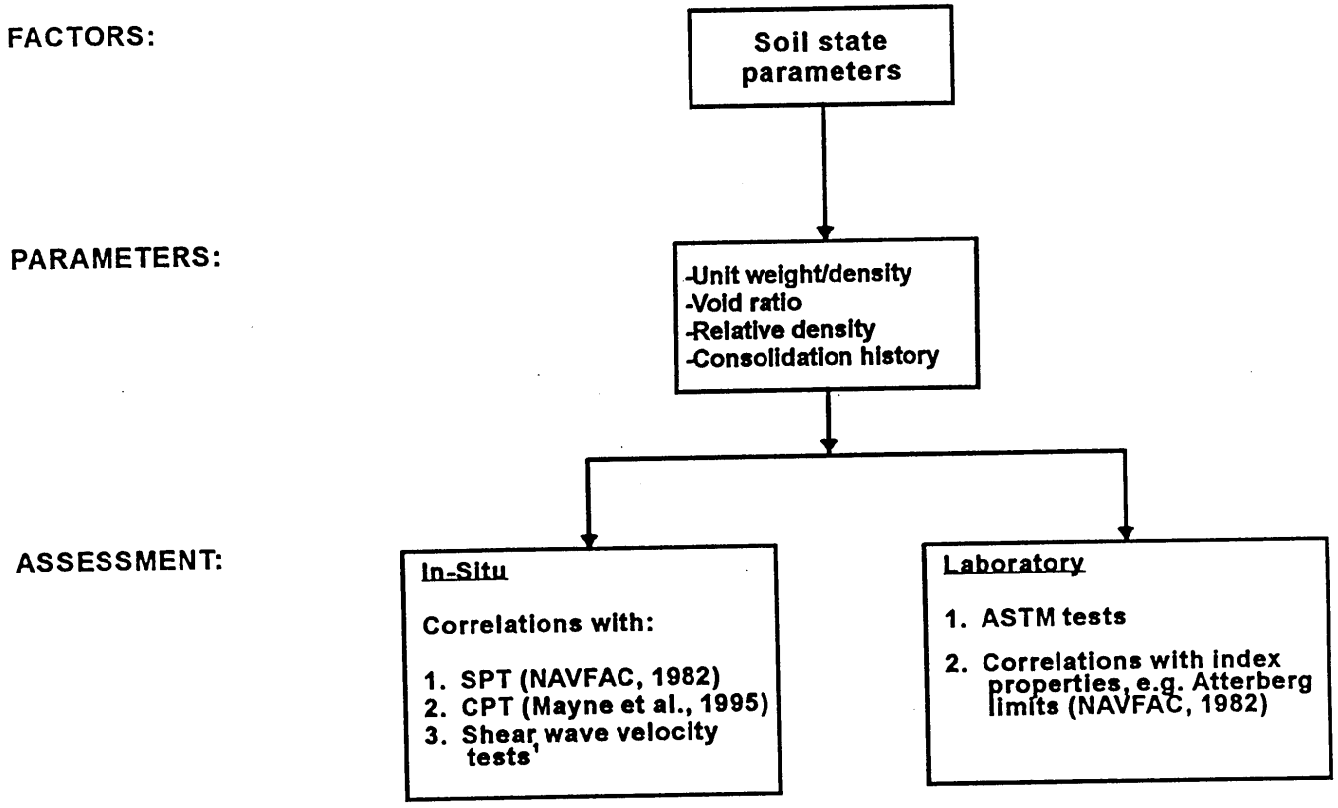
FIGURE 15 Assessment Methods for Earthquake Characteristic Parameters for Preliminary Evaluation of Site Conditions and Design/Performance Requirements



Notes:

1. Refer to EM 1110-2-1420, Hydrologic Engineering Requirements for Reservoirs
2. NOAA - National Oceanic and Atmospheric Admisistration
3. NWS - National Weather Service
4. SCS - Soil Conservation Service
5. USGS - United States Geological Survey
6. USFS - United States Forest Service; BLM - Bureau of Land Management
7. WATSTORE - National Water Data Storage and Retrieval System

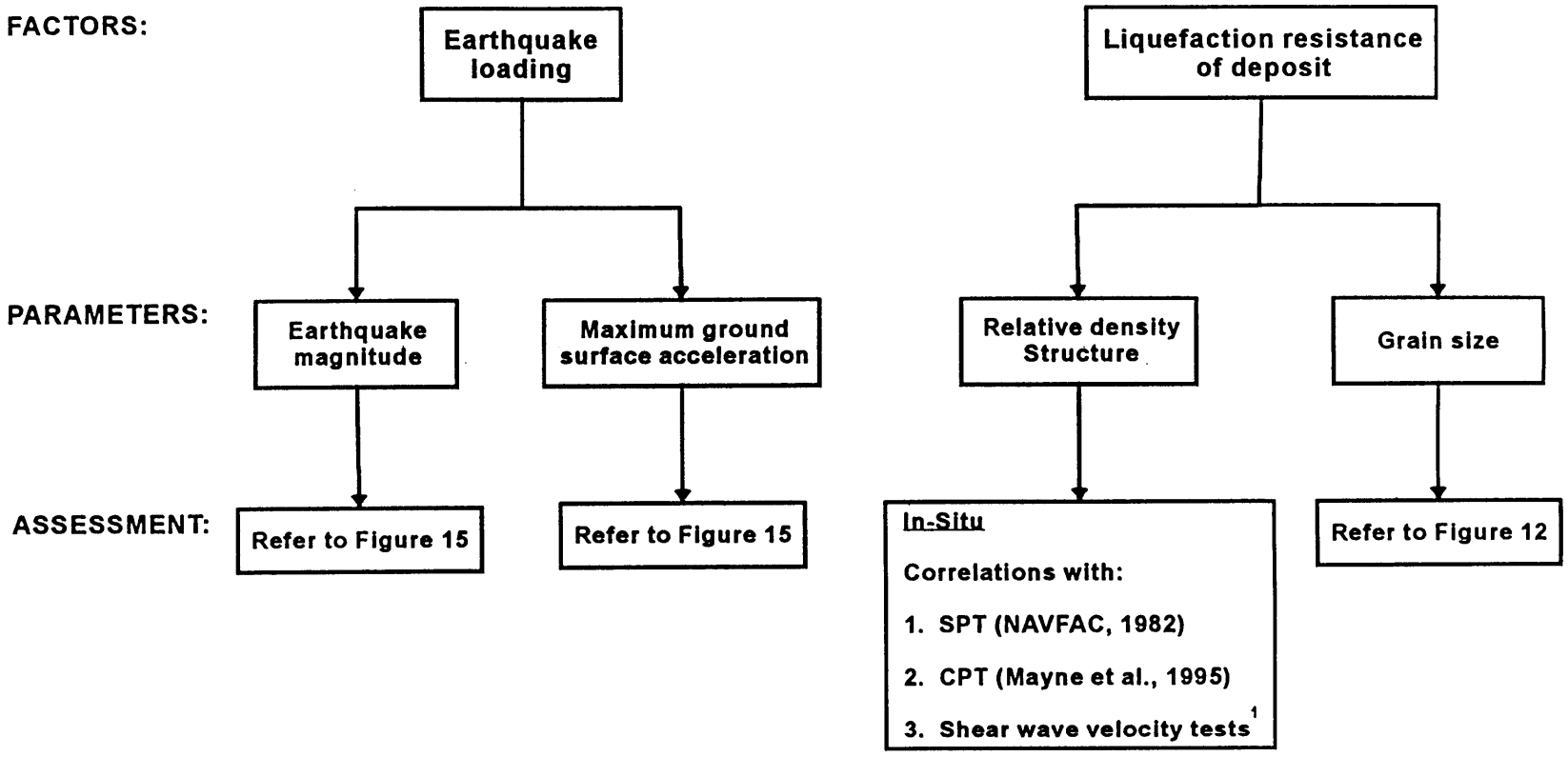
FIGURE 16 Assessment Methods for Flood Parameters for Preliminary Evaluation of Site Conditions and Design/Performance Requirements



Notes:

1. Correlation of shear wave velocity with void ratio, relative density or unit weight is not well established. Therefore, CPT or SPT tests should be used in conjunction with shear wave velocity tests to help evaluate these parameters.

FIGURE 17 Assessment Methods for Soil State Parameters for Difficult Soils, Slope Stability, and Seepage Evaluations



Notes:

1. Correlation of shear wave velocity with relative density is not well established. Therefore, CPT or SPT tests should be used in conjunction with shear wave velocity tests to help evaluate this parameter.

FIGURE 18 Assessment Methods for Earthquake Loading and Liquefaction Resistance Parameters for Evaluation of Liquefaction Potential

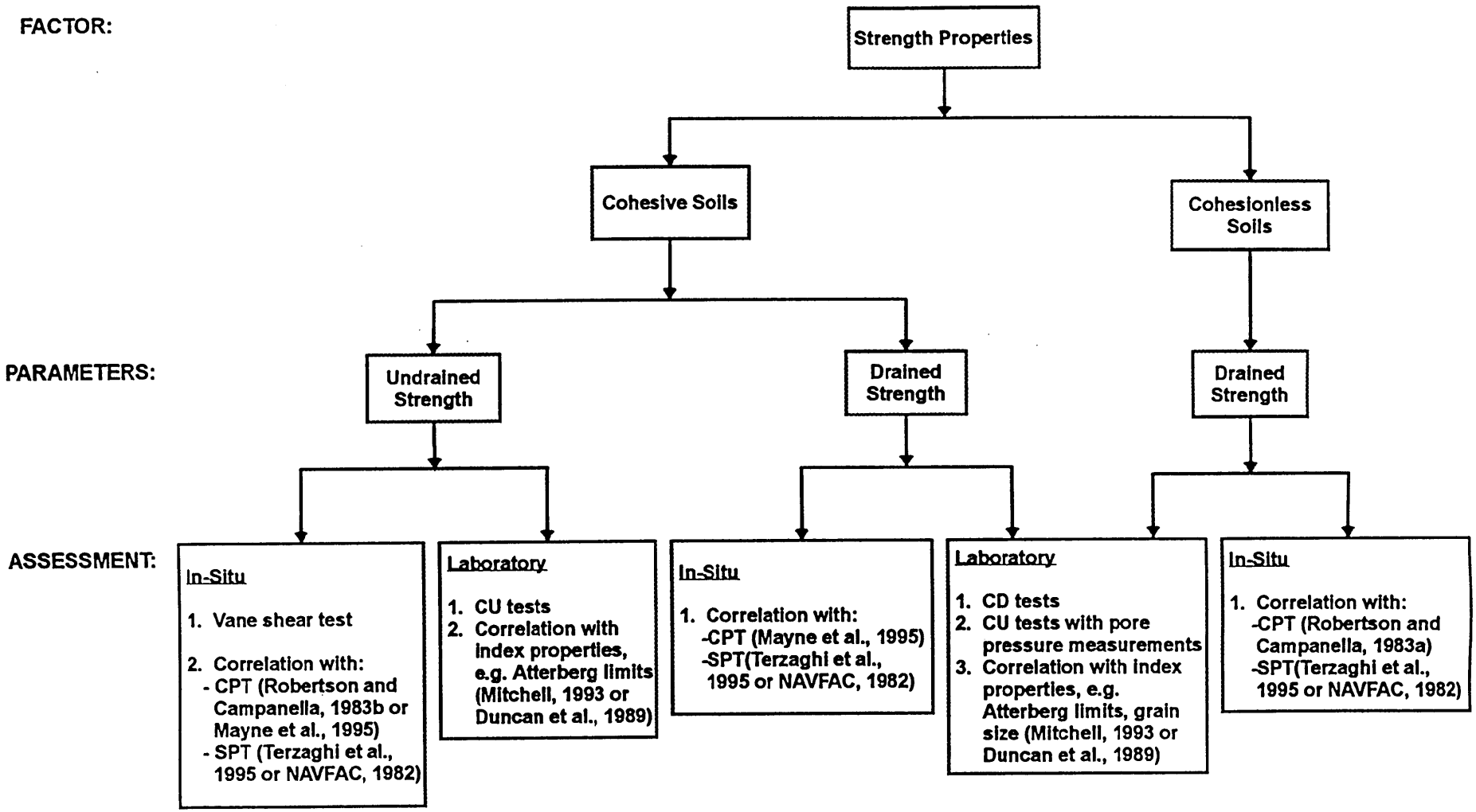


FIGURE 19 Assessment Methods for Strength Properties for Slope Stability and Bearing Capacity Evaluations

Factors	Parameters	Assessment
Earthquake loading	Pseudostatic earthquake coefficient	For most cases, $k_h = 0.5a_{max}/g$ Refer to text for additional guidance
Static Loading	Structural loads Hydraulic loads Soil loads Surcharge loads	Refer to Figure 14
Soil/rock parameters	Classification Soil state parameters Strength parameters Fill compaction characteristics	Refer to Figure 12 Refer to Figure 17 Refer to Figure 19 From laboratory tests
Site conditions	Boundary conditions Geologic conditions Rate of fill placement	Refer to Figure 13 From geotechnical investigation report From construction schedule

FIGURE 20 Parameter Assessment Methods for Slope Stability, Bearing Capacity and Settlement Evaluations

Factors	Parameters	Assessment
Soil/rock parameters	Permeability	From laboratory or field tests
	Joints or fractures	From geotechnical investigation report
Site boundary conditions	Confined/unconfined flow	Refer to Figure 13
	Layers with high/low permeability	Refer to Figure 13
	Hydraulic gradient	From construction plans
	Chemical composition of water	From laboratory tests
Seepage boundary conditions	Impervious boundaries	Refer to Figure 13
	Line of seepage	From flow net or FE* analysis
	Seepage face	From flow net or FE analysis
	Entrances/exits	From flow net or FE analysis
Seepage design factors	Seepage control measures	From construction plans
	Allowable seepage quantity	From performance requirements
	Allowable uplift pressures	From design requirements

* Finite element

FIGURE 21 Parameter Assessment Methods for Seepage Evaluation

Factors	Parameters	Assessment
Earthquake loading	Earthquake magnitude	Refer to Figure 15
	Cyclic Stress Ratio (CSR)	Refer to Figure 15 for a_{max}; $CSR = 0.65(a_{max}/g)*(\sigma_{vo}/\sigma_{vo}')r_d$
Soil parameters	Relative density	Refer to Figure 18
	Grain Size	Refer to Figure 12
Earthquake characteristics	Moment magnitude	Refer to Figure 15
	Distance from site	Refer to Figure 15
Soil and slope parameters	Soil	
	Grain size	Refer to Figure 12
	Relative density	Refer to Figure 18
	Liquefiable layer thickness	Soil borings or CPT soundings
	Slope	
	Grade and geometry	1. Construction plans 2. Field reconnaissance

FIGURE 22 Parameter Assessment Methods for Liquefaction Evaluation - Gross Deformation Estimates

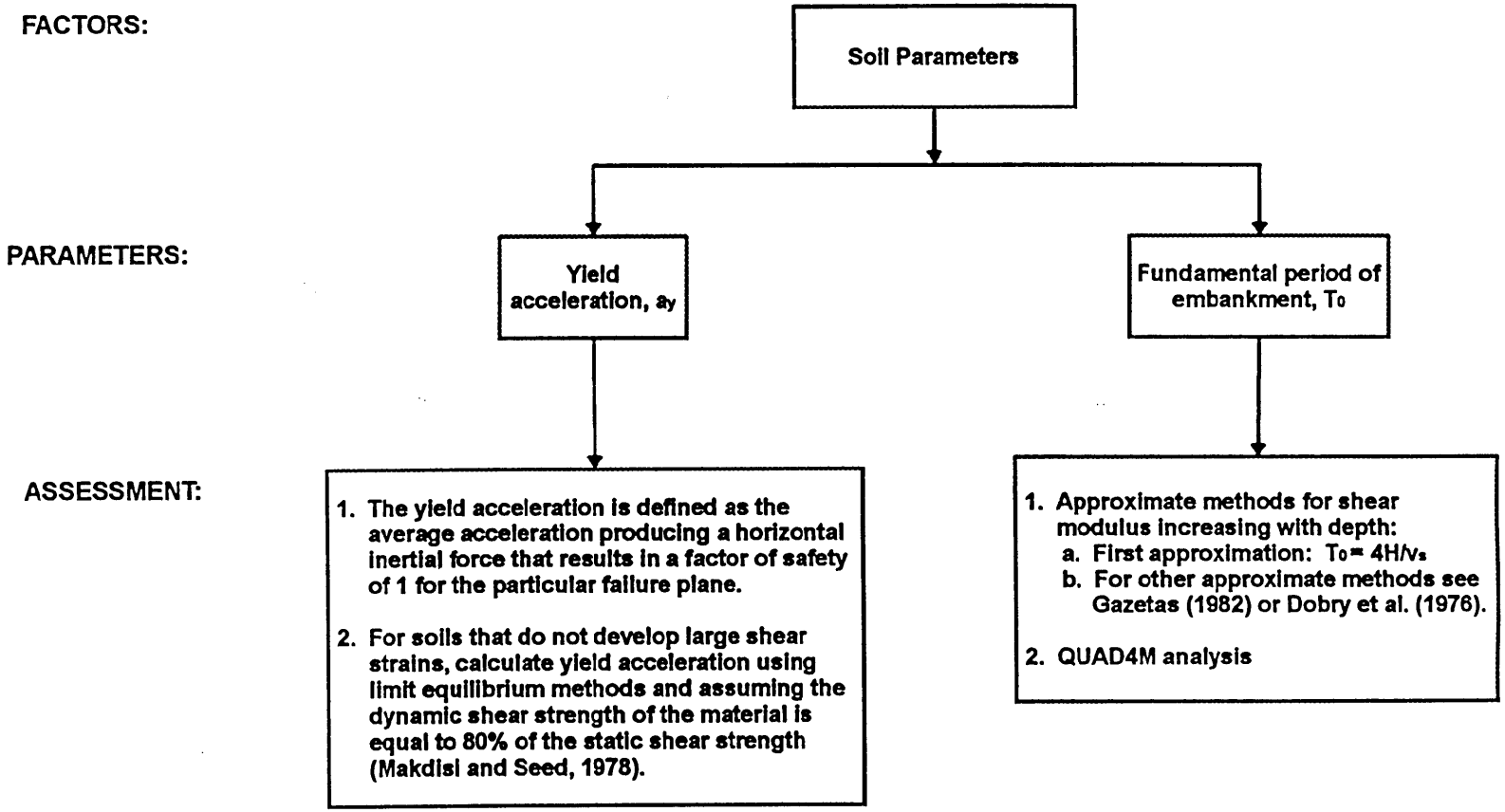
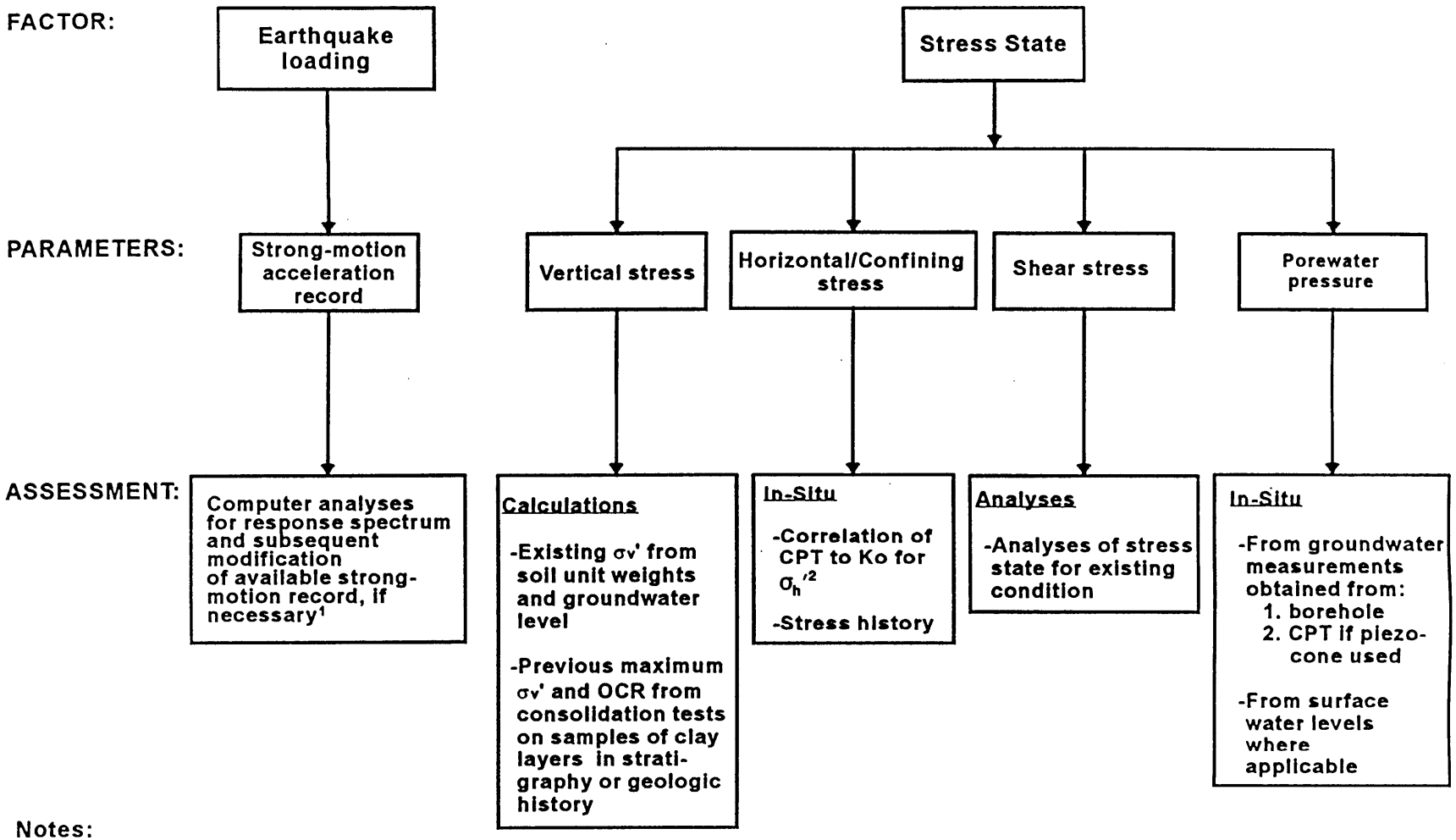


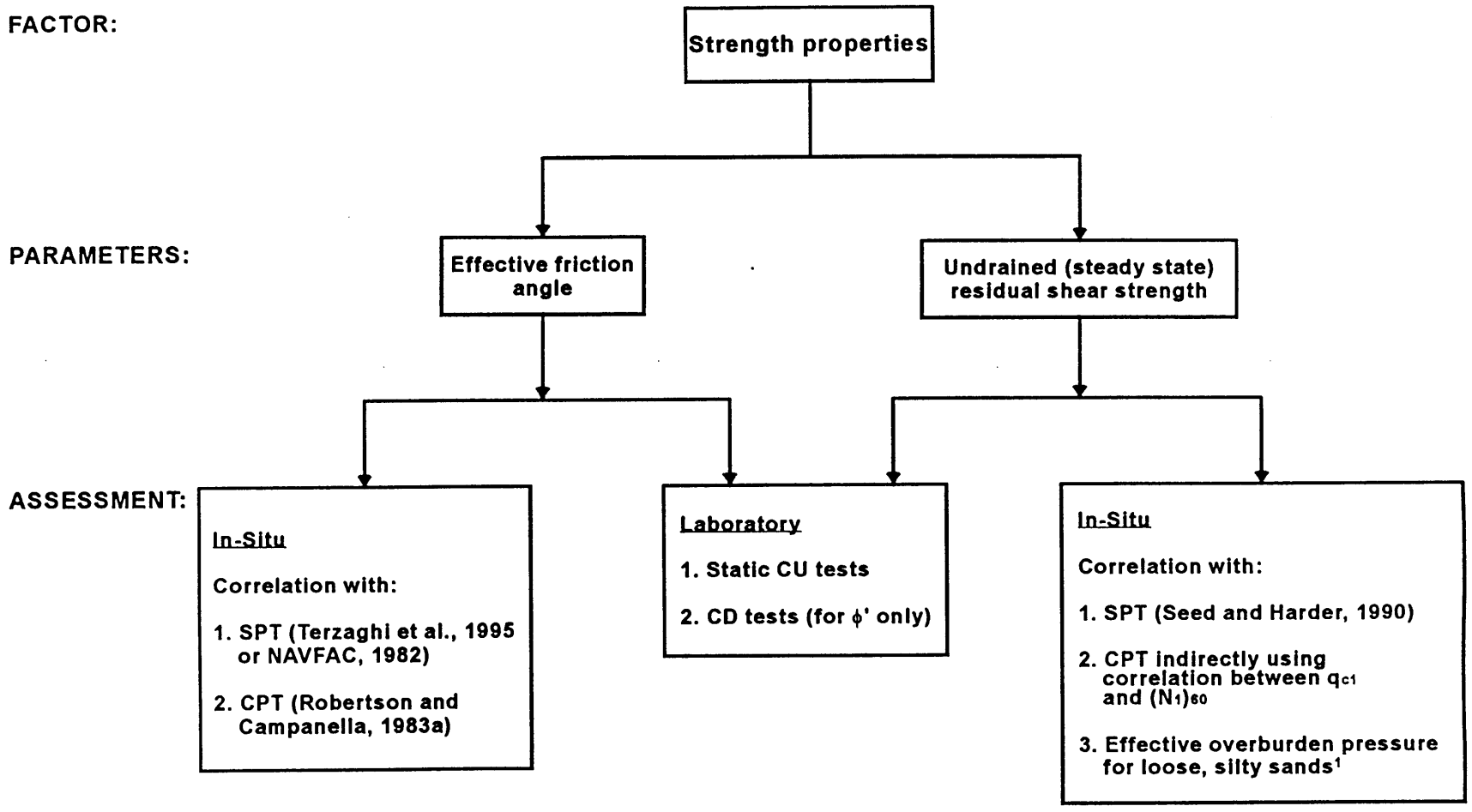
FIGURE 23 Assessment Methods for Soil Parameters for Slope Stability Evaluation - Gross Deformation Estimates



Notes:

1. Computer analyses to determine response spectrum from strong-motion bedrock record can be performed using SHAKE, QUAD4M or equivalent analysis. Modify strong-motion record as necessary to obtain desired design response spectrum.
2. Reference Kulhawy and Mayne (1990).

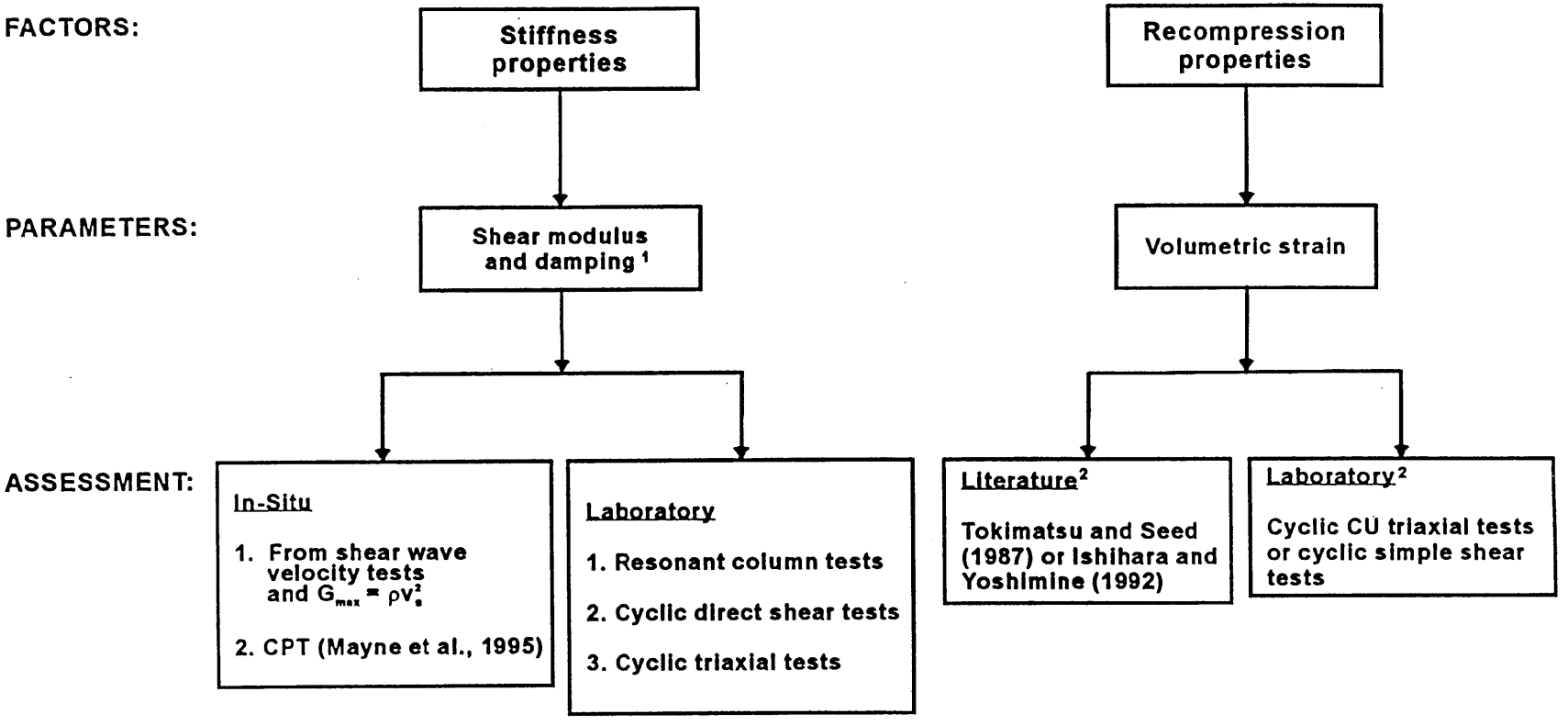
FIGURE 24 Assessment Methods for Earthquake Loading and Stress State Properties for Refined Deformation Estimates for Liquefaction and Slope Stability Evaluations



Notes:

1. Recommended Su/p ratio per Baziar et al. (1995): $Su/p \approx 0.145$ or $Su/p \approx 0.11 + 0.0037(PI)$

FIGURE 25 Assessment Methods for Strength Properties for Refined Deformation Estimates for Liquefaction and Slope Stability Evaluations



Notes:

1. Variation of shear modulus and damping with shear strain for "clean" sands can be determined using curves by Idriss (1990). Curves by Vucetic and Dobry (1991) can be used for plastic soils.

2. Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1992) procedures developed for "clean" sands. For other soil types susceptible to liquefaction, volumetric strains can be estimated in the laboratory using results from cyclic CU triaxial tests on "undisturbed" samples subjected to cyclic stress levels causing liquefaction. Samples are reconsolidated after liquefaction to obtain volumetric strain data.

FIGURE 26 Assessment Methods for Stiffness and Recompression Properties for Refined Deformation Estimates for Liquefaction and Slope Stability Evaluations