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.

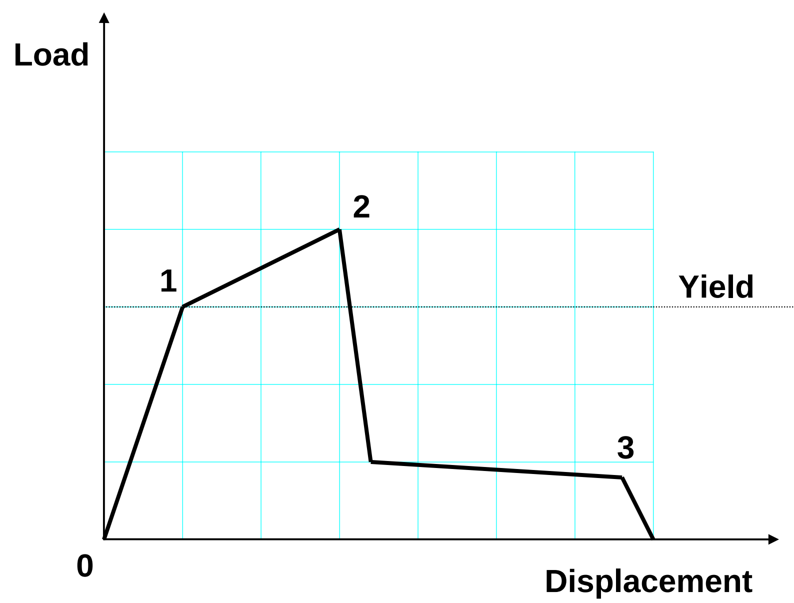

Figure 2-1. Ductile Behavior Curve (From FEMA 273)

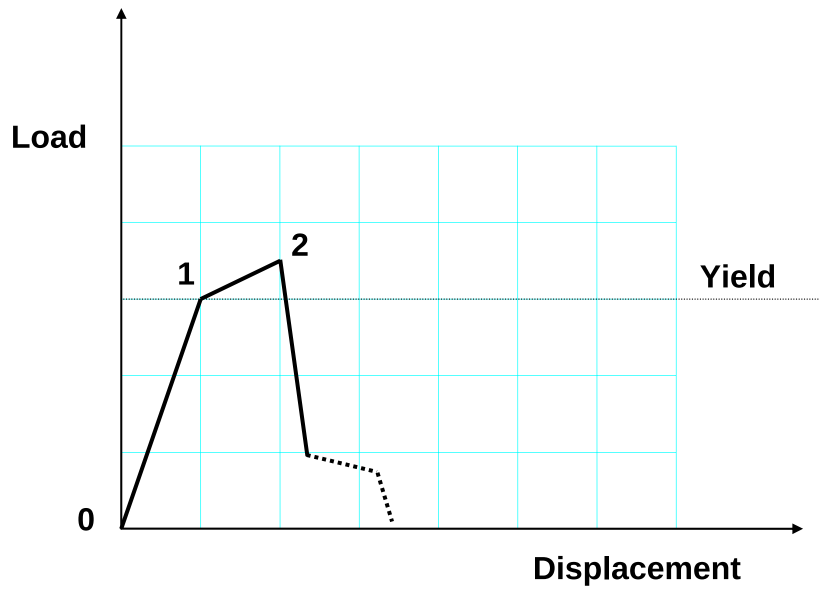

Figure 2-2. Limited-ductile Behavior Curve (consistent with FEMA 273)

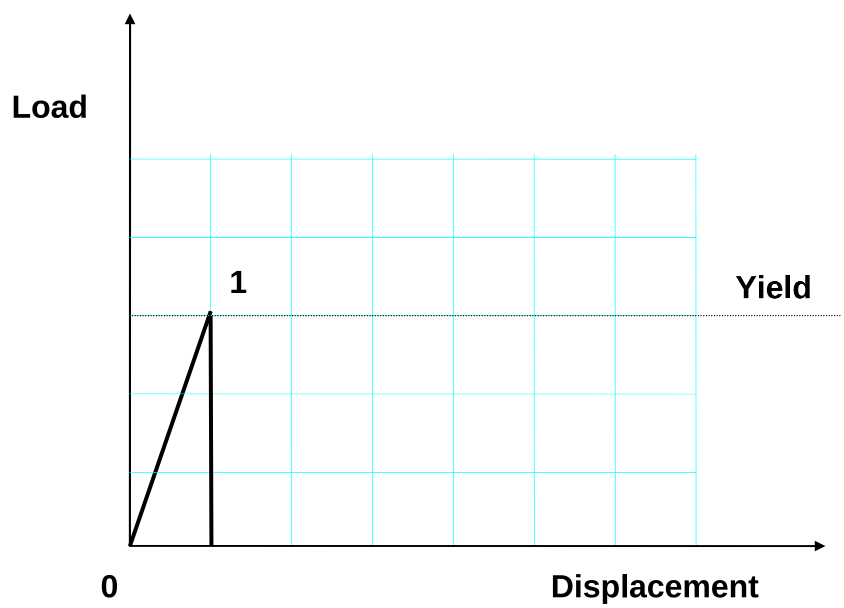

Figure 2-3. Brittle Behavior Curve (From FEMA 273)

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.

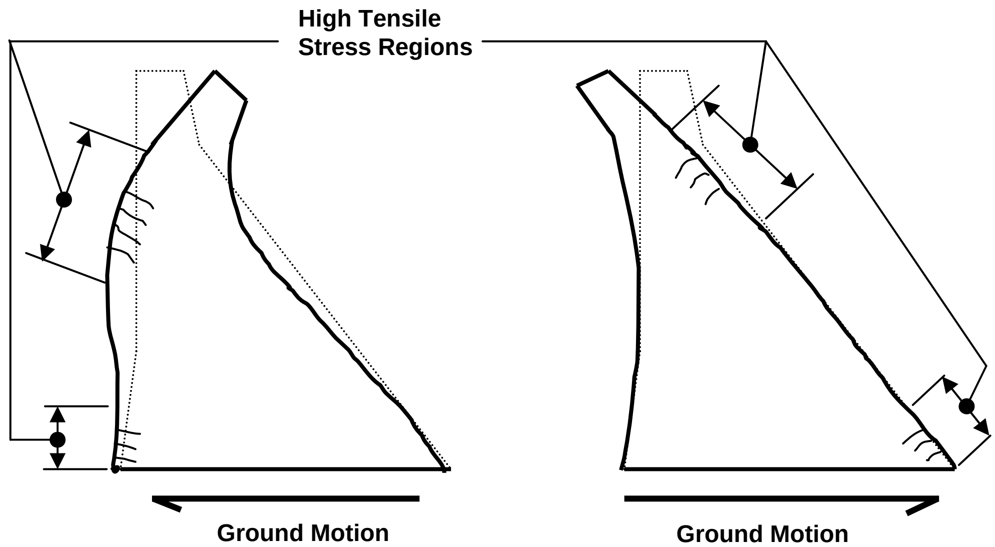

Figure 2-4. Gravity Dam Subjected to Earthquake Ground Motions

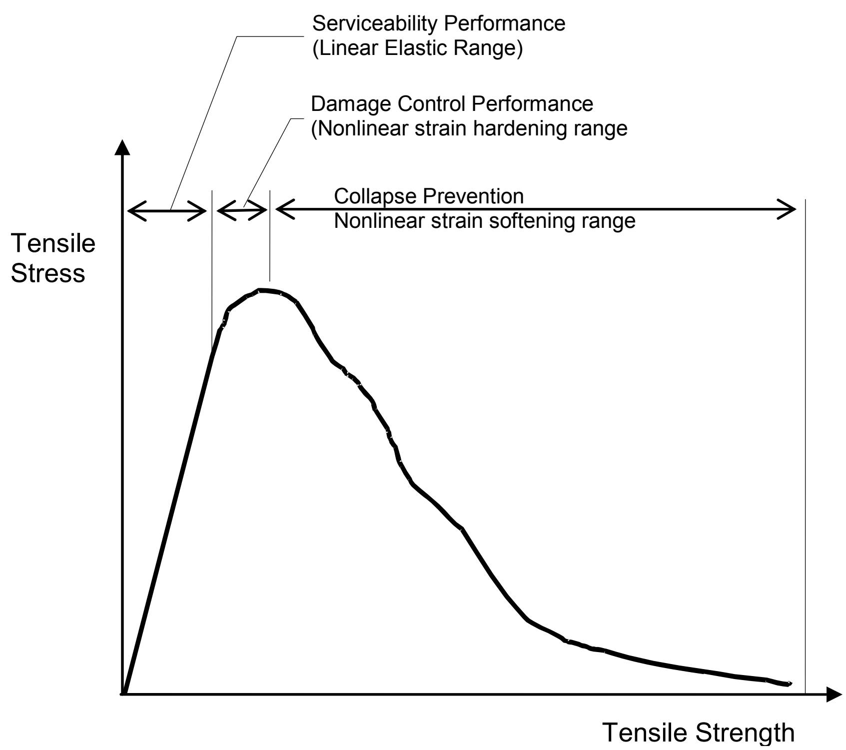

Figure 2-5. Stress – Strain Relationship for Plain Concrete Structures

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.

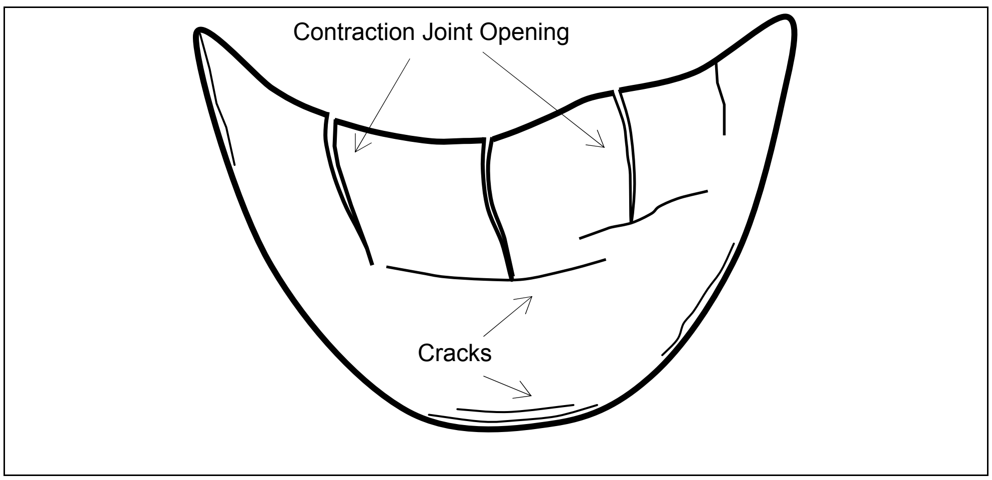

Figure 2-6. Response of Arch Dams to Major Earthquakes

(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.

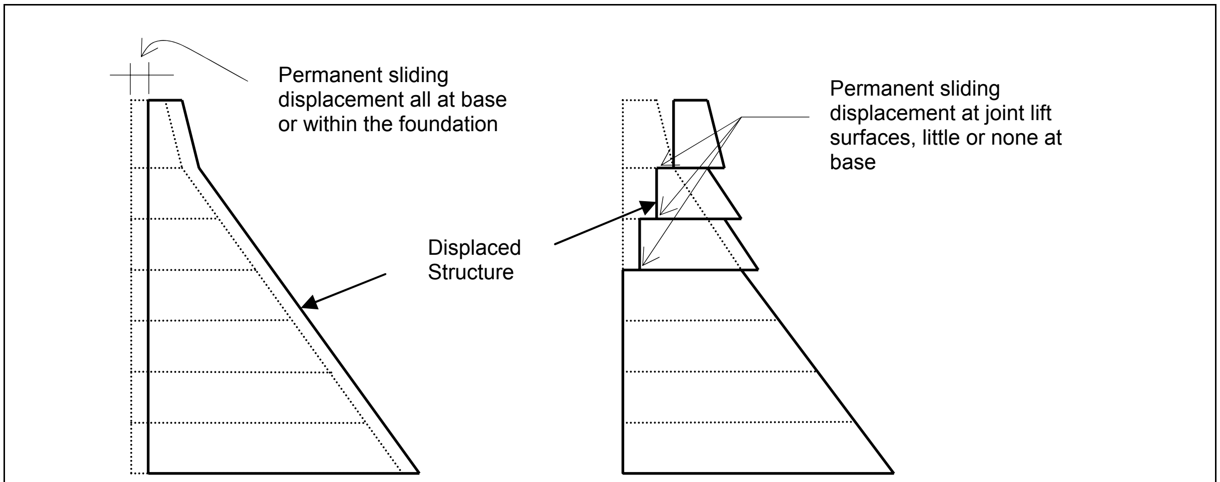

Figure 2-7. Dam Permanent Sliding Displacements

(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.

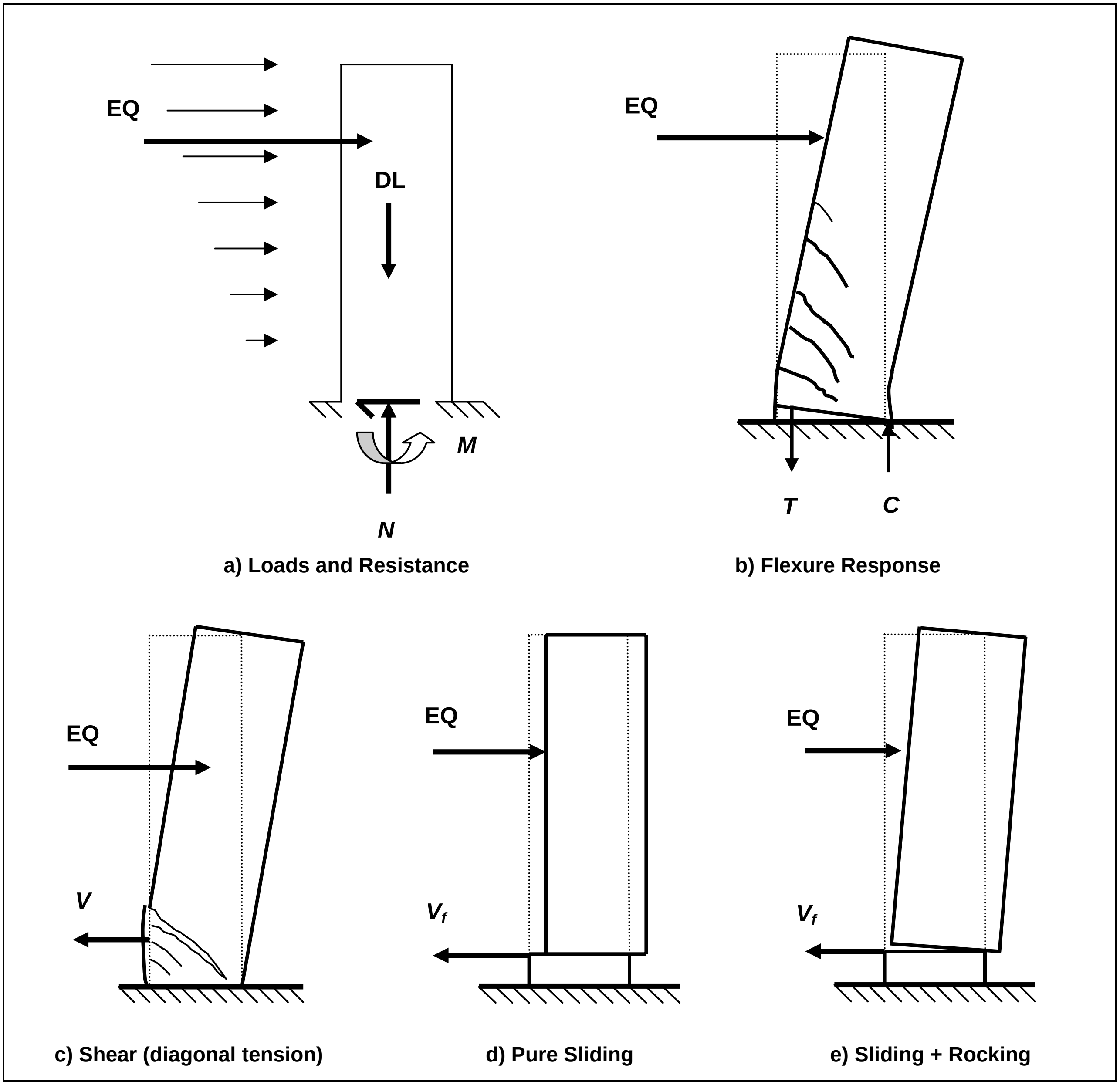

Figure 2-8. Response of a Free Standing Intake Tower to Earthquake Ground Motions

(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:

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

Where:

P = Axial load.

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

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

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.

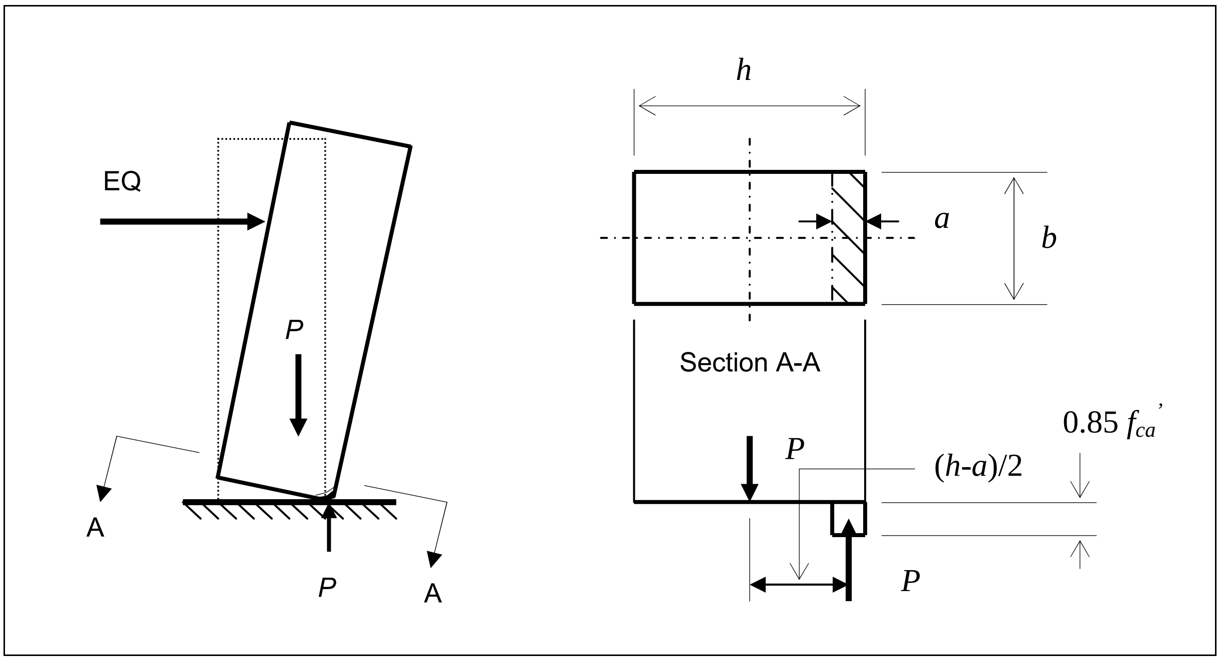

Figure 2-9. Toe Crushing Response of a Free Standing Intake Tower

(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.

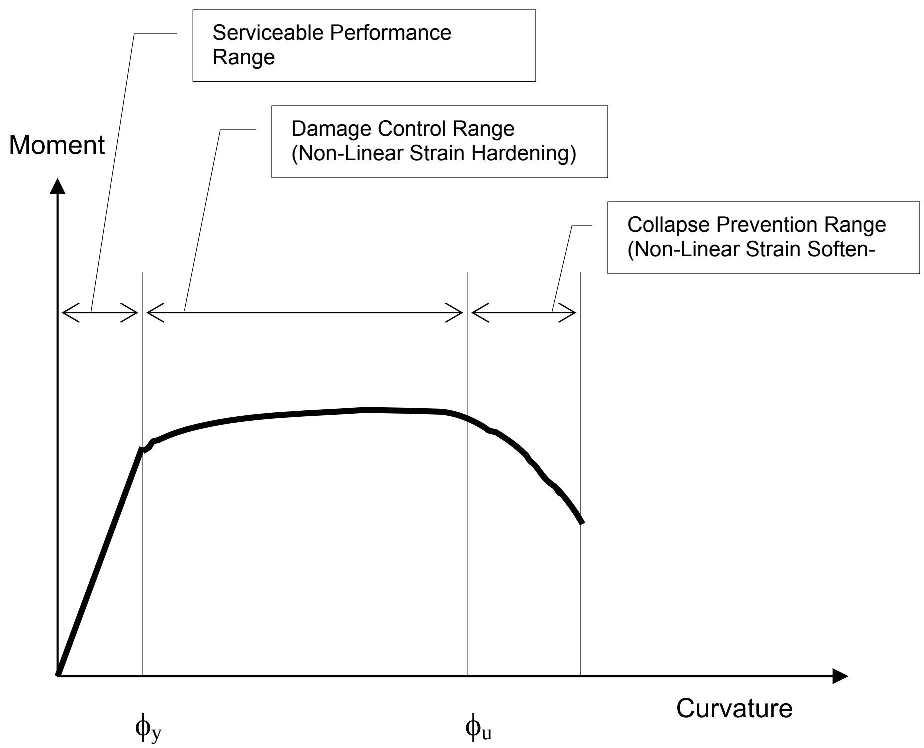

Figure 2-10. Moment-Curvature Diagram for Reinforced Concrete

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.