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.

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

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 $T \geq T_0$), $C_1 = 1.0$.

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

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

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:

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

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:

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

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.

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

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.

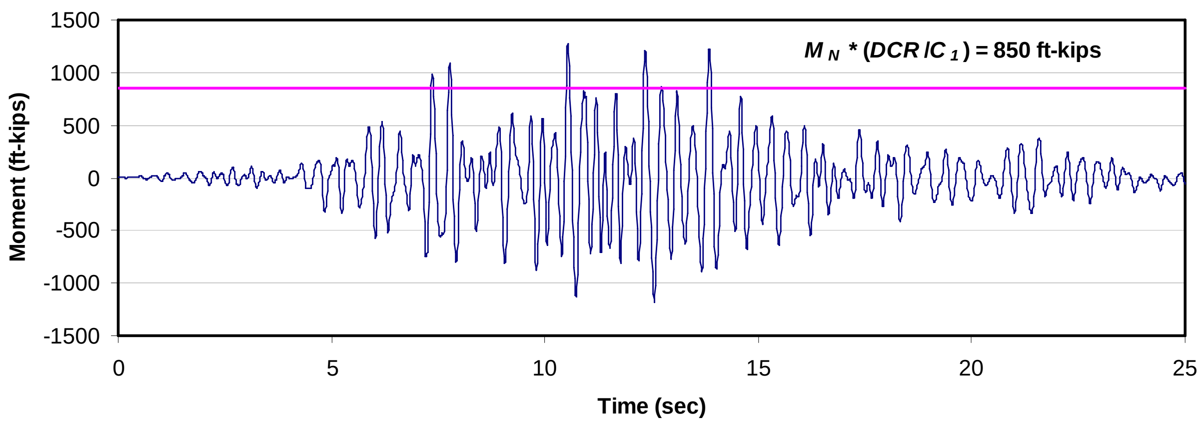


Figure 6-1. Moment Time History Evaluation

(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.

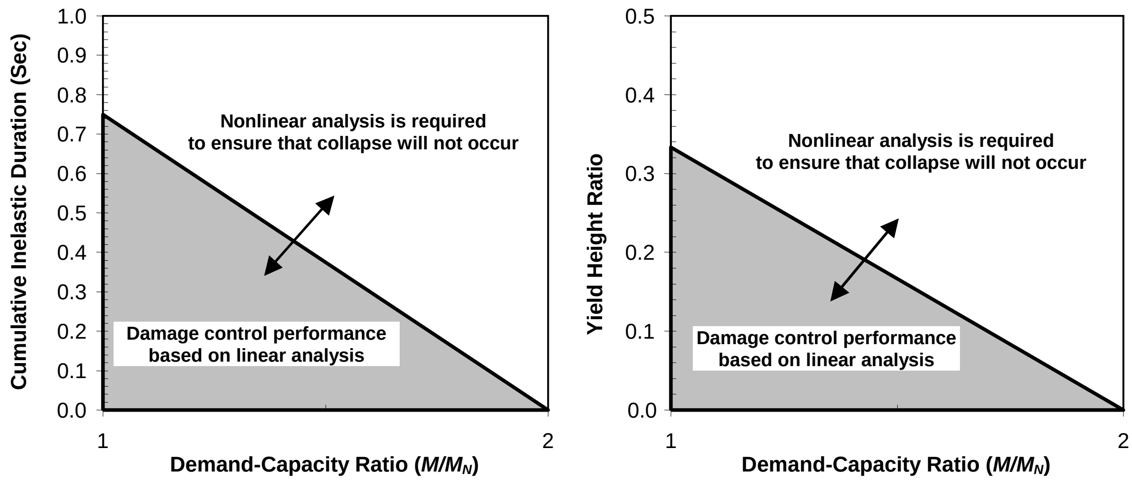


Figure 6-2. Acceptance Criteria for Freestanding Intake Towers

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.

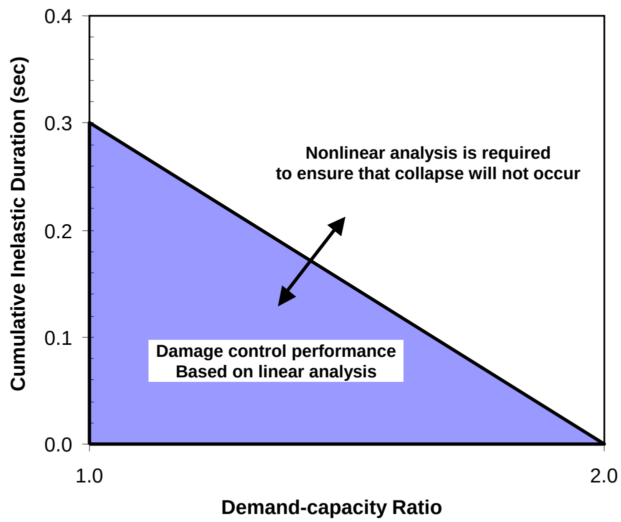


Figure 6-3. Performance Curve for Concrete Gravity Dams

(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.

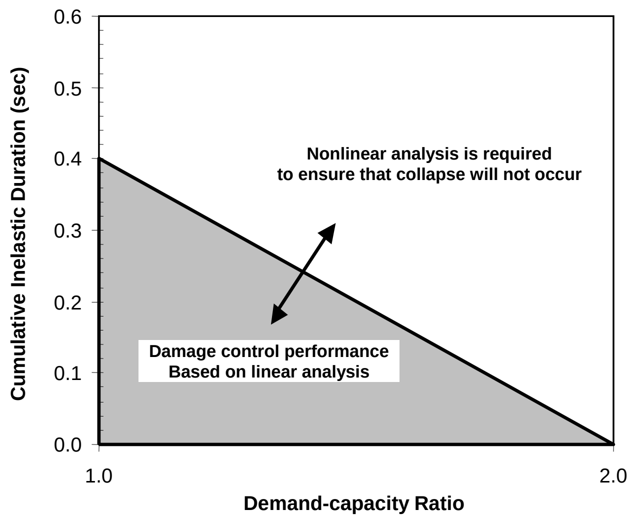


Figure 6-4. Performance Curve for Concrete Arch Dams

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

$$\mu_D = \frac{\delta_D}{\delta_y} \qquad (6\text{-}6)$$

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

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

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

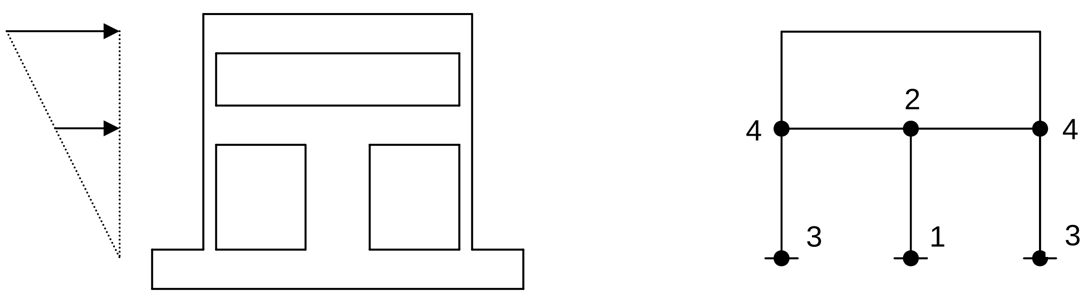
$$\mu_D = \left[\frac{M_D}{M_N} - 1\right]\frac{T_0}{T} + 1 \qquad (6\text{-}8)$$

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

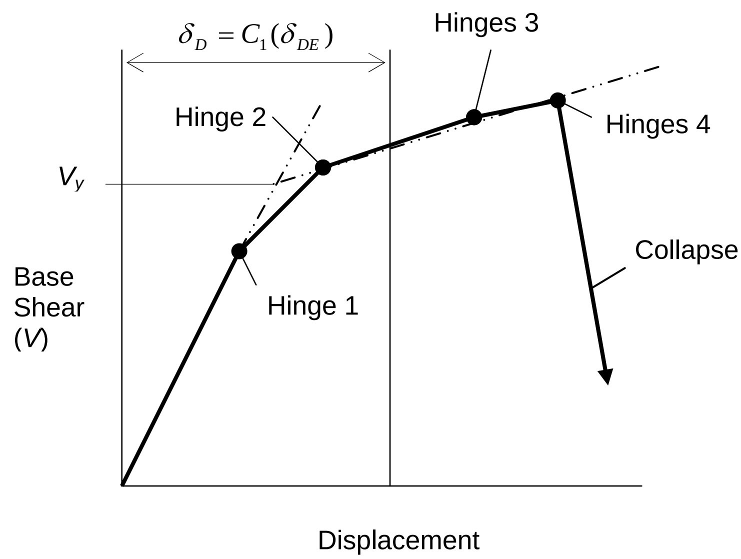


Figure 6-6. Force – Displacement from Pushover Analysis

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:

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

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

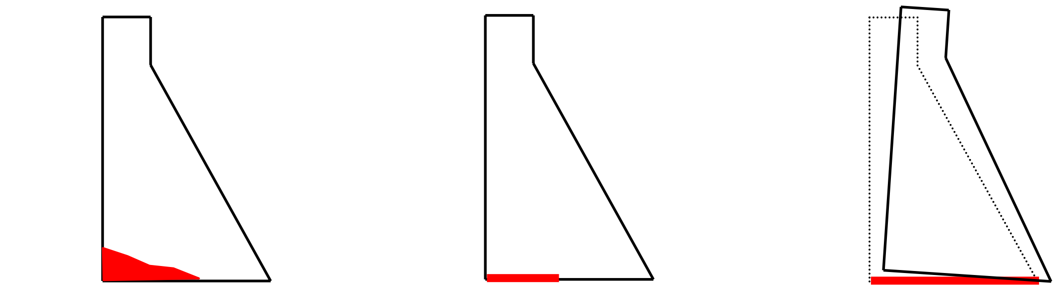
a. General. Under nonlinear dynamic analysis procedure, the induced displacements, stresses, and section forces (seismic demands) are obtained from the step-by-step solution of the equations of motion including nonlinear force-displacement relationship. Seismic demands in the form of response histories 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.