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Earthquake Design and Evaluation of Concrete Hydraulic Structures

ENGINEER MANUAL

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DEPARTMENT OF THE ARMY
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Engineering and Design
EARTHQUAKE DESIGN AND EVALUATION OF CONCRETE HYDRAULIC STRUCTURES

1. Purpose. This manual provides guidance for performance-based design and evaluation of concrete hydraulic structures (CHS). It introduces procedures that show how to design or evaluate a hydraulic structure to have a predictable performance for specified levels of seismic hazard. Traditional design and evaluation procedures may still be used for feasibility and screening purposes. However, for critical facilities, they should be followed by the procedures of this manual to prevent sudden collapse even thought the structure may suffer severe damage, to limit damage to a repairable level, or to maintain functionality immediately after the earthquake.

2. Applicability. This manual applies to all USACE commands having responsibilities for civil works projects.

3. Distribution. This manual is approved for public release. Distribution is unlimited.

4. Discussion. This manual covers requirements for the seismic design and evaluation of plain and reinforced concrete hydraulic structures. The types of concrete hydraulic structures addressed in this manual include dams, U- and W-frame locks, gravity walls, and intake/outlet towers. The guidelines are also applicable to spillways, outlet works, hydroelectric power plants, and pumping plants. The structures may be founded on rock, soil, or pile foundations and may or may not have backfill soil.

FOR THE COMMANDER:

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Chapter 1
Introduction

1-1. Purpose

This manual provides guidance for performance-based design and evaluation of concrete hydraulic structures (CHS). It introduces procedures that show how to design or evaluate a hydraulic structure to have a predictable performance for specified levels of seismic hazard. Traditional design and evaluation procedures may still be used for feasibility and screening purposes. However, for critical facilities, they should be followed by the procedures of this manual to prevent sudden collapse even though the structure may suffer severe damage, to limit damage to a repairable level, or to maintain functionality immediately after the earthquake.

1-2. Applicability

This manual applies to all USACE commands having responsibilities for civil works projects.

1-3. References

Required and related publications are listed in Appendix A.

1-4. Distribution Statement

This manual is approved for public release with unlimited distribution.

1-5. Mandatory Requirements

Engineers performing seismic design and evaluation of concrete hydraulic structures are required to satisfy specific mandatory requirements. The purpose of mandatory requirements is to assure that the structure meets minimum safety and performance objectives. Mandatory requirements usually pertain to critical elements of the design and evaluation, such as loads and load combinations, to analytical procedures used to determine force and displacement demands, and to methods used to determine member strength and displacement capacities. Mandatory requirements pertaining to the guidance contained in a particular chapter are summarized at the end of that chapter. No mandatory requirements are identified in the appendices. Instead, any mandatory requirements pertaining to information contained in the appendices is cited in chapters that reference those appendices. Where other Corps guidance documents are referenced, the engineer must review each document to determine which of its mandatory requirements are applicable to the design or/ evaluation of the project. Engineers performing the independent technical review must ensure that the designers and/or analysts have satisfied all mandatory requirements.

1-6. Scope

This manual covers requirements for the seismic design and evaluation of plain and reinforced concrete hydraulic structures. The types of concrete hydraulic structures addressed in this manual include dams, U- and W-frame locks, gravity walls, and intake/outlet towers. The guidelines are also applicable to spillways, outlet works, hydroelectric power plants, and pumping plants. The structures may be founded on rock, soil, or pile foundations and may or may not have backfill soil.

Chapter 2
Design Criteria

2-1. Design Earthquakes

a. General. Earthquake ground motions for the design and evaluation of Corps CHS are the Operating Basis Earthquake (OBE) and the Maximum Design Earthquake (MDE) ground motions. Seismic forces associated with the OBE are considered unusual loads. Those associated with the MDE are considered extreme loads. Earthquake loads are to be combined with other loads that are expected to be present during routine operations.

b. Operating Basis Earthquake. The OBE is a level of ground motion that is reasonably expected to occur within the service life of the project, that is, with a 50-percent probability of exceedance during the service life. (This corresponds to a return period of 144 years for a project with a service life of 100 years).

c. Maximum Design Earthquake. The MDE is the maximum level of ground motion for which a structure is designed or evaluated. As a minimum, for other than critical structures, the MDE ground motion has a 10 percent chance of being exceeded in a 100-year period, (or a 1000-year return period). For critical structures, the MDE ground motion is the same as the maximum credible earthquake (MCE) ground motion. Critical structures, by ER 1110-2-1806 definition, are structures that are part of a high hazard project and whose failure will result in loss of life. The MCE is defined as the largest earthquake that can reasonably be expected to occur on a specific source, based on seismological and geological evidence.

2-2. Performance Levels

a. General. Various performance levels are considered when evaluating the response of CHS to earthquake ground motions. The performance levels commonly used are serviceability performance, damage control performance, and collapse prevention performance.

b. Serviceability performance. The structure is expected to be serviceable and operable immediately following earthquakes producing ground motions up to the OBE level.

c. Damage control performance. Certain elements of the structure can deform beyond their elastic limits (non-linear behavior) if non-linear displacement demands are low and load resistance is not diminished when the structure is subjected to extreme earthquake events. Damage may be significant, but it is generally concentrated in discrete locations where yielding and/or cracking occur. The designer should identify all potential damage regions, and be satisfied that the structure is capable of resisting static loads and if necessary can be repaired to stop further damage by non-earthquake loads. Except for unlikely MCE events, it is desirable to prevent damage from occurring in substructure elements, such as piling and drilled piers, and other inaccessible structural elements.

d. Collapse prevention performance. Collapse prevention performance requires that the structure not collapse regardless of the level of damage. Damage may be unrepairable. Ductility demands can be greater than those associated with the damage control performance. If the structure does not collapse when subjected to extreme earthquake events, resistance can be expected to decrease with increasing displacements. Collapse prevention performance should only be permitted for unlikely MCE events. Collapse prevention analysis requires a Nonlinear Static Procedure (NSP) or Nonlinear Dynamic Procedure (NDP) in accordance with the guidance in Chapter 6.

2-3. Performance Goals

a. General. Both strength and serviceability must be considered in the design of structures. For plain concrete structures, the consequences of inadequate strength can be failure by shear, flexure, tension, or compression. The same consequences exist for reinforced concrete structures except that additional failure mechanisms such as bond failure and buckling and tensile failure of reinforcing steel are also possible. Lack of adequate strength can result in loss of life and severe economic loss. Structures must also be serviceable under sustained and frequent loads. Serviceability for usual static load conditions is a matter of limiting structural displacements. For unusual earthquake loading (i.e. OBE), the serviceability requirement is to assure the project will function without interruption, with little or no damage. For new structures, the additional cost of designing for linear elastic performance during OBE events is usually low. However, the cost of strengthening an existing structure to obtain the same performance objective may be high. The cost of seismic strengthening of an existing structure for serviceability purposes must be weighed against the cost of repairing the structure after it has experienced an OBE event. The performance goals for concrete hydraulic structures are demonstrated using idealized force displacement curves (Figures 2-1 through 2-3) representing ductile, limited-ductile, and brittle failure behavior. Using procedures described in Chapters 5 and 6, a capacity curve is constructed. With this curve serviceability, damage control, and collapse prevention performance regions are identified. To properly assess the performance of complex structures it is necessary to understand the loading history, the changes in system stiffness and damping as yielding and cracking occur, the redistribution of resisting loads, and the path the structure follows from the initial elastic state to a collapse prevention limit state. This is done using nonlinear static analysis and/or nonlinear dynamic analysis if sufficient information is known about the nonlinear properties of the system. For most structures, a combination of engineering analysis and judgment must be used to determine if performance objectives have been met.

b. Ductile behavior. Ductile behavior is illustrated in Figure 2-1. It is characterized by an elastic range (Point 0 to Point 1 on the curve), followed by a plastic range (Points 1 to 3) that may include strain hardening or softening (Points 1 to 2), and a strength degradation range (Points 2 to 3) in which some residual strength may still be available before collapse occurs. Building frame systems designed according to FEMA or ACI provisions exhibit this type of behavior in flexure. Shear and bond mechanisms, however, exhibit limited-ductile or brittle behavior and therefore these failure modes must be suppressed if overall ductile behavior as illustrated by Figure 2-1 is to be achieved. When subjected to MDE ground motion demands, ductile structures should have sufficient strength to assure performance will remain within the strain hardening region (Points 1 to 2), an inelastic region where strength increases with an increase in strain. In addition, in case of OBE ground motion demands, all elements of the structure should perform within the linear elastic range (Points 0 to 1). Designers of new reinforced concrete structures should establish a hierarchy in the formation of failure mechanisms by allowing flexural yielding to occur while at the same time suppressing shear, and other brittle or limited-ductile failure mechanisms. Such a design produces ductile behavior. Reinforced concrete structures designed by older codes do not provide the quantity of reinforcement (flexural and confinement), or the proper details needed to assure ductile behavior. For those structures, it is necessary to evaluate all three types of brittle, limited-ductile, and ductile failure mechanisms in order to determine which mode of behavior can be expected.

c. Limited-ductile behavior. Limited-ductile behavior (Figure 2-2) is characterized by an elastic range and limited plastic range that may include strain hardening or softening, followed by a complete loss of strength. Plain concrete structures and lightly reinforced concrete structures such as intake/outlet towers (structures with cracking moment capacities equal or greater than nominal strength) generally exhibit this type of behavior in flexure, although the plastic range may be limited. It should be recognized that some residual capacity, as indicated in Figure 2-2, may still exist in concrete gravity dams and in other plain and lightly reinforced concrete structures. This residual capacity occurs due to dead load effects that contribute to shear-friction resistance and to overturning resistance. This residual capacity exists even though cracks have propagated through the structure, or in the case of reinforced concrete structures, even though the principal reinforcing steel has fractured. Limited ductile structures when subjected to MDE ground motion demands should also have sufficient strength to assure performance will be within the inelastic region where strength increases with an increase in strain (strain hardening region). All elements of the structure when subjected to OBE ground motion demands should perform within the linear elastic range.

d. Brittle behavior. An elastic range of behavior, followed by a rapid and complete loss of strength, characterizes brittle, or non-ductile, behavior. Certain failures such as reinforcing steel buckling failures, reinforcing steel splice failures and anchorage failures exhibit this type of behavior under earthquake loading conditions. Sudden failure occurs because the concrete is not adequately confined to prevent spalling which in turn leads to a rapid loss of bond strength, and to buckling of the reinforcing steel. Brittle failure mechanisms should be avoided for the OBE and MDE. In other words the behavior controlled by such mechanisms should remain within the elastic range.

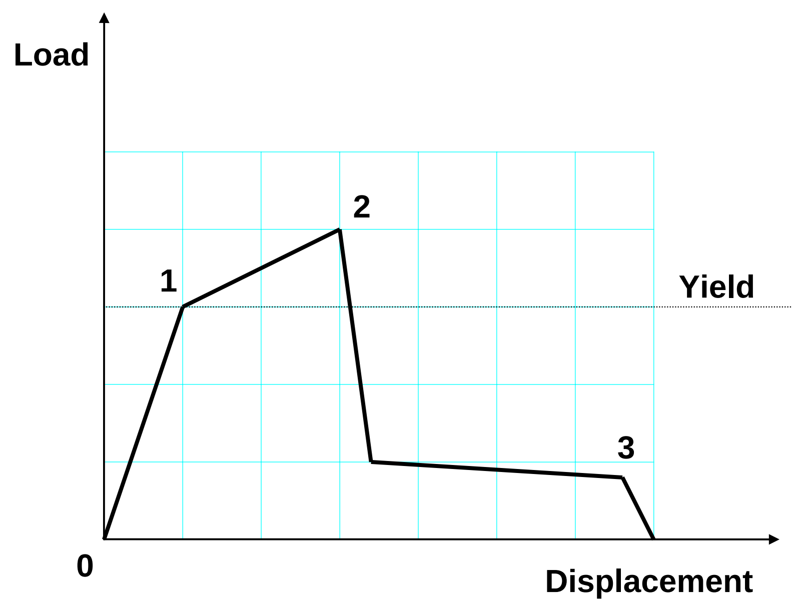


Figure 2-1. Ductile Behavior Curve (From FEMA 273)

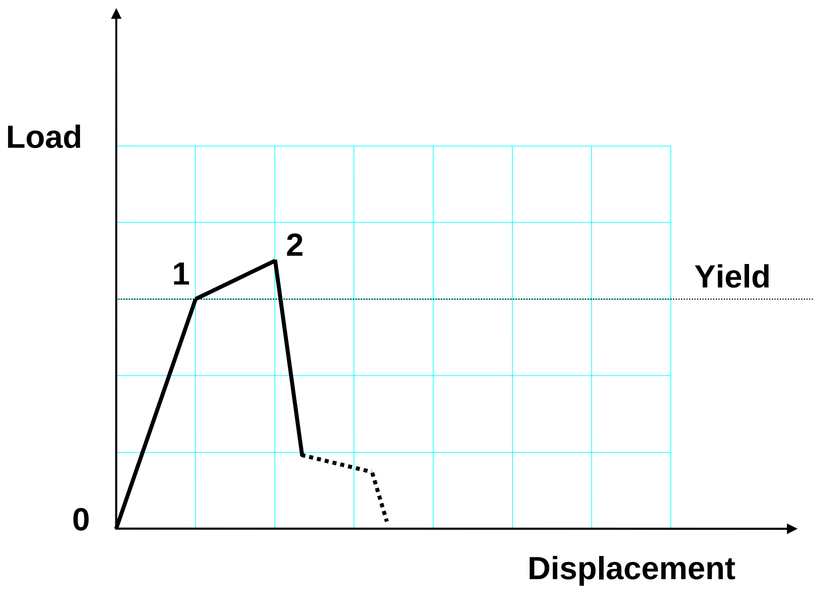


Figure 2-2. Limited-ductile Behavior Curve (consistent with FEMA 273)

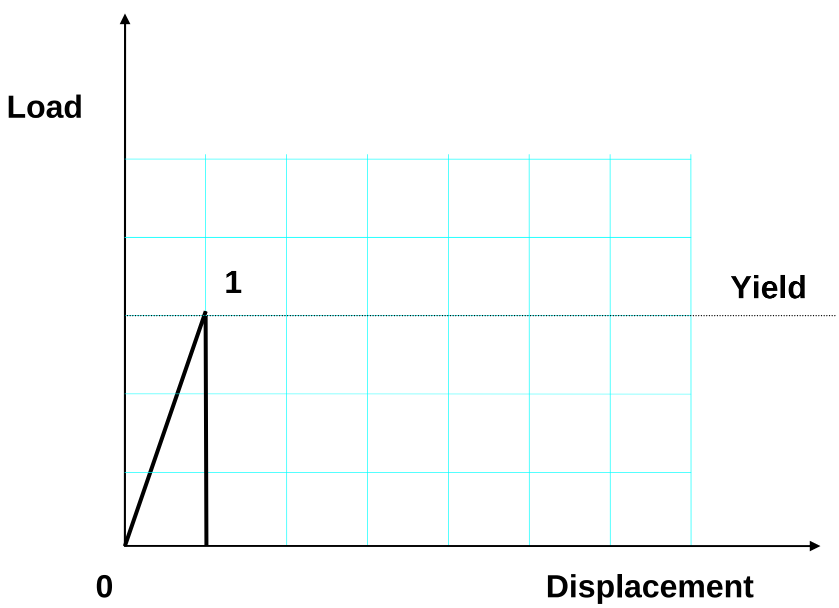


Figure 2-3. Brittle Behavior Curve (From FEMA 273)

2-4. Design Requirements

a. Strength design. Strength design for CHS subjected to earthquake ground motions is achieved by reducing the probability of structure collapse to an acceptable level. This is accomplished by selecting an appropriate design basis earthquake event to be used in combination with specific design and evaluation procedures that assure the structure will perform as intended. Seismic design and evaluation is most often based on linear-elastic response-spectrum or time-history analysis procedures, although nonlinear analysis procedures can be used for evaluation of certain nonlinear mechanisms. The design basis earthquake event used for strength evaluation of CHS is the Maximum Design Earthquake (MDE).

b. Serviceability design. Serviceability design for CHS subjected to earthquake ground motions is achieved by reducing the possibility of structure damage to a negligible level. As for strength performance, this is accomplished by selecting an appropriate design basis earthquake event to be used in combination with appropriate design and evaluation procedures. Evaluation is based on linear-elastic response spectrum analysis or time history analysis procedures. The design basis earthquake event used for serviceability evaluation of CHS is the Operating Basis Earthquake (OBE).

c. Loading combinations. The following loading combinations establish the ultimate strength and serviceability requirements for the design and evaluation of both plain and reinforced concrete hydraulic structures. The loading combinations represent the total demand (dead load + live load + earthquake) for which the structure must be designed or evaluated.

(1) *Earthquake strength design loading combination.* The following strength design loading combination shall be used to determine the total static plus earthquake demand on concrete hydraulic structures for Maximum Design Earthquake (MDE) conditions:

$$Q_{DC} = Q_D + Q_L + Q_{MDE} \tag{2-1}$$

where;

Q_{DC} = Combined action due to dead, live, and maximum design earthquake loads for use in evaluating damage control performance

Q_D = Dead load effect

Q_L = Live load effect + uplift

Q_{MDE} = Earthquake load effect from MDE ground motions including hydrodynamic and dynamic soil pressure effects

The live load effect is the structure response to live loads such as hydrostatic, earth pressure, silt, and temperature loads. Live loads to be considered are those that are likely to be present during the design earthquake event. The earthquake load effect is the response of an elastic structure to design earthquake ground motions. The earthquake load may involve multi-component ground motions with each component multiplied by +1 and -1 to account for the most unfavorable earthquake direction.

(2) *Serviceability loading combination.* The following serviceability design loading combination shall be used to determine the total earthquake demand on concrete hydraulic structures for Operating Basis Earthquake (OBE) conditions:

$$Q_S = Q_D + Q_L + Q_{OBE} \qquad (2\text{-}2)$$

where;

Q_S = Combined action due to dead, live, and OBE loads for use in evaluating serviceability performance

Q_D = Dead load effect

Q_L = Live load effect + Uplift

Q_{OBE} = Earthquake load effect from OBE ground motions including hydrodynamic and dynamic soil pressure effects

Live loads to be considered are those that are likely to be present during the OBE earthquake event.

2-5. Performance Evaluation

a. Plain Concrete Structures

(1) *General.* Although resistance to compressive and shear stresses are evaluated, the safety and serviceability of large plain concrete structures is usually controlled by the tensile behavior and cracking of the concrete. The actual response of massive concrete structures to earthquake ground motions is very complex. Loading histories and rapid seismic strain rates have an important influence on structural performance. The ultimate tensile strength of concrete is especially sensitive to strain rate. Most often, a concrete gravity dam or arch dam is evaluated based on the linear-elastic finite-element method (FEM) of analysis. The resulting stress demands from the FEM combined with engineering judgment and past experience are used to assess the performance. The assessment process requires knowledge on how the tensile strength might vary with loading history, strain rates, and construction methods (especially with respect to construction and contraction joints), and on how cracking might propagate as a result of repeated excursions beyond the tensile strength. The assessment is facilitated by using the stress demand-capacity ratios in conjunction with spatial extent of overstressed regions and cumulative duration of excursions beyond the tensile strength of the concrete. The demand-capacity ratios are obtained from division of computed stress demands by the static tensile strength of the concrete.

(2) *Response to internal force or displacement controlled actions*

(a) The response of a gravity dam to earthquake ground motions is illustrated in Figure 2-4. For earthquake motion cycles in the upstream direction, the potential cracking usually occurs at the heel of the dam at the maximum expected water levels. For earthquake motion cycles in the downstream direction, the potential cracking usually occurs at the slope discontinuity under the minimum expected water level conditions and near the toe of the dam. As earthquake motion cycles swing toward the upstream direction, the potential cracking shifts to the upper part and the base of the dam. In general, the tensile stress-strain results from linear elastic FEM are used to determine if the structure meets established project performance requirements. Performance under OBE loading conditions should be in the linear elastic range (Serviceability Performance) as illustrated on the tensile stress-strain diagram of Figure 2-5.

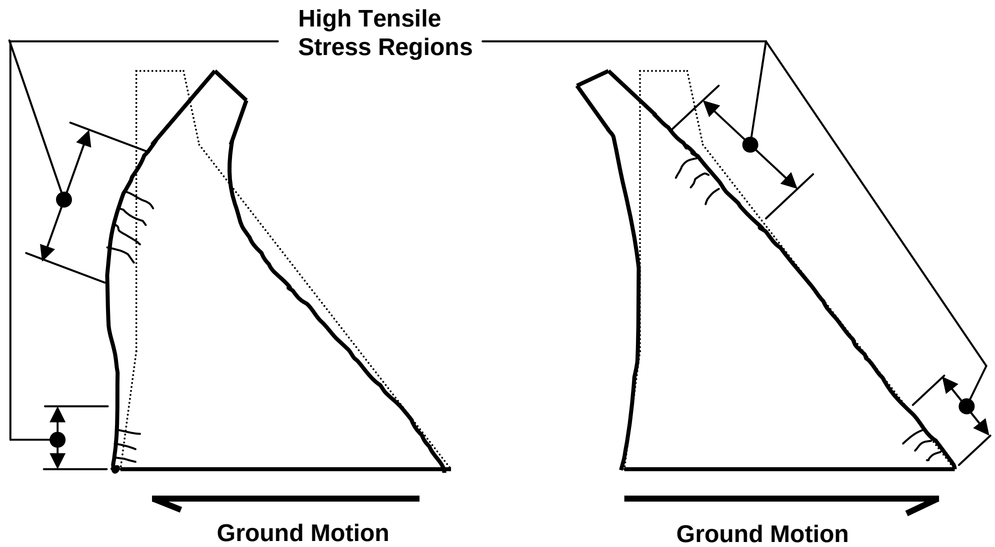


Figure 2-4. Gravity Dam Subjected to Earthquake Ground Motions

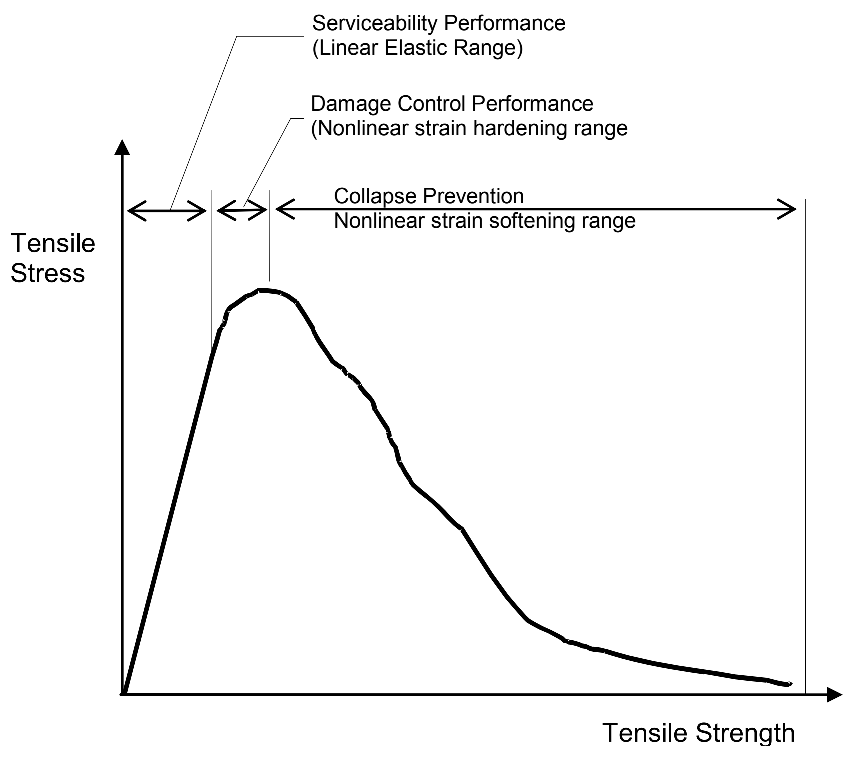


Figure 2-5. Stress – Strain Relationship for Plain Concrete Structures

Performance under MDE loading conditions should be within the non-linear strain hardening range (Damage Control). The strain softening range provides reserve capacity against collapse and represents the concrete capacity to absorb additional energy demands from earthquake ground motions. Additional information on the tensile capacity of plain concrete structures can be found in Chapter 5.

(b) The response of an arch dam to earthquake ground motion is shown in Figure 2-6. Arch dams are generally built as independent cantilever monoliths separated by vertical contraction joints. Since contraction joints can only transfer limited tensile stresses in the horizontal arch direction, the joints can be expected to open and close repeatedly as the dam vibrates in response to severe earthquake ground motion. The contraction joint opening releases tensile arch stresses but increases compressive stresses and vertical cantilever stresses by transferring forces to the cantilevers. The increased compressive stresses could lead to concrete crushing, especially due to impact of joint closing. The increased vertical cantilever stresses could exceed tensile strength of the lift lines (or horizontal joints); in which case tensile cracking is likely to occur along the horizontal lift lines. High tensile stresses also develop along the dam-foundation interface and could cause cracking along the dam-foundation contact or could be absorbed by minor displacements of the jointed foundation rock.

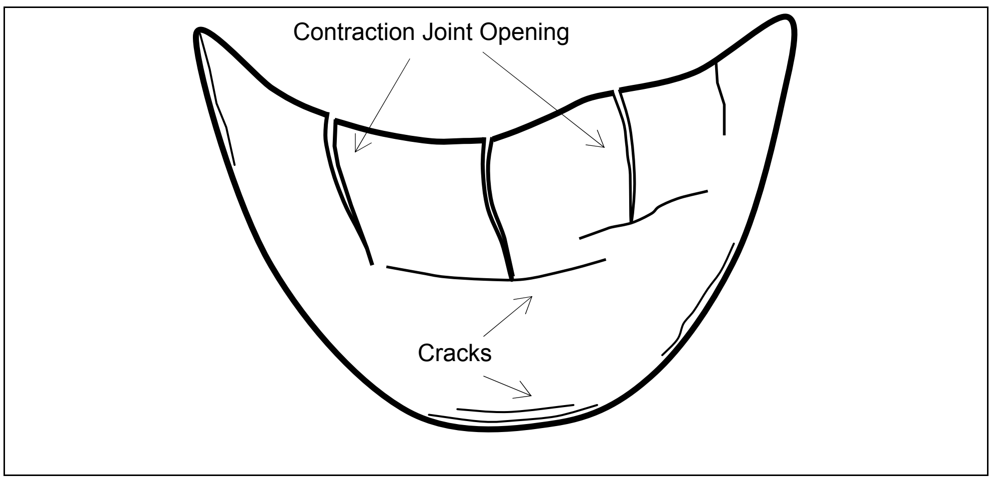


Figure 2-6. Response of Arch Dams to Major Earthquakes

(3) *Response to stability controlled actions.* Once cracking has propagated through the structure along potential failure planes, or along joints where the tensile strength can be substantially less than that of the parent concrete, the structural stability against sliding or rotation should be considered. Rotational failures of a massive concrete hydraulic structure are highly unlikely due to wedging action that limits the rotation. Note that rotational stability assessed based on a 2D analysis ignores additional resistance that might exist due to 3D effects. Even in a straight gravity dam, especially if built in a narrow valley, each monolith can draw resistance from adjacent blocks to remain stable. Evaluation of this mode of failure is discussed in following paragraphs of this Chapter and in Chapter 7. Sliding due to shear failure can occur, leading to unacceptable permanent displacements. The sliding displacements should be evaluated whenever the shear demands along potential failure planes exceed the sliding resistance (shear-friction capacity). An estimate of the permanent displacement can be made using the upper bound sliding displacement methods described in Chapters 4 and 7. Non-linear analysis methods are also available for determining the permanent displacement (Fronteddu et al., 1998; Chavez and Fenves, 1993).

(a) For gravity dams, the sliding may occur along the construction joints, cracked sections within the dam, dam-foundation interface, weak planes within the foundation, or any combination of these. As long as the permanent displacements at construction joint surfaces are within acceptable limits, the sliding response that occurs at joint locations can actually reduce permanent displacements at the dam-foundation interface, and in the case of arch dams can reduce the potential for block shear failure. The sliding response of a gravity dam to earthquake ground motions with joints as strong as the parent concrete may take place at the dam-foundation interface or along weak planes within the foundation. As illustrated in Figure 2-7 (adapted from Fronteddu, Leger, and Tinawi, 1998), the weaker joint condition can cause sliding in the body of the dam thereby reducing the displacement demands at the base of the dam. A rocking response can also have beneficial effects provided such a response does not lead to a rotational stability failure.

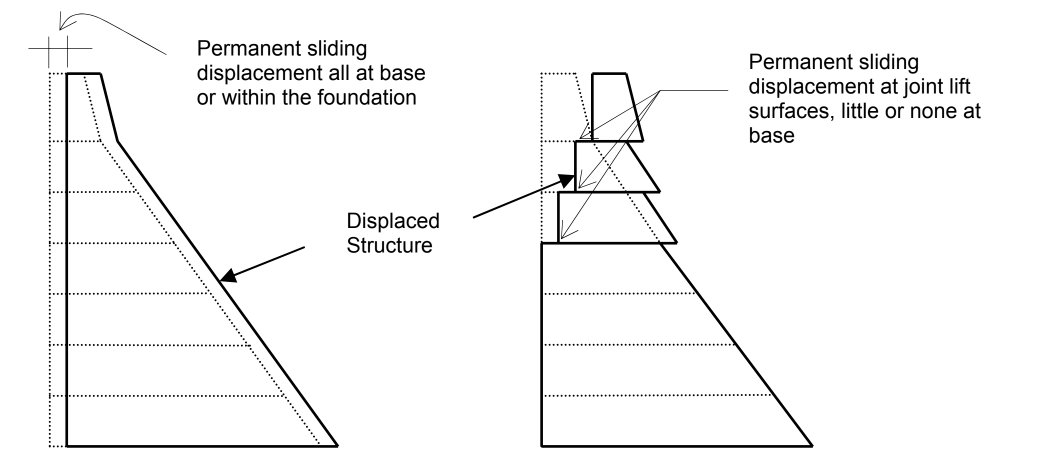


Figure 2-7. Dam Permanent Sliding Displacements

(b) In arch dams, potentially opened contraction joints and cracked lift lines may subdivide the monolithic arch structure into partially free cantilever blocks, capable of transmitting only compressive or frictional forces. In this situation, any failure mode of the arch structure would more likely involve sliding stability of the partially free cantilevers. For small and moderate joint openings, the partially free cantilever blocks, bounded by opened joints, may remain stable through interlocking (wedging) with adjacent blocks. The extent of interlocking depends on the depth and type of shear keys and the amount of joint opening. If potentially dangerous blocks can be shown to be incapable of moving because of friction, tapering, gravity, or orientation consideration, their stability is of no concern. A shear key of rectangular shape would permit only normal opening, but no sliding. Triangular or trapezoidal shear keys allow both opening and some sliding. Hence, the depth of the shear keys controls the maximum amount of joint opening for which adjacent blocks would remain interlocked; deeper shear keys permit larger joint openings. When the partially free cantilevers are treated as rigid blocks, the maximum joint opening with active interlocking can be estimated from rigid block geometry. Therefore, for nonlinear response behavior, the magnitude of compressive stresses, the extent of joint opening or cracking, and the amplitude of non-recoverable movements of concrete blocks bounded by failed joints will control the overall stability of the dam, rather than the magnitude of calculated tensile stresses.

b. Reinforced Concrete Structures

(1) *General.* Under major earthquakes, reinforced concrete structures perform satisfactorily if they are detailed to provide adequate ductility and designed to possess sufficient strength to prevent shear failure. Most existing reinforced concrete hydraulic structures do not conform to modern code detailing and strength requirements, but since they are massive, they may still perform adequately during major earthquakes. Although large diameter steel reinforcing bars are used in construction of these structures, the ratio of the steel area to concrete area is small. They are therefore, classified as lightly reinforced concrete structures, for which the cracking moment capacity is greater than the nominal strength. Code provisions applicable to buildings and bridges may not be directly applicable to lightly reinforced hydraulic structures because of significant differences in reinforcement ratio and axial load ratio. Issues specific to the performance of lightly reinforced concrete structures are presented in Chapter 5. General issues and potential modes of failure that must be examined in seismic response evaluation of reinforced concrete hydraulic structures are discussed below. Failures can occur when:

- Flexural displacement demands exceed flexural displacement capacity
- Shear demands exceed shear (diagonal tension) capacity
- Shear demands exceed sliding shear capacity
- Moment demands exceed overturning capacity (rocking)

Figure 2-8 illustrates types of responses that can lead to one of the above failures.

(2) *Response to internal forces or displacement controlled actions*

(a) *Flexural response.* The flexural response illustrated in Figure 2-8b can lead to a flexural failure if rotation demands in plastic hinge regions (where yielding occurs) exceed the rotational capacity of reinforced concrete. Rotational capacity is a function of curvature capacity and plastic hinge length. In lightly reinforced concrete structures, the curvature capacity is often limited by the ultimate strain capacity of the reinforcing steel. For members with high reinforcement ratios and large axial loads, the curvature capacity will be limited by the compressive strain capacity of the concrete. Low reinforcement ratios limit plastic hinge length and thus flexural rotation capacity. The capacity of bar anchorage lengths and lap splices must be evaluated as part of a flexural response analysis to assure that bond and splice failures, which could limit flexural ductility, do not occur. There is a potential for splice failure under repeated cycles of inelastic rotation where lap splicing occurs in plastic hinge regions, or where lap splices are not suitably confined by transverse reinforcement.

(b) *Shear (diagonal tension).* A shear (diagonal tension) response is illustrated in Figure 2-8c. Since shear failure is a brittle sudden failure, energy dissipation as a result of yielding should take place through a flexural response rather than a shear response. To assure this, it is desirable to provide shear capacity equal to the shear demands the structure would experience if it remained elastic. As a minimum, the shear capacity of the structure should be greater than the shear forces associated with the development of the member flexural capacity, with consideration of possible flexural over-capacity due to strain hardening of the reinforcing steel. The shear capacity of reinforced concrete members includes contributions from the concrete due to aggregate interlock, from the transverse steel reinforcement due to truss action, and from axial load due to arching action. For typical lightly reinforced concrete hydraulic structures, the major contribution to shear capacity comes from the aggregate interlock. The shear capacity diminishes as the flexural ductility demand in the plastic hinge region increases. Shear capacity and its sensitivity to flexural ductility demand are described in Chapter 5.

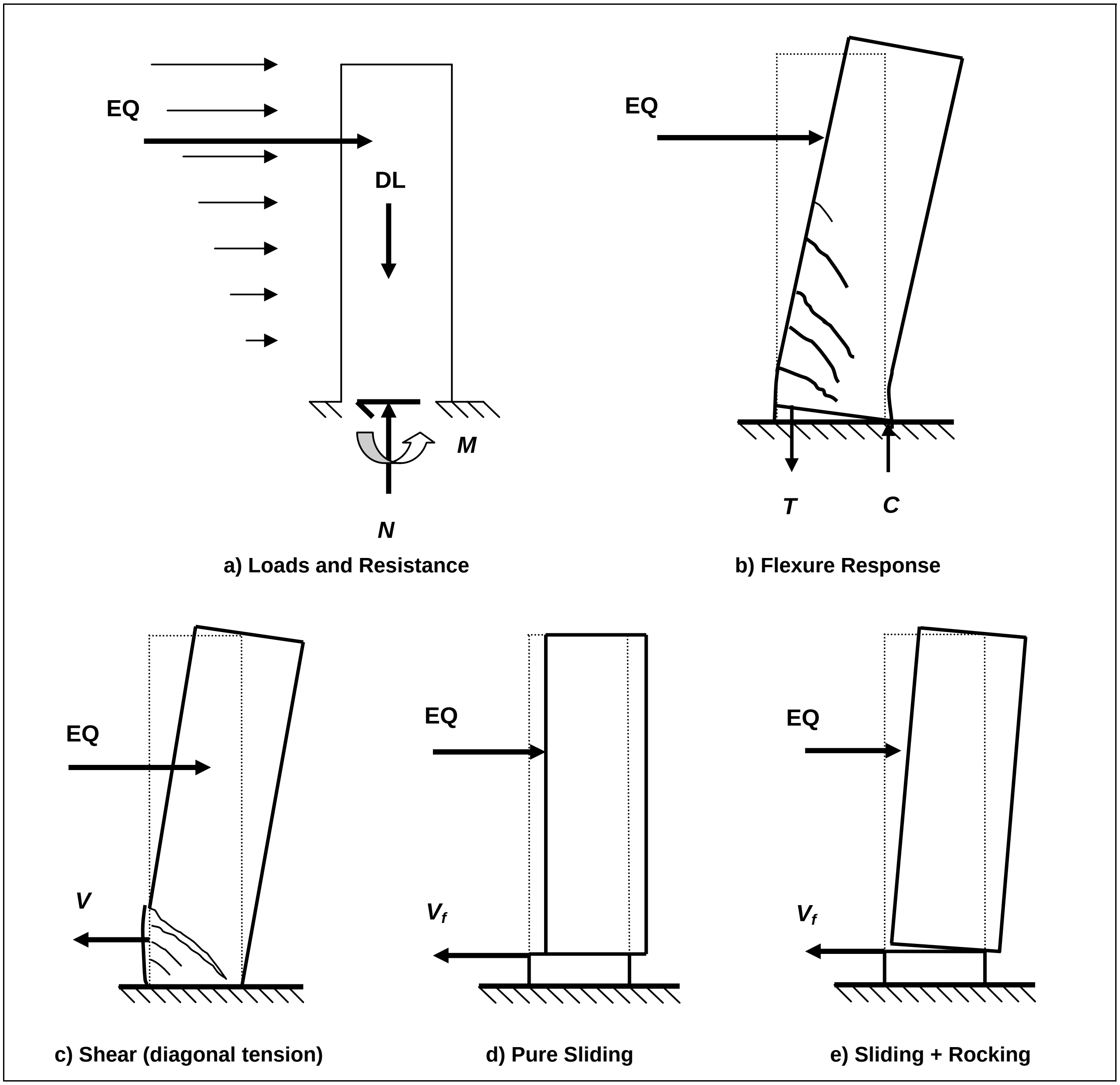


Figure 2-8. Response of a Free Standing Intake Tower to Earthquake Ground Motions

(c) *Sliding shear response.* Figure 2-8d illustrates a sliding shear response. This may occur in the upper part of the tower as well as at its base if the overturning moment is not large enough to cause rocking. When evaluating a sliding shear response within the structure, the capacity of the structure to resist shear will be based on shear-friction concepts ($V_f = N \tan\phi$) with the normal force (N) having contributions from the longitudinal reinforcing steel and axial dead load.

(d) *Sliding plus rocking response.* Figure 2-8e illustrates a sliding plus rocking response. A pure sliding shear may not occur at the structure-foundation interface due to earthquake load distribution that could produce large overturning moment. In this situation a pure rocking or combined rocking plus sliding seems more plausible.

(3) *Response to stability controlled actions*

(a) *General.* Lightly reinforced concrete structures are most vulnerable to failure by fracturing of the flexural reinforcing steel. Once the flexural reinforcing steel fractures, the seismic evaluation becomes one of determining if the residual capacity of the cracked structure with ruptured reinforcing steel is adequate to prevent a failure by sliding instability, or by rotational instability. For sliding, the residual capacity is the shear-friction resistance of the concrete with no consideration given to the shear-friction resistance provided by the reinforcing steel. For rotation, the residual capacity or stabilizing moment is that provided by the moment resisting couple formed between the axial load and the concrete compressive stress zone formed at the extremity of the concrete section (Figure 2-9).

(b) *Sliding stability.* The sliding response will be as illustrated in Figure 2-8d. The capacity to resist sliding will be based on shear-friction principles except that the shear-friction contribution from reinforcing steel crossing the failure plane will be ignored. In cases where the sliding shear demand exceeds the sliding resistance (shear-friction capacity), an estimate of the permanent displacement can be made using the upper bound sliding displacement method described in Chapters 4 and 7. Non-linear analysis methods are also available for determining the permanent displacement that might occur as the result of the fracturing of the flexural reinforcing steel (Fronteddu, Leger, and Tinawi, 1998).

(c) *Rotational stability.* Once a tower has suffered a through crack at its base due to high seismic moments, it could undergo rocking response if the moment demands exceed the restoring or resisting moment of Equation 2-3. For the purpose of rocking response, the tower may be considered a rigid block. Depending on the magnitude and form of the ground motion, the tower may translate with the ground, slide, rock, or slide and rock. Assuming that the angle of friction is so large that sliding will not occur, the tower initially rotates in one direction, and, if it does not overturn, it will then rotate in the opposite direction, and so on until it stops. There are fundamental differences between the oscillatory response of a single-degree-of-freedom (SDOF) oscillator and the rocking response of a slender rigid block (Makris and Kostantinidis, 2001). Rocking structures cannot be replaced by “equivalent” SDOF oscillators. The rocking response of structures should be evaluated by solving equations that govern the rocking motion, as described in Chapter 7. The quantities of interest for a rocking block are its rotation, θ, and its angular velocity $\dot{\theta}$. Similar to the response spectra of SDOF oscillators, rocking response spectra which are plots of the maximum rotation and angular velocity vs. the frequency parameter of geometrically similar blocks can be produced for rocking response. The rocking response spectra can then be used directly to obtain the maximum uplift or rotation of the block for a given ground motion. A comparison of the estimated maximum rotation with the slenderness ratio (i.e. α in Figure 7-5) of the block will indicate whether the block will overturn in accordance with the procedure described in Chapter 7.

(d) *Toe crushing.* In rocking mode the entire weight of the tower is exerted on a small region called the toe of the tower. The resulting compressive stresses in the toe region could be high enough to either crush the concrete or the foundation rock below. In either case this has the effect of reducing the moment lever arm from (h/a) to (h-a)/2, as illustrated in Figure 2-9. Should this happen the stabilizing or resisting moment (M_r) discussed in paragraph 2-5b(3)(c) should be computed as follows:

$$M_r = P\left(\frac{h-a}{2}\right) \quad (2\text{-}3)$$

Where:

P = Axial load.

h = Dimension of the section in the direction of the earthquake load.

$$a = \frac{P}{0.85 f'_{ca}(b)} \quad (2\text{-}4)$$

The response to the forces causing seismic rotational instability is shown in Figure 2-9. In Equation (2-4), f'_{ca} is the best estimate of concrete compressive strength at the base of the structure.

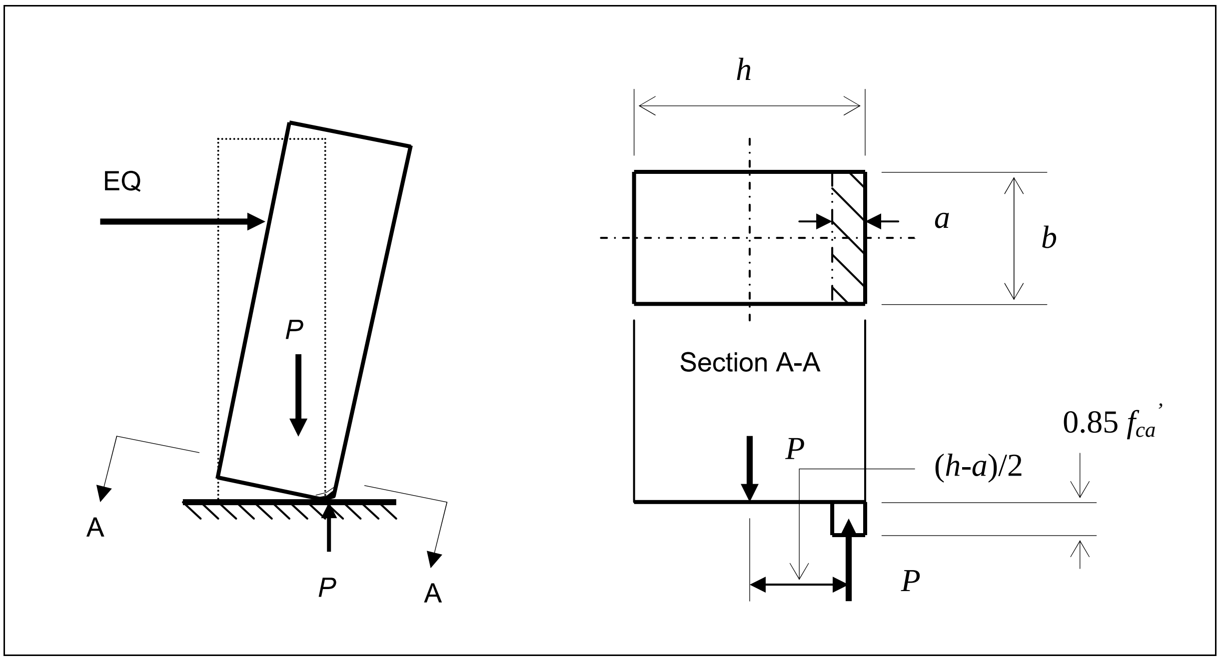


Figure 2-9. Toe Crushing Response of a Free Standing Intake Tower

(4) *Performance evaluation -- DCR allowable values*

(a) Demand to capacity ratios (DCR) are used to evaluate the seismic performance of reinforced concrete structures. Depending on whether the response is a force controlled action (shear) or a displacement controlled action (flexure), demands and capacities will be expressed in terms of forces, displacement ductility ratios, or displacements. Capacities are determined in accordance with procedures described in Chapter 5. The various methods of analysis used to determine demands are covered in Chapter 6. The most common method is the Linear Dynamic Procedure (LDP) in which seismic demands are computed by response-spectrum or time-history analysis methods. Under the Linear Dynamic Procedure, performance goals are met when all DCR ratios are less than or equal to allowable values established in Chapter 6.

(b) In addition to the DCR method, flexural response or displacement-controlled actions can be evaluated using a displacement-based approach where displacement capacities are compared to displacement demands. The moment-curvature diagram in Figure 2-10 illustrates the flexural performance requirements for reinforced concrete structures. Under OBE loading conditions the structure should respond within the serviceable performance range and under MDE within the damage control range. Reserve capacity against collapse is provided in part by reserve energy capacity contained in the strain softening range. Performance under shear and other brittle failure mechanisms is evaluated using DCR procedure in accordance with Paragraph 2-5b(4). The shear capacity needed for this evaluation is obtained as described in Chapter 5.

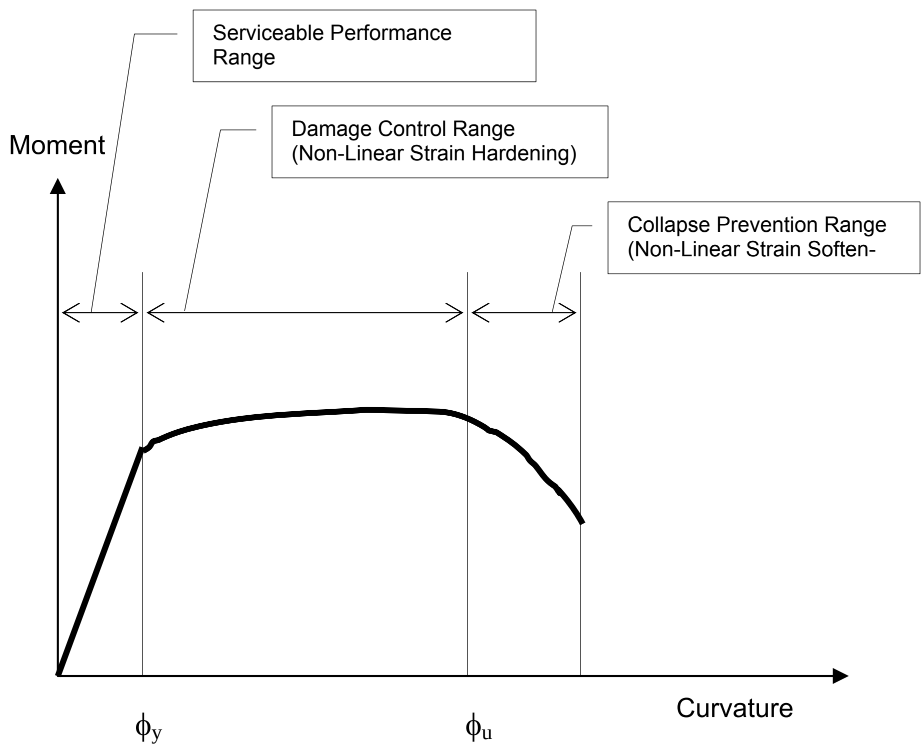


Figure 2-10. Moment-Curvature Diagram for Reinforced Concrete

2-6. Mandatory Requirements

a. Earthquake loading combinations for strength and serviceability of concrete hydraulic structures shall be in accordance with Equations 2-1 and 2-2.

b. Performance-based evaluation of CHS structures shall follow the methodology and goals established in Paragraph 2-5.

Chapter 3
Estimating Earthquake Ground Motion Demands

3-1. Specification of Earthquake Ground Motions

a. General. The earthquake ground motions for design and evaluation of CHS are generally characterized in terms of response spectra and acceleration time histories. Information on response spectra can be found in EM1110-2-6050, “Response Spectra and Seismic Analysis for Concrete Hydraulic Structures." Information on earthquake acceleration time histories and time history analysis can be found in EM 1110-2-6051, “Time History Analysis for Concrete Hydraulic Structures.” ER 1110-2-1806, “Earthquake Design and Evaluation for Civil Works Projects,” provides guidance and direction for the seismic design and evaluation of all civil works projects.

b. Using response spectra for earthquake design and analysis. Acceleration response spectra represent the peak acceleration response of a number of single-degree-of-freedom (SDOF) oscillators to particular earthquake ground motions. Information on response spectra can be found in Technical Report ITL-92-4 (Ebeling 1992) and EM1110-2-6050. Earthquake response spectra can be site specific or standard (non-site specific). Standard response spectra are based on spectral shapes developed for recorded ground motions with similar subsurface characteristics. The standard spectral shapes are defined with respect to effective peak ground accelerations or spectral accelerations taken from the national seismic hazard maps. Although a response spectrum represents the maximum response of SDOF systems, the response of multi-degree of freedom systems (MDOF) can also be obtained from the response spectrum by applying the mode-superposition technique. According to this technique the linear earthquake response of a MDOF system can be obtained by combining responses of several SDOF systems, each of which represents a mode of vibration of the MDOF system. The dynamics of MDOF systems are described in Technical Report ITL-94-4 (French et al. 1994) and EM 1110-2-6050.

c. Standard response spectra. Guidance is provided in Appendix B for constructing standard acceleration response spectra based on the most recent national seismic hazard data. Appendix B provides a procedure for developing standard acceleration response spectra and effective peak ground accelerations for use in the seismic design and evaluation of structural features of USACE projects as required by ER 1110-2-1806. Standard response spectra are based on a general characteristic shape that is defined in terms of effective peak accelerations or spectral accelerations. The effective peak ground acceleration is obtained from division of the spectral ordinate (for a 5%-damped spectrum) at period of 0.2 seconds by a standard value of 2.5. Examples of standard response spectra and determination of effective peak ground accelerations are given in Appendix C. The standard response spectra can be used as a starting point for developing conceptual designs and performing evaluations, determining if the earthquake loading controls the design, and establishing the need for more refined analysis and the impact the earthquake loading might have on construction cots.

d. Site-specific response spectra. Earthquake ground motions are dependent on source characteristics, source-to-site transmission path properties, and site conditions. The source characteristics include stress conditions, source depth, size of rupture area, amount of rupture displacement, rise time, style of faulting, and rupture directivity. The transmission path properties include the crustal structure and shear-wave velocity and damping characteristics of the crustal rock. The site conditions include the rock properties beneath the site to depths up to 2 km, the local soil conditions at the site up to a hundred meters or more in depth, and the topography of the site. All these factors are considered in detail in a site-specific ground motion study, rather than in a general fashion as occur in the standard response spectra methodology. Also, due to regional differences in some of the factors affecting earthquake ground motions, different attenuation relationships exist. There are two basic approaches to developing site-specific response spectra: deterministic and probabilistic. In the deterministic approach, typically one or more earthquakes are specified by magnitude and location with respect to a site. Usually, the earthquake is taken as the Maximum Credible Earthquake (MCE), and assumed to occur on the portion of the source closest to the site. The site ground motions are then estimated deterministically, given the magnitude and source-to-site distance. In the probabilistic approach, site ground motions are estimated for selected values of probability of ground motion exceedance in a design time period or for selected values of the annual frequency or return period of ground motion exceedance. A probabilistic ground motion assessment incorporates the frequency of occurrence of earthquakes of different magnitudes on the various seismic sources, the uncertainty of the earthquake locations on the various sources, and the ground motion attenuation including its uncertainty. Guidance for developing site-specific response spectra and for using both the deterministic approach and the probabilistic approach can be found in EM 1100-2-6050.

e. Acceleration Time Histories. EM 1110-2-6051 describes the procedures for developing site-specific acceleration time-histories of ground motion for dynamic analysis of hydraulic structures. The overall objective is to develop a set (or sets) of time-histories that are representative of site ground motions that may be expected for the design earthquake(s) and that are appropriate for the types of analyses planned for specific structures. The following steps are included in this process:

(1) Initially selecting recorded time-histories that are reasonably consistent with the tectonic environment of the site; design earthquake (magnitude, source-to-site distance, type of faulting); local site conditions; and design ground motion characteristics (response spectral content, duration of strong shaking, and special characteristics, e.g. near-source characteristics). If sufficient recorded motions are not available, simulated recorded time-histories can be developed using ground motion modeling methods.

(2) Modifying time-histories selected in (1) above to develop the final set(s) to be used in dynamic analysis. Two approaches that can be used in this process are simple scaling of time-histories (by constant factors) so that a set of time-histories has spectral values that, on average, are at the approximate level of the design response spectrum; and spectrum matching, which involves modifying the frequency content of a given time-history so that its response spectrum is a close match to the design response spectrum.

(3) Further modifying the time-histories for site response effects, if the site is a soil site and the time-histories have been developed for outcropping rock conditions.

(4) Further modifying the time-histories for spatial variations of ground motion, if it is desired to incorporate effects of wave passage and incoherence in the ground motions that would arrive beneath a very large or long structure.

f. Selection of records for deterministically defined and probabilistically defined earthquakes. Application of the above guidelines is straightforward when design earthquakes are expressed deterministically, i.e., in terms of magnitude, faulting type, and source-to-site distance. However, the application of the guidelines is less straightforward when the design earthquake ground motions (typically the response spectrum) are derived from a probabilistic ground motion analysis (often termed a probabilistic seismic hazard analysis or PSHA). From this type of analysis, which is described in detail in EM 1110-2-6050, the design response spectrum for a certain selected probability of exceedance in a design time period (or, equivalently, for a design return period) reflects the contribution of different earthquake magnitudes and distances to the probabilities of exceedance. Therefore, when the design response spectrum is probabilistically based, the PSHA should be deaggregated to define the relative contributions of different magnitudes and distances to the ground motion hazard. Furthermore, the de-aggregation should be done for probability values or return periods that correspond to those of the design earthquake and for response spectral periods of vibration of significance for seismic structural response because the relative contributions of different magnitudes and distances may vary significantly with return period and period of vibration. The dominant magnitude and distance is then considered as representative in selecting time histories and defining strong motion duration.

3-2. Multi-Directional Effects

a. General. Two-dimensional structures are generally analyzed for one or two components of the earthquake ground motion (horizontal only, or horizontal + vertical). The ground motions may be defined as multi-component response spectra or acceleration time histories. Some three-dimensional structures such as navigation locks and straight gravity dams can be idealized as two-dimensional structures. While others such as arch dams must be evaluated using three-dimensional models requiring three components of ground motion (two horizontal + vertical). In certain cases, such as freestanding intake/outlet towers, the vertical component of earthquake ground motion may be ignored if it contributes very little to the total response. Structures must be capable of resisting maximum earthquake ground motions occurring in any direction. In response spectrum analysis, the secondary component of horizontal ground motion is usually set equal to the primary component. This is somewhat conservative but eliminates the need to determine the ground motion direction of attack that produces the greatest demand to capacity response (DCR). However, in time history analysis it may be necessary to apply horizontal components in either horizontal direction in order to obtain the largest response in accordance with Paragraph *e* below. The orthogonal components of earthquake ground motion are commonly applied along the principal axes of the structure. In time history analysis the total response due to all components of the ground motion are obtained by algebraic summation of responses due to each individual component. In response-spectrum analysis the total response due to multiple earthquake components are estimated as described in the following paragraphs.

b. Percentage combination method. In a response-spectrum analysis, the percentage combination method can be used to account approximately for the simultaneous occurrence of earthquake forces in two perpendicular horizontal directions. For rectangular structures with clearly defined principal directions, this method yields approximately the same results as the SRSS method described in Paragraph 3-2*c* below. However, for non-rectangular and complex three-dimensional structures the percentage method can under estimate structural responses. The percentage combination is accomplished by considering two separate load cases for both the OBE and MDE. Illustrating for the MDE loading combination, Equation 2-1, the two load cases would be as follows:

Load Case 1:

$$Q_{DC} = Q_D + Q_L \pm Q_{MDE(X1)} \pm \alpha Q_{MDE(X2)} \qquad (3\text{-}1)$$

Load Case 2:

$$Q_{DC} = Q_D + Q_L \pm \alpha Q_{MDE(X1)} \pm Q_{MDE(X2)} \qquad (3\text{-}2)$$

Generally α is assumed to be 0.30 for rectangular structures, and 0.40 for circular structures.

Q_{DC} = peak value of any response quantity (forces, shears, moments, stresses, or displacements) due to the effects of dead load, live load, and the MDE

$Q_{MDE(X1)}$ = effects resulting from the X_1 component of the MDE ground motion occurring in the direction of the 1st principal axis of the structure.

$Q_{MDE(X2)}$ = effects resulting from the X_2 component of the MDE ground motion occurring in the direction of the 2nd principal axis of the structure.

c. Square Root of the Sum of the Squares Method (SRSS). A better way to combine structural responses for the multi-component earthquake response spectra is the use of the SRSS method. This method is applicable to rectangular and complex three-dimensional structures. For any response quantity of interest, e.g. moment or shear at a particular location, the results from the separate application of each component of ground motion are combined by the square root of the sum of the squares to obtain the total response. Note that since response-spectrum stresses, forces, or moments have no sign, the combination should consider response-spectrum quantities to be either positive or negative. Illustrating for the MDE loading combination, Equation 2-1, the SRSS demand would be as follows:

$$Q_{DC} = Q_D + Q_L \pm \left[(Q_{MDE(X1)})^2 + (Q_{MDE(X2)})^2 + (Q_{MDE(Z)})^2 \right]^{1/2} \qquad (3\text{-}3)$$

d. Critical Direction of Ground Motion. The directions of ground motion incidence are usually assumed along the fixed structural reference axes. Considering that the ground motion can act along any horizontal direction, there could be a different direction of seismic incidence that would lead to an increase of structural dynamic response. The maximum structural response associated to the most critical directions of ground motions has been examined in several publications (Wilson and Button 1982, Smedy and Der Kiureghian 1985, Wilson et al. 1995, Lopes and Torres 1997, and Lopez et al. 2000). These investigations indicate that the critical direction of the horizontal ground motion components yielding the maximum structural response depends on the two horizontal spectra and also on the structural response parameters but not on the vertical spectrum. For the special case of identical spectra along the two horizontal directions, the structural response does not vary with the angle of incidence, i.e. any direction is a critical direction. The response value for this special case is the upper bound to all possible structural responses due to any combination of spectral ratios and angle of incidence. A conservative approach is therefore to analyze the structure with the same horizontal spectrum applied simultaneously along the two horizontal structural reference directions, and the vertical spectrum.

e. Load combination cases for time history analysis. 3D time-history analysis of CHS with response in the damage control range should be evaluated for three or more sets of three-component earthquake ground motions plus the effects of usual static loads. For each set of three-component earthquake ground motions the static loads and earthquake ground motion components should be combined in accordance with Table 3-1. In general, a complete permutation of all three components with positive and negative signs may be required to obtain the most critical directions that would cause the largest structural response (EM 1110-2-6051). 2D time-history analysis of CHS with response in the damage control range is conducted in a similar fashion requiring three or more sets of two-component acceleration time-history records, except that for two-component excitation a total of 4 permutations will be required.

Table 3-1
Load Combination Cases for Combining Static and Dynamic Stresses for Three-component Excitation

	Seismic Loads			
Case	**Horizontal (H1)**	**Vertical (V)**	**Horizontal (H2)**	**Static Loads**
1[1]	+	+	+	+
2	+	+	-	+
3	+	-	+	+
4	+	-	-	+
5	-	+	+	+
6	-	+	-	+
7	-	-	+	+
8	-	-	-	+

Note: The (+) and (-) signs indicate the loads are multiplied by +1 (zero phase) or -1 (180 phase) to account for the most unfavorable earthquake direction.
[1] Case-1: Static + H1 + V + H2

3-3. Earthquake Demands on Inelastic Systems

a. General. The inelastic response of a structure to earthquake ground motion is different than the elastic response. The difference occurs because the vibration characteristics of the structure change as the structure yields. The predominant change is a shift in the fundamental period of vibration. In most cases, a reduction in earthquake demand occurs as the period of the structure lengthens. In Figure 3-1, a capacity spectrum is used to illustrate the inertia force reduction (or spectral acceleration reduction) that occurs when a structure yields. In Figure 3-1, earthquake demands for an elastic system, as represented by a response spectrum, are reconciled with the elasto-plastic load/displacement characteristics for a ductile structure. Point "*A*" represents the earthquake demand assuming the structure remains elastic with the line "*O-A*" representing the linear elastic response. Point "*B*" represents the earthquake demands for elasto-plastic behavior with the line "*O-B-C*" representing the load/displacement characteristics of the elasto-plastic system. As can be seen, the earthquake force demand on the elasto-plastic system is substantially less than that of the elastic system. It should also be recognized that as a structure yields, damping increases significantly, thus leading to further reduction of earthquake demands. However, if the fundamental period of the structure falls in the ascending portion of the response spectrum, a shift in period may actually increase earthquake demands. This condition is likely to occur for stiff structures founded on soft soils, but can also occur for stiff structures founded on rock. In such cases, the structure should be designed to remain elastic.

b. Inelastic displacement demands. Knowing displacements in yielding structures, the level of damage or yielding can be controlled by limiting displacements to predetermined values for a specified level of earthquake shaking. Simple techniques have been developed for estimating inelastic displacements from elastic displacements. Structures with a period of vibration between zero and 0.75 T_0, where T_0 is characteristic ground motion period (period corresponding to the peak acceleration response), exhibit an equal acceleration response which offers no benefit from yielding. Structures with periods of vibration greater than 0.75 T_0, however, will benefit from structure displacement ductility. The inelastic response of structures with fundamental periods of vibrations between 0.75 T_0 and 1.5 T_0, can be estimated using equal energy principles. The inelastic response of structures with fundamental periods of vibration greater than 1.5 T_0, can be estimated using equal displacement principles.

(1) Equal acceleration response. Rigid structures, with a period of vibration (T) equal or less than 0.04 sec (or between 0 and 0.75 T_0), will exhibit an equal acceleration response. In this case, force or acceleration is conserved regardless of any ductile properties attributed to the structure. Structures exhibiting an equal force or acceleration response should therefore be designed to remain elastic.

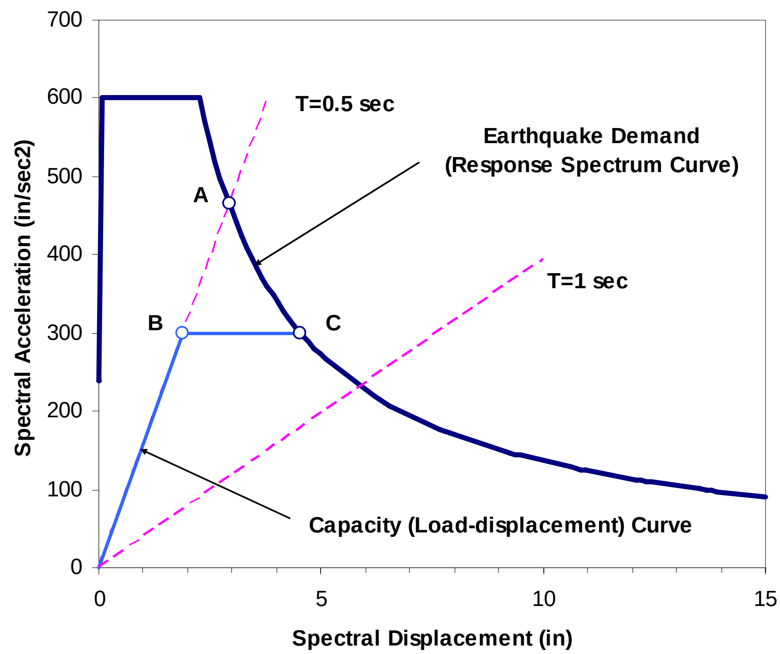


Figure 3-1. Earthquake Demands on Inelastic Structures

(2) Equal energy response. Structures with fundamental periods of vibration between 0.75 T_0 and 1.5 T_0 will exhibit an equal energy response. The characteristic ground motion period (T_0) generally varies between 0.2 seconds and 0.7 seconds depending on site conditions, with firm sites having shorter characteristic periods than soft sites. The structure must have sufficient displacement ductility to provide the reserve inelastic energy capacity needed to resist earthquake ground motion demands. The equal energy response concept is presented in Figure 3-2. For a given displacement ductility (μ_δ), the inelastic (yield) capacity (F_Y) must be sufficient to produce an equal energy response. Equating the energy for a linear elastic response to that for an inelastic response (hatched area under the nonlinear portion of the load displacement curve equal to the hatched area under the linear elastic curve), it can be determined that the yield capacity of the structure must be equal to or greater than the capacity required of the structure if it were to remain elastic (F_E) divided by $\sqrt{2\mu_\delta - 1}$, or:

$$F_Y \geq \frac{F_E}{\sqrt{2\mu_\delta - 1}} \qquad (3\text{-}4)$$

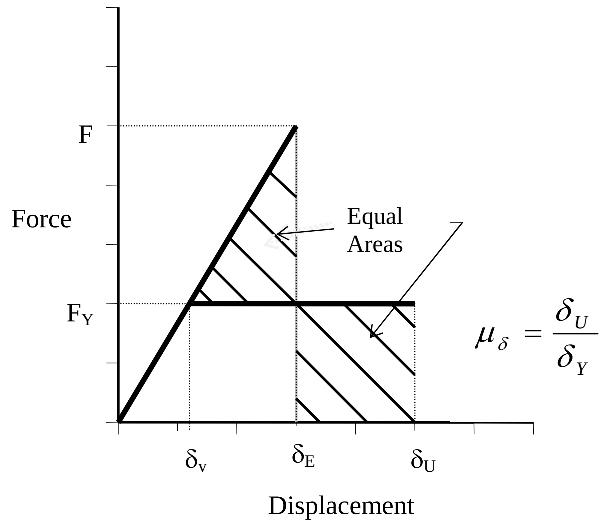


Figure 3-2. Equal Energy Response

(3) Equal displacement response. Structures with fundamental periods of vibration greater than 1.5 T_0 will exhibit an equal displacement response. An equal displacement response means that to perform as intended, the displacement ductility capacity must be sufficient to provide a structure displacement capacity equal to, or greater than, the peak displacement the structure will experience during the design earthquake. The equal displacement response concept is presented in Figure 3-3. From Figure 3-3 it can be determined that the yield capacity of the structure must be equal to or greater than the capacity required of the structure if it were to remain elastic (F_E) divided by μ_δ, or:

$$F_Y \geq \frac{F_E}{\mu_\delta} \qquad \text{(3-5)}$$

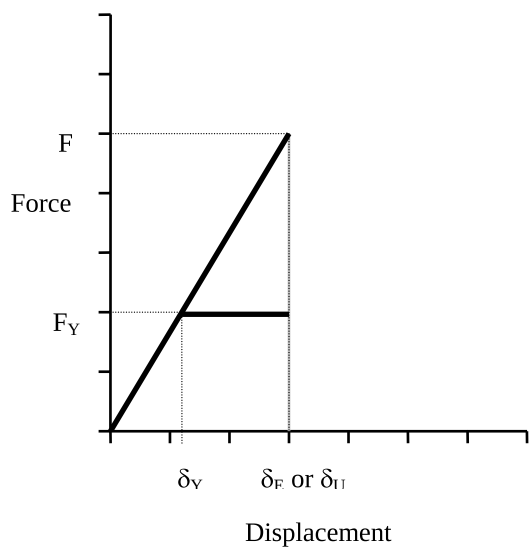


Figure 3-3. Equal Displacement Response

(4) General relationship between required yield strength (F_Y) and elastic demand (F_E). A general relationship has been developed (Paulay and Priestley, 1992) for relating required yield strength to elastic demand. This relationship provides a smooth transition from an equal acceleration response ($\frac{F_E}{F_Y} = 1$ regardless of μ_δ) at T = 0, through the equal energy approximation ($\frac{F_E}{F_Y} = \sqrt{2\mu_\delta - 1}$) at about T = 0.75 T_0, to the equal displacement approximation ($\frac{F_E}{F_Y} = \mu_\delta$) for $T \geq 1.5\ T_0$. The relationship for the smooth transition is:

$$\frac{F_E}{F_Y} = 1 + \frac{(\mu_\delta - 1)T}{1.5T_0} \geq \mu_\delta \qquad (3\text{-}6)$$

Using Equation 3-6; for a known level of displacement ductility (μ_δ) and a given elastic earthquake demand (F_E), the required yield capacity of a structural component (F_Y) can be determined.

3-4. Mandatory Requirements

a. Standard spectra. Standard spectra used in the preliminary design and evaluation of Corps hydraulic structures shall be developed in accordance with the procedures described in Appendix B.

b. Site-specific spectra. Site-specific spectra used in the preliminary design and evaluation of Corps hydraulic structures shall be developed in accordance with the procedures described in EM 1110-2-6050.

c. Acceleration time histories. Acceleration time histories for dynamic analysis of CHS shall be selected and developed in accordance with EM 1110-2-6051.

d. Multi-directional effects. Multi-directional effects shall be considered when designing or evaluating concrete hydraulic structures for earthquake ground motions. General information on multi-directional effects can be found in Paragraph 3-2.

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:

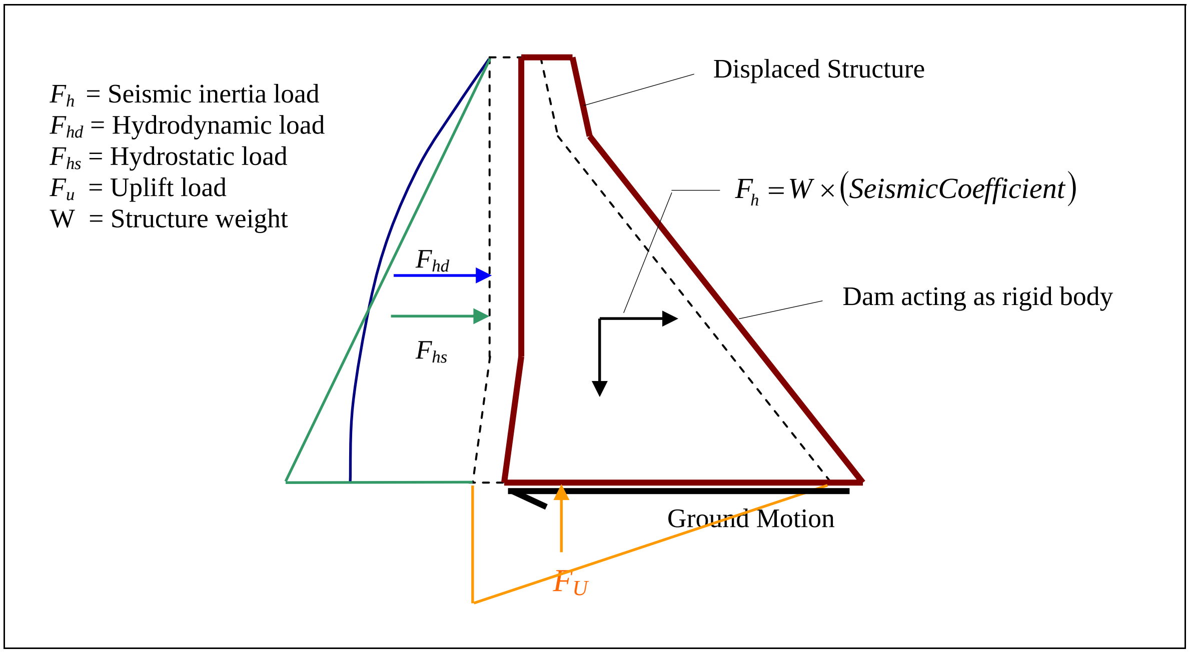


Figure 4-1. Gravity Dam Sliding on Foundation

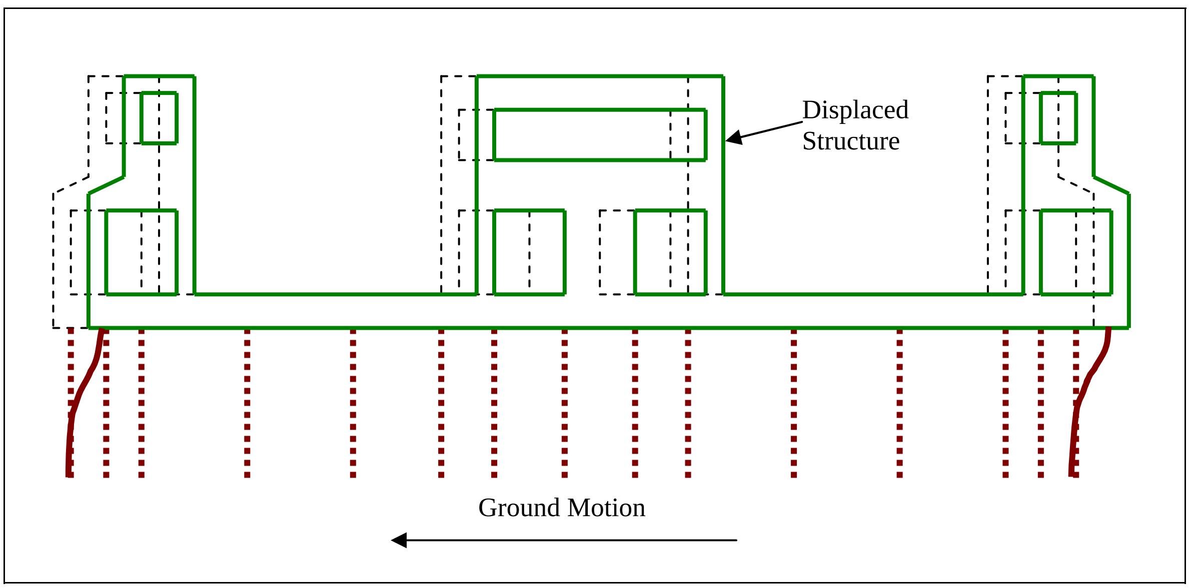


Figure 4-2. Navigation Lock on Flexible Pile Foundation

(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.

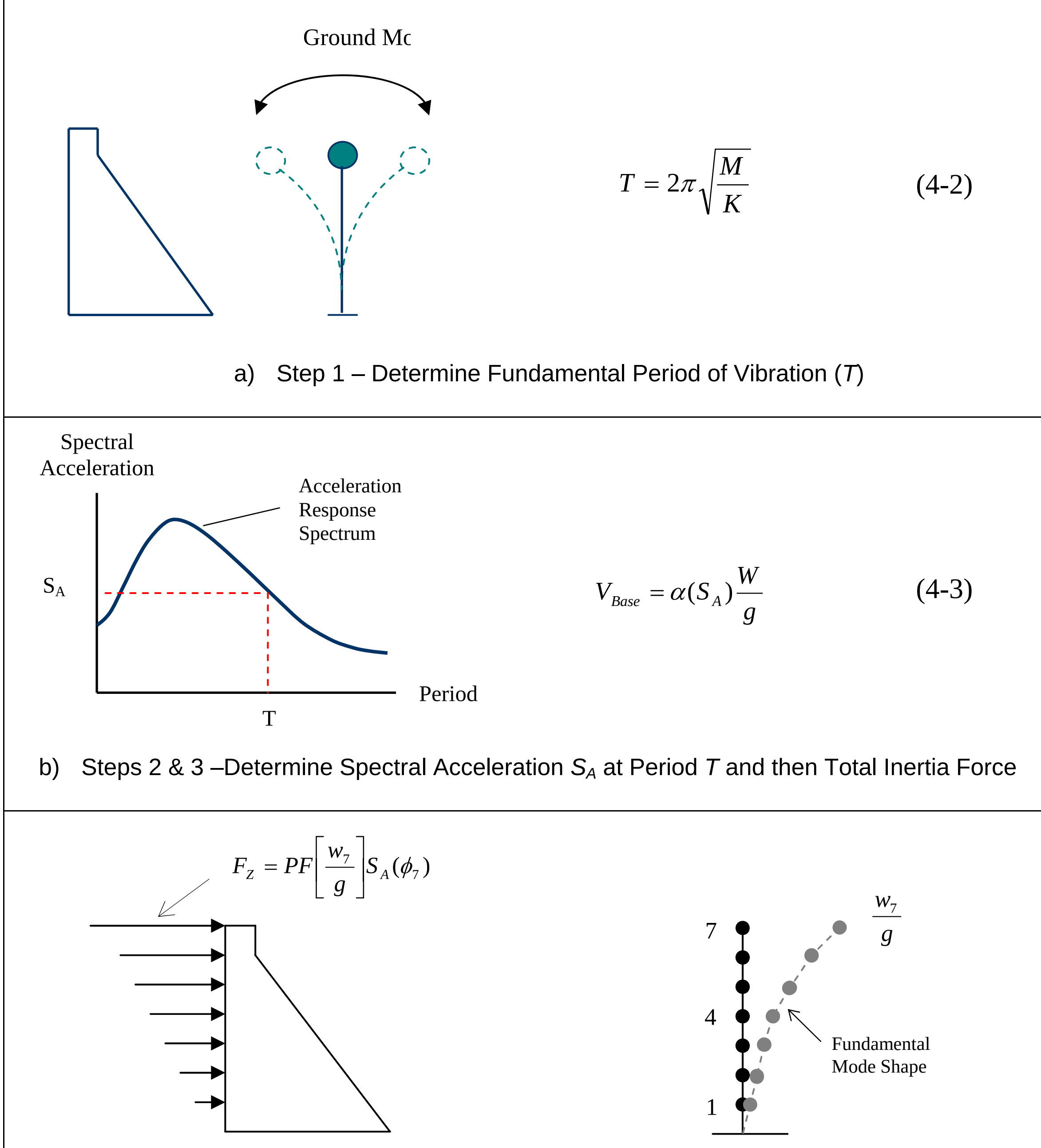
(3) Once the spectral acceleration (S_A) has been determined, the total inertial force on the structure due to the design ground motions (represented by the design response spectrum shown in Figure 4-3b) can be estimated using Equation (4-3) in Figure 4-3b. The spectral acceleration and the total mass 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:

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

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.



$$T = 2\pi\sqrt{\frac{M}{K}} \qquad (4\text{-}2)$$

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

$$V_{Base} = \alpha(S_A)\frac{W}{g} \qquad (4\text{-}3)$$

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).

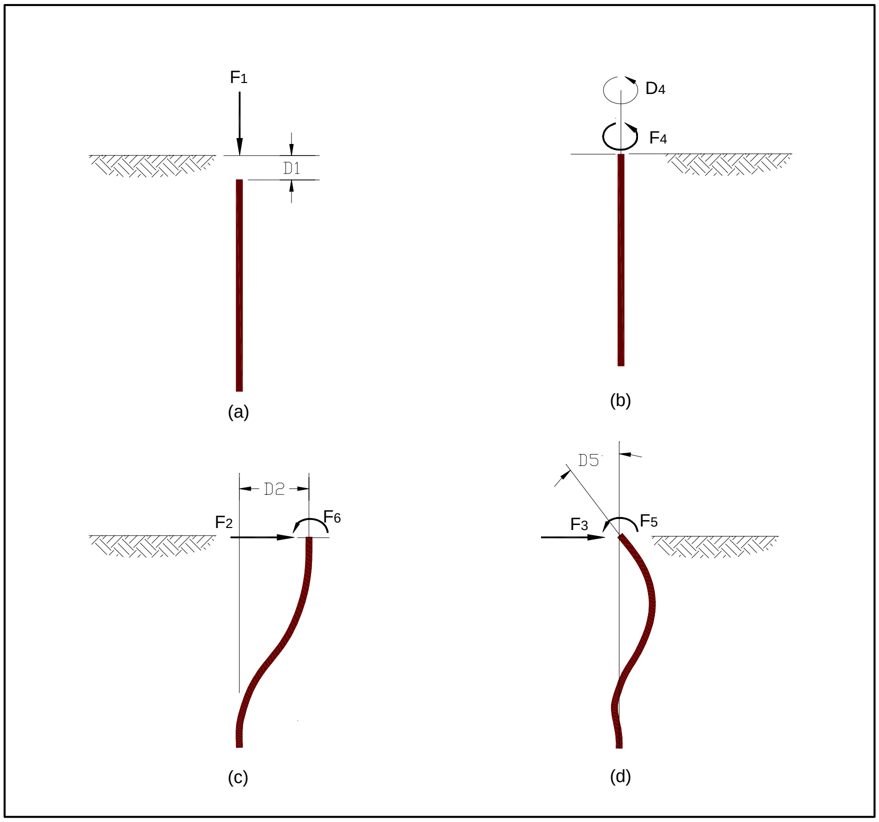


Figure 4-4. Pile Behaviors

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

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

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)

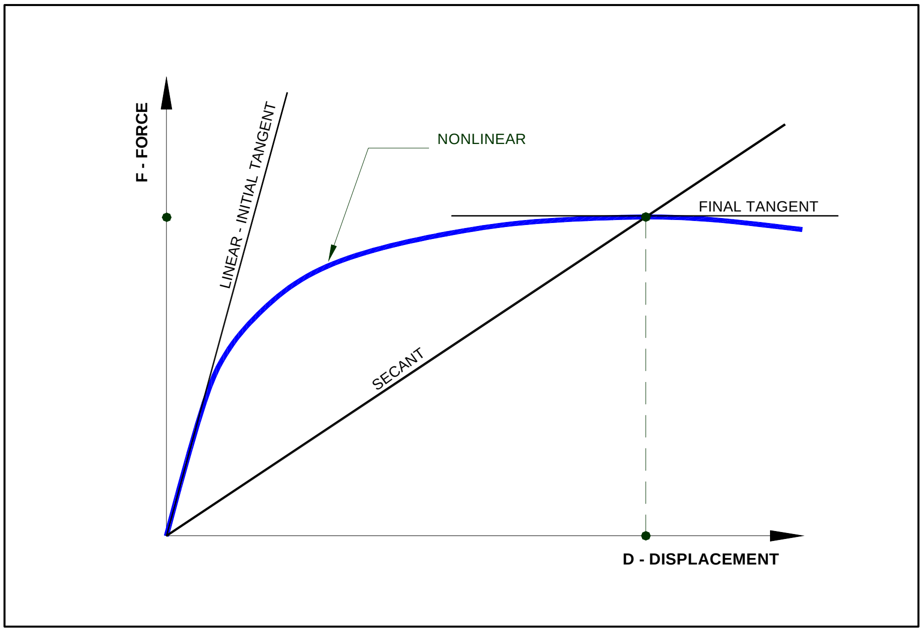


Figure 4-6. Secant stiffness value selected at design level displacement from nonlinear soil-pile force-displacement curve (after Kriger and Wright 1980)

(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.

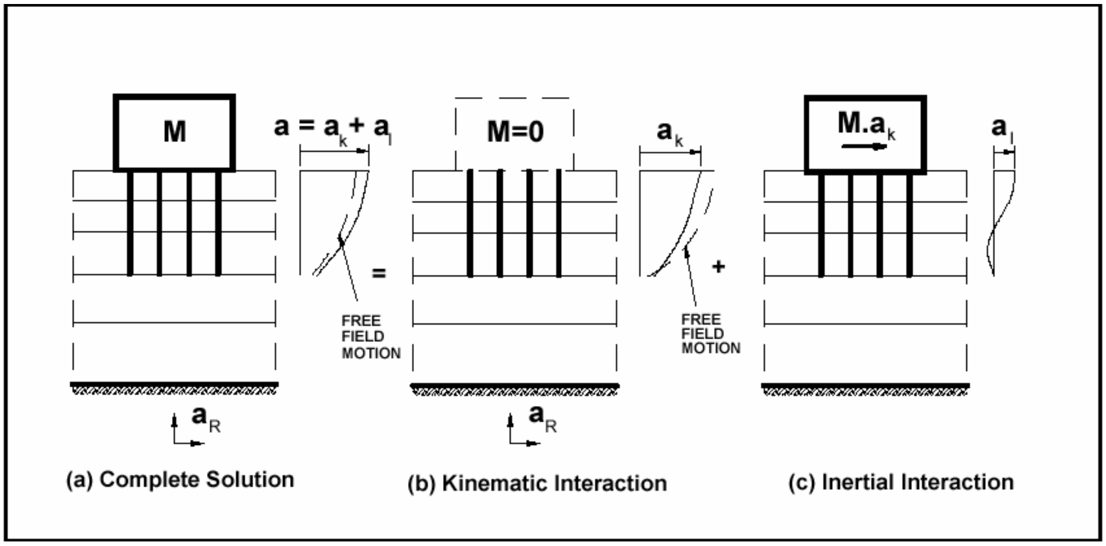


Figure 4-7. Substructuring concept: Decomposition of the problem into kinematic and inertia interaction problems.

(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.

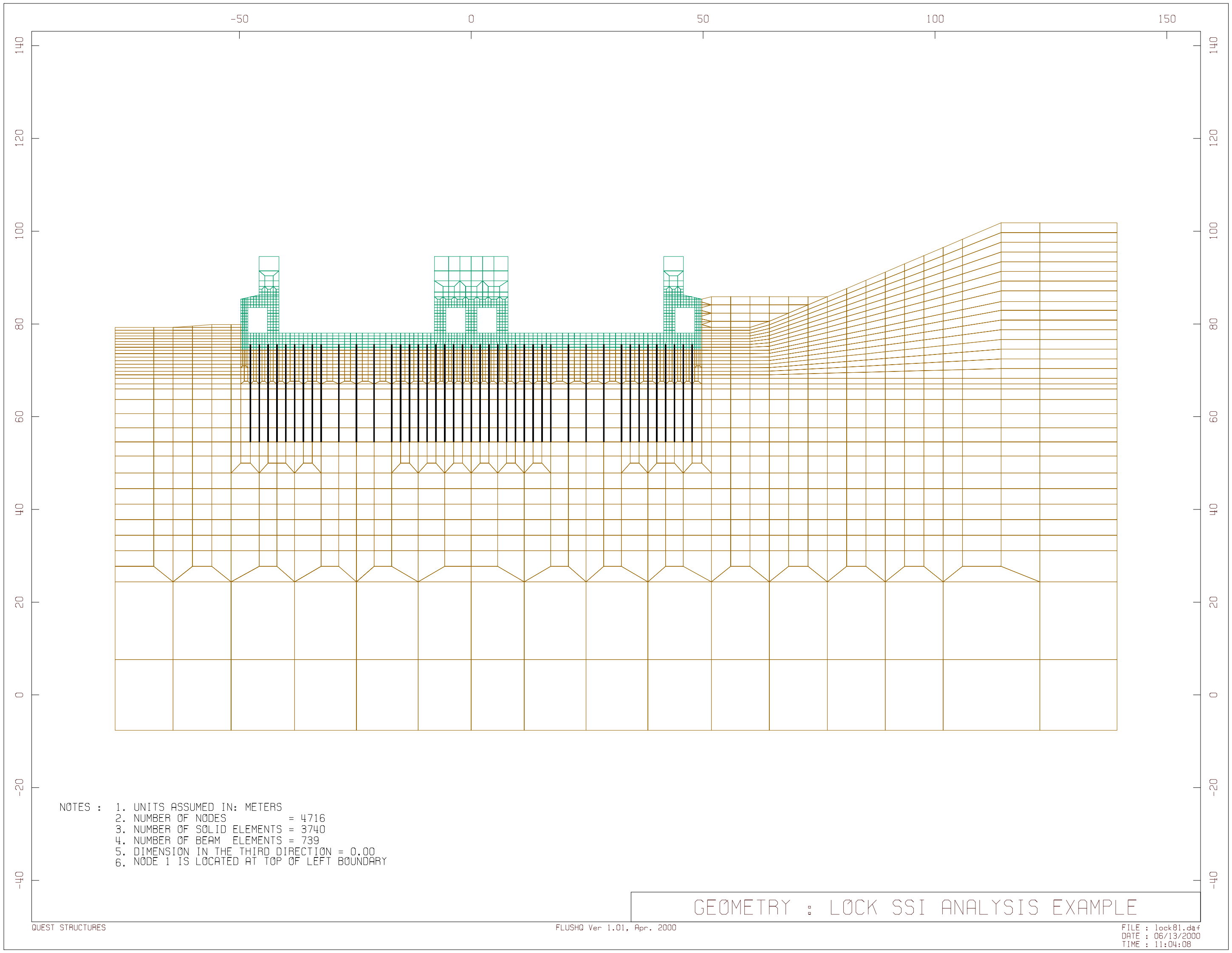


Figure 4-8. Q-FLUSH Two-dimensional Soil-Pile-Structure-Interaction Model of Olmsted Lock Chamber Monolith

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).

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

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:

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

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

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).

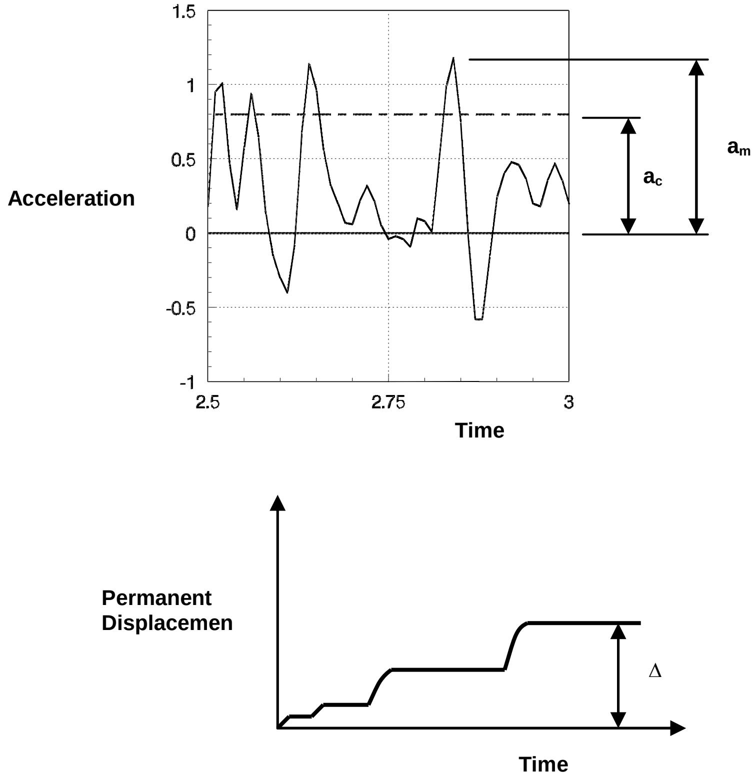


Figure 4-9. Permanent Sliding Displacement

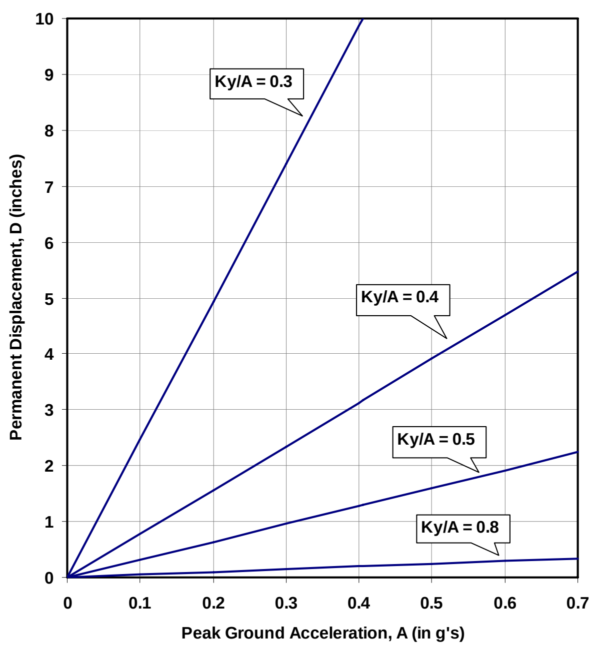


Figure 4-10. Permanent Displacement as a Function of k_y and A

4-8. Mandatory Requirements

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

Chapter 5
Concrete Properties and Capacities

5-1. Plain Concrete Structures

a. General. The concrete properties important in the seismic design and evaluation of concrete dams are the unit weight, compressive, tensile, and shear strengths, modulus of elasticity, and Poisson's ratio. Properties of mass concrete at high rate of loading are higher than those under static loading conditions. Therefore, concrete properties used in the seismic analysis should reflect the effects of high deformation rates and cyclic loading response that the dam would experience under earthquake shaking. In general, the performance of a dam under earthquake loading is controlled by the tensile strength of the concrete, and by tensile crack propagation. However, the actual tensile strength used in performance evaluation of the dam should be determined by taking into account the effects of lift joints. The actual tensile strength across the poorly constructed lift joints of some older dams could be markedly lower than that for the homogeneous concrete. Thus it is important that such weaknesses in the mass concrete are accounted for in the seismic safety evaluation, and that the actual reduced strength at lift joints is determined by material testing. The properties of concrete for the final design and evaluation should also be determined by testing.

b. Testing. A comprehensive laboratory testing program is required to obtain the design mixture proportions for concrete strength and workability, to obtain the material properties important to structural analysis and thermal studies, and to validate in-place concrete strengths of both the parent concrete and lift joints. A measure of tensile strength of the concrete can be obtained from direct tension, modulus of rupture, or splitting tension tests. The direct tension tests of concrete are seldom carried out due to difficulties associated with the specimen holding devices. The modulus of rupture test is not favored for existing structures because of its beam specimen requirement. The most commonly used test for estimating the tensile strength of the concrete is the ASTM 496 splitting tension test, which uses a cylindrical specimen. Relationships between the tensile strength obtained from splitting tensile tests and direct tensile strength, for both conventional concrete and RCC, are given in EM 1110-2-2200.

c. Concrete Coring and Specimen Parameters. A concrete coring program to obtain test specimens should start with a random coring or non-destructive tests to establish the overall quality and uniformity of concrete, and to locate problem areas in existing structures. Once potential areas have been discovered, coring can concentrate in these areas to better define properties. While average values of strength and elastic modulus of the concrete are of some value for structural analysis, coring and testing should focus on "weak links" since these problem areas are more likely to govern performance of the structure, than the average properties. Another important factor in establishing the concrete properties is that sufficient number of specimens are taken and tested so that the uncertainty in the estimated parameter values are reduced to an acceptable level. The number of tests needed to establish the concrete properties depends on statistical considerations and cost. While for few tests (say less than 5), adding an additional test results in significant reduction in the uncertainty, for many tests, the reduction obtained by using an additional test is small. So the engineer must answer the question: "Is the additional precision obtained by using another test worth the additional expense?" As a general guideline the minimum number of tests for a specific parameter is about six, while more than nine tests would probably not be economical.

d. Dynamic Properties. Strength and elastic properties are strain rate sensitive. In the absence of test data, the following relationships between static and dynamic properties may be assumed (Bruhwiler, 1990):

(1) The dynamic modulus is equal to 1.15 times the static modulus.

(2) The dynamic Poisson's Ratio is equal to 0.70 times the static ratio.

(3) The dynamic compressive strength is equal to 1.15 times the static compressive strength.

(4) The dynamic tensile strength is equal to 1.50 times the static tensile strength.

(5) The dynamic shear strength is equal to 1.10 times the static shear strength.

e. Capacity (strength). The ultimate tensile strength of the parent concrete (concrete without joints) obtained from static load testing must be adjusted to account for lower strength at construction joints and strain rate effects. It is not reasonable to expect the bonding at construction joints to be equal to that of the parent concrete. The tensile strength of conventional mass concrete joints cleaned by high pressure water jet is approximately 70-percent of the tensile strength of the parent concrete (WES, 1973). This relationship is also applicable to roller compacted concrete (RCC) when lift joints are properly cleaned and covered with a mortar bedding (EP 1110-2-12). However, test results at some exiting dams show that tensile strengths of deteriorated or poorly construction joints could be more than 50% lower than that of the parent concrete. Raphael (Raphael, 1984) discusses the effects of dynamic loading (high strain rates) on the tensile strength of concrete, and the effect of nonlinear strain at failure on the results of linear elastic finite element analysis. According to Raphael, the static tensile strength of the concrete should be increased by a factor of 1.50 to obtain the dynamic tensile strength. As discussed in Chapter 6, a DCR allowable value equal to this ratio will be used for evaluation of the linear-elastic response-spectrum analysis. However, for the linear-elastic time-history analysis the ratio of the apparent dynamic tensile strength to the static tensile strength in conjunction with other parameters will be used for evaluation of the performance. Beyond yield levels apparent tensile stresses are higher than the actual tensile stresses. According to Raphael, the apparent dynamic tensile strength of the concrete is twice the static tensile strength.

5-2. Reinforced Concrete Structures

a. General. Earthquake related catastrophic failures have occurred in major civil works structures, reinforced concrete building and bridge structures. As a result structural codes have been revised dramatically in the past 20-years. Many of the earthquake related deficiencies in buildings and bridges designed by older codes also exist in most USACE structures. These deficiencies however should be examined with respect to the unique characteristics of major civil works structures. The major differences between major civil works structures and buildings / bridge type structures are that:

(1) Major civil works structures are lightly reinforced with reinforcement percentages generally less than 0.5-percent

(2) Major civil works structures have low axial load ratios

(3) Major civil works structures because of large cross-sectional dimensions have large shear capacities

(4) In major civil works structures the concrete protection (cover) and reinforcing bar spacing exceeds that found in bridge and building type structures

(5) Major civil works structures are generally of massive wall-slab construction rather than beam-column construction

b. Compressive strains in CHS. In most major civil works structures the compressive strains in the concrete are low and earthquake demands are usually not sufficient to cause a shear failure. Bond deterioration under cyclic loading only occurs if the maximum compressive strain at the location of reinforcing bar splices reaches levels where longitudinal micro-cracking develops. When compressive strains are below 0.2-percent (0.002) the chance for micro-cracking and bond deterioration that could lead to reinforcing steel splice failure is low. When compressive strains are below 0.4-percent the chance for concrete spalling is low. This means that in most civil works structures spalling would not occur, and that the disastrous consequences of spalling, such as the loss of concrete cover, the loss of confinement reinforcement, and the buckling of reinforcing steel would be unlikely.

c. Potential modes of failure. Performance requirements for reinforced concrete structures are met if all brittle modes of failure (all failure modes other than flexure) are suppressed. Brittle modes of failure include shear (diagonal tension), sliding shear (shear-friction) and fracture of flexural reinforcing steel. Inelastic flexural response will limit shear demands. Therefore, it is only necessary to provide shear strength equal to or greater than the shear demand associated with the maximum flexural strength. Fracturing of reinforcing steel is unique to lightly reinforced concrete members and will occur when strains in the reinforcing steel exceed 5-percent. This mode of failure can be prevented by limiting the displacement ductility capacity of members to that which will produce reinforcing steel strains less than 5-percent. Reinforcing steel used to resist flexural demands must also have splice and anchorage lengths sufficient to develop the maximum bar strength including strain hardening effects. The capacity of reinforced concrete members can be determined using the procedures described below. The capacity of members available to resist brittle modes of failure is discussed first. Brittle modes of failure are considered to be force-controlled actions (FEMA 273, 1997). For force-controlled actions, the capacity (nominal or ultimate strength) of the member at the deformation level associated with maximum flexural ductility demand must be greater than the force demands caused by earthquake, dead, and live loads (as represented by Equations 2-1 and 2-2). The flexural mode of failure is considered to be a displacement-controlled action. In a displacement-controlled action moment demands can exceed moment capacities, however, the displacement capacity of members must be greater than the inelastic displacement demands placed on the structure due to earthquake, dead, and live loads. The flexural displacement capacity will usually be limited either by the compressive strain in the concrete (a maximum of 0.02 % if bond deterioration is to be prevented), or by the tensile strain in the reinforcing steel (a maximum of 5% if fracture of the reinforcing steel is to be prevented). Another important potential mode of failure relates to piles supporting a navigation lock. As indicated in Example D2, performance of the lock structure is governed by yielding of the piles. In this particular example yielding should be limited to less than 10 percent of piles.

d. Shear (diagonal tension)

(1) General. Since shear failure is a brittle failure, it is necessary to inhibit shear failure by ensuring that shear strength exceeds the shear demand corresponding to that associated with

the maximum feasible flexural strength. Shear strength in plastic hinge regions is a function of the flexural displacement demand. As plastic-hinge rotations increase, shear cracks widen, and the capacity of the concrete to transfer shear by aggregate interlock decreases, as illustrated in Figure 5-2.

(2) Shear capacity. In order to meet damage control performance requirements for MDE loadings, the capacity of the reinforced concrete hydraulic structures in shear shall be equal to or greater than the lesser of:

- The full elastic demand placed on the member by the design earthquake, or
- The shear corresponding to 1.5 times the shear associated with the nominal flexural strength.

The capacity of the concrete in shear may be considered as the summation of shear due to aggregate interlock and the shear strength enhancement as the result of axial load (V_C), and to a lesser extent due to the shear resistance available from the transverse reinforcing (traditional truss mechanism), V_S. The total ultimate shear strength (V_U) can be expressed as:

$$V_U = \phi(V_C + V_S) = 0.85(V_C + V_S) \qquad (5\text{-}1)$$

The concrete component of shear strength is given by:

$$V_C = 2\left[k + \frac{P}{2000A_g}\right]\sqrt{f_c'}\,(A_e) \qquad \text{(psi units)} \qquad (5\text{-}2)$$

$$V_C = 0.17\left[k + \frac{P}{2000A_g}\right]\sqrt{f_c'}\,(A_e) \qquad \text{(MPa units)}$$

where,

P = Axial load on section

f_c' = Actual concrete compressive strength (The actual concrete compressive strength, which may be as high, or higher than 1.5 times the design compressive strength, should be used when calculating the shear capacity.)

A_g = Gross concrete area

A_e = $0.8(A_g)$

In Equation 5-2, k = 1 for flexural displacement ductility demand μ = 1, and k = 0.5 for μ = 2.0, with linear interpolation between these values for μ greater than 1.0 but less than 2.0. The relationship between concrete shear strength and flexural displacement ductility is illustrated in Figure 5-1.

For rectangular sections the contribution of shear steel to the total shear capacity is:

$$V_S = \frac{A_h(f_y)(0.8d)}{s} \qquad (5\text{-}3)$$

Where *d* is the section dimension in the direction of the seismic shear forces, f_y is yield strength of steel, s is spacing of reinforcement, A_h is the reinforcement cross section area, and V_S is the contribution from the shear reinforcement.

For circular sections the contribution of the shear reinforcement is given by:

$$V_S = \frac{\pi A_h (f_y)(0.8d)}{2s} \tag{5-4}$$

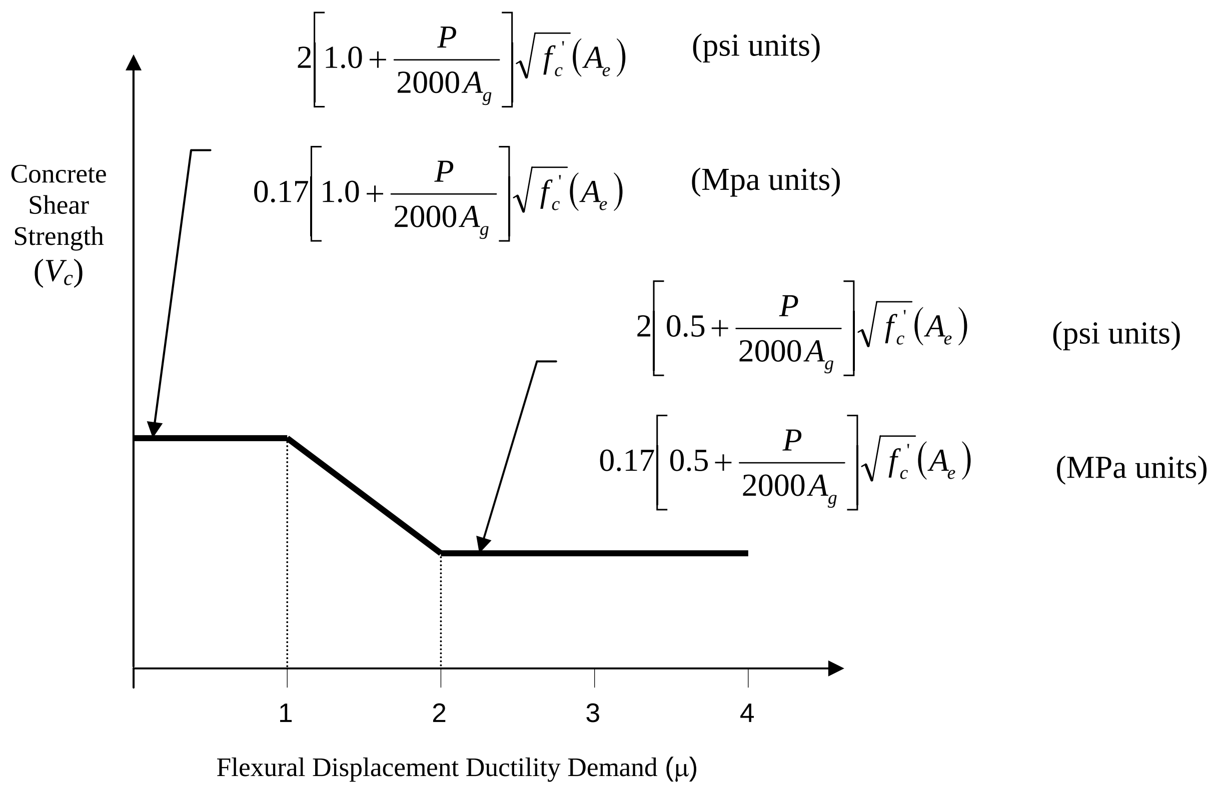


Figure 5-1. Concrete Shear Strength vs. Ductility

e. Sliding Shear. Sliding shear (shear friction) along the base of a structure or structural member should also be investigated. The shear friction shear capacity (V_{SF}) can be determined by the following expression:

$$V_{SF} = \mu_{SF}\ (P + 0.25\, A_s f_y) \tag{5-5}$$

Where:

μ_{SF} = sliding shear coefficient of friction, per ACI 318.

P = Axial load on section.

A_s = Area of the longitudinal reinforcing steel across the potential failure plane.

f_y = yield strength of the reinforcing steel.

f. Reinforcing Steel Anchorage. The flexural strength of a structure will deteriorate during a major earthquake if the vertical reinforcement provided for bending is not adequately anchored. For straight bars, the anchorage length provided (l_a) should be greater than:

$$l_a = \frac{f_y (d_b)}{13.8} \quad \text{mm} \qquad \text{(MPa units)} \qquad (5\text{-}6)$$

$$l_a = \frac{f_y (d_b)}{2000} \quad \text{inches} \qquad \text{(psi units)}$$

Where:

f_y = yield strength of reinforcing steel
d_b = diameter of reinforcing steel

g. Reinforcing Steel Splices.

(1) The lap splice length provided should not be less than:

$$l_s = \frac{A_b f_y}{0.94\sqrt{f'_{ca}}(c + d_b)} \quad \text{mm} \qquad \text{(Mpa units)} \qquad (5\text{-}7)$$

$$l_s = \frac{A_b f_y}{11.31\sqrt{f'_{ca}}(c + d_b)} \quad \text{inches} \qquad \text{(psi units)}$$

Where:

f'_{ca} = Actual concrete compressive strength

c = the lesser of the clear cover over the reinforcing bars, or half the clear spacing between adjacent bars

A_b = Area of reinforcing bars

For existing structures, the actual compressive strength rather than the design compressive strength should be used when evaluating splice lengths and anchorages.

(2) Deterioration of bond and splice strengths of reinforcing bars is one of the major problems in the design of earthquake-resistant reinforced concrete structures. Transverse reinforcement provides the best protection against splice strength degradation. For new structures, adequate transverse reinforcing steel should be provided at all splice locations where concrete compressive strains are expected to exceed 0.002 in. /in. Perimeter transverse confinement reinforcement using smaller bars at close spacings is better than using larger bars

at wide spacings. However, close spacing of transverse reinforcement may not leave enough room for concrete placement. In situations like this, a design modification may be in order. Splice performance will be greatly improved if splices are located away from regions where yielding is expected to occur, and if lap splice locations are staggered (i.e., no more than half of the bars spliced at any horizontal plane). This is usually the case, but if yielding occurs near splicing, transverse reinforcement should be provided giving due consideration to concrete placement.

h. Fracture of reinforcing steel. Fracture of reinforcing steel can be prevented if enough flexural reinforcing steel is provided to produce a nominal moment strength equal to, or greater than, the 1.2 times the cracking moment capacity of the section. Existing massive concrete structures rarely meet this requirement. Even for new designs, the cost to provide this amount of flexural reinforcement may be prohibitive, but could be justified if seismic loading controls the design. Existing structures can be considered to meet MCE damage control performance requirements, if it can be demonstrated that brittle modes of failure will not occur. Alternatively, a displacement-based evaluation (Paragraph 5-3) can be performed to show that reinforcing steel strains are below 5-percent.

i. Flexure. The nominal moment strength of reinforced concrete members can be determined in accordance with EM 1110-2-2104 requirements. The nominal strength is the capacity to be used in determining demand to capacity ratios (DCR's) for the linear static and linear dynamic analysis methods described in Chapter 6. When DCR's exceed allowable values (see Chapter 6), a displacement-based evaluation can be performed to assess the inelastic flexural response of the structure (Paragraph 5-3).

5-3. Reinforced Concrete Displacement Capacities

When DCR values exceed allowable limits, the inelastic response of the structure is considered to be significant and should be assessed using a displacement-based analysis. The purpose of a displacement analysis is to ensure that flexural displacement capacities (elastic plus inelastic) are greater than flexural displacement demands of the earthquake ground shaking. Displacement-based analysis refers to either a nonlinear static pushover analysis described in Chapter 6, or an equivalent linear dynamic analysis procedure described in EM 1110-2-2400 for intake towers. In pushover analysis, a pushover or capacity curve is developed that shows structure displacement capacities at various stages of inelastic response. In the EM 1110-2-2400 displacement-based analysis, the earthquake displacement demands are computed by a response-spectrum analysis in which an effective stiffness is utilized for the plastic region at the base of the tower. The estimated displacement demand is then compared with the ultimate displacement capacity of the tower. The ultimate displacement capacity is related to the height, the length of plastic hinge, and the fracture strain of the reinforcement. The fracture or ultimate strain of the reinforcement is obtained over a standard 8-in gage length, and on the average can be taken as 18-percent.

5-4. Mandatory Requirements

a. Plain concrete structures.

(1) The tensile capacity of the concrete used in evaluation shall be representative of the concrete at construction joints.

(2) The tensile capacity values used in final design or evaluation shall be based on test results.

b. Reinforced concrete structures.

(1) Flexural capacity used in seismic design and evaluation shall be the nominal moment capacity determined in accordance with EM 1110-2-2104.

(2) Shear capacity shall be determined in accordance with Equation 5-1 with the shear contribution from the concrete (aggregate interlock) determined using Equation 5-2.

Chapter 6
Analysis Procedures and Evaluation of Results

6-1. Introduction

Four procedures are presented for the seismic evaluation of concrete hydraulic structures (CHS). They include: linear static, linear dynamic, nonlinear static, and nonlinear dynamic analyses. The linear static and linear dynamic analyses are commonly used in the design and evaluation of CHS. However, the use of nonlinear analysis in evaluation of existing structures under damaging earthquakes and in design of new structures with potential nonlinear response is gaining recognition. For existing structure, application of nonlinear analysis can eliminate unnecessary remediation and for new structures it can substantiate and verify the design. With respect to earthquake loadings, concrete hydraulic structures are designed and evaluated for both the Operational Basis Earthquake (OBE) and the Maximum Design Earthquake (MDE) earthquake loads in combination with the usual dead and live loads that occur during normal operating conditions. The seismic analysis may start with a seismic coefficient method or equivalent lateral force analysis (linear static procedures) and progress to a linear-elastic response-spectrum or linear-elastic time-history analysis (linear dynamic procedure). In some instances a nonlinear static or nonlinear dynamic analysis may be performed to assess the actual capacity or the level of damage that a structure may experience. The linear static and linear dynamic procedures are described in Paragraph 6-3, the nonlinear static procure in Section 6.5, and the nonlinear dynamic procedure in Paragraph 6-6.

6-2. Seismic Design and Evaluation Using DCR Approach

a. General. A demand to capacity comparison, utilizing a demand to capacity ratio (DCR) as a performance indicator, establishes the basis for performance evaluation of plain and reinforced concrete structures subjected to earthquake ground motions. For reinforced concrete structure, DCR is defined as the ratio of force or moment demand to force or moment capacity. For plain concrete structure, DCR is defined as the ratio of stress demands to static tensile strength of the concrete. Maximum permissible values of the DCR are established to assure serviceability and damage control performance objectives are met. The DCR approach is used in conjunction with linear analysis procedures to evaluate:

(1) Damage control performance for deformation-controlled actions (flexure) under Maximum Design Earthquake (MDE) loading conditions.

(2) Damage control performance for force-controlled actions (shear) under Maximum Design Earthquake (MDE) loading conditions.

(3) Serviceability performance for displacement-controlled actions (flexure) under Operational Basis Earthquake (OBE) loading conditions.

(4) Serviceability performance for force-controlled (shear) actions under Operational Basis Earthquake (OBE) loading conditions.

b. Flexural performance for MDE. In the linear procedures, a linear-elastic model of the structure is subjected to lateral forces of the design earthquake to determine displacements, stresses, and forces developed in the model. If the structure responds nonlinearly, as is often the case for MDE loading conditions, the lateral displacements and corresponding internal forces (or stresses) will exceed yield values. The degree to which the calculated internal forces exceed flexural strength (or tensile stress capacity) is used as a measure of the extent of nonlinear deformations that develop in the member. The acceptance criteria for deformation-controlled actions, as expressed by Equation 6-1, are based on this concept. In Equation 6-1, the moment demands for damage control performance as represented by Equation 2-1 may exceed nominal moment capacity, i.e. the demand to capacity ratio (DCR) can be greater than one. For flexural demands associated with the MDE, the DCR allowable value in Equation 6-1 provides a measure of the displacement ductility capacity required of the member to meet damage control performance objectives. The DRC allowable values for damage control performance in flexure are listed in Table 6-1. These are based on the yield and performance characteristics of the concrete and reinforcing steel.

c. Shear performance for MDE. Shear failures are to be suppressed because they are brittle failures that involve rapid strength deterioration. Therefore under MDE loading conditions the shear demand should not exceed the shear capacity of the structure, or the shear DCR should be less than or equal to one. The allowable DCR values for damage control performance in shear are listed in Table 6-1.

d. Flexural performance for OBE. To keep yielding in flexure to levels that will not impair serviceability, the DCR should be equal to or less than one. The allowable DCR values for serviceability performance in flexure are set in Table 6-1.

e. Shear performance for OBE. The DCR for shear should be less than one to assure shear strength deterioration will not occur at levels of shear demand equal to, or slightly greater than, the OBE. The allowable DCR values for serviceability performance in shear are given in Table 6-1.

6-3. Linear Static and Linear Dynamic Procedures

a. General evaluation process. Concrete hydraulic structures must be designed and evaluated for unusual and extreme earthquake ground motion conditions represented by the OBE and MDE, respectively. It is not usually economical or practical to design new CHS to remain elastic during the MDE, nor can it be expected that existing CHS will respond elastically to an MDE event. In the linear static procedure, the inertia forces of the OBE and MDE are estimated by either the seismic-coefficient method or the equivalent lateral force (ELF) method. In the linear dynamic procedure, the seismic forces are determined by either a linear-elastic response-spectrum analysis or a linear-elastic time-history analysis. Information on the seismic coefficient method, the equivalent lateral force method, response-spectrum analysis, and time-history analysis can be found in Chapter 4.

b. Evaluation process for plain concrete structures. Plain concrete structures, such as dams, are usually evaluated for earthquakes using either a linear-elastic response-spectrum analysis, or a linear-elastic time-history analysis. A finite-element model (FEM) is used to represent the structure and its interaction with the foundation and water (Paragraph 4-3*a*(3) and (4)), and the results are output in the form of concrete stresses with tensile stresses as the primary quantity of interest. The peak tensile stresses obtained from the FEM analysis are those the structure would experience if it remained elastic. Evaluation is accomplished by comparison of elastic earthquake demands (tensile stress demands) to tensile capacity of the concrete. On the basis of linear-elastic analysis, performance is considered acceptable if the resulting demand to capacity ratios are less than the allowable values listed in Table 6-1 and that for the linear-elastic time-history analysis spatial extent and duration of high stresses also meet the specific criteria set forth for each type of structure in Paragraphs 6-3*d*(2), 6.3*e*(2)*, and 6.3f*(2) below.

c. Evaluation process for reinforced concrete structures. Reinforced concrete structures, such as intake towers, navigation locks, and spillway piers, are commonly evaluated for earthquake ground motion effects using a linear-elastic response-spectrum analysis. Linear-elastic time-history modal analysis is also used to evaluate post-yield response with respect to cumulative duration of moment excursions exceeding the moment capacity as well as the spatial extent of yield region. Depending on the complexity of its geometry, an intake tower may be evaluated using an FEM model with frame or solid elements (Paragraph 4-3*a*(2) and (4)). A lock structure is usually evaluated by a 2D or 3D FEM using solid elements (Paragraph 4-3*a*(3), (4), and (5)) and by frame elements to perform pushover analysis (Paragraph 4-3*a*(2)). The results for FEM models with frame elements are output as forces (moments, shears, and axial load) rather than stresses. Force quantities facilitate the evaluation because they can be compared directly to capacity (nominal strength) of reinforced concrete members. Such comparison will show which part of the structure and to what extent, if any, will experience nonlinear behavior in the form of yielding of reinforcing steels and cracking of the concrete. For this purpose demand-capacity ratios for the bending moments, axial, and shear forces should be computed and compared with the allowable values in Table 6-1. The section force demands should also be compared with the axial force-bending moment interaction diagrams to account for the axial force-bending moment interaction effects. The results for FEM models using solid elements are output as element stresses, which must be converted into forces and moments at critical sections and then compared with section capacities. The evaluation should investigate all potential modes of failure (Paragraph 5-2*c*). Brittle failure mechanisms include shear failure, reinforcing steel anchorage failure, and reinforcing steel splice failure (Paragraphs 5-2*d* to *g*), for which the structure should respond elastically. Flexural failures (Paragraph 5-2*h*) are generally considered to be ductile failures. Performance is considered acceptable provided all brittle modes of failure are suppressed and demand to capacity ratios are less than the allowable values listed in Table 6-1.

d. Evaluation process for gravity dams. Gravity dams subjected to OBE ground motions should perform within the linear-elastic range to assure that very little or no tensile cracking will occur. Under MDE ground motion demands, gravity dams may respond in the inelastic range provided that the performance is within the strain hardening range (i.e. damage control range in Figure 2-5). Stresses in excess of the ultimate tensile stress capacity are assumed to initiate and propagate cracking. The results of the time-history analysis can be used to assess the damage potential of stress excursions that exceed the tensile capacity of the concrete and the effects that cracking might have on the performance of the dam.

(1) Response-Spectrum Analysis. A linear-elastic response-spectrum analysis is generally the first step in the evaluation process (Paragraph 4-2*c*). The earthquake demands in terms of stresses are computed and compared with the stress capacity of the concrete to assess whether the resulting DCR ratios are lower than allowable values listed in Table 6-1b. In cases where tensile stress demands exceed the allowable tensile capacity of the concrete (DCR's exceed acceptable limits), a linear-elastic time history analysis is generally performed.

(2) Linear Time-History Analysis. Linear time history analysis of gravity dams should be conducted and evaluated in accordance with procedures discussed in Paragraph 6-4*d*(1). A systematic interpretation and evaluation of the results of time-history analysis in terms of the demand-capacity ratios, cumulative inelastic duration, spatial extent of overstressed regions,

and consideration of possible modes of failure form the basis for estimation of probable level of damage or acceptable level of nonlinear response.

e. Evaluation process for arch dams. (From EM 1110-2-2201) The earthquake response analysis of arch dams is generally based on the linear-elastic dynamic analysis using finite-element procedures. It is assumed that the concrete dam and the interaction mechanisms with the foundation rock and impounded water exhibit linear-elastic behavior. Using this method, the arch dam and foundation rock are treated as 3-D systems. The analysis is performed using the response-spectrum modal superposition or time history method. The earthquake performance is evaluated using the numerical results obtained from such analyses. The results of linear analysis provide a satisfactory estimate of the dynamic response to low or moderate intensity OBE earthquake motions for which the deformations of the dam are within the linear-elastic range. In this case, the performance evaluation is based on DCR allowable values listed in Table 6-1b. Under MDE ground motions, it is possible that the calculated stresses would exceed the allowable values, and that some damage would occur. In such extreme cases, the dam may suffer significant damage but should retain the impounded water without rupture. Evaluation for the MDE should start with the DCR approach and progress to the linear time-history analysis and possibly to nonlinear time history analysis, as needed.

(1) Response-spectrum Analysis. The response-spectrum method of analysis (EM 1110-2-6050, EM110-2-2201) uses a response-spectrum representation of the seismic input motions to compute the maximum response of an arch dam to earthquake loads. Three orthogonal components of response spectra are used as the seismic input. This method provides an efficient procedure for the preliminary analyses of new and existing arch dams. It may also be used for the final analyses, if the calculated stress values meet the DCR allowable values listed in Table 6-1b. Otherwise, linear time-history analysis, and if needed, nonlinear time-history analysis should be considered. Using the response-spectrum procedure, the maximum response of the dam is obtained by combining the maximum responses for several modes of vibration computed separately.

(2) Linear Time-History Analysis. Linear time-history analysis of arch dams is conducted and evaluated in accordance with procedures and load combinations described in EM 1110-2-6051. It involves a systematic interpretation and evaluation of the results of time-history analysis in terms of the demand-capacity ratios, cumulative inelastic duration, and spatial extent of overstressed regions as described in Paragraph 6-4*d*(2).

f. Evaluation process for intake towers. Earthquake loadings generally govern the design of intake towers. Performance is considered acceptable if all brittle modes of failure are suppressed, and demand to capacity ratios are less than the allowable values listed in Table 6-1. The DCR requirements for flexure limit the ductility demand to levels acceptable for lightly reinforced structures. The DCR requirements for brittle modes of failure will suppress shear and sliding shear failures. Shear capacity for computation of DCR should be selected consistent with the level of displacement ductility demand associated with the peak flexural response (see Figure 5-3).

(1) Response-spectrum Analysis. The response-spectrum analysis of intake towers is carried out in accordance with EM 1110-2-2400 and EM 1110-2-6050. A model of the tower is developed as described in Paragraph 4-3 and is subjected to one horizontal and the vertical (2D model) or two horizontal and the vertical (3D model) components of response spectra. Section forces and moments are computed and combined in accordance with Paragraph 3-2, and compared with section capacities and axial force-bending moment diagram following the

procedure described in Paragraph 6-3*c*. In cases where force demand capacity ratios exceed the allowable values listed in Table 6-1a, a linear-elastic time history analysis may be performed to assess the level of damage.

(2) Linear Time-History Analysis. The damage for lightly reinforced freestanding intake towers is evaluated on the basis of demand-capacity ratios (DCR) and cumulative inelastic duration described in Paragraph 6-4*c*(2).

g. Evaluation process for navigation locks. The earthquake performance of reinforced concrete navigation locks is evaluated on the basis of demand-capacity ratios computed for the foundation piles when they are present) and the concrete sections in accordance with EM 1110-2-6051. The computation of earthquake demands starts with linear-elastic analysis using response-spectrum and/or time-history method.

(1) Response-spectrum analysis. Lock structures founded on rock with no backfill soil can adequately be analyzed using the response-spectrum modal superposition method, as described in EM 1110-2-6050. The performance is evaluated by computing and comparing force and moment DCRs with the allowable values listed in Table 6-1a. The total force and moment demands are obtained for the combined effects of static plus earthquake loads. The section shear capacity is determined according to Paragraph 5-2*d*(2). The section moment capacities are obtained from the axial force-bending moment interaction diagrams characterizing the strength of a reinforced concrete section.

(2) Time-history analysis. Locks founded on soil or pile foundation and with backfill soil may require SSI time-history analysis, as described in 4.3*c.* The earthquake performance of the lock is evaluated on the basis of demand-capacity ratios computed for the foundation piles and the concrete sections in accordance with EM 1110-2-6051. If all computed demand-capacity ratios are less than or equal to 1.0, then the lock structure and piles are expected to respond elastically with no damage. Otherwise demand-capacity ratios of greater than 1.0 show the structure will experience nonlinear behavior in the form of yielding of steel members and cracking or crushing of the concrete. The acceptability of the level of damage and nonlinear behavior will be determined on the basis of performance curves provided in EM 1110-2-6051. Performance of the pile-foundation under is evaluated using interaction factors or demand-capacity ratios computed in accordance with Equation 6-6. For the OBE excitation, the piles should respond within the linear elastic range of behavior. Under the MDE excitation, the piles interaction factor, I_p, should generally be less than or equal to 1. However, for severe and damaging earthquakes the pile interaction factor could approach 1.1 for less than 10 percent of the piles, provided that nonlinear pushover analysis is conducted to ensure that permanent later displacements of the pile foundation, if any, is small.

6-4. Acceptance Criteria for Linear Procedures

a. ELF and response-spectrum analysis. The earthquake load effects calculated in accordance with Chapter 4 combined with the effects of dead and live loads as specified in Equations 2-1 and 2-2 are used to calculate total demands on the structure. The expected capacity or strength of the structure is determined in accordance with Chapter 5, and demand to capacity ratios (DCRs) are calculated for each structural component of interest and for each potential failure mechanism (Flexure, shear, etc.). The seismic performance of the structure is considered acceptable if the DCR for each component and potential failure mechanism is less than or equal to the *allowable value* for that particular component and failure mechanism.

$$\text{DCR} \le \textit{Allowable Value} \qquad (6\text{-}1)$$

The DCRs allowable values for concrete hydraulic structures are provided in Table 6-1. In some cases the forces obtained from the linear-elastic analysis are not sufficient to displace the structure to the maximum inelastic displacements expected in response to the design earthquake ground motions. This could occur in the case of an equal energy response (Paragraph 3-3*a*(2)). Therefore, for the flexural response when earthquake moment demands exceed nominal moment capacity, the moment demands from the ELF, or response spectrum analysis must be multiplied by a C_1 factor, where:

C_1 = Modification factor to relate expected maximum inelastic displacements to displacements obtained from the linear-elastic response

The C_1 factor is based on the FEMA 273 formulation for C_1, with the term SR in Equation 6-2 representing R in the FEMA formulation. Studies (Whittaker et al., 1998) suggest that for strength ratios (nominal flexural strength to moment demand) of 0.5 the use of the FEMA C_1 factor will produce inelastic displacements that are representative of mean elastic displacements obtained from linear elastic analyses. This however is not true for strength ratios lower than 0.5 where the inelastic displacements can substantially exceed the C_1 adjusted mean elastic displacements. The strength ratios for hydraulic structures are generally in the 0.5 range, and therefore the FEMA 273 formulation for C_1 is considered to be appropriate.

For an equal displacement response (or for $T \ge T_0$), $C_1 = 1.0$.

For an equal energy response (or for $T \le T_0$):

$$C_1 = \left[1.0 + (SR - 1)\frac{T_0}{T}\right]\left(\frac{1}{SR}\right) \le 1.5 \qquad (6\text{-}2)$$

Where:

$$SR = \frac{M_D}{M_N} \qquad (6\text{-}3)$$

M_D = Elastic moment demand from linear analysis

M_N = Nominal moment capacity

Table 6-1a
DCR *Allowable Values* for Reinforced Concrete Hydraulic Structures

Action In terms of forces	Performance Objectives	
	Damage Control (MDE)	Serviceability (OBE)
Flexure	2.0	1.0
Shear	1.0	0.8
Sliding Shear	1.0	0.8

Table 6-1b
DCR Allowable Values for Response-Spectrum Analysis of Plain Concrete Hydraulic Structures

Action In terms of stresses	Performance Objectives	
	Damage Control (MDE)	Serviceability (OBE)
Tension due to flexure	1.5	1.0
Diagonal tension due to shear	0.9	0.8
Shear due to sliding	1.0	0.8

Table 6-1a is based on the assumptions that:

1. Concrete hydraulic structures are lightly reinforced
2. Beams, slabs, walls and other load carrying members are controlled by flexure
3. The members are non-conforming meaning they do not meet confinement steel and other seismic detailing requirements of ACI 318
4. Wall and other vertical load carrying members have axial load ratios $\frac{P}{A_g f_c'}$ less than 0.1
5. DCR *allowable values* for conditions other than those described above can be selected from Tables 6-10 and 6-11 of FEMA 273

(1) Illustrating the use of Table 6-1a for flexure. For damage control requirements, DCR allowable values for the flexural response in a reinforced concrete structure must be less or equal to 2. This means that the ratio of the elastic moment demand (modified by C_1) to the nominal moment capacity, must be less than or equal to 2:

$$\frac{M_{DC}(C_1)}{M_N} \leq 2.0 \tag{6-4}$$

Where:

M_{DC} = Total moment demand (See Equation 2-1) obtained from a linear-elastic response spectrum or time history analysis.

C_1 = Modification factor to relate maximum inelastic displacements to displacements obtained from linear elastic response (see EQ 6-2)

M_N = Nominal moment capacity.

(2) Illustrating the use of Table 6-1b for flexure. For damage control requirements, DCR allowable values for the flexural response in a plain concrete structure must be less than 1.5. This means that the ratio of the flexural tensile stress demand from the linear-elastic response-spectrum analysis to the splitting static tensile stress capacity, must be equal to, or less than 1.5, or:

$$\frac{\sigma_{ta(DC)}}{f_t^s} \leq 1.5 \qquad (6\text{-}5)$$

Where:

$\sigma_{ta(DC)}$ = Total tensile stress demand (See Equation 2-1) obtained from a linear-elastic response- spectrum analysis for MDE ground motions

f_t^s = Static tensile strength (see Chapter 5)

In other words, Equation 6-5 is the same as requiring the tensile stress demand obtained from a linear elastic FEM analysis to be equal to or less than the dynamic tensile strength.

b. Pile interaction factors (demand-capacity ratios). Performance of the pile-foundation under the MDE loading combination is evaluated using interaction factors or demand-capacity ratios computed in accordance with Equation 4-1.

$$I_p = \left(\frac{f_a}{F_a} + \frac{m_x}{M_x} + \frac{m_y}{My}\right)_{static} + \left(\frac{f_a}{F_a} + \frac{m_x}{M_x} + \frac{m_y}{M_y}\right)_{dynamic} \qquad (6\text{-}6)$$

Where:

I_p = pile interaction factor
f_a, m_x, m_y = the axial force and bending moments (force and moment demands) computed either from the static or dynamic analysis
F_a = allowable axial force (force capacity) for combining with allowable moment (moment capacity)
M_x, M_y = allowable moments (moment capacities), respectively, about the strong and weak axes of the pile

c. Time history analyses – reinforced concrete structures

(1) FEMA 273 approach. In Table 6-1a, the DCR allowable values for flexure are based on the assumption that the structure has the capacity to resist three fully reversed deformation cycles at the deformation levels represented by the allowable values, in addition to similar cycles at lesser deformation levels. When time history analyses are used to verify performance acceptability, the evaluations shall be relative to acceptable moment demand levels represented by the DCR allowable values. The acceptable moment demand level is equal to (*DCR Allowable Value/ C_1*) times M_N, which is illustrated as being 850 ft-kips in Figure 6-1. The moment

demand response as represented by Figure 6-1 suggests that there are five cycles above the acceptable moment demand level (850 ft-kips), indicating the structure may not perform to expectations. With a longer duration earthquake of similar magnitude the performance would surely be unacceptable. Short period structures subjected to long duration earthquakes are particularly vulnerable to numerous cycles at the deformation levels represented by the DCR allowable values. In some cases, the energy contained in each cycle may not be sufficient to impair strength. In other cases, the increased number of cycles can lead to reductions in force and deformation capacity. The effects on strength and deformation capacity of more numerous cycles beyond what is considered the ductility capacity of the structure should be considered. In those cases where the number of cycles, beyond what is considered to be acceptable moment demand levels exceed three, the cumulative duration approach as described in EM 1110-2-6051 should be used.

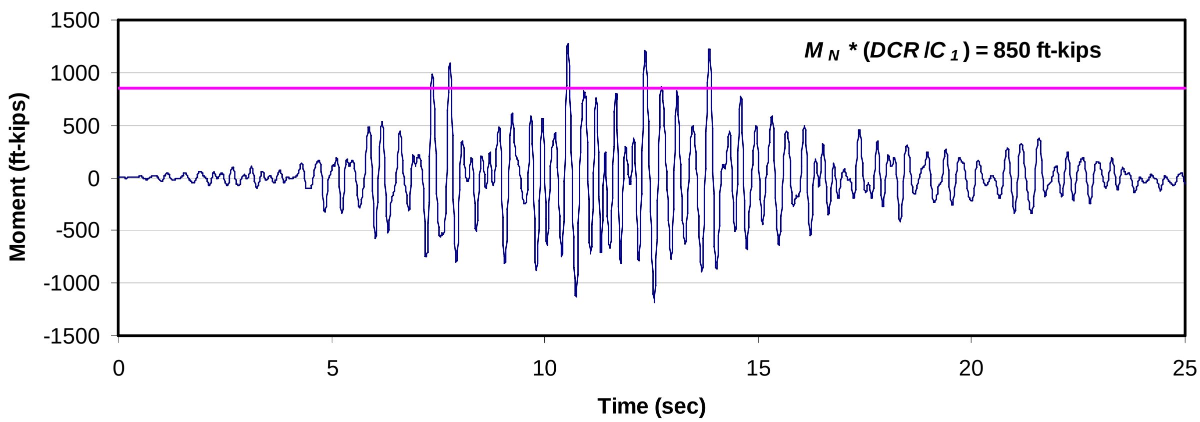


Figure 6-1. Moment Time History Evaluation

(2) EM 1110-2-6051 cumulative duration approach. The acceptance criteria for linear-elastic time-history evaluation of lightly reinforced freestanding intake towers is assessed on the basis of demand-capacity ratios (DCR) and cumulative duration, as described in EM 1110-2-6051. The basic procedure is to perform linear time-history analysis with appropriate amount of damping to obtain bending moment DCR ratios for all finite elements. Initially a damping ratio of 5 percent is used and then increased to 7 percent if DCR ratios are approaching 2 and to 10 percent if they exceed 2. After adjustment for the damping, the damage is considered moderate and acceptable if the following conditions are met:

- Bending moment DCR ratios computed on the basis of linear time-history analysis remain less than 2
- Cumulative duration of bending-moment excursions above DCR ratios of 1 to 2 fall below the acceptance curve given in Figure 6-2
- The extent of yielding along the height of tower (i.e., plastic hinge length for DCR ratios of 1 to 2) is limited and falls below the acceptance curve.

If DCR ratios exceed 2.0 or the cumulative duration and the yield lengths rise above the acceptance curves, the damage is considered to be severe and should be assessed using nonlinear analysis procedures. The term cumulative inelastic duration is defined as the total time of bending-moment excursions above a particular capacity corresponding to DCR ratios of

1 to 2. The yield height ratio refers to the yielded length of tower normalized with respect to the tower height. To keep the damage to a moderate level, the yield length should be less than one-third of the tower height, as shown in Figure 6-2.

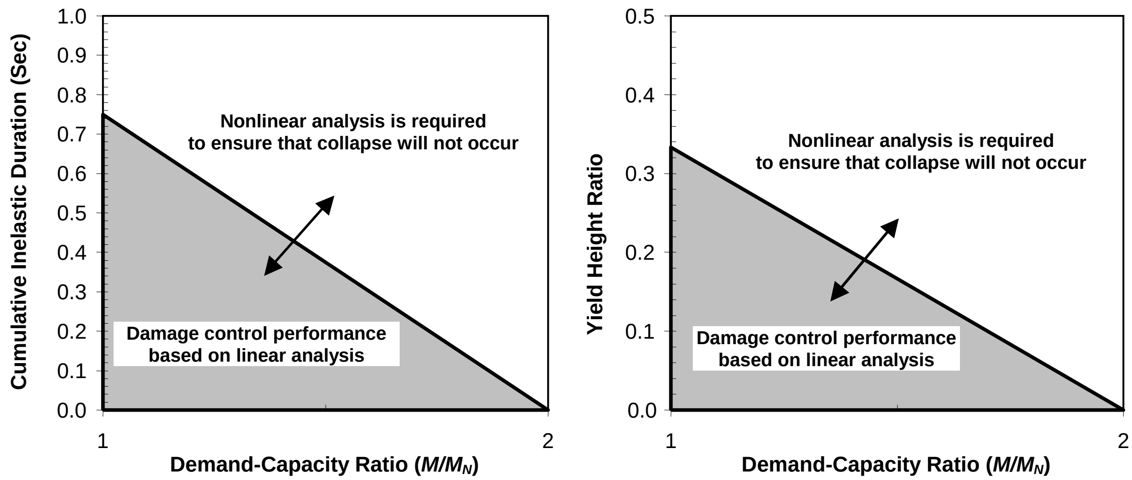


Figure 6-2. Acceptance Criteria for Freestanding Intake Towers

d. Time history analyses – plain concrete structures

(1) Concrete gravity dams. The acceptance criteria for linear-elastic time-history analysis of gravity dams is based on DCRs and cumulative inelastic duration described in EM 1110-2-6051. DCR for plain concrete structures are computed as the ratio of stress demands to static tensile strength of the concrete. A systematic interpretation and evaluation of the results of time history analysis in terms of the demand-capacity ratios, cumulative inelastic duration, spatial extent of overstressed regions, and consideration of possible modes of failure form the basis for estimation of probable level of damage or acceptable level of nonlinear response. The dam response to the MDE is considered to be within the linear-elastic range of behavior with little or no possibility of damage if the computed stress demand-capacity ratios are less than or equal to 1.0. The dam would exhibit nonlinear response in the form of cracking of the concrete and/or opening of construction joints if the estimated stress demand-capacity ratios exceed 1.0. The level of nonlinear response or cracking is considered acceptable if demand-capacity ratios are less than 2.0 and limited to 15 percent of the dam cross-sectional surface area, and the cumulative duration of stress excursions beyond the tensile strength of the concrete falls below the performance curve given in Figure 6-3. Consideration should also be given to relation between the fundamental period of the dam and peak of the earthquake response spectra. If lengthening of the periods of vibration due to nonlinear response behavior causes the periods to move away from the peak of the spectra, then the nonlinear response would reduce seismic loads and improve the situation by reducing stresses below the values obtained from the linear time-history analysis. When these performance conditions are not met, or met only marginally with the nonlinear response increasing the seismic demand, then a nonlinear time-history analysis might be required to estimate the damage more accurately.

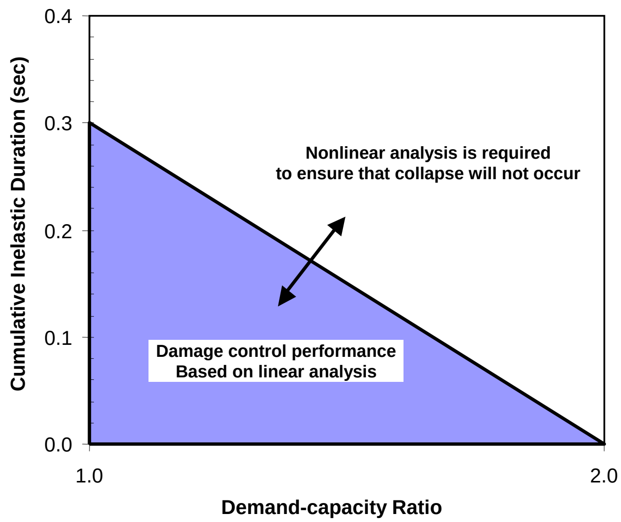


Figure 6-3. Performance Curve for Concrete Gravity Dams

(2) Concrete arch dams. The acceptance criteria for the linear-elastic time-history analysis of arch dams are based on procedures and load combination cases described in EM 1110-2-6051. It involves a systematic interpretation and evaluation of the results of time history analysis in terms of the demand-capacity ratios, cumulative inelastic duration, spatial extent of overstressed regions, and consideration of possible modes of failure that form the basis for estimation of probable level of damage or acceptable level of nonlinear response. The dam response to the MDE is considered to be within the linear elastic range of behavior with little or no possibility of damage if computed demand-capacity ratios are less than or equal to 1.0. Considering that the ability of contraction joints to resist tension is limited, the joints may still open even if demand-capacity ratios are less than or equal to 1.0. The amount of contraction joint opening at a DCR ≤ 1, however, is expected to be small with negligible or no effects on the overall stiffness of the dam. The dam is considered to exhibit nonlinear response in the form of opening and closing of contraction joints and cracking of the horizontal joints (lift lines) if the estimated demand-capacity ratios exceed 1.0. The level of nonlinear response or opening and cracking of joints is considered acceptable if DCR < 2, overstressed region is limited to 20 percent of the dam surface area, and the cumulative inelastic duration falls below the performance curve given in Figure 6-4. The relation between the fundamental period of the dam and peak of the response spectra should also be considered to determine whether the nonlinear response behavior would increase or decrease the seismic demand. If these performance criteria are not met, or met marginally with increasing demand due to nonlinear behavior, then a nonlinear analysis would be required for more accurate estimate of the damage.

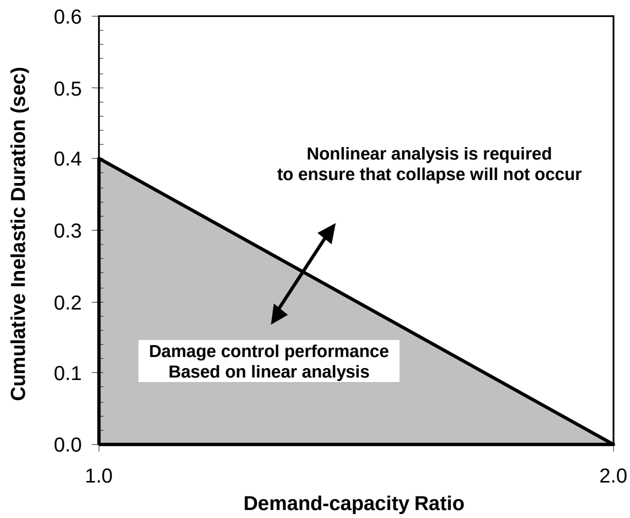


Figure 6-4. Performance Curve for Concrete Arch Dams

6-5. Nonlinear Static Procedures

Two different nonlinear static procedures are available for the evaluation of reinforced concrete structures. They are the displacement ductility evaluation and the pushover analysis. The displacement ductility approach at this time is limited to cantilever type structures as described in Chapter 5. The pushover method is applicable to all types of structures, as long as structural members of the reinforced concrete structure can be represented by frame elements for which nonlinear response behavior has been established.

a. Displacement ductility evaluation. In displacement ductility evaluation the displacement ductility capacity of the structure is determined as described in Chapter 5. Displacement ductility demands are estimated based on a linear elastic response-spectrum analysis. Since all inelastic action will be due to the flexural response, the elastic moment demand (M_D) and the nominal moment capacity of the section (M_N) are used to determine the displacement ductility demand (μ_D) on the structure. The ductility demand will depend on whether the structure exhibits an equal energy response or an equal displacement response (see Chapter 3). A formulation for displacement ductility demand can be developed using Formula 6-2 based on the following:

(1) The displacement ductility demand (μ_D) is equal to the displacement demand (δ_D) divided by the displacement at yield (δ_y), or

$$\mu_D = \frac{\delta_D}{\delta_y} \tag{6-6}$$

(2) Since in linear analysis displacements are proportional to forces, Equation (6-6) can be expressed:

$$\mu_D = \frac{M_D(C_1)}{M_N} \tag{6-7}$$

(3) Recalling that SR = M_D / M_N (see Equation 6-3), the displacement ductility demand can be calculated using the following equation:

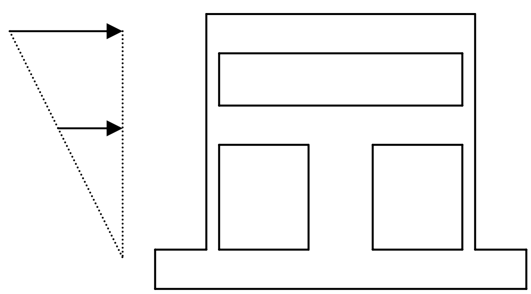
$$\mu_D = \left[\frac{M_D}{M_N} - 1\right]\frac{T_0}{T} + 1 \tag{6-8}$$

This should neither be less than M_D/M_N for equal displacement response, nor greater than 1.5 M_D/M_N for equal energy response. The above formulation provides a smooth transition between the equal energy and equal displacement regions of the response spectrum. The structure is considered to perform acceptably if all modes of brittle failure are suppressed and the displacement ductility capacity exceeds the displacement ductility demand. Shear should be reexamined to make sure the shear capacity has not been reduced to unacceptable levels due to high flexural ductility demand. A pushover analysis is required when it becomes necessary to evaluate collapse prevention performance. Collapse prevention performance could be the performance objective for critical structures where the MDE is equal to the Maximum Credible Earthquake (MCE).

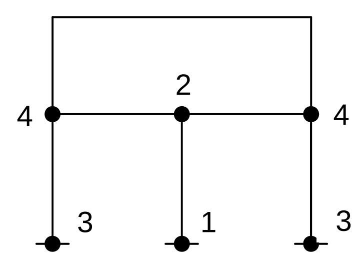
b. Pushover method. For complex structures where plastic hinges can form in several locations, a pushover type analysis (collapse mechanism analysis) should be used to assess the actual performance of the structure. Pushover analysis is a nonlinear static procedure in which the magnitude of loading is increased incrementally according to a predefined pattern. The analysis with increasing loads continues until the structure is displaced to a large enough displacement (target displacement) capable of mobilizing principal nonlinear modes of behavior up to collapse of the structure. The computer model of the structure incorporates inelastic material response, thus allowing for redistribution of forces and deformations as structural members undergo nonlinear response in the form of yielding of reinforcing steel and cracking of the concrete. The pushover analysis is conducted using a load controlled or displacement controlled procedure. Load-controlled procedure involves incremental application of a monotonic load to the structure until the maximum load is reached or the structure collapses, whichever occurs first. Force control should be used when the magnitude of load is known (such as gravity), and the structure is expected to support the load. Displacement-controlled procedure involves incremental application of a monotonic load until the control displacement is reached a pre-specified value or the structure collapses, whichever occurs first. Displacement control is used when the value of applied load is not known in advance, or when the structure is expected to lose strength. Since the final value of earthquake load can not be determined precisely in advance, the displacement-controlled method is usually employed. In addition to the load and displacement-controlled procedures, the capacity spectrum method is also available for seismic evaluation of structures with multiple plastic hinge regions. The capacity spectrum is an approximate nonlinear static procedure that predicts the inelastic displacement demand of the structure by combining structural capacity obtained from a pushover analysis with seismic demand represented by response spectra (ATC-40 and example in Appendix D).

(1) The displacement-controlled approach, also known as the target displacement approach, is the pushover analysis procedure selected in FEMA 356 (2000) for seismic assessment of building structures. The same approach is also applicable to hydraulic structures, as demonstrated in examples in Appendices D and E. In pushover procedure, a series of nonlinear static analyses carried out to develop a capacity or pushover curve for the structure. With increasing the magnitude of loading during the pushover analysis, the structural members undergo nonlinear response, and thus weak links and failure modes of the structure are found. The yield regions are monitored as lateral loads representing inertial forces in an earthquake

are increased until the ultimate rotational capacity is reached, or the structure fails due to instability. The displacement capacity, as represented by rotational failure or by structure instability, is then compared with the earthquake displacement demand to determine if the displacement capacity of the structure is sufficient to prevent failure. The pushover method, using the target displacement approach of FEMA 356, is illustrated by Figure 6-5 and 6-6. Figure 6-5 represents a center wall section of a navigation lock. The structural idealization is shown along with potential yield regions (plastic hinge zones). The pushover loads are distributed as shown, and are proportioned in accordance with the first mode shape. The structure is displaced by increasing the lateral load (keeping the same distribution) until the force demand equals the capacity of the most critical member. Assuming shear modes of failure are suppressed, the most critical member is the one where the nominal moment capacity is reached first. The displacement at the top of the structure is plotted vs. the total lateral load. A plastic hinge element is then inserted in the model. The stiffness of the plastic hinge element is based on its load-rotation characteristics. Generally stiffness equal to 5-percent of the effective section stiffness is used. The new model (model with the plastic hinge) is pushed in a similar manner until the next critical element or elements are located. The load and displacement increment associated with the formation of the new hinge is plotted and the process continues until the rotation of one of the plastic hinges reaches its ultimate rotational capacity, or until instability occurs. The displacement capacity as determined from the capacity curve is compared to the displacement demand (δ_D) to see if performance is satisfactory.



Center Wall Section



Structure Idealization

Figure 6-5. Pushover Model for Navigation Lock Center Wall

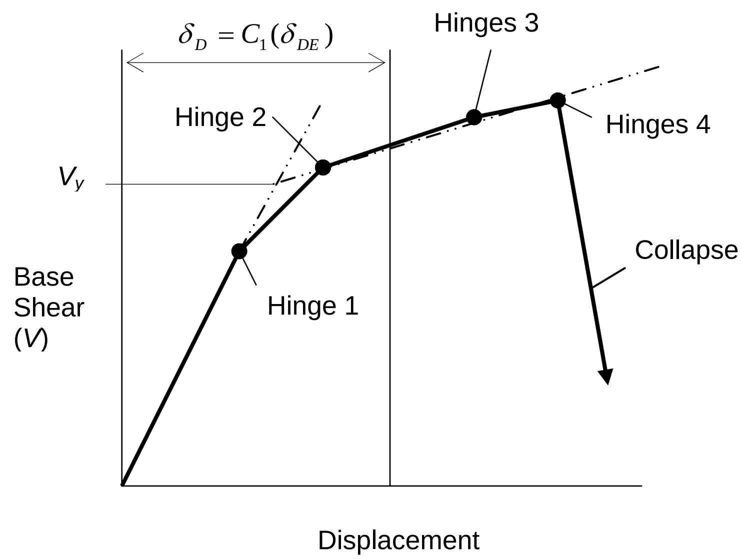


Figure 6-6. Force – Displacement from Pushover Analysis

The displacement demand is determined using the following equation:

$$\delta_D = C_1(\delta_{D1}) \tag{6-9}$$

Where:

δ_{D1} = The first mode displacement demand of the structure. Obtained from linear elastic response-spectrum analysis, or by the following equation:

$$\delta_{D1} = C_0(S_a)\frac{T^2}{4\pi^2} \tag{6-10}$$

Where:

S_a = Response spectrum acceleration for the first mode of vibration.

T = First mode period of vibration

C_0 = Modification factor to relate spectral displacement to top of structure displacement. Use first mode participation factor, or assume $C_0 = 1.5$.

C_1 is the modification factor to relate expected maximum inelastic displacements to displacements obtained from the linear elastic response. For an equal displacement response (or for $T \geq T_0$), $C_1 = 1.0$. For $T \leq T_0$:

$$C_1 = \left[1.0 + (SR-1)\frac{T_0}{T}\right]\left(\frac{1}{SR}\right) \leq 1.5 \qquad \text{(Repeat of Equation 6-2)}$$

Where:

$$SR = \text{Strength ratio} = C_0 \frac{S_a(m_T)}{V_y} \qquad (6\text{-}11)$$

And where:

m_T = Total Mass (structure mass + added water mass + added soil mass)

V_y = Structure yield strength calculated from the pushover analysis assuming an idealized bilinear load-displacement relationship (see Figure 6-6).

The term S_a (m_T) represents the total shear demand on a single degree of freedom system with a period (T), spectral acceleration (S_a), and mass (m_T). Performance is acceptable if the relationship between the displacement demand and the displacement capacity meets project performance objectives, and provided all brittle modes of failure are suppressed.

6-6. Nonlinear Dynamic Procedure

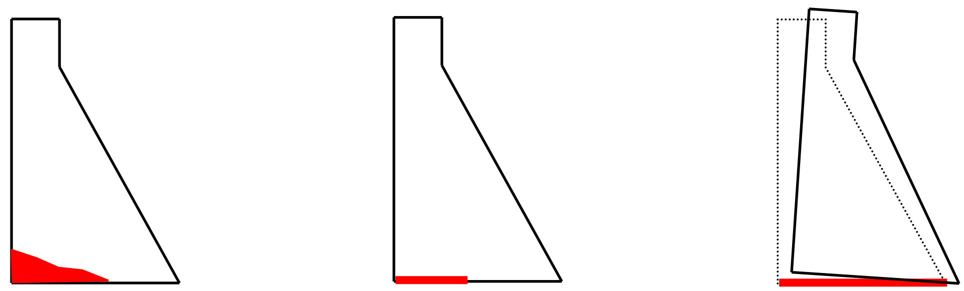
a. General. Under nonlinear dynamic analysis procedure, the induced displacements, stresses, and section forces (seismic demands) are obtained from the step-by-step solution of the equations of motion including nonlinear force-displacement relationship. Seismic demands in the form of response histories are computed using ground motion acceleration time histories as the seismic input, and then compared with the structure capacity to determine if the desired performance has been achieved. The nonlinear dynamic analysis for plain concrete structures is carried out using a nonlinear finite-element representation of the structure. Performance is evaluated by investigating the formation and propagation of tensile cracking to determine whether or not the cracking would lead to failure of the structure. The failure mechanisms may involve sliding along the joints and cracked sections, rotational instability, or both. Prediction of crack patterns in mass concrete can be accomplished using fracture mechanics techniques. For crack analysis of 3D structures both linear elastic fracture mechanics (LEFM) and nonlinear fracture mechanics (NLFM) may be considered. These methods are based on various material models that depend on critical values of parameters characterizing the crack-tip stress and strain fields. The application of these methods to concrete structures has somewhat been limited to static loading conditions. Conducting a meaningful non-linear fracture mechanics analysis of a concrete gravity dam is extremely difficult and should be undertaken only under the supervision of experts in the field of fracture mechanics, and with approval by and in consultation with CECW-CE. In the case of concrete hydraulic structures such as dams, the nonlinear behavior mostly involves opening and closing of the vertical joints and tensile cracking along the horizontal lift lines and the dam-foundation interface. These conditions can be identified using the linear-elastic dynamic analysis described earlier and then analyzed for structural stability using nonlinear dynamic procedures described in the following paragraphs. The performance of dams for the MDE is considered satisfactory if the cracks that develop during intense ground shaking have not opened to the extent that significant leakage through the dam can occur, or to the extent that significant permanent irrecoverable displacements within the dam or foundation occur.

b. Gravity dams

(1) While it is possible to model material and other sources of nonlinearity in analysis of gravity dams, the required parameters are either not known or well defined. For this reason the nonlinear dynamic analysis of a gravity dam should focus on capturing the potential failure modes that would have the most impact on the stability of the dam. A typical gravity dam is built as individual monoliths separated by vertical joints, and construction of each monolith involves placement of concrete in lifts that produces horizontal joints whose tensile strength could be less than that of the parent concrete. Consequently, in a major earthquake it is likely that the vertical joints would open and close repeatedly and tensile cracking would occur along the lift lines, at the dam-foundation interface, and at the change of slope in upper part of the dam where stress concentration occurs. The nonlinear performance evaluation of gravity dams therefore starts with a linear-elastic time-history analysis to identify overstressed regions that would experience cracking, followed by nonlinear dynamic analyses incorporating slippage and rotation with respect to opened joints and cracked sections, as well as post-earthquake analyses for static loads and after-shock excitations.

(2) The results of nonlinear analysis will include sliding displacement and rotation demands that must be sufficiently small not to jeopardize safety of the dam during the main event as well as during the after shocks. This means that after the level of damage has been established for the main event, the damaged structure should be tested against the probable aftershock that could be one to two magnitudes smaller than the main shock. In addition, post-earthquake static stability analyses should be carried out so that the ability of the damaged structure to resist the operating loads can be demonstrated.

(3) For example, a linear-elastic dynamic analysis may indicate that the gravity dam shown in Figure 6-7 will experience high tensile stresses at the dam-foundation interface and that the dam does not pass the acceptance criteria set forth in Paragraph 6-4c(2). In subsequent nonlinear dynamic analyses gap-friction elements are introduced at the high tensile-stress region of the base to allow formation and propagation of cracks, which are found to extend through the entire base of the dam. The results may indicate that the dam fully cracked at the base will undergo sliding and rocking leading to a permanent displacement (offset) at the end of the shaking. The magnitude of the permanent sliding displacement is estimated and compared with operational and safety requirements. The performance of dams for the MDE is considered satisfactory if the cracks that develop during intense ground shaking have not opened to the extent that significant leakage through the dam can occur, or to the extent that significant permanent irrecoverable displacements within the dam or foundation occur. Appendix H provides an example of nonlinear time-history analysis and performance evaluation of a non-overflow gravity dam section.



a) Linear-elastic analysis shows high tensile stresses at the base

b) Nonlinear analysis with gap-friction elements indicates cracks will extend to entire base

c) Results indicate that dam will undergo sliding and rocking with a permanent offset

Figure 6-7. Example of sliding and rocking of a typical gravity dam

c. Arch Dams

(1) Arch dams are generally built as independent cantilever monoliths separated by vertical contraction joints. Since contraction joints cannot transfer substantial tensile stresses in the arch direction, the joints can be expected to open and close repeatedly as the dam vibrates in response to severe earthquake ground motions. Construction of arch dams also involves horizontal construction joints known as lifts that may exhibit lower tensile strength than the mass concrete. Consequently opening of contraction joints and cracking of lift joints are the most likely nonlinear mechanisms that could occur in arch dams. Such conditions can be modeled and analyzed using QDAP (Quest Structures, 2001) or other finite-element programs with nonlinear joint capabilities. As in the case of linear analysis the concrete arch and the foundation rock are discretized using standard 3D solid elements, but joints and fractures in the dam, at the dam-foundation interface, or within the foundation are represented by nonlinear 3D joint elements. Therefore the only nonlinear effects considered for the response of the dam are those associated with the opening, closing, and sliding of the along the joints and cracked sections. Since opening of the contraction joint and cracking of the lift joints relieve high tensile stresses, the traditional stress-based criteria will not be applicable to the QDAP results. Instead, under nonlinear dynamic analysis, the magnitude of compressive stresses, the extent of joint opening or cracking, and the amplitude of non-recoverable movements of concrete blocks bounded by failed joints that control the overall stability of the dam should be assessed, as opposed to the magnitude of calculated tensile stresses. Appendix F provides an example of nonlinear time-history analyses conducted to assess the earthquake performance of an arch dam.

(2) The nonlinear dynamic analysis of arch dams should also assess stability of potentially moveable blocks in the abutments if there are adversely jointed rock blocks directly beneath the dam. This problem is best handled as a coupled dynamic problem in which the moveable blocks are modeled as part of the dam finite-element model to allow joint slippage in the abutments and the effects it might have on the stability of the dam. The block joints can be modeled using 3D joint elements discussed above which resist bearing and shear but not tension. The sensitivity of the results to shear strength of the joints and strength degradation with movement and uplift pressures should be investigated.

d. Reinforce Concrete Hydraulic Structures. Similar to static nonlinear analysis, reinforced concrete hydraulic structures such as intake towers and lock structures can be evaluated using nonlinear dynamic analysis. Nonlinear evaluation of reinforced concrete hydraulic structures involves step-by-step solution of equations of motion with nonlinear force-displacement relationship for members exhibiting nonlinear behavior. A nonlinear dynamic analysis of reinforced hydraulic structures may be carried out when the contribution of higher modes are significant. In situation like this, the pushover analysis, which simulates only the effects of the fundamental mode, is not appropriate. The nonlinear dynamic analysis of reinforced concrete hydraulic structures should be attempted when the structure can be idealized by frame elements for which nonlinear behavior is well defined. The structural models for the nonlinear dynamic analysis will be similar to those considered for the pushover analysis (see Examples D1 and D2), except that the seismic input will consist of two or three-component acceleration time histories. Accordingly the results will include displacement, stress, and force demand histories.

6-7. Design vs. Evaluation

Most existing hydraulic structures have been designed for load combinations that considered very little or no provisions for the effects of earthquake ground motions. For these structures, earthquake ground motion effects are often part of an evaluation process rather than the original design process. However, for design of new structures in highly active seismic regions earthquake loads could control the design. In situations like this, the structure must be designed for the effects of seismic loads and performance levels prescribed in Chapter 2. To accomplish this, the structural configurations and dimensions should be adjusted until the desired performance is achieved. In addition to such adjustments the design of reinforced concrete hydraulic structures should also involve practices that ensure ductile behavior while suppressing brittle failure modes. Every attempt should be made to:

(1) Meet minimum reinforcing steel requirements

(2) Provide adequate confinement at splice locations

(3) Provide adequate splice and anchor lengths

(4) Avoid locating splices in inelastic regions

(5) Provide direct and continuous load paths

6-8. Minimum Steel Design Requirements for New Reinforced Concrete Hydraulic Structures

For economy reason, structures located in high seismic regions should be designed to perform inelastically for the MDE ground motions. To ensure ductile performance, the flexural steel provided should as a minimum result in nominal moment strength equal to 1.2 times the cracking moment (i.e. $M_N/M_{cr} \geq 1.2$). This will provide displacement ductility greater than that exhibited by existing structures whose nominal moment capacities are less than cracking moment capacities. Structures detailed in accordance with modern seismic detailing practices and designed to have $M_N/M_{cr} \geq 1.2$ will have flexural displacement ductilities greater than those indicated by the DCR allowable values in Table 6-1a. The selection and use of higher DCR allowable values for such structures should be in consultation with CECW-ET. For concrete elements with deep cross-section dimensions, i.e. spillway piers for instance, the amount of reinforcing steel needed to satisfy minimum steel requirements may be excessive. When it is impractical to design for $M_N/M_{cr} \geq 1.2$, sufficient quantities of reinforcing steel should still be provided to ensure nearly elastic performance under the MDE. Meeting minimum reinforcing requirements ensures that plastic hinge cracking is not limited to a discrete location; rather it is spread out to improve plastic hinge rotational capability and displacement ductility.

6-9. Mandatory Requirements

a. Linear static and linear dynamic evaluations. Tables 6-1a and 6-1b establish maximum permissible demand to capacity ratios (DCR's) for the linear static and linear dynamic evaluations. The use of higher DCR allowable values must be justified on a project by project basis in consultation with CECW-ET.

b. Nonlinear static and nonlinear dynamic evaluations. Nonlinear static and nonlinear dynamic evaluations shall be performed when performance can not be assured by the linear static and linear dynamic evaluations (when DCR's exceed the allowable values). Nonlinear static and nonlinear dynamic evaluations shall also be performed when it becomes necessary to evaluate collapse performance for critical structures where the MDE demands are those of the Maximum Credible Earthquake (MCE).

c. Minimum reinforcing steel requirements. New reinforced concrete structures where the design is controlled by earthquake loadings shall be reinforced as required by Paragraph 6-7.

Chapter 7
Methods to Evaluate the Seismic Stability of Structures

7-1. Introduction

Structures must be evaluated with respect to sliding and rotation to ensure that they remain stable during an earthquake. Sliding stability of CHS under earthquake loading is evaluated using the limit equilibrium method (seismic coefficient) and permanent sliding displacement approaches (EM 1110-2-6050). Rotational stability of CHS under earthquake loading is evaluated using the energy-based formulation and the limit equilibrium method (EM 1110-2-6050). In addition to these methods, a new method based on rocking spectrum is introduced for assessment of rotational stability after a tipping of the structure has been indicated. All of these stability methods assume rigid structural behavior. This assumption is reasonable for most massive hydraulic structures, because the period of a sliding or rocking structure is much longer than the vibration period of the flexural response of the structure. However, the effects of structure flexibility on sliding and rotation could be important for more flexible and less massive structures and should be investigated. The structure flexibility can significantly affect the earthquake demands, which are used to determine whether or not sliding or rotation would take place. Sliding or rocking of a structure during an earthquake may not lead to failure of the structure. For a sliding failure to occur, the sliding displacement of the structure must be of sufficient magnitude to impair lateral load carrying capacity or life safety protection (for example, uncontrolled release of water from a reservoir). For a rotational stability failure to occur, the ground motion energy imparted to the structure after tipping occurs must be sufficient to cause rotational instability, or otherwise impair lateral load carrying capacity and life safety protection. Since bearing pressures can increase significantly as the resultant moves towards the edge of the base during a rotational response to earthquake ground motions, the load carrying capacity of the structure can be impaired due to a foundation bearing failure.

7-2. Rigid Structure vs. Flexible Structure Behavior

While a rigid structure will be subjected to a maximum acceleration equal to the peak ground acceleration (PGA) during earthquake ground shaking, a flexible structure will experience an average acceleration that depends on vibration period of the structure and on characteristics of the earthquake ground motion. This is illustrated by the acceleration response spectrum in Figure 7-1. The figure represents the typical acceleration responses of single-degree-of-freedom (SDOF) systems on a rock or firm soil site. Although most structures are not SDOF, a similar relationship can be assumed for the first-mode acceleration response of multi-degree of vibration systems. From Figure 7-1 it can be seen that only very rigid structures, with vibration period close to zero seconds, can be expected to experience peak accelerations equal to the PGA. For structures with periods between 0.02 seconds and 1 second (the typical range for most concrete hydraulic structures)

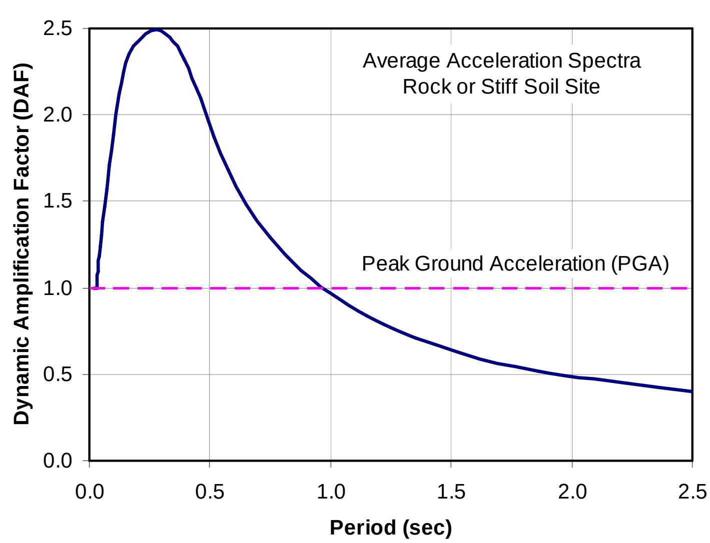


Figure 7-1. Dynamic Amplification Effects

the average structure acceleration will be greater than the PGA, with Dynamic Amplification Factors (DAF) as high as two to three.

7-3. Sliding Stability

a. Seismic coefficient method. In the limit equilibrium or seismic coefficient method, the sliding stability is expressed in terms of a prescribed factor of safety. A seismic coefficient, equal to 2/3 the peak ground acceleration divided by the acceleration of gravity (g), is used by the Corps to evaluate the potential for sliding. This coefficient when multiplied by the effective weight (structure weight + hydrodynamic added weight) provides the total lateral inertial force on the structure due to earthquake ground motions. The total lateral inertial force when added to static lateral forces, if any, provides the total driving force for the sliding stability analysis. The Maximum Design Earthquake (MDE) is considered an extreme load condition requiring a safety factor of 1.1 against sliding failure (Refer to EM 1110-2-2100 for stability requirements). A permanent sliding displacement analysis is required for structures that do not meet the required sliding factor of safety determined by the seismic coefficient method.

b. Permanent sliding displacement approach

(1) Upper bound estimate - rigid behavior. Sliding of a structure on its base will not occur until the total driving force exceeds the resisting force, or in other words when the sliding factor of safety is less than one. The total driving force can be due to static earth pressures, hydrostatic pressures, earthquake inertia forces, and earthquake induced hydrodynamic forces. Hydrodynamic forces are commonly determined by the Westergaard's added hydrodynamic mass (EM 1110-2-6051). The total mass of the system is therefore represented by the sum of the structure mass plus the hydrodynamic added mass. The static component of the driving force can easily be determined. The maximum inertia force for a rigid structure is a product of the total mass times the peak ground acceleration. The peak ground acceleration that will initiate sliding (i.e. when the driving force equals the resisting force) is defined as the critical acceleration. If the critical acceleration is greater than the peak ground acceleration of the design earthquake then the structure will not slide. Conversely, if the critical acceleration is less than the peak ground acceleration the structure will slide. An upper bound estimate of the permanent sliding displacement can be made using Newmark's rigid block analysis procedures (Newmark, 1965) or by methods developed by Richards and Elms (Richards and Elms, 1977). The Newmark procedure has been incorporated into the Corps program CSLIP. Newmark developed rigid block analysis procedures for rigid structures that slide in one direction only (dams, retaining walls, etc.) and for structure, which have the potential to slide equally in both directions (intake towers, lock monoliths, etc.). Newmark's sliding block analysis is demonstrated for a concrete gravity dam in Chopra and Zhang (1991), and the results from the Newmark analysis are compared to those obtained from a response history analysis. The potential for sliding, and the upper bound estimate of permanent sliding displacements can be reasonably determined using a Newmark-type sliding block analysis provided that the foundation sliding resistance is based on a best estimate (mean value) of the foundation shear strength, and that foundation shear strength parameters are adjusted for dynamic loading effects. Although the permanent sliding displacement is to be based on a mean shear strength value, permanent-sliding displacements should also be calculated using upper and lower bound estimates of foundation shear strength parameters.

(2) Upper bound estimate - flexible behavior. An approximate method based on rigid block analysis procedures (Chopra and Zhang, 1991) has been developed to estimate upper bound permanent displacements for flexible behavior. The analysis is similar to that used in the rigid

block analysis except that the sliding potential and estimate of upper bound displacements are based on the average peak structure acceleration rather than the peak ground acceleration. The average peak structure acceleration will generally be larger than the peak ground acceleration (see Figure 7-1) and therefore the upper bound permanent sliding displacement will be larger for a flexible structure than it is for a rigid structure. The average peak acceleration of the structure can be estimated by dividing the total first mode inertial force (base shear) obtained from a linear elastic response spectrum analysis by the total mass. Procedures for estimating average peak structure accelerations for flexible structures are provided in Chopra and Zhang, 1991.

c. Response history analysis procedures

(1) Linear time-history analysis – instantaneous factor of safety*.* The results of linear-elastic time-history analysis can be used to compute time-history or instantaneous sliding factor of safety along any desired sliding plane(s). The instantaneous factor of safety for the earthquake loading condition is obtained by combining the interface (i.e. sliding plane) force histories due to the earthquake loading with the interface forces due to the static usual loads plus the uplift. At each time step, the static and dynamic nodal forces are combined and then resolved into a resultant force having components normal and tangential to the sliding plane. The resisting forces are obtained from the normal component of the resultant force using the Mohr-Coulomb law, and the driving force is computed from vector summation of tangential components of the resultant force. The time-history of factor of safety is then obtained from the ratio of the resisting to driving forces at each time step. Figure 7-2 is an example of instantaneous factors of safety. The time-history starts at value equal to static factor of safety and then oscillates as the structure responds to the earthquake ground shaking. Under earthquake excitation, the stability is maintained and sliding does not occur if the factor of safety is greater than 1. However, a factor of safety of less than one indicates a transient sliding, which if repeated numerous times, could lead to excessive permanent displacement that could undermine safety of the structure. For example, Figure 7-1 shows that the factor of safety repeatedly falls below one, an indication that sliding of the structure could be expected. The magnitude of sliding displacement and its impact on the stability of the structure need to be evaluated by performing a nonlinear sliding displacement analysis discussed next.

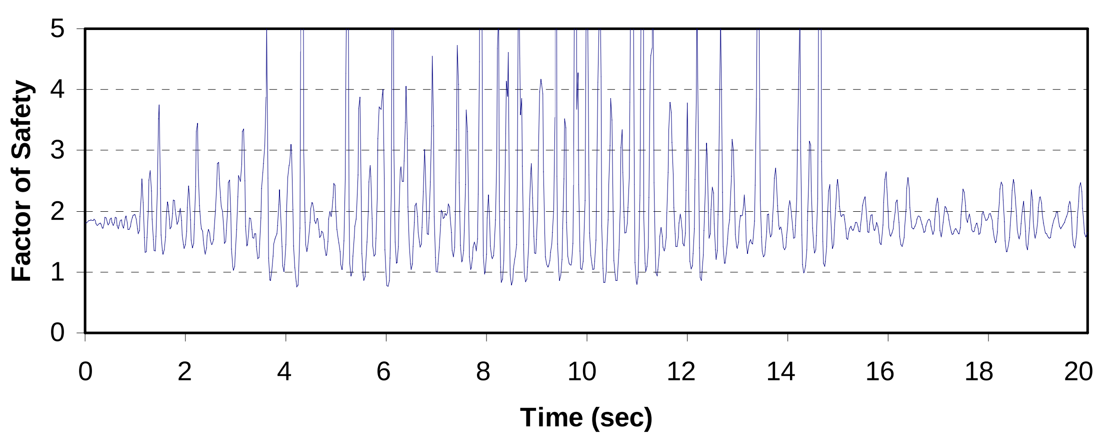


Figure 7-2. Time-history or instantaneous factors of safety

(2) Nonlinear time history analysis -- permanent sliding displacement. In nonlinear time-history analysis, governing equations of motion for the sliding structure are derived with respect to time and solved using step-by-step procedures (Chopra and Zhang 1991, Chavez and Fenves, 1993). A sliding structure is subjected to the ground acceleration plus the acceleration associated with the sliding displacement. If the sliding structure is assumed to be rigid, the governing equations involve dynamic equilibrium of inertia and static forces in the direction of sliding. Sliding is initiated when the acceleration reaches a critical or yield acceleration, i.e. a value at which the driving and resisting forces are equal; and the sliding ends when the sliding velocity becomes zero and the ground acceleration falls below the critical acceleration. If the sliding structure is flexible, two sets of governing equations will represent the sliding phase: 1) equations representing equilibrium of forces for the portion of the structure above the sliding plane, and 2) equations representing equilibrium for the entire sliding structure including all forces acting on the sliding plane. The structure's total permanent sliding displacement is then obtained by step-by-step solution of these coupled sets of equations. Alternatively, the nonlinear sliding behavior can be estimated using gap-friction elements along the sliding plane followed by a direct step-by-step integration of the equations of motion to obtain the total permanent sliding displacement.

7-4. Rotational Stability

a. General. A structure will tip about one edge of its base when earthquake plus static overturning moment (M_o) exceed the structure restoring moment capacity (M_r), or when the resultant of all forces falls outside the base. Depending on the magnitude of the peak ground acceleration, duration of main pulses, and slenderness of the structure, different rotational or rocking responses can be expected. As with sliding stability the inertia forces are likely to be larger for flexible structures than they are for rigid structures. Rotational or rocking responses to ground motions may include:

(1) No tipping because $M_o < M_r$

(2) Tipping or uplift because $M_o > M_r$, but no rocking due to insufficient ground motion energy

(3) Rocking response ($M_o > M_r$) that will eventually stop due to the energy loss during impact

(4) Rocking response that leads to rotational instability (extremely unlikely).

The likelihood of tipping can be determined by the following simple tipping potential evaluation. Even if tipping occurs, it is unlikely that it would result in rotational instability for the massive concrete hydraulic structures (Paragraph 7-4*d*). However, high bearing pressures can develop during tipping and rocking responses. A bearing failure evaluation is required to determine whether bearing pressures associated with the tipping and rocking responses could lead to foundation failure. Rocking spectrum and nonlinear time-history procedures are available to evaluate the potential for rotational instability (Paragraph 7-4d).

b. Tipping Potential Evaluation. Hydraulic structures subjected to large lateral forces produced by earthquakes may tip and start rocking when the resulting overturning moment becomes so large that the structure breaks contact with the ground. For a nearly rigid structure as shown in Figure 7-3, or for a flexible structure idealized as an equivalent single degree of freedom system, the tipping occurs when the overturning moment exceeds the resisting moment due to the weight of the structure. Note that in both cases it is assumed that the structure is not bonded to the ground, but it may be keyed into the soil with no pulling resistance. This condition is expressed by:

$$M_o > Mr$$

$$m\ S_a\ h > m\ g\ b \qquad \text{or,} \qquad S_a > g\ (b/h) \qquad (7\text{-}1)$$

where:

M_o = overturning moment

M_r = resisting moment

S_a = spectral acceleration

g = gravitational acceleration

b = one-half width of the structure

h = distance from the base to the center of gravity

m = mass of the structure

This expression can also be used for hydraulic structures, except that the moments due to hydrostatic and hydrodynamic forces should be included and that the added hydrodynamic mass of water be also considered in determination of the structure's center of mass.

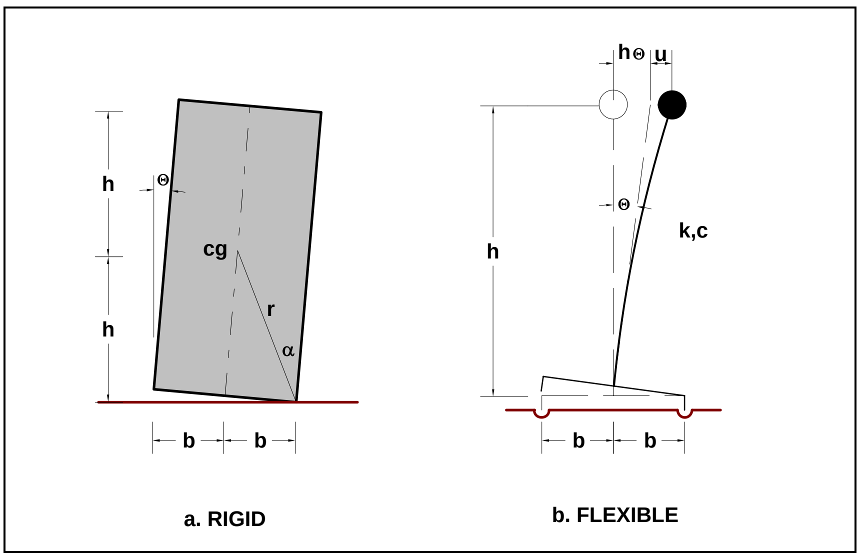


Figure 7-3. Rigid block and SDOF models for rigid and flexible structures

c. Energy Based-Rotational Stability Analysis. The structure will eventually overturn if the moment $M_o > M_r$ is applied and sustained. However, under earthquake excitation large overturning moments occur for only a fraction of second in each cycle, with intermediate opportunities to unload. Although rocking occurs, the structure may not become unstable rotationally if the energy loss during impact results in reduction of the angular velocity when the rotation reverses. By comparing the earthquake average energy input with the required average energy for overturning the structure, Housner provided the following approximate relationship as a criterion for the overturning stability of a rocking structure (Housner 1963):

$$\alpha = S_v \sqrt{\frac{mr}{gI_o}} \tag{7-2}$$

where:

α = angle between the vertical and the line segment R as illustrated in Figure 7-2.

r = distance from the center of gravity to the corner about which rotation occurs.

I_o = mass moment of inertia about that corner.

S_v = spectral velocity of the earthquake ground motion.

Based on the average energy formulation used, this equation is interpreted as stating that for a given spectral velocity S_v, a block having an angle α given by Equation 7-2 will have approximately a 50 percent probability of being overturned (Housner 1963). For slender structures such as intake towers Equation 7-1 can be approximated by:

$$\alpha = \frac{S_V}{\sqrt{gr}} \tag{7-3}$$

By combining Equations 7-1 and 7-3 and using the relationships among the spectral acceleration, velocity, and displacement, R. E. Scholl (ATC-10-01, 1984) found that consideration of one spectral parameter alone as the earthquake demand is not sufficient for evaluating overturning and suggested the following relationships:

$$S_d = b \quad \text{when} \quad S_d = g\frac{b}{h} \tag{7-4}$$

These equations show that when S_a is just sufficient to cause tipping, the structure will start rocking, but its displacement approximated by spectral displacement S_d must reach the value b before it can overturn. These equations also demonstrate why larger structures such as buildings do not overturn during earthquakes, whereas smaller rigid blocks having the same aspect ratio are expected to overturn. This is because, in general, S_d is never large enough to tip over a building, but it can approach one-half the base width (i.e. b) of smaller rigid blocks such as tombstones. A better and more accurate procedure for evaluation of rocking response is the use of rocking spectra and nonlinear time-history method described next.

d. Time-history and rocking spectrum procedures

(1) Time history and rocking spectra can be used to estimate the uplift or overturning of hydraulic structures that tend to undergo rocking motion (Makris and Konstantinidis, 2001). There are distinct differences between a SDOF oscillator and the rocking motion of a rigid block, as shown in Figure 7-4. As such, an equivalent SDOF oscillator and standard displacement and acceleration response spectra should not be used to estimate rocking motion of structures. For example, the restoring mechanism of the SDOF oscillator originates from the elasticity of the structure, while the restoring mechanism of the rocking block from gravity. The SDOF oscillator

has a positive and finite stiffness, k, and energy is dissipated as the force-displacement curve forms closed loops. The rocking block, on the other hand, has infinite stiffness until the magnitude of the applied moment reaches the restoring moment, and once the block is rocking, its stiffness decreases and reaches zero when the of rotation of the block becomes equal to α (the block slenderness). The vibration frequency of a rigid block is not constant because it depends on the vibration amplitude (Housner 1963). The vibration frequency $p = (3g/4R)^{1/2}$ is a measure of the dynamic characteristic of the block. It depends on the size of the block, R, and the gravitational acceleration, g. This indicates that rocking response cycles of larger block is longer than the corresponding rocking response-cycles of the smaller block.

(2) Governing equations. The governing equations of rocking motion under horizontal ground acceleration are given by Yim et al. 1980, Makris and Roussos 2000, among others):

$$I_o \ddot{\theta}(t) + m\,g\,R\,\boldsymbol{sin}(-\alpha-\theta) = -m\ddot{u}_g(t)R\,\boldsymbol{cos}(-\alpha-\theta), \qquad for \quad \theta < 0 \qquad (7\text{-}5)$$

$$I_o \ddot{\theta}(t) + m\,g\,R\,\boldsymbol{sin}(\alpha-\theta) = -m\ddot{u}_g(t)R\,\boldsymbol{cos}(\alpha-\theta), \qquad for \quad \theta > 0 \qquad (7\text{-}6)$$

which in its compact form can be expressed as:

$$\ddot{\theta}(t) = -p^2\left\{\boldsymbol{sin}\left[\alpha\,\boldsymbol{sgn}[\theta(t)] - \theta(t)\right] + \frac{\ddot{u}_g}{g}\boldsymbol{cos}\left[\alpha\,\boldsymbol{sgn}[\theta(t)] - \theta(t)\right]\right\} \qquad (7\text{-}7)$$

where for rectangular blocks;

α = $\tan^{-1}(b/h)$

I_o = $(4/3)\, m\, R^2$

P = $(3g/4R)^{1/2}$

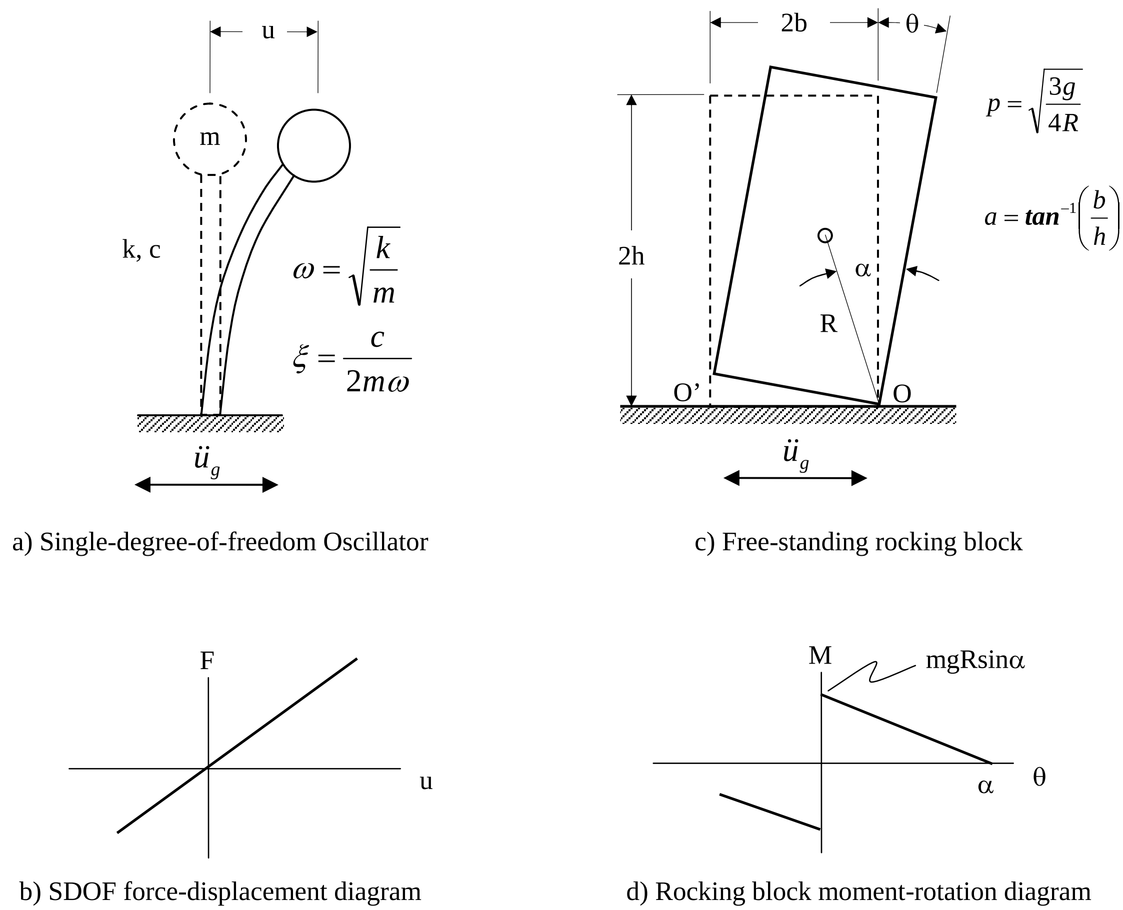


Figure 7-4. Comparison of a single-degree-of-freedom oscillator with a freestanding block in rocking motion (adopted from Makris and Konstantinidis, 2001)

(3) Time-history solution. The solution of Equation 7-7 is obtained by step-by-step numerical procedures. The rocking response quantity of interest include the block rotation, θ, and its angular velocity, $\dot{\theta}$. The resulting time-histories of θ and $\dot{\theta}$ will indicate how many impacts the block will experience and whether or not it will overturn (i.e. θ becomes greater than α).

(4) Rocking spectra. Same as the standard response spectra, one can generate rotational and angular velocity spectra (rocking spectra) as a function of the "period" $T=2\pi/p$ for different values of slenderness (damping), $\alpha = \tan^{-1}(b/h)$. This can be accomplished by solving Equation 7-7 for the maximum rotation of similar blocks of different sizes subjected to a given earthquake acceleration time history. This was done for similar blocks with $\alpha = 15°$ subjected to Pacoima Dam motion recorded during the 1971 San Fernando earthquake. The resulting rocking spectrum and the input acceleration record are shown in Figure 7-5. In the rocking spectrum, as $2\pi/p$ increases, the size of the block becomes larger. Larger values of the slenderness α correspond to larger amount of energy lost during impact. Figure 7-4 indicates that any block with slenderness α =15° that is small enough so that $2\pi/p<3.3$ sec (*or* $R<6.7$ ft) will overturn when subjected to the Pacoima Dam record. Larger blocks with $2\pi/p>3.3$ sec (*or* $R>6.7$ ft), will

uplift, but the maximum rotation is only a fraction of their slenderness. From this example, it should be obvious that rocking spectra provides a powerful and accurate tool for assessment of overturning potential of hydraulic structures. New research and development in this area are necessary to develop computation tools needed to make such assessments.

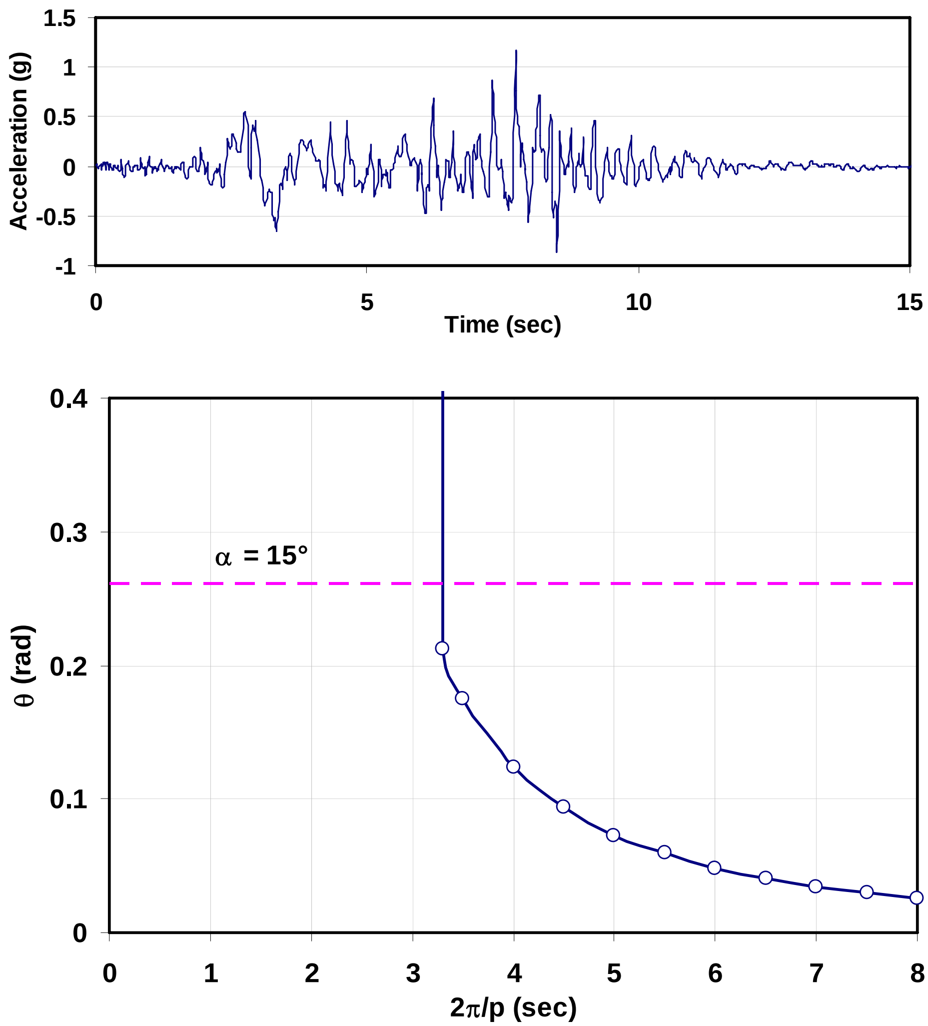


Figure 7-5. Pacoima Dam motion recorded during the 1971 San Fernando earthquake (top) and rocking spectrum of similar blocks with α = 15° (bottom).

7-5. Mandatory Requirements

a. Performance requirements for stability shall be in accordance with EM 1110-2-2100, Stability Analysis of Concrete Structures.

b. Seismic stability evaluation other than seismic coefficient method shall be in accordance with procedures discussed in this chapter.