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:

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

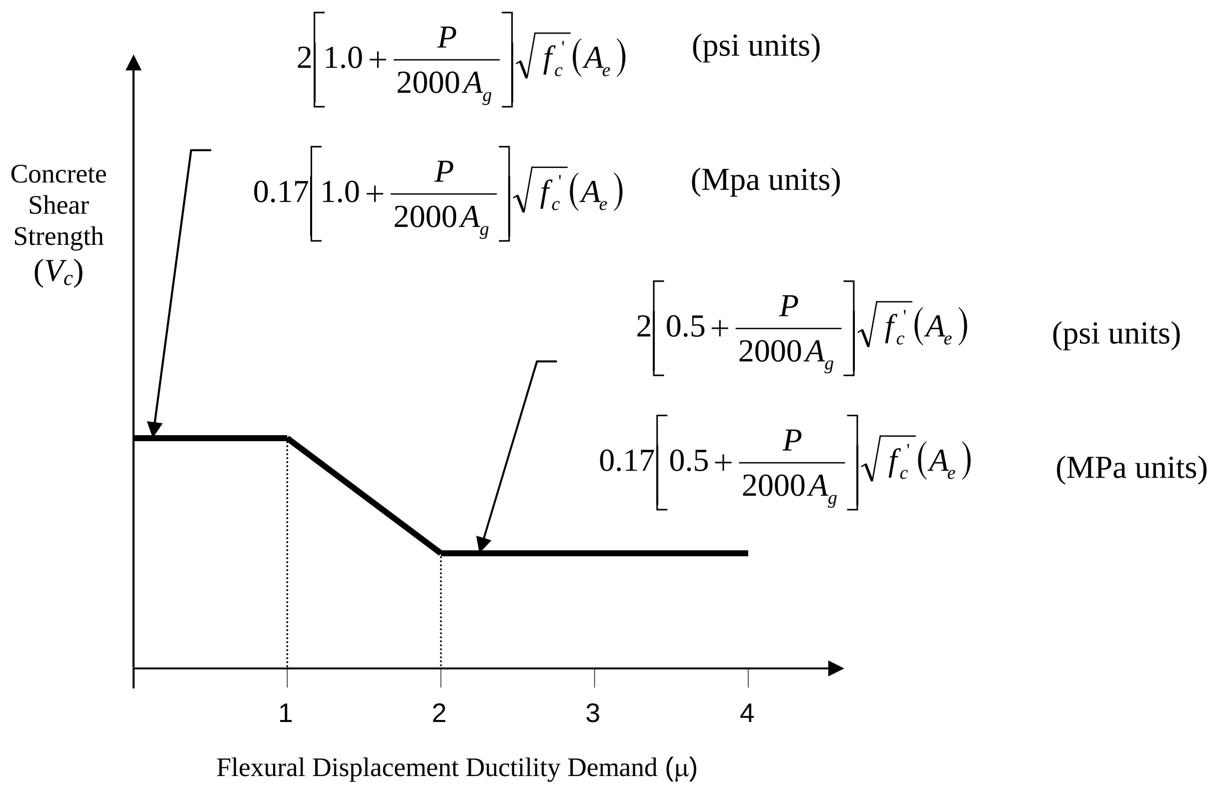

Figure 5-1. Concrete Shear Strength vs. Ductility

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:

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

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

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:

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

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

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.