Chapter 4
Methods of Seismic Analysis and Structural Modeling

4-1. Progressive Analysis Methodology

The evaluation of structures for earthquake ground motions should be performed in phases in order of increasing complexity progressing from simple equivalent lateral force methods, to linear elastic response-spectrum and time-history analysis, to nonlinear methods, if necessary. The following paragraphs describe the various analytical methods used to assess earthquake ground motion effects beginning with the simplest method and progressing to the more complex methods. In each analysis procedure idealized models of structures are used to estimate the dynamic response of structures to earthquakes.

4-2. Methods of Analysis

a. Seismic coefficient method. Seismic coefficient method has traditionally been used to evaluate seismic stability of structures. According to ER 1110-2-1806 this method may still be used in the preliminary design and stability analyses. In the seismic coefficient method, earthquake forces are treated simply as static forces and are combined with the hydrostatic pressures, uplift, backfill soil pressures, and gravity loads. The analysis is primarily concerned with the rotational and sliding stability of the structure treated as a rigid body. The inertia forces acting on the structure are computed as the product of the structural mass, added-mass of water, and the effects of dynamic soil pressures, times a seismic coefficient. The magnitude of the seismic coefficient is often taken as a fraction of the peak ground acceleration expressed as a decimal fraction of the acceleration of gravity. In representing the effects of ground motion by static lateral forces, the seismic coefficient method neither accounts for the dynamic characteristics of the structure-water-soil system nor for the characteristics of the ground motion. The method however can give reasonable results when the structure primarily acts as a rigid body, such as sliding response of a gravity dam depicted in Figure 4-1. As the most probable sliding response, this failure mechanism is commonly used to determine a factor of safety against sliding. Note that prior to the sliding evaluation the dam should be analyzed as a flexible structure to determine stresses and the extent of cracking that might lead to such a sliding failure, as shown in Figure 2-4. Another instance where the structure may be analyzed as a rigid body is the case where the massive concrete structure is supported on a flexible foundation, such as the pile founded navigation lock monolith shown in Figure 4-2. In this case, accelerations will be nearly uniform from the base to top of the monolith, if the piles are relatively flexible with respect to the structure. This is similar to the response of base isolated stiff buildings. Under this condition the seismic coefficient method may be used for the preliminary design and evaluation of the lock system. The method, however, should be used with caution if the interaction between the structure and soil-pile foundation is significant. The seismic coefficient method is part of the Linear Static Procedure (LSP) described in Chapter 6.

b. Equivalent lateral force method. The equivalent lateral force method (ELF) is commonly used for the seismic design of buildings. Assuming that the structure response is predominantly in the first mode, similar procedures have also been developed for preliminary seismic analysis of gravity dams (Fenves and Chopra, 1986) and intake towers (EM 1110-2-2400) using standard mode shapes and periods. In such cases, the first mode of vibration could contribute as much as 80-percent or more to the total seismic response of the structure. Therefore, the period and general deflected shape of the first mode are sufficient for estimating inertia forces or equivalent lateral loads needed for the seismic design or evaluation. The ELF Method is illustrated in Figure 4-3. The steps in the analysis are described as follows:

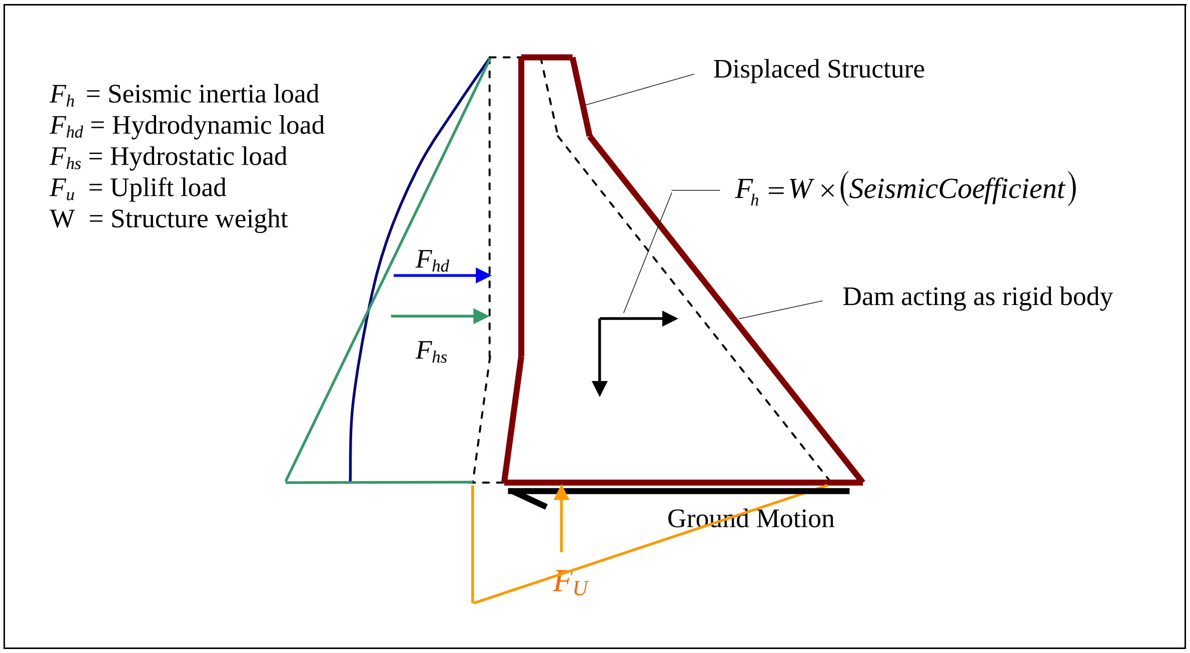


Figure 4-1. Gravity Dam Sliding on Foundation

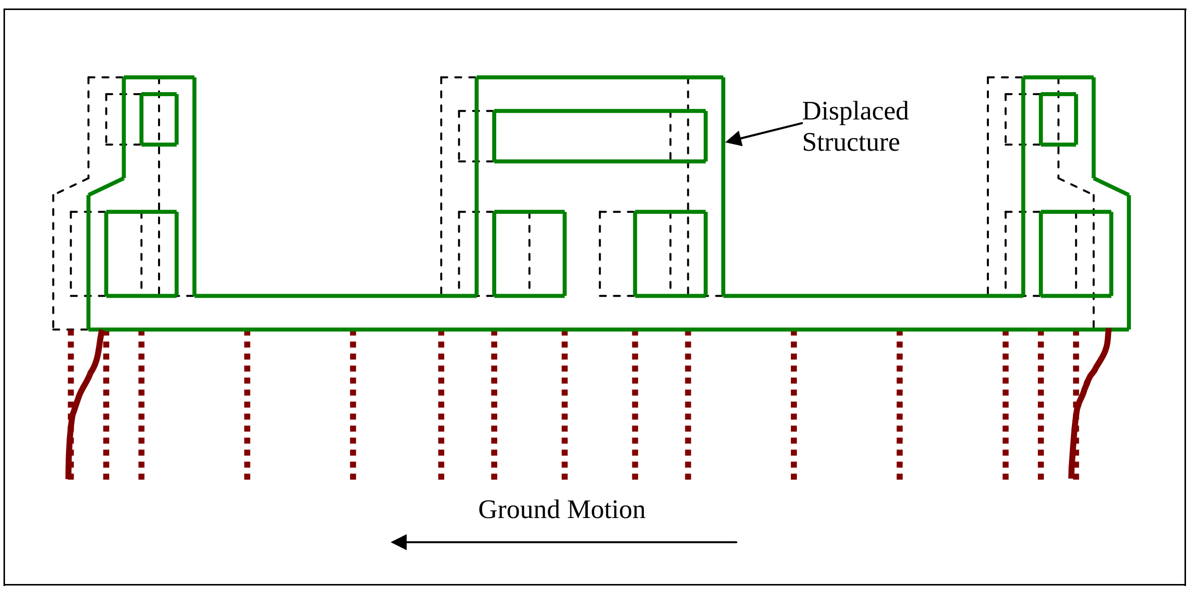


Figure 4-2. Navigation Lock on Flexible Pile Foundation

(1) The first step as illustrated in Figure 4-3a is to estimate the period of vibration of the first mode. This can be done using a general formula developed for the particular structure under consideration based on what is known about the stiffness of the structure-foundation system (*K*) and the total system mass (M = structure mass + added hydrodynamic mass + backfill.) The formula will be of the general form shown in Figure 4-3a.

(2) The second step is to determine the spectral acceleration (S_A) for an equivalent single-degree-of-freedom system. This can be done using the period of vibration determined in Step 1 in combination with a standard or site-specific acceleration response spectrum. This step is illustrated in Figure 4-3b. In some cases, as for buildings, the spectral acceleration will be represented by a standard spectrum in equation form as part of a base shear formula. The base shear formula will also account for the base shear participation factor described in the following step.

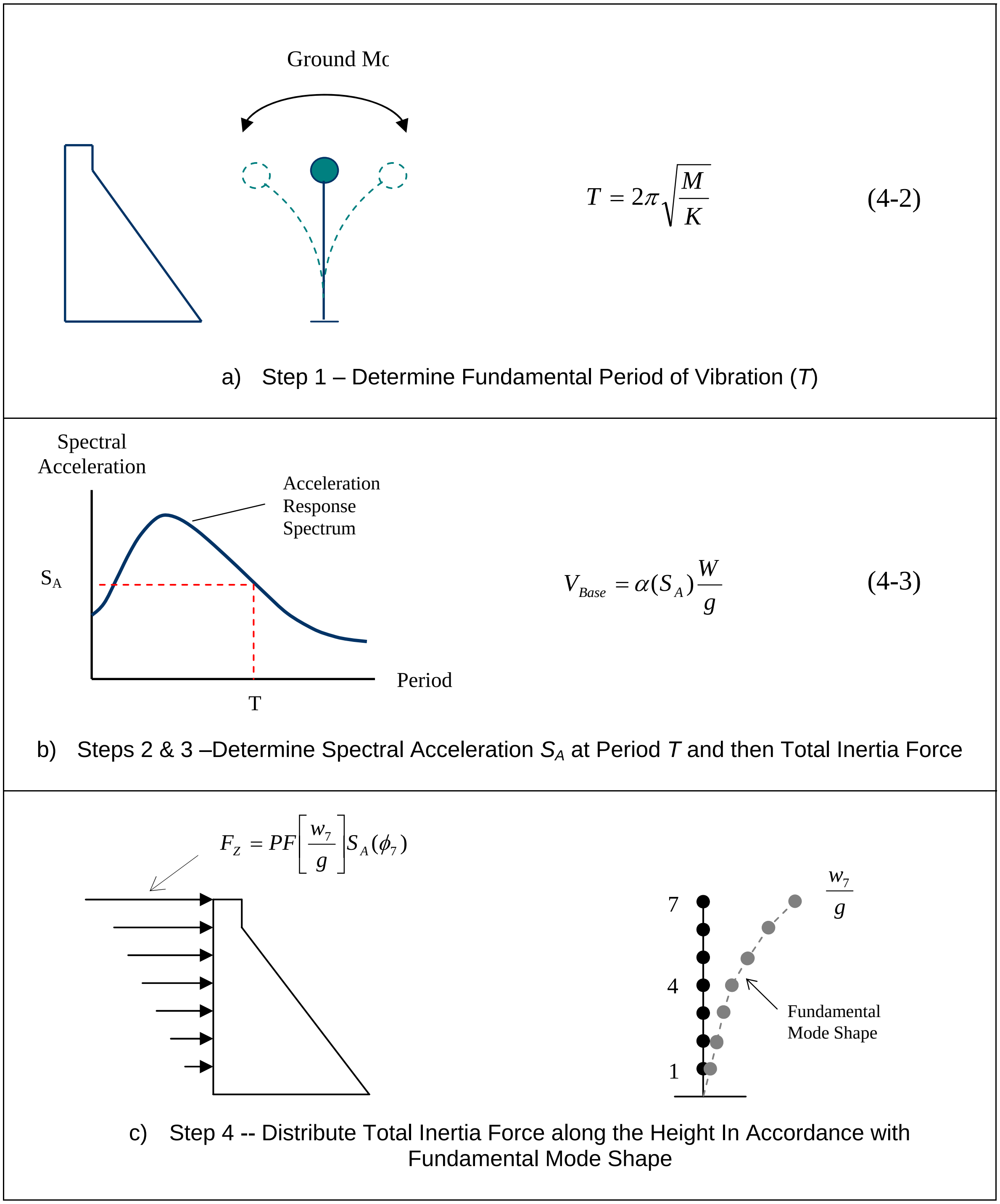
(3) Once the spectral acceleration (S_A) has been determined, the total inertial force on the structure due to the design ground motions (represented by the design response spectrum shown in Figure 4-3b) can be estimated using Equation (4-3) in Figure 4-3b. The spectral acceleration and the total mass being known, the base shear participation (α) can be estimated from the structure deflected shape and mass distribution.

(4) The analytical model of the structure is represented by a series of lumped masses as shown in Figure 4-3d. The total inertial force (base shear) is then distributed along the height of the structure at location of each lumped mass. The magnitude of each inertial force is obtained from the product of the mass participation factor (PF), times the lumped mass (w/g), times the spectral acceleration (S_A), times the value of the mode shape (ϕ_Z) at the lumped mass location, or:

$$F_Z = PF\left[\frac{w_Z}{g}\right]S_A(\phi_Z) \tag{4-1}$$

The first-mode mass participation factor (PF) is the same for each lumped mass location (Z) and can be determined based on the mass distribution and deflected shape. The mode shape value (ϕ_Z) will either be provided in the ELF Method, or will be incorporated in a base shear distribution formula as done for buildings. Once all the inertial forces have been determined, the analysis can proceed in the same fashion as any static analysis.

Because the dynamic characteristics of the structure are considered when determining earthquake demands and distributing inertia forces to the structural system, the ELF method is an excellent static force method. ELF methods in some cases, as for the evaluation of dams and intake towers, have the capability of including higher mode effects. The ELF method is part of the Linear Static Procedure (LSP) described in Chapter 6.



a) Step 1 – Determine Fundamental Period of Vibration (*T*)

b) Steps 2 & 3 –Determine Spectral Acceleration S_A at Period *T* and then Total Inertia Force

c) Step 4 -- Distribute Total Inertia Force along the Height In Accordance with Fundamental Mode Shape

Figure 4-3. Illustration of Equivalent Lateral Force Method

c. Response spectrum – modal analysis procedure. The response spectrum–modal analysis procedure is similar to that described above for the ELF method, except that it is the most basic and truly dynamic method of analysis. In this method, the peak responses of linear elastic structures to earthquake ground motions characterized by response spectra are determined. The number of modes required varies for each analysis however; all modes with significant contribution to the total response of the structure should be included. Usually the numbers of modes are adequate if the total mass participation of the modes used in the analysis is at least within 90-percent of the total mass of the structure. Modal analysis is usually performed using computer software capable of determining the periods of vibration and mode shapes for all contributing modes. Most structural analysis programs have this capability, and for many reasons this type of analysis is preferred over the ELF method of analysis which is limited to a single mode. The response-spectrum modal analysis procedure however has limitations and time history analysis is usually recommended for final design and evaluation in conformance with ER 1110-2-1806 when:

- The computed response-spectrum stresses or section forces exceed the allowable values, thus indicating nonlinear response might occur

- Soil-structure interaction, water-structure interaction, and reservoir bottom absorption effects are controlling response of the structure and could impact a new design or evaluation of an existing structure

- An estimate of the level of nonlinear behavior and thus damage is necessary to assess acceptability of the design or seismic safety of an existing structure (EM 1110-2-6051)

Detailed information on the response spectrum–modal analysis procedure can be found in EM 1110-2-6050 (1999). The response spectrum – modal analysis is part of the Linear Dynamic Procedure (LDP) described in Chapter 6.

d. Time history-modal analysis procedure. This procedure is similar to that described for the response spectrum – modal analysis procedure, except that earthquake demands are in the form of acceleration time histories, rather than response spectra and the results are in terms of displacement and stress (or force) histories. Peak values of various response quantities are extracted from the response histories. Time history-modal analysis provides valuable time-dependent information that is not available in the response spectrum–modal analysis procedure. Especially important is the number of excursions beyond displacement levels where the structure might experience strength degradation (strain softening). As with the response spectrum–modal analysis procedure, the time history-modal analysis is limited to a linear elastic response. The nonlinear response of a structure is computed by the time-history method using the direct integration procedure described in 4-2*e* below. Detailed information on the time history–modal analysis procedure can be found in EM 1110-2-6051 (2000). The time history–modal analysis is part of the Linear Dynamic Procedure (LDP) described in Chapter 6.

e. Nonlinear Time history – direct integration procedure. This type of time history analysis, described in Paragraph 6-6, involves the direct integration of the equations of motion, and therefore is the most powerful method available for evaluating the response of structures to earthquake ground motions. It is a step-by-step numerical integration procedure, which determines stresses (or forces) and displacements for a series of short time increments from the initiation of loading to any desired time. The time increments are generally taken of equal length for computational convenience. The condition of dynamic equilibrium is established at the beginning and end of each time increment. The motion of the system during each time increment is evaluated on the basis of an assumed response mechanism. The advantage of this method is that it can be used for both linear and nonlinear analyses. In the case of nonlinear analyses, structure properties (including nonlinear behavior) can be modified during each time increment to capture response behavior appropriate to that deformed state. The application of nonlinear analysis to concrete hydraulic structures is limited to cases for which experimental or observational evidence of nonlinear behavior is available and that validity of the numerical models have been demonstrated. These include certain nonlinear behavior such as joint opening mechanisms in arch dams, tensile cracking of gravity dams, sliding and rotational stability of blocks isolated by opened joints and cracked sections, and yielding and cracking of free-standing intake towers.

4-3. Modeling of Structural Systems

a. Structural models. Structural models for dynamic analyses are developed much in the same manner as for static analyses. However, distribution of mass and stiffness and dynamic interaction between the structure and water and between the structure and foundation as well as with the backfill soil should be established accurately. The response of a structure under severe ground shaking may approach or exceed the yield/cracking state. This means that in a linear-elastic dynamic analysis the use of effective stiffness (Paragraph 4-4) is more appropriate than the initial elastic stiffness used in the static analysis, and that the damping should be selected consistent with the expected level of deformation and the extent of nonlinear behavior. Furthermore, concrete deterioration and cracking can reduce structural stiffness of an existing structure; thus these effects should be considered in estimation of a representative effective stiffness. The dynamic interaction with the foundation introduces flexibility at the base of the model and could provide additional damping mechanisms through material and radiation damping. Various foundation models suitable for concrete hydraulic structures are discussed in Paragraph *b* below. A hydraulic structure also interacts with the impounded, surrounding, or retained water through hydrodynamic pressures at the structure-water interface. This interaction is coupled in the sense that motions of the structure generate hydrodynamic pressures that affect deformations (or motions) of the structure, which in turn influence the hydrodynamic pressures. Various structure-water interaction models with varying degrees of sophistication are described in Chapter 2 of EM 1110-2-6051. They include models as simple as the added-mass concept to more vigorous finite-element formulation that accounts for water compressibility and boundary absorption effects. Often structure-foundation interaction and structure-water interaction effects can be accommodated in the special finite-element models developed for gravity and arch dam evaluation. Since foundation properties, structural properties, and boundary conditions can vary, it is advisable to systematically vary parameters that have a significant effect on structure response until the final results cover a reasonable range of possible responses the structure could experience during the design earthquake. Properties of concrete for use in seismic analysis are described in Chapter 5. Following is a brief description of structural idealization for seismic analysis of concrete hydraulic structures. For detailed discussions and more information refer to EM 1110-2-6050, EM 1110-2-6051, and respective manuals for a particular structure.

(1) A variety of models are used to represent different types of hydraulic structures or to capture certain modes of behavior. For example, the model may be as simple as a rigid block to perform sliding stability analysis of a dam section (Figure 4-1), a frame model to compute earthquake response of a freestanding intake tower, a 2D finite-element mesh for stress analysis a gravity dam, or a more elaborate 3D finite-element mesh with nonlinear joint elements to simulate contraction joints opening in an arch dam (see example in Appendix F).

(2) Frame type models. A frame type or stick model is composed of beam-column elements with nodal lumped masses for analyzing regular freestanding intake towers, or possibly U-frame or W-frame lock sections. Frame models are generally preferred for reinforced concrete where the earthquake demands are expressed in section moments, shears, and axial loads; the parameters needed to design and evaluate reinforced concrete members. The frame models could be developed in two or three dimensions. In 2D representation one horizontal component of the ground motion and sometimes also the vertical component will be used as the seismic input. Appendices D and E present two examples of frame models: a freestanding intake tower and a W-frame or dual-chamber navigation lock, respectively. One advantage of the frame models is that the beam-column elements include plastic hinge capability for modeling nonlinear behavior. In the examples cited here, this capability was used to conduct nonlinear static pushover analyses.

(3) 2D models. 2D finite-element idealization is used to model planar or very long structures such as gravity dams, lock structures, retaining walls, and outlet tunnels. These structures are usually built of independent segments separated by construction joints, and the loads perpendicular to the long axis are assumed not to change along each segment. In situations like this, the structure may be modeled as a 2D slice using either the plane stress or plane strain elements depending on whether the stress or strain can be ignored in the out-of-plane direction. In either case the foundation model is idealized using plane-strain elements. A 2D model is usually analyzed for two components of ground motion applied in the vertical and horizontal directions. Examples in Appendices G and H illustrate application of this type of modeling to a lock gravity wall with backfill soil and a non-overflow gravity dam section.

(4) 3D models. 3D finite-element idealization is used to analyze structures with complex or irregular geometry or nonuniform loading. Arch dams, inclined intake towers supported by the abutment foundations, irregular freestanding towers with significant torsional behavior, gravity dams built in narrow canyons, and certain lock monoliths with complicated components and loading conditions fall in this category. A 3D model is usually constructed using 3D solid elements, but shell elements may also be used for relatively thin sections of the structure. The seismic input for a 3D model includes three orthogonal components of ground motion, two horizontal and the vertical, applied along the principal axes of the structure. Appendix F provides an example of 3D modeling applied to linear and nonlinear earthquake analyses of an arch dam.

(5) Soil-structure-interaction (SSI) models. An SSI model refers to a case where interaction between the structure and its foundation requires special consideration in terms of the ground motion at the base of the structure and the flexible support provided by the soil foundation. At soil sites the bed rock motion is affected by the local soil conditions as it travels to the ground surface, and the presence of the structure produces a further change to this motion due to kinematic constraints. Furthermore, the foundation interacts with the structure by elongating periods of vibration and providing additional damping. An SSI condition requires a model which includes both the structure and foundation together (direct method) or separately (substructure method). These methods are briefly described in Paragraphs *c*(3) and *c*(4) below. Further discussions are provided in EM 1110-2-6050 and EM 1110-2-6051. An SSI analysis may be conducted using 2D or 3D models.

b. Foundation models. Foundation-structure interaction introduces flexibility at the base of the structure and provides additional damping mechanisms through material damping and radiation. The flexible foundation tends to lengthen the period of vibration and the material and radiation damping in the foundation region has the effect of reducing the structural response.

Such interaction effects generally introduce frequency-dependent interacting forces at the structure-foundation interface requiring more elaborate analysis. In practice however, simplified models that include only the flexibility of the foundation and not its inertia and damping are more common.

(1) Massless rock foundation model. Generally arch dams, gravity dams, and sometimes lock walls and intake towers are built on competent rock foundations. In these situations a massless finite-element model can adequately represent the effects of rock region supporting the structure. The size of foundation model need not be very large so long as it is comparable with dimensions of the structure. The earthquake input is applied directly at the fixed boundaries of the massless foundation model.

(2) Viscoelastic rock foundation model. The simplified massless foundation model discussed above accounts only for the flexibility of the foundation thus ignores its inertia and damping effects. This assumption may not be appropriate for rock sites whose elastic moduli are substantially lower than the massive concrete that they support. In such cases if similar rocks can be assumed to extend to large depths, the foundation may be idealized as a viscoelastic model. A viscoelastic model is represented by impedance functions whose terms are complex and frequency-dependent. The real component of the impedance function represents the stiffness and inertia of the foundation and the imaginary component characterizes its radiation and material damping. Two such viscoelastic models have been developed for the 2D analysis of gravity dams (Dasgupta and Chopra 1979) and 3D analysis of arch dams (Zhang and Chopra 1991).

(3) Finite-element soil-structure interaction (SSI) model. The interaction between the soil and structure can be fully accounted for by developing a direct SSI model, which includes both the structure and the supporting soil. The structure is modeled using frame and/or solid elements with linear material properties. The soil medium is represented by solid elements with strain-dependent soil properties. The two-dimensional direct method of SSI analysis can be carried out using the computer program FLUSH (Lysmer et al. 1975) or Q-FLUSH (Quest Structures 2001). These programs conduct SSI analyses in the frequency domain, where the nonlinear soil behavior is approximated by the equivalent linear method (Seed and Idriss 1969) and the response is evaluated by iteration. The iteration involves updating the stiffness and damping values in accordance with the prescribed strain-dependent material curves until the solution converges.

(4) Lumped-parameter soil foundation model. The soil-structure interaction effects can also be represented using a lumped-parameter model of the soil. A complete form of the lumped-parameter model consists of frequency-independent springs, dampers, and masses that closely reproduce the actual response of the soil. The simplest model that can be developed for each degree of freedom of a rigid basemat includes a spring and a damper connected to the basemat with a fictitious mass of the soil added to mass of the structure. The frequency-independent coefficients of this SDOF system are obtained by a curve-fitting procedure such that a good agreement between the dynamic stiffness of the SDOF model and that of the actual soil is achieved. Appendix B of EM 1110-2-6051 provides lumped-parameter models for a disk supported by a homogeneous half space, an embedded cylinder, an embedded prism, and a strip supported on the surface of a homogeneous half space. For application to finite-element analysis, distributed soil springs, dampers, and masses can be obtained by dividing the total soil parameters by the base area, and then assigning them to individual nodes according to the tributary area of each node.

c. Pile foundation models. Several analytical methods have been developed for the seismic-load analysis of soil-pile systems. The static-load "*p-y*" method of pile analysis, originated in the offshore industry, have been modified and extended to cyclic loading conditions, and is now routinely applied to dynamic or earthquake loading cases. At the same time, dynamic soil-pile analysis methods (elastic continuum solution) have been developed for single piles and pile groups embedded in homogenous and non-homogenous soil media. Such methods are more theoretically sound than the *p-y* method, and along with the finite-element method provide reasonable solutions for the soil-pile-structure interaction analysis. However these methods do not allow for the adequate characterization of the localized yielding at the soil-pile interface, and are generally suitable for relatively low levels of seismic loading. The results of dynamic pile analyses include seismic response as well as the dynamic stiffness of piles that can be used in the subsequent soil-pile-structure interaction analysis. In practice, four levels of soil-pile-structure-interaction (SPSI) analysis progressing from simple to complete interaction can be employed as follows:

(1) Single-pile kinematic seismic response analysis. This basic pseudo-static analysis incorporates nonlinear response and is performed as pile integrity evaluation. A pseudo-static method for pile integrity consists of transforming the horizontal profile of soil displacement (derived from a free-field site response analysis) to a curvature profile, and comparing peak values to allowable pile curvatures. This method assumes piles perfectly follow the soil, and that no inertial interaction takes place. Alternatively, a displacement time history may be applied to nodal points along the pile in a dynamic pile integrity analysis.

(2) Pile-head stiffness or impedance functions. In the second level of analysis, pile head stiffness or impedance functions may be obtained from linear or nonlinear soil-pile analyses and assembled into a pile-head stiffness matrix for use in a global response analysis (Figures 4-4 and 4-5). Secant stiffness values at design level deformations are normally prescribed from nonlinear soil-pile response analyses (Figure 4-6).

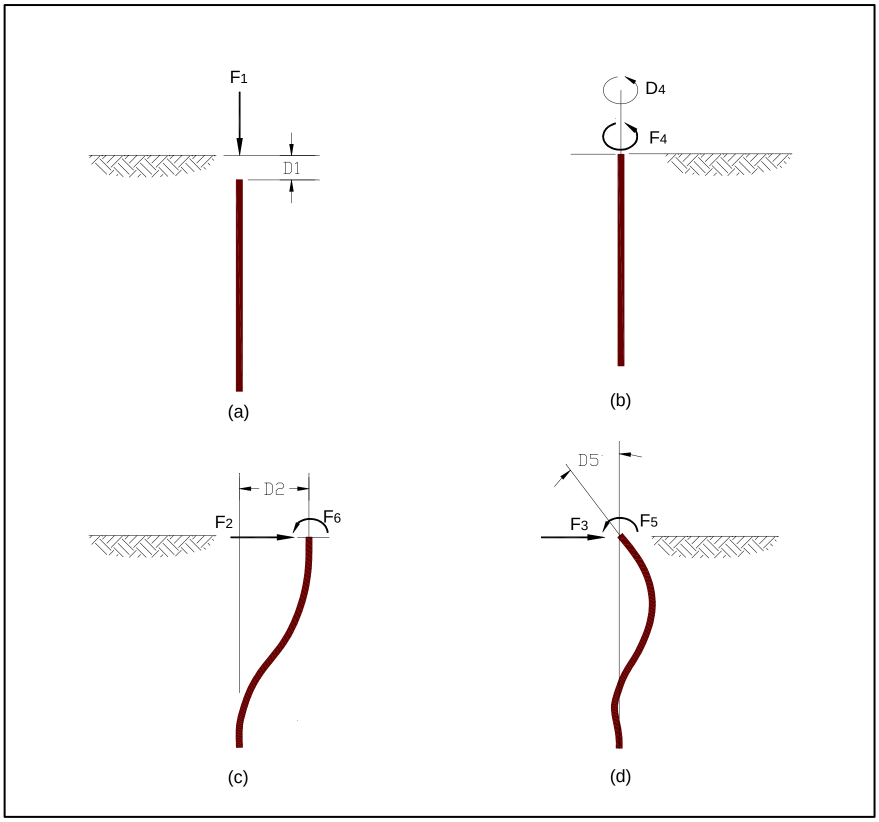


Figure 4-4. Pile Behaviors

$$\begin{bmatrix} K_{11} & 0 & 0 & 0 & 0 & 0 \\ 0 & K_{22} & 0 & 0 & 0 & K_{26} \\ 0 & 0 & K_{33} & 0 & K35 & 0 \\ 0 & 0 & 0 & K_{44} & 0 & 0 \\ 0 & 0 & K53 & 0 & K_{55} & 0 \\ 0 & K_{62} & 0 & 0 & 0 & K_{66} \end{bmatrix} \begin{Bmatrix} D_1 \\ D_2 \\ D_3 \\ D_4 \\ D_5 \\ D_6 \end{Bmatrix} = \begin{Bmatrix} F_1^1 \\ F_2^2 + F_6^2 \\ F_3^3 + F_5^3 \\ F_4^4 \\ F_5^5 + F_3^5 \\ F_6^2 + F_6^6 \end{Bmatrix} = \begin{Bmatrix} F_1 \\ F_2 \\ F_3 \\ F_4 \\ F_5 \\ F_6 \end{Bmatrix}$$

$$\underline{K} \times \underline{D} = \underline{F_{dir}} + \underline{F_{coup.}} = \underline{F}$$

where:

$\underline{K} = Stiffness$

$\underline{D} = Displacement$

$\underline{F_{dir}} = Direct\ Force$

$\underline{F_{coup.}} = Coupled\ Force$

$\underline{F} = Total\ Force$

Figure 4-5. Flexible Pile Stiffness Matrix (after Kriger and Wright, 1980)

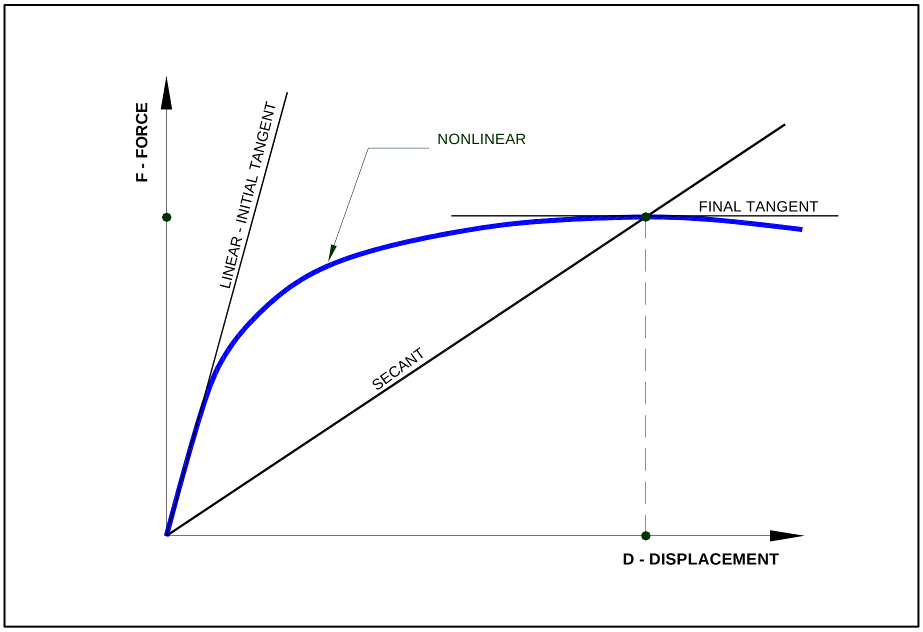


Figure 4-6. Secant stiffness value selected at design level displacement from nonlinear soil-pile force-displacement curve (after Kriger and Wright 1980)

(3) Substructure method. Both inertia and kinematic interaction may be evaluated from a substructuring type analysis to determine pile head impedance and foundation level input motions (Figure 4-7). As described in EM1110-2-6050, the SPSI analysis may be performed in two steps consisting of the kinematic and inertia parts. The kinematic interaction is accomplished by setting mass of the superstructure to zero and obtaining the foundation level input motions (kinematic motions) for the subsequent inertia interaction analysis. The inertia interaction analysis is carried out in two steps. First pile-head impedance or dynamic stiffness is determined from a separate analysis of the soil-pile foundation system, and then used as spring supports in the inertia analysis of superstructure subjected to kinematic motions.

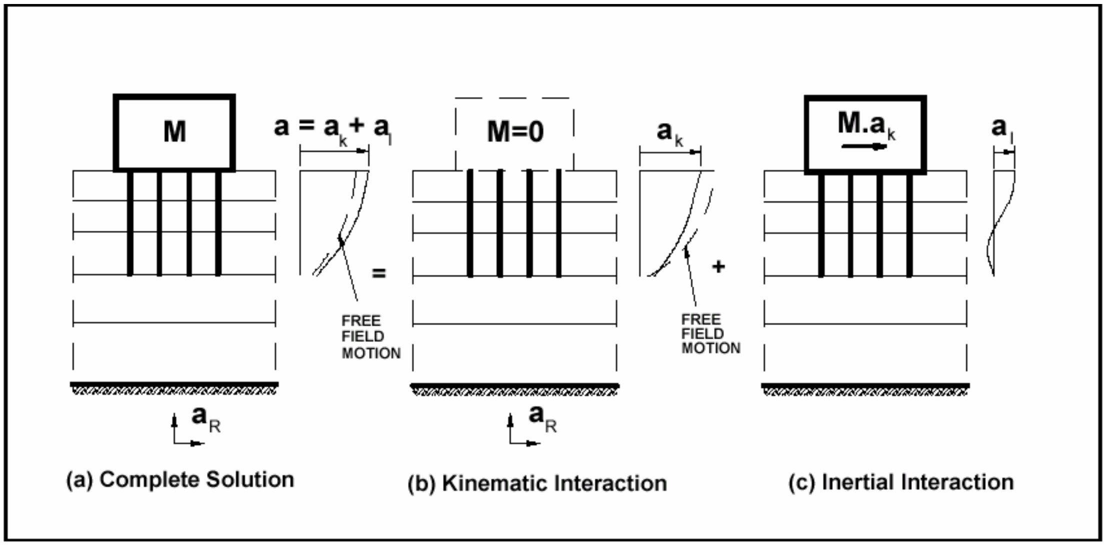


Figure 4-7. Substructuring concept: Decomposition of the problem into kinematic and inertia interaction problems.

(4) Complete or direct method of analysis. Finally a fully coupled SPSI analysis may be carried out to determine the complete system response. This can be accomplished by developing a complete finite-element model consisting of the structure and the soil-pile foundation and analyzing for a prescribed input motion. A 2D approximation of the soil-pile-structure system can be evaluated using the computer program FLUSH (Lysmer et al. 1975) or its enhanced web-based version Q-FLUSH at www.webdams.com (QUEST Structures 2001). Application of the SPSI analysis to lock structures is fully described in EM 1110-2-6051 and shown in Figure 4-8. An important aspect of the SPSI analysis is that large shear deformations that occur in soils during strong earthquake shaking introduce significant nonlinear behavior in the foundation region and must be considered in the analysis. In the FLUSH program the nonlinear response of the soil is approximated by the equivalent linear method (Seed and Idriss 1969). A similar 3D approximation of SPSI model can be evaluated using the computer program SASSI (Lysmer et al. 1981). However, the number of piles that can be included in 3D SASSI models is limited.

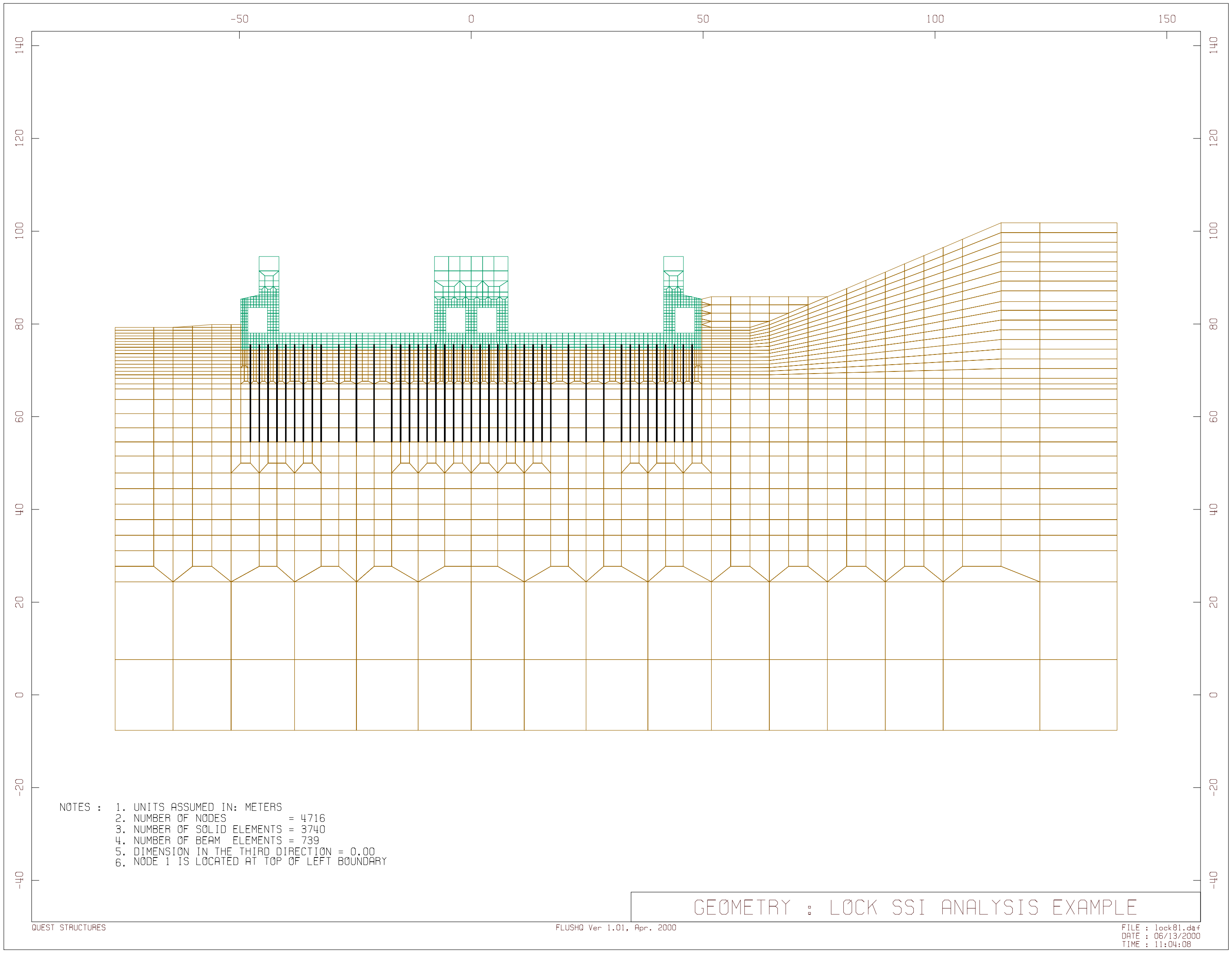


Figure 4-8. Q-FLUSH Two-dimensional Soil-Pile-Structure-Interaction Model of Olmsted Lock Chamber Monolith

d. Fluid-structure interaction. A hydraulic structure and water interact through hydrodynamic pressures at the structure-water interface. In the case of concrete dams, the hydrodynamic pressures are affected by the energy loss at the reservoir boundary. The complete formulation of the fluid-structure interaction produces frequency-dependent terms that can be interpreted as an added force, an added mass, and an added damping (Chopra 1987). The added hydrodynamic mass influences the structure response by lengthening the period of vibration, which in turn changes the response spectrum ordinate and thus the earthquake forces. The added hydrodynamic damping arises from the radiation of pressure waves and, for dams, also from the refraction or absorption of pressure waves at the reservoir bottom. The added damping reduces the amplitude of the structure response especially at the higher modes. Hydrodynamic effects for concrete hydraulic structures including dams, locks, and intake towers are fully described in EM 1110-2-6051. If the water is assumed incompressible, hydrodynamic effects are simply represented by added mass coefficients. Depending on the level of sophistication needed the added hydrodynamic mass may be computed using Westergaard, velocity potential, or finite-element procedures (EM 1110-2-6051). For high dams refined dam-water interaction analysis including water compressibility and reservoir-boundary absorption effects may be required (Hall and Chopra 1980; Fenves and Chopra 1984b; Fok and Chopra 1985.

e. Backfill-structure interaction effects. The interaction between the structure and backfill, and structure and surrounding water, as stated above, can be approximated using added mass concepts. It should be realized however, these interactions are complex and in some cases it may be necessary to use analytical methods, which deal with the interaction effects directly. Also important is the interaction between the structure and foundation. This interaction too is complex. In general, the effect of the foundation is to lengthen the fundamental period of the structure-foundation system, and to increase energy absorption due to energy radiation and material damping that occurs in the foundation material.

4-4. Effective Stiffness

When analyzing concrete hydraulic structures for static loads, it is generally acceptable to use stiffness values associated with the un-cracked section properties and to ignore the stiffening contribution of reinforcing steel. However, under seismic loads it is important that distribution of stresses and member forces be based on stiffness values that are representative of the near yield /cracking conditions. This is because the effective stiffness of CHS at near yield/cracking conditions can be significantly less than that represented by gross section properties. For reinforced concrete structures, the effective stiffness should be used in dynamic analyses to ensure that the hierarchy of member yielding conforms to assumed distributions, and that member plastic deformations are reasonably distributed through the structure. A reasonable estimate of the member stiffness is also required in computation of the structure period and hence seismic forces and displacements.

a. Plain concrete structures. Under severe earthquake ground shaking, it is probable that the elastic capacity of plain concrete structures such as gravity and arch dams would be exceeded, indicating some cracking with possible reduction in global stiffness of the structure. If cracking occurs near surfaces of these massive structures it will have minimal effects on the overall stiffness of the structure. Consequently in linear elastic analysis it is acceptable to use stiffness properties associated with the un-cracked sections. However, if cracking is pervasive and significant, its extent should be estimated and mapped so the stability of the cracked structure can be evaluated. Ideally, the evaluation should be conducted using nonlinear analyses if appropriate tools and procedures are available. Otherwise several approximate equivalent-linear analyses may be attempted, each with reduced stiffness and resistance characteristics assigned to all finite-elements that have reached their tensile capacities. Such approximate analyses however are valid for static loading condition and not for the earthquake loading which is oscillatory. The stiffness modification and analysis of the modified structure are repeated until no further cracking would occur or the structure reaches a limit state indicating failure. Approximate equivalent-linear analyses must be carried out based on a rational interpretation of the results and sound engineering. The loss or reduction of stiffness should be applied in the direction perpendicular to the cracks. For each element the amount of stiffness reduction should be estimated approximately proportional to the area covered by cracks.

b. Reinforced concrete structures. To obtain a best estimate of force and displacement demands the stiffness of cracked members (effective stiffness) should be used rather than the gross stiffness. The effective stiffness used is an average value for the entire member accounting for the distribution of cracking along the member length. The effective stiffness of reinforced concrete structures can be estimated based on the relationship between the cracking moment (i.e., the moment required to initiate cracking while ignoring the reinforcing steel) and the nominal moment capacity of the reinforced concrete section. The nominal moments and cracking moments are estimated at regions of the maximum positive or negative moments. Once the cracking moment (M_{CR}) and the nominal moment capacity (M_N) have been determined, the ratio of the effective stiffness (I_E) to the gross stiffness (I_G) can be estimated from:

$$\frac{I_E}{I_G} = 0.8 - 0.9\left[\frac{M_N}{M_{CR}} - 1\right] \tag{4-4}$$

The ratio of I_E / I_G should neither be greater than 0.8, nor less than 0.35 for walls reinforced with 40-grade steel, nor less than 0.25 for walls reinforced with 60-grade steel (EM 1110-2-2400). The nominal moment strength can be determined in accordance with standard ACI-318 procedures. The cracking moment (M_{CR}) can be determined from the following expression.

$$M_{CR} = \frac{I_G}{C}\left(\frac{P}{A_G} + f_r\right) \tag{4-5}$$

Where:

f_r = Modulus of Rupture = $0.62\sqrt{f_c^{'}}$ (MPa units)

= $\left\{7.5\sqrt{f_c^{'}}\right\}$ (psi units)

P = Axial Load

A_G = Gross Section Area

C = Distance from neutral axis to extreme fiber

4-5. Damping

a. An effective damping of 5-percent of the critical provides a reasonable estimate of the dynamic response of concrete hydraulic structures at or near yield and cracking. However, damping could be as low as 2 to 3 percent for loads far below the yielding and cracking and higher than 5 percent if the structure is showing energy dissipation through joint opening, tension cracking, and yielding. In situations where such nonlinear responses could develop, a damping value as high as 10 percent can be justified in performing linear response analyses.

However, after increasing the damping to 10 percent, if the structure is still showing further nonlinear behavior, then a nonlinear response analysis should be performed.

b. Dynamic interaction between the structure and foundation could increase the effective damping if the subsurface condition suggests potential energy dissipation through radiation and the foundation deforms far enough to offer energy loss through hysteretic behavior. In addition dynamic interaction between the structure and impounded, surrounding, or retained water can also increase the effective damping due to energy radiation and absorption at fluid boundaries. Unless such interaction effects are significant, the damping value should be limited to 5-percent. Higher effective damping values between 5 to 10 percent could be justified if interaction effects of the foundation and impounded are significant but have not explicitly been included in the analysis.

4-6. Interaction with Backfill Soil

a. General. In addition to the foundation and water interaction effects discussed in Section 4.3*,* the soil behind the lock and retaining walls also affects earthquake response of the wall. During an earthquake, a lock wall is subjected to dynamic soil pressures caused by motions of the ground and the wall. Depending on the magnitude of wall movements the backfill soil is said to be in yielding, nonyielding, or intermediate state. Accordingly, the available methods of design and analysis of the backfill soil pressures also fall into similar categories.

b. Dynamic pressures of yielding backfill. Yielding backfill condition means wall movements due to earthquake ground motions are sufficient to fully mobilize shear resistance along the backfill wedge creating limit state conditions. The dynamic earth forces will then be proportional to the mass in the failure wedge times the ground accelerations. When designing retaining walls with yielding backfill conditions for earthquake ground motions, the Mononobe-Okabe (Mononobe and Matuo 1929; Okabe 1924) approach and its several variations are often used. Procedures for determining the failure wedge and dynamic soil pressure effects for active, at-rest and passive conditions are described in the US Army Technical Report No. ITL-92-11, “The Seismic Design of Waterfront Retaining Structures”, (Ebeling and Morrison, 1992). The resulting dynamic pressures expressed in terms of equivalent added-mass coefficients are then added to the nodal masses of the wall in the dynamic analysis of the wall system as described in Section 4.3 above.

c. Dynamic pressures of non-yielding backfill. For massive structures with soil backfill, it is unlikely that movements sufficient to develop backfill yielding will occur during an earthquake. In this situation the backfill soil is said to be nonyielding and is treated as an elastic material. If idealized as a semi-infinite uniform soil layer, the dynamic soil pressures and associated forces for a nonyielding backfill can be estimated using a constant-parameter SDOF model (Veletsos and Younan 1994)) or a more elaborate MDOF system (Wolf 1995). The dynamic soil pressures for a more general nonyielding backfill soil can be determined by the finite-element procedure similar to that discussed in Paragraph 4-3*c*(4).

d. Intermediate case. The intermediate case in which the backfill soil undergoes nonlinear deformations can be represented by the finite element procedures using a soil-structure-interaction computer program such as QFLUSH. Figure 4-8 is an example of this approach where the lock structure, pile foundation, and the backfill soil are included in the model. The foundation and backfill soil are represented using plane-strain 2-D soil elements whose shear modulus and damping vary with level of shearing strains, and the nonlinear behavior is approximated by the equivalent linear method.

4-7. Permanent Sliding Displacement

a. Retaining walls and dams that are stable under static loading conditions may slide under severe earthquake ground motions, if the combined static plus seismic shear demands exceed sliding resistance along any potential sliding planes. The acceleration that generates sufficient force to initiate sliding is termed the critical acceleration (a_c). Every time the ground acceleration exceeds the critical acceleration the structure will slide. The ratio of the critical acceleration to the acceleration of gravity (a_c/g), is termed the yield coefficient (k_y). The ratio of the peak ground acceleration (a_m) to the acceleration of gravity (a_m/g), is termed the seismic coefficient (A). The expected permanent displacement of a retaining wall or dam treated as a rigid block can be estimated using the Newmark sliding block analogy (Newmark, 1965).

b. As shown in Figure 4-9, each time the ground acceleration exceeds the critical acceleration (a_c), some displacement at the structure-foundation interface will occur, and these will add up throughout the ground shaking and result in a final sliding permanent displacement. The total permanent sliding displacement will be a function of the earthquake characteristics such as duration and intensity, with the major factor being the number of times the critical acceleration is exceeded. As a part of extensive parametric studies, Richards and Elms have suggested the following equation for estimating permanent displacement, (Richards and Elms, 1977).

$$\Delta = 0.087\frac{v_g^2}{Ag}\left[\frac{k_y}{A}\right]^{-4} \tag{4-8}$$

Where:

v_g = The peak ground velocity of the earthquake.

For preliminary design purposes the peak ground acceleration can be assumed equal to:

$v_g = 0.30A$ (distance in inches)

$v_g = 0.75\ A$ (distance in meters)

The relationship described above then can be simplified to:

$$\Delta = 0.2\frac{A^5}{k_y^4} \quad \text{(inches)} \tag{4-9a}$$

$$\Delta = 5\frac{A^5}{k_y^4} \quad \text{(mm)} \tag{4-9b}$$

A plot of the above relationship is shown in Figure 4-10. This plot can be used as a preliminary evaluation tool for estimating permanent displacement in retaining walls and dams. Additional information relative to the sliding displacement of dams can be found in Zhang and Chopra (1991).

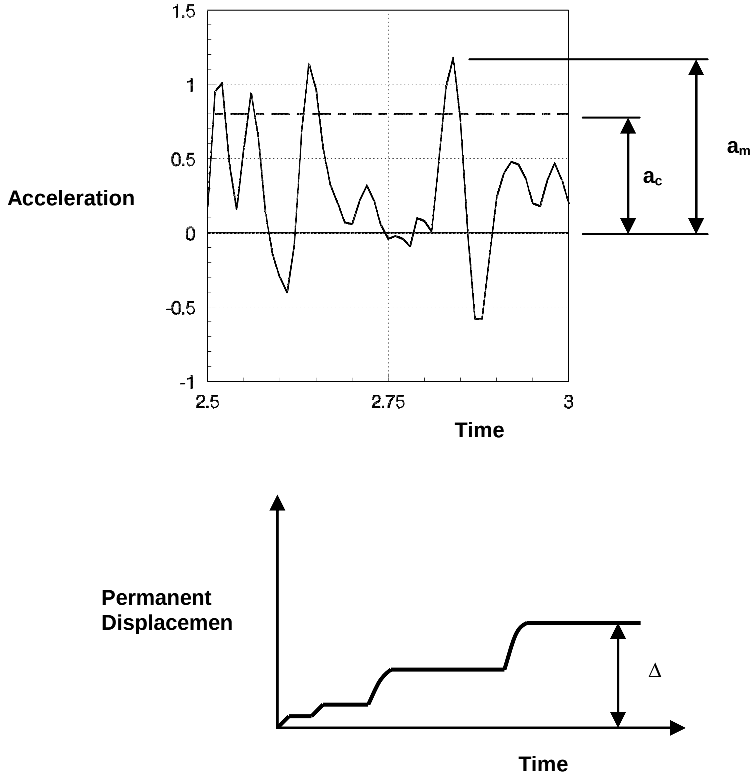


Figure 4-9. Permanent Sliding Displacement

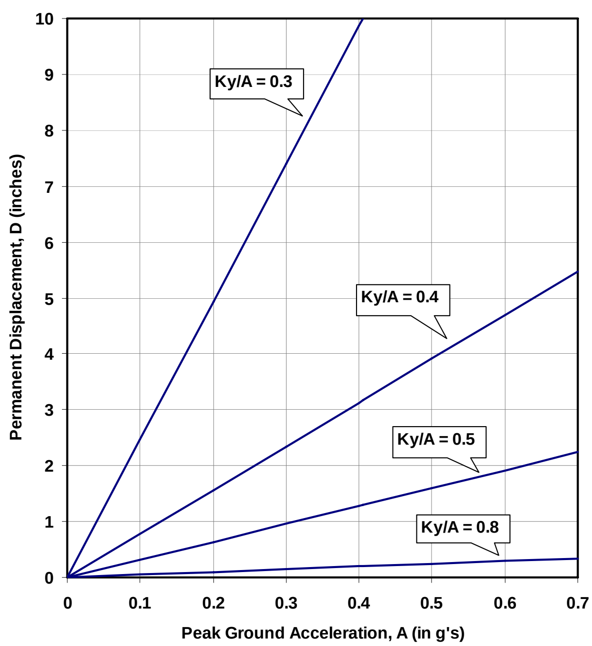


Figure 4-10. Permanent Displacement as a Function of k_y and *A*

4-8. Mandatory Requirements

Seismic evaluation of CHS should follow the progressive analysis methodology described in this chapter.