CHAPTER X

OTHER DAMS

October 1997

Chapter X
Other Dams

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10-1 Introduction

10-1.1 Purpose and Scope

The guidelines presented in this chapter provide staff engineers with recommended procedures and criteria to be used in reviewing and evaluating the safety of the following types of dams:

- buttress dams;
- concrete dams on pile foundations;
- concrete dams on soil foundations;
- timber dams;
- inflatable dams;
- stone masonry dams;
- water retaining power plant structures;
- cellular sheet pile structures; and
- mechanically stabilized earth dams;

The staff engineers' review will be conducted to ensure that all decisions, methods, and procedures performed by licensees/exemptees, or their consultants, are sound regarding dam safety, and to ensure that the Commission's Dam Safety Program objectives as stated in Part 12 of the Commission's Regulations are consistent with accepted up-to-date procedures (the term licensees also refers to exemptees or applicants for license or exemptees from license where appropriate).

Safety evaluation of both new and existing dams presents special and unique problems. Existing dams may prove difficult to analyze especially in those instances where the dam was designed before the development of modern design and construction technology or where adequate records of design, construction, and material properties are not available, such as for timber dams, masonry dams, and piles. Existing dams should be viewed in light of knowledge of studies and reports on similar dams of the same vintage to gain an understanding of probable design and construction methods.

The objective set forth in this chapter is to provide systematic procedures for performing staff evaluations. It is not intended to generate any new philosophy or theories on the methods used to analyze these structures. Existing Bureau of Reclamation and Corps of Engineers literature as well as other readily available references have been used.

10-1.2 Review Procedures

Staff review of analyses performed by licensees or their consultants should concentrate on the justification of the assumptions used in the analysis. The basis for critical assumptions such as allowable stresses, shear strengths, seepage control, and loading conditions should be carefully examined. The consultant's reports, exhibits, and supplemental information must provide justification for these assumptions by way of foundation exploration and testing, material testing, instrumentation data, and records maintained during the actual construction of the project. Methods of analysis should conform to conventional procedures and standards used in the engineering profession.

As mentioned in the preface to the Engineering Guidelines, considerable engineering judgement must be exercised by staff when evaluating procedures or situations not specifically covered herein. Unique problems or unusual solutions may require deviations from the criteria and/or procedures outlined in this chapter. In these cases, such deviations must be evaluated on an individual basis in accordance with Chapter I, Section 1-4.

10-1.3 Forces

Forces to be considered in the design of the various dam structures included in this Chapter are discussed in detail in Chapter III - Gravity Dams of these Guidelines.

Forces generally consist of:

- dead loads (including post-tensioning);
- external hydrostatic loads;
- internal hydrostatic loads;
- earth and silt loads;
- earthquake loads;
- ice loads;

- wind loads; and

- temperature loads.

Many of the forces that must be considered in the design of the dam structure are of such a nature that an exact determination cannot be made. The intensity, direction, and location of these forces must be estimated by the designer after consideration of all available facts and, to a certain extent, must be based on judgement and experience.

Only exceptions, clarifications, and additions are included in the following sections.

10-1.4 Loading Combinations

Loading combinations and requirements suitable in general for gravity type dams are discussed in detail in Chapter III of these Guidelines. These include:

- Case I Usual Loading Combination - Normal Operating Condition.

- Case II Unusual Loading Combination - Flood Discharge.

- Case IIA Unusual Loading Combination - Ice.

- Case III Extreme Loading Combination - Normal Operating with MCE.

- Case IIIA Extreme Loading Combination (Unconstructed Dams Only) - Construction Condition with Earthquake.

Where a new dam is founded on compressible clay, a Case IV should be added.

- Case IV End of Construction Combination (Unconstructed Dams Only) - Dams founded on soil, or rock with clay layers.

For embankment type dams, loading combinations are discussed in detail in Chapter IV - Embankment Dams of these Guidelines. These include:

- end of construction;

- sudden drawdown;

- partial pool with steady seepage;

- steady seepage;
- earthquake; and
- appropriate flood surcharge pool.

Only exceptions, clarifications, and additions are included in the following sections.

10-2 Buttress Dams

Concrete buttress dams consist of a sloping upstream face supported by a series of triangularly shaped buttresses. Buttress dams evolved from concrete gravity dams. Since concrete gravity dams require only about 25 to 35 percent of the concrete to safely carry stresses to the foundation, buttress dams eliminate the extra concrete and at the same time eliminate most of the uplift pressures. If there is a base slab that spans between the buttresses, the uplift is not eliminated. Stability is attained by sloping the upstream face so that the weight of water acts as a stabilizing load. The design of buttress dams is discussed in several references (Boggs, Jansen, and Tarbox 1988; Burroughs 1969; Copen, Lindholm, and Tarbox 1977; Corns, Tarbox, and Schrader 1988; Creager 1917; Creager and Justin 1927; Creager, Justin, and Hinds 1945; Davis 1969a, b, c; Houk and Wengler 1969; Legas 1988; Marcello 1969; Thomas 1976; Wegeman 1927).

Buttress dams are economical when 1) labor and forming are inexpensive, 2) concrete is expensive, 3) materials need to be hauled to a remote site, or 4) construction water is scarce. Since construction of new buttress dams is generally not economical in modern times, this section focuses on evaluation of existing buttress dams.

10-2.1 Types of Buttress Dams

The most common types of buttress dams are discussed below. Many different names have been used for various types of buttress dams and some of them are listed in parentheses to help reduce confusion when consulting the literature.

Flat Slab Dam (a.k.a. Ambursen Dam, Deck Dam). A buttress dam in which the upstream face is a relatively thin flat slab usually made of reinforced concrete. Typically, the slabs are simply supported and are not integral with the buttresses making the structure relatively flexible. As a result, ordinary foundation movements have little effect on stress distributions. A compressible material and waterstop are normally provided between the slab and the corbel of the buttresses to allow movement yet remain watertight. The slab is continuous in some designs, making the structure more rigid.

<u>Multiple Arch Dam</u>. A buttress dam in which the upstream face is a series of arches spanning between buttresses. The arches are generally semi-circular with central angles between 100 to 180 degrees, though non-circular arches have been used. The arches may be unreinforced or reinforced. The arches may be integral with the buttresses, making the structure rigid and susceptible to damage from even small foundation movements. Alternatively, the arches may be structurally independent of the buttresses, making the structure somewhat flexible. One variation that has been used is a multiple dome buttress dam, in which the arch is curved both in plan and section.

<u>Hollow Gravity Dam</u>. In modern usage, a dam that has the external appearance of a gravity dam, but which has large open areas inside. These are usually buttress dams in which the downstream portion is covered with a reinforced concrete slab. They are often used for overflow spillway sections of buttress dams or as housing for a powerhouse.

<u>Massive Head Buttress Dam</u> (a.k.a. Round Head, Diamond Head, Massive Buttress, Cored Gravity, Hollow Gravity [archaic definition], Tee Head, Hammer Head). A buttress dam in which the upstream end of each buttress (head) is enlarged to span the gap between buttresses. Although common in Europe, there are very few massive head buttress dams in the United States and none under FERC jurisdiction. Therefore, the guidance in this section applies specifically to slab and buttress or multiple arch buttress dams and not necessarily massive head buttress dams.

There are many variations within the basic types of buttress dams that can have significant influences on the structural behavior of the dams. Therefore, the structural details of each dam must be established before the safety of the dam can be properly evaluated. Typical buttress dam variations include:

- thin upstream faces with thin, closely-spaced buttresses to massive upstream faces, with massive, widely-spaced buttresses;

- completely unreinforced upstream faces and buttresses to heavily reinforced upstream faces and buttresses;

- constant thickness upstream faces and buttresses to upstream faces and buttresses that increase in thickness with depth;

- upstream faces structurally independent of the buttresses to upstream faces integral with the buttresses;

- straight upstream faces to upstream faces that have a steeper slope near the top;

- single wall buttresses to double wall buttresses with internal bracing;

- laterally unbraced buttresses to buttresses braced with pilasters, flanges, struts, and shear walls;

- monolithic buttresses to buttresses with inclined or longitudinal joints that separate the buttress into a series of inclined columns; and

- buttresses founded directly on rock to buttresses founded on spread footings or continuous slabs.

10-2.2 Typical Problems

Many buttress dams were designed and built over 50 years ago using outdated methods, loads, and acceptance criteria. As a result, a number of these structures have been, or will need to be, modified to overcome inadequate stability or excessive stresses. The most critical areas tend to be concrete deterioration from freeze-thaw damage, poor quality concrete, inadequate design, and inadequate seismic stability. Case histories of problems and resolutions to the problems are common in the literature (Garland, Waters, Focht, and Rutledge 1995; Rohde and Zuccolotto 1995; Reynolds, Joyet, and Curtis 1993; Niziol and Paolini 1993; Lamar, Ivarson, and Tenke-White 1991; Wheelock and Wilkins 1991; and Bengtson 1989).

Freeze-thaw damage is common in cold climates because the upstream face is generally saturated and the downstream side of the upstream face is exposed to seasonal temperatures. Non air-entrained concrete was commonly used in older buttress dams and contributes to freeze-thaw damage. Severe freeze-thaw damage can jeopardize the safety of the dam because of the relative thinness of the upstream faces.

The strength of the concrete has been found to be quite low (less than 2,000 psi) and variable in some buttress dams. These conditions have been attributed to insufficient cement, excessive water, and poor quality control during construction. Such low and variable strengths significantly impact buttress dam safety because the concrete is relatively highly stressed.

Thin, unreinforced, upstream faces and buttresses cannot tolerate significant deflection or deterioration and are, therefore, highly susceptible to damage from a variety of common conditions.

Multiple arch buttress dams, in which central angles of individual arches are less than 180 degrees, impose significant lateral load on buttresses. This load is resisted in part by adjoining arches. As a result, failure of one arch could cause catastrophic failure of the entire dam.

Buttresses, especially unreinforced or unbraced buttresses, are susceptible to damage from lateral earthquake loading.

10-2.3 Forces

Forces for the analysis of buttress dams are generally the same as for concrete gravity dams, which are discussed in Chapter III, Section 3-2 of these Guidelines. Only exceptions, clarifications, and additions are discussed below.

10-2.3.1 Dead Loads

Dead loads include the weight of the upstream and downstream face and the weight of the buttresses. Dead load is carried by the buttresses regardless of the connection detail, because the normal pressure on the sloping contact times any reasonable friction angle is sufficient to prohibit sliding of the slab with respect to the buttress. Consider the example shown in Figure 10-2.1. Note that the angle of the contact force between the slab and the buttress is less than 14°. This means the contact force alone will keep the slab pinned against the buttress.

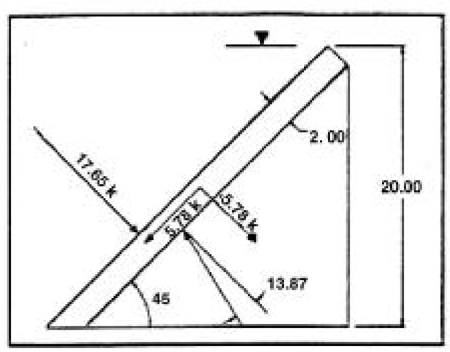

Figure 10-2.1 Angle of Contact Force Between Slab and Buttress

10-2.3.2 External Hydrostatic Pressures

The effects of hydrodynamic pressures on a slab and buttress dam must be considered with more care than one would typically apply when considering a gravity dam. As water flows over the surface of an ogee crest, hydrodynamic forces are produced which can be much greater than the depth of flow, or which can exert a negative pressure which tends to lift the slab off of the buttresses (Figure 10-2.2). Bucket pressures can approach the value of the jet stagnation pressure, and negative pressures near the crest of the dam can be sub-atmospheric. While these forces pose no problem to the mass concrete of a gravity dam, the thin slabs of a slab and buttress dam may not have been sufficiently reinforced to resist the large hydrodynamic pressures associated with flood loading. In addition, since these dams are hollow, proper air venting of the interior of the dam must be assured. During a flood event with a quickly rising tailwater, an air pocket can be formed within the dam that will exert upward pressure on the dam (Figure 10-2.3). This also can cause the slabs to be pulled off the buttresses, and this upward load has a detrimental effect on overall stability.

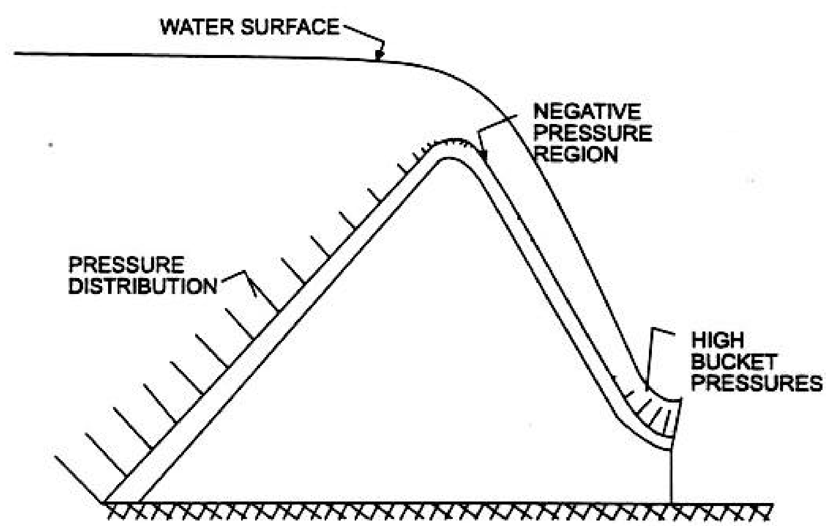

Figure 10-2.2 Hydrodynamic Pressures on a Buttress Dam

10-2.3.3 Internal Hydrostatic Loads

If a buttress dam is founded on a continuous slab, uplift pressures at the concrete-to-rock interface should be treated as discussed for concrete gravity dams in Chapter III, Section 3-2.4 of these Guidelines.

If the buttress es are founded directly on rock or on spread footings, the uplift at the concrete-to-rock interface will be substantiall y reduced because the open areas between the buttresses provide drainage. For these cases,

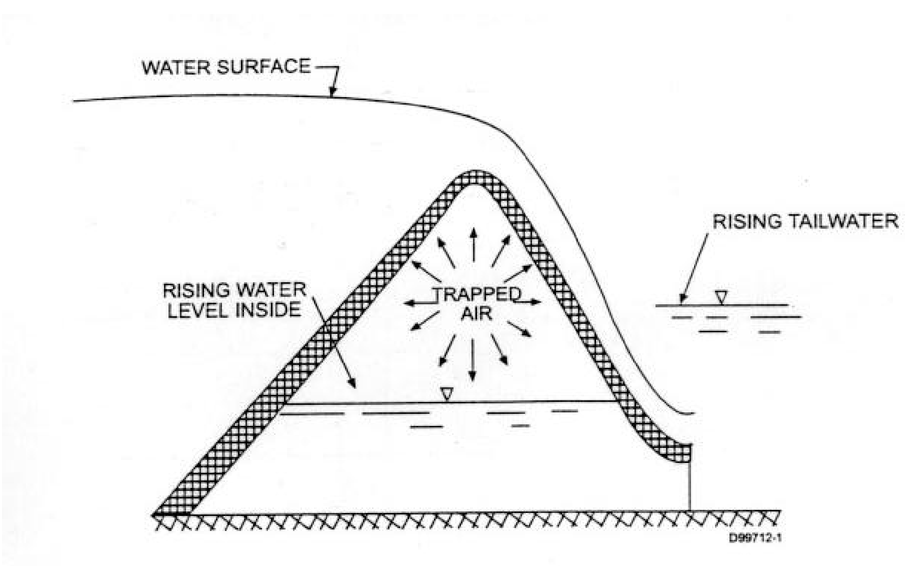

Figure 10-2.3 Entrapped Air in a Buttress Dam

uplift may be assumed to vary from headwater pressure at the upstream face to tailwater pressure at the downstream edge of the upstream face slab or arch. Uplift pressure beneath the remaining portion of the buttress or buttress footing may be assumed to be tailwater pressure.

When examining failure planes within the foundation, the uplift pressures should be treated similar to uplift pressures for concrete gravity dams. If shallow, subhorizontal discontinuities exist, uplift pressures should be calculated using cracked base type analyses.

10-2.3.4 Earthquake Forces

Earthquake forces are the same as for concrete gravity dams. Separate analyses should be run with the horizontal accelerations applied in the transverse (upstream-downstream) and longitudinal (cross-valley) directions. Vertical acceleration should also be considered. For application of the seismic coefficient method to buttress dams, the upstream slope of the dam effects the hydrodynamic forces resulting from an earthquake. Zangar (1952) provides curves for determination of increased pressure for various upstream slopes.

Figure 10-2.4 is reproduced from Engineering Monograph No. 11, US Bureau of Reclamation, Hydrodynamic Pressures on Dams Due to Earthquake Effects, by Zangar, May 1952. This figure provides the maximum value of coefficient C and the values of the coefficient at the base of dams for various inclinations of the upstream face from the vertical, as defined on Figure 10-2-4.

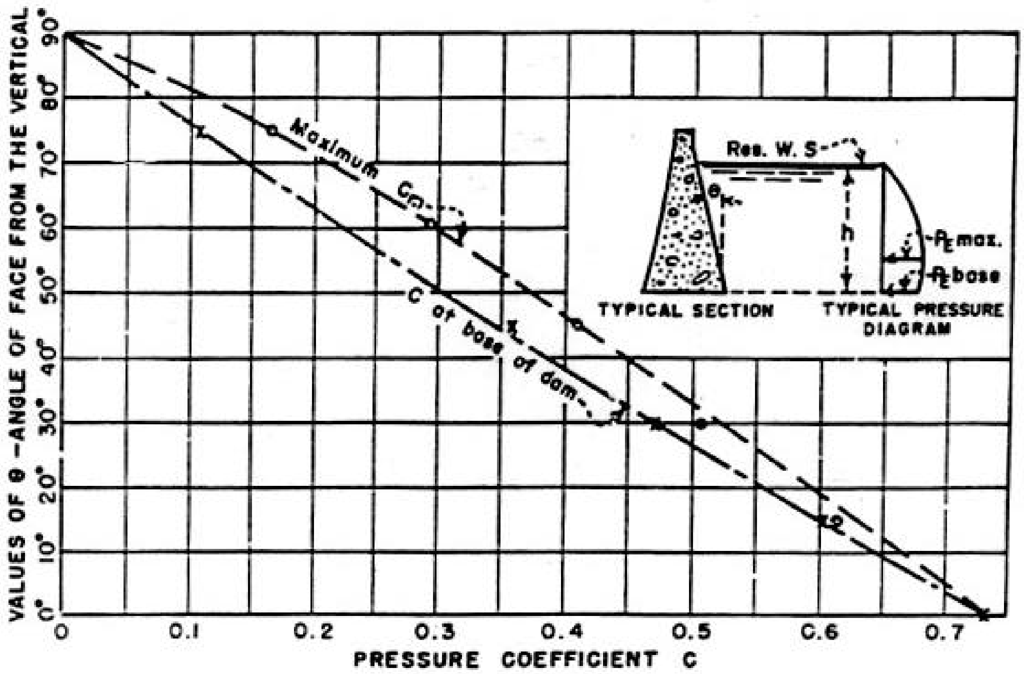

Figure 10-2.4 Increased Pressure Coefficients For Constant Sloping Faces

For dams with constant upstream slopes, the increase in pressure due to a horizontal earthquake may be estimated, according to Zangar (1952), from:

$$P_e = \tfrac{1}{2} \propto w h \;\; Cm \; \left[y/h(2-y/h) + \sqrt{y/h\,(2-y/h)} \;\right]$$

Where:

P_e is the increase in pressure in pound per square foot,

∝ is the horizontal earthquake intensity,

w is unit weight of water, pounds per cubic feet,

h is the height of dam above the base in feet,

y is the depth at which the pressure increase is being determined, and

Cm is the maximum value of C obtained from Figure 10-2.4.

At the base of dam, y = h, and the expression for P_b (P_e at the base) becomes:

$$P_b = \propto w\ h\ Cm$$

And the total horizontal force, in pounds per foot, above the base is:

$$V_b = 0.726\ P_b\ h$$

And the total over-turning moment, M_b, in foot-pound per foot, is:

$$M_b = 0.299\ P_b\ h^2.$$

10-2.3.5 Ice Pressures

The sloping upstream faces of the buttress dams tend to cause ice to fail by bending rather than crushing and as a result the magnitude of the ice forces are significantly reduced. A study of ice forces on sloping structures shows that ice forces depend on the ice strength, ice thickness, coefficient of friction between ice and concrete, and inclination of the slope as illustrated by Figures 10-2.5 and 10-2.6. Reduced ice forces can be computed according to the simple two-dimensional theory presented by Croasdale (1980):

$$\frac{H}{b} = \sigma_f \left(\frac{\rho_w\ g\ t^5}{E} \right)^{1/4} C_1 + Z\ t\ \rho_i\ g\ C_2$$

where:

H	=	horizontal force
b	=	width of structure
σ_f	=	flexural strength of ice
ρ_w	=	density of water
g	=	force of gravity
t	=	ice thickness
E	=	elastic modulus of ice
Z	=	height of ice on slope
ρ_i	=	density of ice

where, C_1 and C_2 are functions only of the slope angle (α), and the coefficient of friction between ice and concrete (μ). Values for the coefficients C_1 and C_2 are plotted in Figures 10-2.7 and 10-2.8 and for typical values of α and μ.

Typical properties of fresh water ice are listed below (Rice 1975):

Modulus of elasticity, E:

Strain rate (s^{-1})	E, psi
1×10^{-8}	400,000
1×10^{-6}	700,000
1×10^{-4}	1,000,000

Poissons ratio: 0.33

Tensile strength: 30 to 150 psi

Compressive strength:

Strain rate (s^{-1})	Max. Stress (psi)
1×10^{-4}	1,400
3×10^{-7}	50

Shear strength: 50 to 150 psi

In the simple two-dimensional theory given above, the first term can be considered to be the force necessary to break the ice, and the second term can be considered to be the force necessary to push the ice pieces up the sloping structure. As a 2-D theory it might be considered accurate for a very wide structure, but it is probably inaccurate for narrow structures.

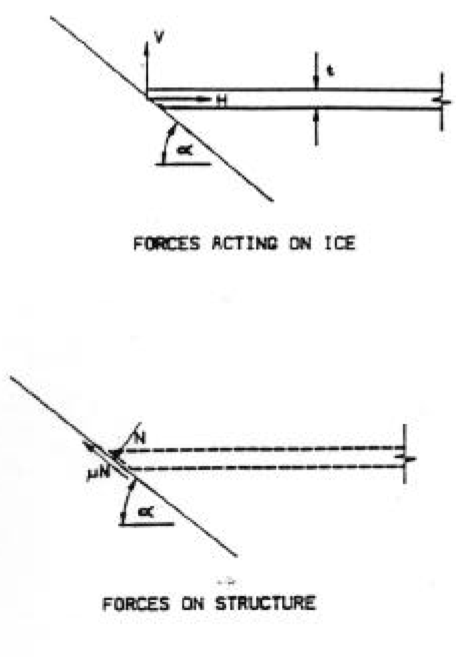

Figure 10-2.5. Initial interaction between ice and sloping structure.

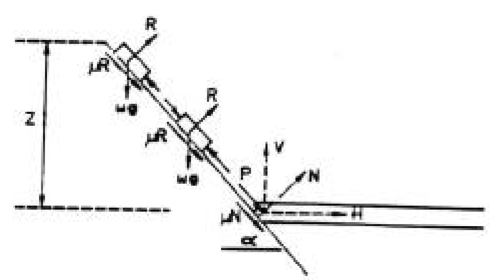

Figure 10-2.6. General interaction between ice and sloping structure.

For example, let:

σ_f	= 700 kPa
$\rho_w g$	= 9.8 kN/m^3
t	= 0.6 m
E	= 7 x 10^6 kPa
Z	= 1.5 m
ρ_i g	= 9.0 kN/m^3
$\propto$	= 45 degrees
μ	= 0.3

then, $C_1 = 1.25$, $C_2 = 2.6$

$$\frac{H}{b} = 700\left(\frac{9.8(0.6)^5}{7x10^6}\right)^{1/4} 1.25 + 1.5(0.6)(9.0)(2.6) = 16 + 21 = 37\frac{kN}{m}$$

10-2.3.6 Temperature

Temperature effects are not significant for many buttress dams, because the relatively flexible nature of buttress dams allows thermal stresses to be relieved through slight deformations. Temperature effects need be considered only if the dam is not reinforced, or there is physical evidence of overstressing from temperature loads.

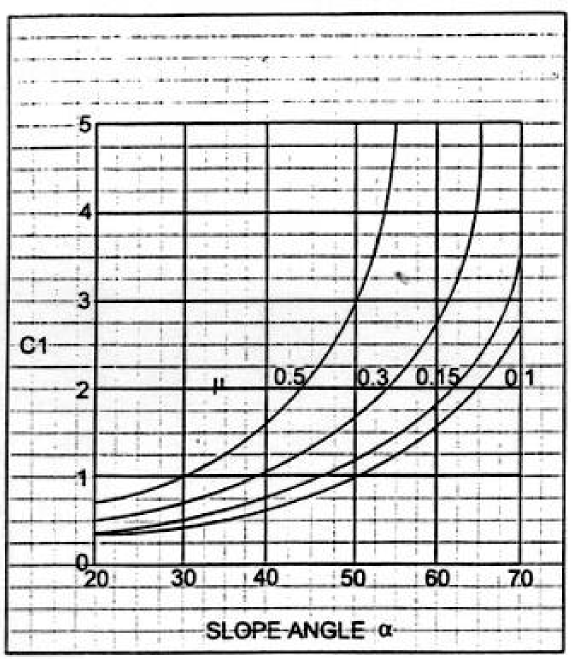

Figure 10-2.7. C_1 vs slope angle and friction.

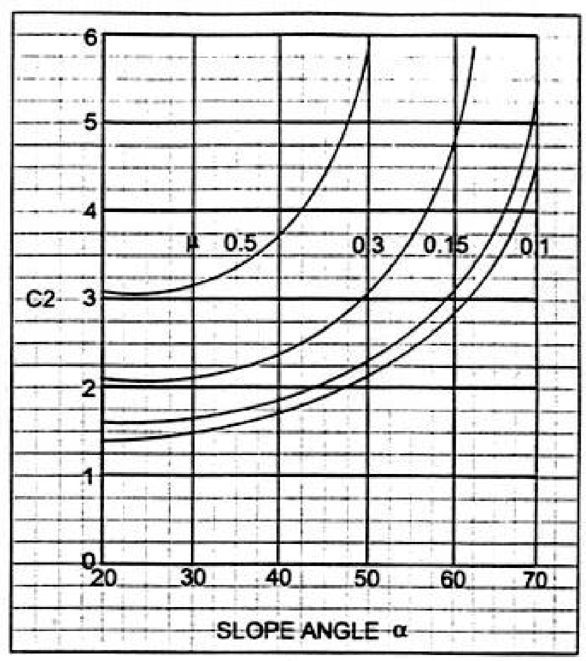

Figure 10-2.8. C_2 vs slope angle and friction.

Source: Croasdale (1980)

Excessive thermal stresses will usually be apparent from cracking at the connection between the buttress and the upstream face slab or arch. Thermal expansion of a buttress tends to cause horizontal cracks in the upstream face slab or arch. Thermal expansion of an upstream face slab or arch tends to cause cracks in the buttress perpendicular to the upstream face.

10-2.4 Loading Combinations

Loading combinations for the analysis of buttress dams are the same as for concrete gravity dams discussed in Chapter III, Section 3-3 of these Guidelines, except that the need to include temperature effects should be evaluated. Where temperature effects may be significant, temperature changes should be included in the load cases in the same manner as is done for arch dams (USBR 1977a and b), except summer ambient on downstream face can be reduced because of shading between buttresses.

10-2.5 Analyses

Analysis methods commonly used in evaluating buttress dams are discussed in this section. Simplified methods such as the gravity method and unit width method or the two-dimensional finite element method are generally sufficient for most structures. More sophisticated methods such as three-dimensional finite element analyses may be justified when simpler analyses indicate unsatisfactory behavior.

Depending on the details of design, a buttress arch or slab and buttress dam may more closely resemble a building. Method of frame analysis used for buildings may be appropriate. The strength and stability on individual components may be more critical than the strength and stability of the dam as a whole.

Typical steps for the evaluation of buttress dams are:

1) Review available project drawings, reports, and data.

2) Inspect the dam for evidence of deterioration, cracking, and leakage.

3) Inspect the dam for offsets between members that could indicate movement.

4) Establish the geometry, loads, and make conservative assumptions regarding the material properties.

5) Perform stability analysis using the gravity method.

6) Perform stress analyses using the gravity and unit width methods or finite element method.

7) If appropriate, perform a dynamic analysis.

8) If the analysis indicates that the structure does not meet safety criteria, or if the analytical results do not appear to be consistent with the historical behavior of the dam, then develop more elaborate field and laboratory investigations to refine assumptions.

10-2.5.1 Stability

Stability analyses of buttress dams are the same as for concrete gravity dams discussed in Chapter III, Section 3-4 of these Guidelines, with only minor differences.

The stability of representative buttresses should be evaluated with loads from contributing portions of the upstream face, rather than evaluating a unit thickness as for gravity dams.

Buttresses with longitudinal joints (or cracks) may not act monolithically and stresses for each column formed by the joints (or cracks) should be analyzed separately.

Stability should be evaluated for potential failure planes within the structure (commonly along lift lines), at the base of the structure, and within the foundation. Foundation failure planes are often more critical for buttress dams than for gravity dams, because buttress dams are lighter than gravity dams and thus mobilize less frictional resistance to sliding for the same height of dam. Care should be taken to identify all potential failure planes in the foundation.

The cracked base type of analysis is not applicable for buttress dams, unless they are founded on continuous slabs or have shallow, subhorizontal discontinuities in the foundation (Figure 10-2.9). This is because uplift is at tailwater level all around the buttresses if the buttresses are founded directly on the foundation. If there is a base slab or a horizontal joint in the foundation rock directly below the dam, the uplift distribution then resembles that of a standard gravity dam. (Refer to Chapter III.)

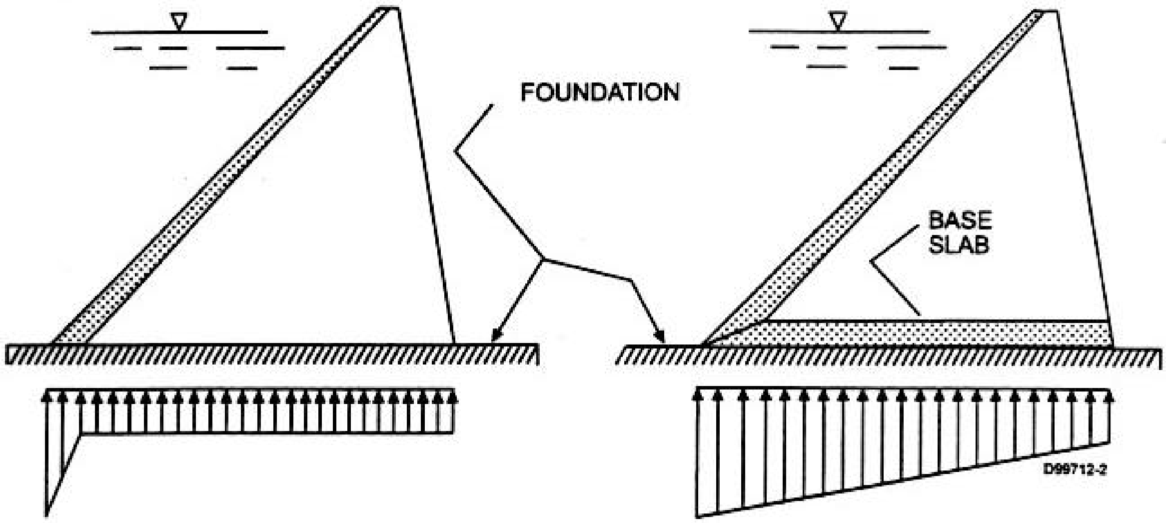

Figure 10-2.9. Uplift Pressure Diagrams For Buttress Dams

10-2.5.2 Stress

Stress analyses of buttress dams should be more detailed than for concrete gravity dams, because stresses are higher and often control the design. Principal stresses and maximum shear stresses must be evaluated in all portions of the structure including the upstream face, buttresses, corbels, upstream face footings, and lateral supports.

In many buttress dams, tensile stresses occur in the buttresses, near the upstream face, due to the geometry of the buttresses and the applied loads.

Damage to the connection between the upstream face and the buttresses from overstressing or inadequate reinforcing is common. Shear and tensile stresses at the connections must be evaluated.

If buttresses have longitudinal joints (or cracks), the buttress may not act monolithically and each column separated by the joints (or cracks) should be evaluated independently.

Gravity Method. The gravity method is generally sufficient for calculation of stresses in most *buttresses*. The basis of the gravity method is that vertical stresses on horizontal planes have a linear variation giving a trapezoidal distribution. This assumption, often referred to as the "trapezoidal law", greatly simplifies the analysis of principal and shear stresses and is applicable for buttresses less than about 200 to 250 feet high (Marcello 1969; Davis 1969c).

Once vertical and shear stresses on horizontal planes are found by the trapezoidal law, principal and maximum shear stresses are calculated by balancing forces on elementary sections and computing the stresses on the incremental areas of the sections. The method is described in various references (Burroughs 1969; Copen, Lindholm, and Tarbox 1977; Creager, Justin, and Hinds 1945; Corn, Tarbox, and Schrader 1988; Davis 1969b; Marcello 1969). Because the method involves small differences in large numbers, the thickness of the horizontal slices must necessarily be small; typically, between 1 and 10 feet have been used. The smaller spacing provides more accurate results and should be used where stresses are critical, as is often the case just under the upstream haunch.

Unit Width Method. In most cases, the stresses in the *upstream face slab or arch* can be adequately evaluated by analyzing an independent unit strip of the slab or arch spanning between the buttresses as shown in Figure 10-2.10. The pressure applied to a unit strip by the reservoir is not uniform because the upstream face is inclined. Unit strips should be evaluated at the maximum depth of a given thickness or reinforcement pattern. Flat slabs should be evaluated as either simply supported or continuous beams or slabs. If this type of analysis shows that an arch or slab strip of unit width can resist all applied loads, then no more rigorous analysis needs to be done.

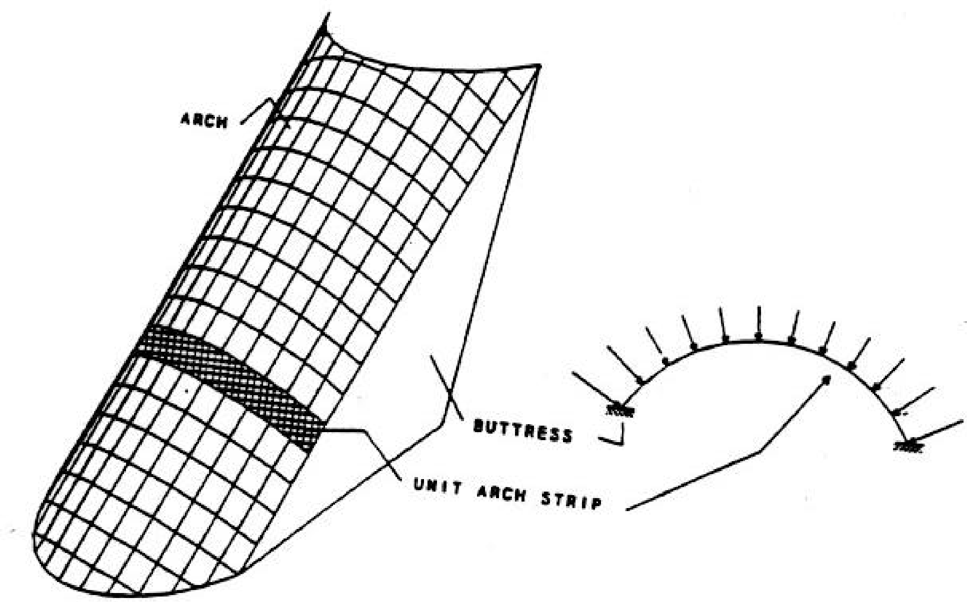

Figure 10-2.10 Unit Arch Strip

Finite Element Method. The finite element method is discussed in Chapter III, Section 3-4 of these Guidelines. Two-dimensional finite element models can often be used to evaluate the upstream face and buttresses separately. Three-dimensional finite element models may be justified where simpler methods do not resolve all concerns or where seismic loading is large.

For evaluation of reinforced concrete structures, the moments, shears, and axial forces of structural members must be calculated by integrating the stresses computed by the finite element model. The resultant loads in the member must be compared to the capacity of the member to evaluate its adequacy.

For most static three-dimensional analyses, there is seldom a need to analyze more than one bay of buttress dams, because cross valley distribution of load typically does not occur and it should not be relied upon if it does occur.

The finite element method is the only practical method available for evaluating the dynamic response of buttress dams. Although two-dimensional models have been used to calculate the dynamic response of individual buttresses (Zienkiewicz and Anderson 1967 and Zienkiewicz 1969), these analyses are typically done with three-dimensional models. Nuss (1995) compares the results of a three-dimensional model of an entire buttress dam with simplified three-dimensional models of slices taken from the complete model. Several boundary conditions should be used to bound a reasonable range of solutions when slice models are used.

10-2.5.3 Analysis of Reinforced Concrete

In dams constructed before the 1940s, square, un-deformed reinforcing bars were sometimes used. Because of the lack of deformation ridges, these bars have longer development lengths and inferior post yield performance. For this reason, the ultimate strength method of concrete analysis currently used by ACI is not appropriate. These structures must be analyzed using the working stress method in ACI 318-95 Appendix A, "Alternate Design Method." The following criteria shall apply for slabs of slab and buttress dams with un-deformed bars:

Allowable Stresses for Concrete Slabs with Plain Bars*	
Tensile Stress in Reinforcing Steel	20 KSI (140 mPa)
Bond Stress Between Concrete and Reinforcing	0.04 F´c (160 psi maximum) (ACI 1936 and ACI 1963)

* These values may be increased by 1.33 for unusual and extreme loading conditions.

The upstream faces of flat slab buttress dams and reinforced buttresses, with deformed reinforcing bars, should be analyzed using the Corps of Engineers "Strength Design for Reinforced Concrete Hydraulic Structures" USACE (1992).

For these analyses, the size, spacing, and strength of reinforcing steel in existing dams should be established. As a first step, conservative assumptions may be used. Steel used in many existing buttress dams may have a lower yield strength than modern steel. If the results of the initial analyses based on conservative assumptions indicate unsatisfactory behavior, or are ambiguous, field investigations may be needed to refine the assumptions. The type, spacing, and condition of existing reinforcement can be established with field investigations and its yield strength can be evaluated by testing.

Large deflections can crack the slabs and expose the reinforcing steel to water. Where large deflections are observed, corrosion of the reinforcing steel should be evaluated. Non-destructive testing can be used to locate reinforcing steel, but coring or excavating is usually required to evaluate the condition of the reinforcing steel.

Slabs on the downstream face are evaluated in the same manner as those on the upstream face.

Corbels or haunches used to support the upstream face on buttresses should have sufficient reinforcement for water loads, upstream face loads, and frictional forces that tend to prevent expansion or contraction of the upstream face from temperatures changes.

Where buckling is suspected of being a problem, buttresses should be evaluated as load bearing plates (or columns if there are longitudinal joints or cracks).

A number of computer software programs are available for use in analyses. Refer to Chapter III, Section 3-4.7 for further discussion.

10-2.5.4 Dynamic Analyses

The seismic coefficient method, discussed in Chapter III, Section 3-2.6.2 of these guidelines is adequate for buttress dams located in seismic zones 0 and 1. Vertical acceleration should also be considered.

Dynamic analyses are required for all buttress dams in zone 2 as well as for those in zones 3, and 4, because of the susceptibility of buttresses to overstressing from lateral loading.

The pseudo-dynamic method discussed in Chapter III, Section 3-2.6.3 of these Guidelines was developed based on typical dynamic characteristics of concrete gravity dams and is therefore not appropriate for buttress dams.

The finite element method is the only practical method available for performing dynamic analyses of buttress dams. This type of analysis is discussed in Chapter III, Section 3-4.4 of these guidelines. Either time history or response spectrum analyses are acceptable. The response of the dam should be evaluated for earthquake loads applied in both the transverse (upstream-downstream) and longitudinal (cross-valley) directions. Dynamic, three-dimensional finite element analyses of multiple arch buttress dams are quite complex and require considerable experience and judgement to perform and evaluate.

Nuss, Payne, and Sozen (1994), Nuss (1995) and Boggs, Jansen, and Tarbox (1988) describe several such analyses and some of the complexities involved.

10-2.5.5 Post Earthquake Stability Analyses

The effects of cross-valley loading and structural response must be considered in the post earthquake stability analysis. Significant rocking of the buttresses with respect to the foundation, may destroy cohesive bonds.

10-2.6 Acceptance Criteria

Criteria for buttress dams are generally the same as for concrete gravity dams discussed in Chapter III of these Guidelines. Only exceptions, clarifications, and additions are discussed below.

10-2.6.1 Static

Sliding stability criteria are the same as for concrete gravity dams discussed in Chapter III of these Guidelines.

Stress criteria for unreinforced concrete are the same as for concrete gravity dams discussed in Chapter III of these Guidelines, except that the major principal, minor principal, and maximum shear stresses should meet the recommended factors of safety in Table 2 "Recommended Factors of Safety" of Chapter III.

For buttresses, the maximum allowable compressive stress may be governed by the slenderness ratios. If bracing, struts, stiffeners, or counterforts exist, they will increase resistance to buckling and should be considered in any analysis.

Stress criteria for reinforced concrete should follow the Corps of Engineers "Strength Design for Reinforced Concrete Hydraulic Structures" (USACE 1992).

10-2.6.2 Dynamic and Post Earthquake

Dynamic stability criteria for buttress dams are the same as for concrete gravity dams discussed in Chapter III, Section 3-5.3 of these Guidelines.

The USBR has applied deflection criteria to evaluate the stability of multiple arches when stress levels were near the allowable dynamic strength limits of the reinforced concrete and substantial cracking was anticipated (Nuss, Payne, and Sozen 1994). The criteria is applicable to reinforced concrete elements whose behavior is dominated by flexural and axial stresses. The calculated deformation of the arches, in a cracked condition, was found to be of the same magnitude as the estimated yield deformation. Since experimental tests have shown that reinforced concrete beams can tolerate displacement on the order of twice the yield displacement, the arches were judged to be adequate.

10-2.6.3 Foundation Stability

Foundation stability criteria for buttress dams are the same as for concrete gravity dams discussed in Chapter III, Section 3-5.6 of these Guidelines. For Buttress dams, the gravity and hydrostatic loads are transferred to the foundation through the buttresses. While gravity dams can bridge over small areas of bad foundation material, each and every buttress must be founded on a competent foundation.

10-2.7 Material Properties

In general, material properties for buttress dams are the same as for concrete gravity dams discussed in Chapter III, Section 3-6 of these Guidelines. Specific concerns for buttress dams are poor quality concrete, deterioration of the concrete, and the condition, size, spacing and strength of any reinforcing steel.

Most buttress dams were constructed before construction quality control programs and concrete air-entrainment were common. As a result, the quality of concrete is quite variable, and often subject to deterioration, especially from freeze-thaw action. All concrete should be thoroughly examined for evidence of deterioration. Where there is a concern about the quality of the concrete, cores should be taken to evaluate the nature and extent of deterioration and representative samples should be tested to evaluate strengths.

Deterioration and cracking of the concrete can expose the reinforcing steel to air and water, resulting in corrosion. Where existing reinforcing steel is structurally necessary, the condition, type, and extent of reinforcing steel should be verified with field investigations. Such field investigations may include coring, geophysical surveys, and excavation and exposure of reinforcing in areas of concern. Where appropriate, the strength of the reinforcing should be evaluated by testing. Concrete faces coated with gunite can be deceiving. Poor quality concrete and corroded reinforcement may be below surface.

10-2.8 References

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10-3 Concrete Dams on Pile Foundations

10-3.1 Introduction

Piles used to found concrete dams and other hydraulic structures are designed to distribute foundation loads, both axial and lateral, into or through loose, soft or compressible soils beneath the structures. This section provides an overview of the evaluations of existing hydraulic structures founded with piles, primarily with respect to lateral forces transmitted from the dams to the foundations. The response of pile foundations to vertical loads is important and should be considered in safety evaluations of hydraulic structures. However, detailed discussions concerning the evaluation of pile resistance to vertical loads are not provided as vertical loading is usually not the critical case in such evaluations. The design and construction of new pile foundations are also not specifically addressed.

Piles are defined as driven, not drilled, members. Piles can include steel H piles, steel pipe piles, precast concrete, cast-in-place and mandrel driven piles and timber piles. Pile details, capacities and allowable stresses are provided in USACE 1991 and many other publications.

10-3.2 Forces

Forces for the analysis of concrete dams on pile foundations are generally the same as for gravity concrete dams discussed in Chapter III, Section 3-2 of these Guidelines.

10-3.3 Loading Combinations

Loading combinations for the analysis of concrete dams on pile foundations are generally the same as for gravity dams discussed in Chapter III, Section 3-3 of these Guidelines. Loading combinations should be established to produce critical combinations. For each loading combination, the effect each load will have on pile forces and on internal forces in the pile cap should be considered. Some loadings may control the internal design of the pile cap even though they may not produce the critical pile forces. Generally, it is important to analyze the load cases with the largest lateral loads in each direction and the cases with the maximum and minimum vertical loads.

10-3.4 Structure and Subsurface Conditions

The evaluation of hydraulic structures on pile foundations involves the solution of complex soil-pile-structure interaction problems. Details concerning the structure, piles and subsurface conditions need to be established regardless of the method of analysis. Simplifying assumptions concerning many of the parameters are required regardless of the analytical technique used for the evaluations.

Information concerning the structure includes the following:

- Structure configuration and dimensions, including plans, sections, expansion joints, etc.
- Pile foundation details, including pile types, sizes, lengths, locations, spacing, depths, attachment conditions to the structure, etc.
- Structure and foundation performance, including horizontal and vertical movements, seepage conditions, the condition of the structure and piles, etc.
- Past, current and anticipated reservoir and tailwater levels and other similar conditions that result in forces and loads on the structure.

Subsurface condition information includes:

- Types and depths of soils and bedrock beneath the structure
- Groundwater levels, uplift pressures and seepage conditions
- Soil and bedrock engineering properties, including unit weight, plasticity, gradation, strength (angle of internal friction, cohesion), consolidation, etc.
- Potential soil settlement, which can create voids under the structure.

Sources of such information include structure and geotechnical data and design reports, construction plans and specifications, construction records, inspection reports, and records documenting structure and reservoir monitoring. Such information may not be readily available, particularly for older structures. Assumptions concerning structure, pile and subsurface conditions must frequently be made for initial evaluations. If so, parametric evaluations of the effect of differing conditions should be considered.

One concern related to hydraulic structures supported on piles is potential settlement of soil under the structure. Existence of such condition, if suspected, should be investigated by installation of piezometers and drilling of exploratory bore holes as necessary. If existence of voids is confirmed, controlled grouting so as not to overstress the structure and impact its design provisions would be required.

10-3.5 Analyses

10-3.5.1 General

Pile foundations are used to distribute foundation loads into or through soils beneath structures to provide increased foundation resistance to anticipated loads and to reduce lateral and/or vertical deformations. The load capacity of a pile is dependent on the properties of the pile, the properties of the soil and/or bedrock around and beneath the pile, and allowable pile movement. Attachment conditions of the pile to the pile cap also affects the lateral load capacity of piles. Design (working) capacities are generally governed by pile movement considerations and are less than ultimate (failure) capacities of the piles.

10-3.5.2 Vertical Capacity

Piles beneath hydraulic structures should be evaluated assuming all vertical loads are carried by the piles, and assuming no uplift (tension) loads in the piles. However, the vertical capacity of piles is generally not the critical condition in safety evaluations as actual pile foundation loads exceeding allowable pile vertical capacity will likely result in increased pile settlement and the pile cap bearing on the underlying soils. While settlement may cause problems, it does not necessarily constitute a safety problem.

Vertical capacity and settlement evaluations for pile foundations are discussed in ASCE 1984, CGS 1985, Davisson 1970, NAFAC 1982, Poulos and Davis 1980, USACE 1991; and Vesic 1977, as well as many other publications.

10-3.5.3 Lateral Capacity

Resistance provided by piles to horizontal forces is the result of complex interaction of the structure, pile and foundation soils. Structure forces are distributed through and by the piles to the surrounding soils. The pile and the soils each provide resistance, the amount of which is a function of the relative stiffness of the pile and the soils, and the magnitude of the load applied to the pile. Soil resistance is a nonlinear function of soil deflection and of the specific type and properties of the soil. Varying soil deflections and resistances with depth cause pile bending which induces shear and bending stresses in the pile. Pile top connections to the structure (pile caps) also influence the distribution of stresses in the pile and soils. Rigid, fixed connections where pile top rotation is prevented or limited cause more bending stresses at the pile top than do pinned connections where the pile top is free to rotate (see Figure 10-3.1).

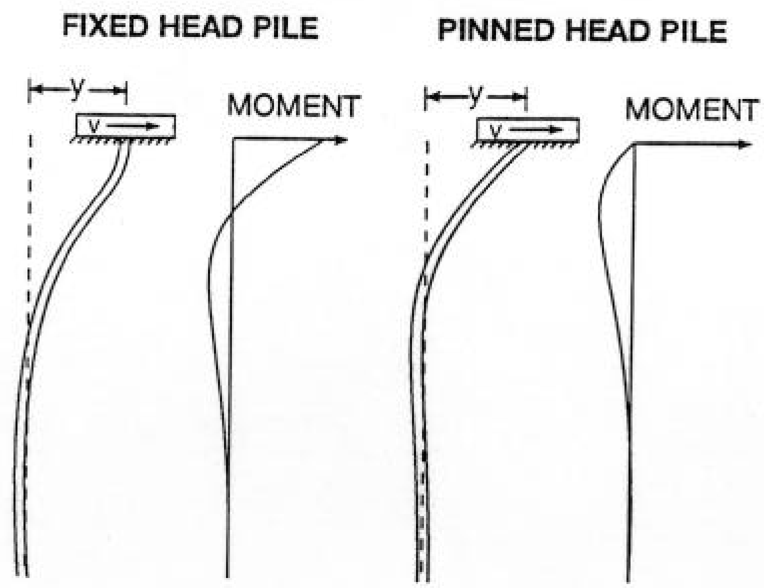

Figure 10-3.1. Effects of Pile Top Fixity

Several procedures have been used to model the response of vertical piles subject to lateral loads. Linear or constant soil responses, rigid, non-bending piles, fixed points of pile rotation and other similar simplifying assumptions have been included in most of the models to facilitate their solution. A more rigorous model is based on the elastic bending of the pile and a nonlinear soil response that is a function of soil properties and soil movement.

Considerable judgement must be applied in selecting and applying analytical techniques to the evaluation of pile foundations due to the complexity of the problems and the simplifying assumptions involved. The techniques used should be selected on a case-by-case basis. Parametric studies where input assumptions and values are systematically varied to test the effect of the various assumptions on the evaluation results should be considered.

The elastic pile method and an ultimate analysis method are discussed below.

Elastic Pile Method

Based on the assumption that the pile is a linearly elastic beam and that the soil reaction can be represented as a line load (Reese 1977), the equation for pile-soil interaction is:

$$EI\frac{d^4y}{dx^4}+P_x\frac{d^2y}{dx^2}-P=0$$

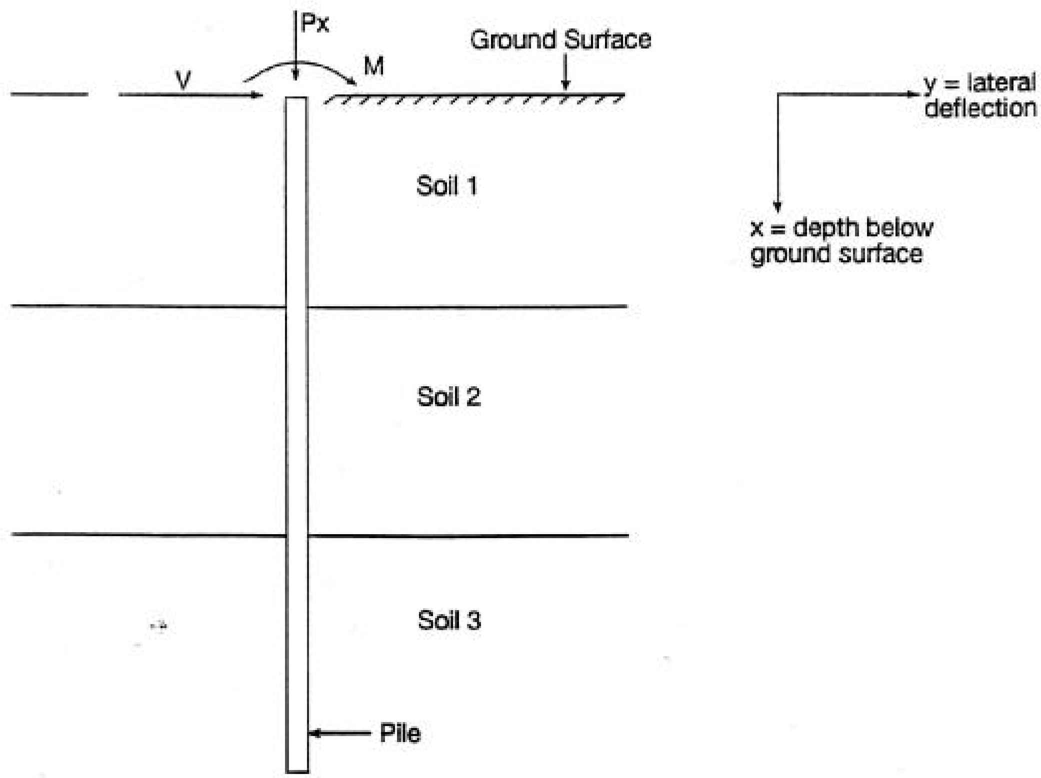

Figure 10-3.2 Typical Pile Nomenclature

where, as indicated on Figure 10-3.2:

E	=	modulus of elasticity of the pile
I	=	moment of inertia of the pile
y	=	lateral deflection of the pile at point x along the pile length
x	=	distance from top of pile
P_x	=	axial load on the pile
P	=	soil reaction per unit length

For most conditions, horizontal forces induced into the pile by the axial load (P_x) are small due to relatively small lateral deflections (y) and can be neglected in the analysis. The equation becomes:

$$EI\frac{d^4y}{dx^4} = P$$

The following can be obtained by integrating the equation:

$$EI\frac{d^3y}{dx^3} = V$$

$$EI\frac{d^2y}{dx^2} = M$$

$$EI\frac{dy}{dx} = S$$

where: V = shear force in the pile at point x
M = bending moment in the pile at point x
S = slope of the deflected shape of the pile at point x

The soil reaction (P) in the equations is a nonlinear function of deflection (y) and varies depending on soil type and properties (see Figure 10-3.3). Analytical procedures have been developed (Reese 1977, Geosoft 1987, and others) to predict the soil reaction (P-y curves) based on typical soil properties, including soil type, cohesion (c), friction angle (ø), and soil unit weight (γ). Computer programs (Reese 1977, Geosoft 1987, and others) are available that develop P-y curves and use iterative techniques to solve the equations for elastic bending of the pile. The fixity or amount or rotation of the pile top can be specified, and layered soil profiles can be used. Output from the programs typically includes moment (M), shear (V), lateral deflection (y) and soil reaction (P) relative to distance below the pile top (x).

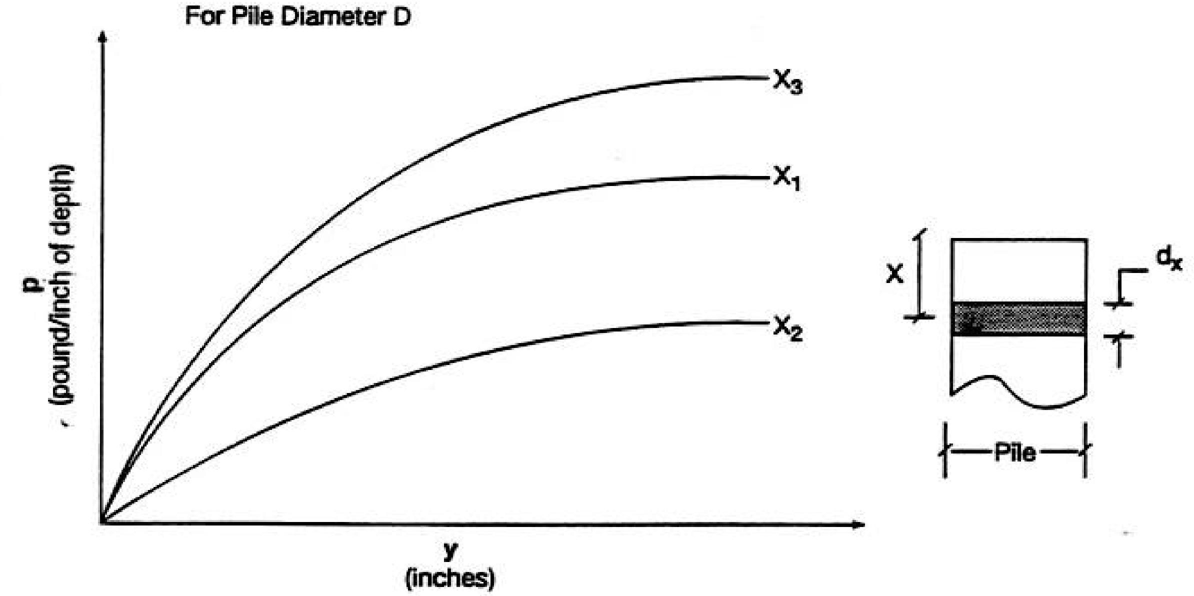

Figure 10-3.3 Typical Soil Load-Deflection (p-y) Curves

The above methods can be used to evaluate resistance of pile foundations to lateral forces. Forces acting on piles can be developed based on the lateral forces acting on the dam, and pile deflections, moments and shear forces can be estimated using the computer programs. Theoretical factors of safety against pile failure or against pile stresses exceeding allowable values can be obtained by dividing ultimate or allowable moments and shear forces obtained from elastic pile properties by the maximum moments and shear forces obtained from the lateral load analysis. Forces from varying loading conditions (usual, unusual or extreme) can be evaluated, and potential total movements of the dam under those forces can be obtained for further consideration in the safety evaluations. If data is available documenting actual dam movements at known loading conditions, the computer model can be calibrated using that information.

An example of the elastic pile method of analyses is summarized in the attached Pile Analysis Example.

Ultimate Pile Strength Method

Another method to evaluate the ultimate capacity of piles and pile foundations is to assume the foundations deflect sufficiently over their entire length to develop the ultimate resistance of the soil surrounding the piles (Poulos and Davis 1980). The ultimate resistance of the soil is (Broms 1964a and 1964b):

For cohesive soils:

From the bottom of the pile cap to 1.5 pile diameters

$$C = 0$$

From 1.5 pile diameters to the bottom of the pile cap

$$C = 9cD$$

For cohesionless soils:

$$K_x = 3K_p\gamma D$$

where:

C	=	ultimate resistance of cohesive soils, lb/ft
c	=	soil cohesion, lb/ft^2
K_x	=	ultimate resistance of cohesionless soils at point x along pile length
k_p	=	coefficient of passive lateral earth pressure
	=	(1 + sin ø)/(1 - sin ø)
ø	=	angle of internal friction (degrees)
γ	=	soil unit weight, lb/ft^3
x	=	vertical distance from ground surface, ft
D	=	pile diameter, ft

From the limit states and pressures shown on Figure 10-3.4, the following ultimate loads for laterally loaded piles can be derived:

For cohesive soils:

Short Pile:

$$V_u = C(L - 1.5D)$$

Medium Pile:

$$V_u = \sqrt{4CM_t + 2(CL)^2 + 4.5(CD)^2} - C(1.5D + L)$$

Long Pile:

$$V_u = \sqrt{2C(M_t + M_b) + 2.25(CD)^2} - 1.5D$$

For cohesionless soils:

Short Pile:

$$V_u = \frac{KL^2}{2}$$

Medium Pile:

$$V_u = K\left[\left(\frac{3M_t}{2K} + \frac{L^3}{2}\right)^{\frac{2}{3}} - \frac{L^2}{2}\right]$$

Long Pile:

$$V_u = [1.125K(M_t + M_b)^2]^{\frac{1}{3}}$$

where:

V_u = ultimate lateral resistance

M = ultimate moment capacity of the pile at the pile top (M_t) and pile tip (M_b)

L = pile length

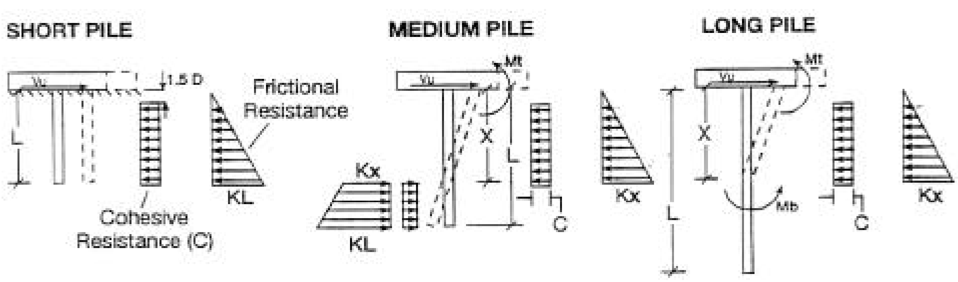

Figure 10-3.4 Limit States for Ultimate Analyses

For a given soil type and fixity condition, the equation that yields the minimum load governs. In cases where the pile is not sufficiently embedded in the cap to be considered fixed, M_t should be assumed to be zero or an evaluation of the percentage of the fixed head moment should be developed for the particuar embedment case.

A procedure for deriving theoretical factors of safety using the ultimate pile strength method is to calculate ultimate pile moment capacities (M_T, M_B) and ultimate pile shear capacity using ultimate strength values for the pile type and size being analyzed. Those ultimate moment capacity values, along with the ultimate soil strength values (C,K) can be substituted into the appropriate equations to calculate the minimum ultimate lateral resistance (Vu) of the piles, as previously described. The factor of safety is the minimum ultimate lateral resistance divided by the sum of the lateral forces acting on the dam, or the maximum ultimate shear capacity of the piles, whichever produces the lower value.

10-3.6 References

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10-3.7 Pile Analysis Example

Consider the pile supported Ambursen dam shown below. The dam is hollow with 2-foot thick interior piers at 20-feet on center.

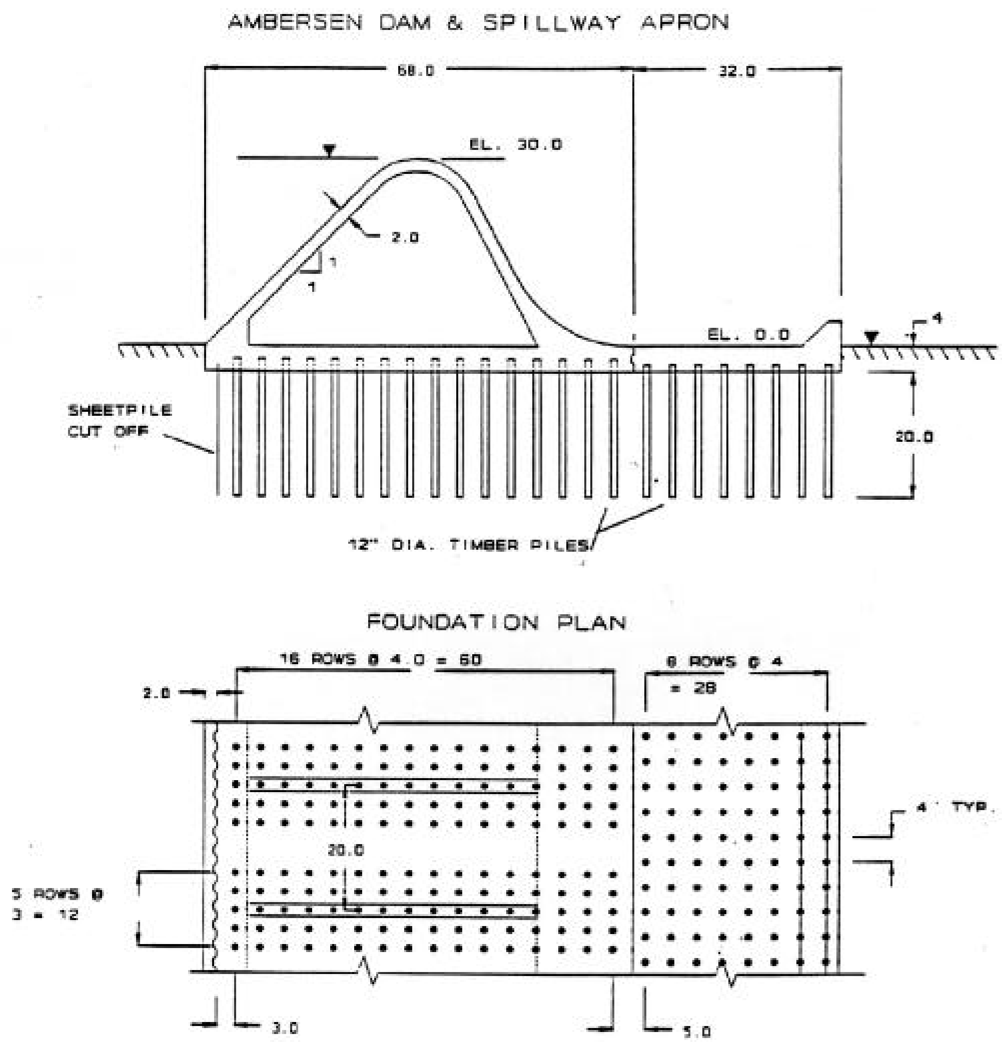

Figure 10-3.5 Ambursen Dam Pile Foundation

As with any stability analysis, the first step is to determine the loads acting on the structure. In this case, since there is a joint between the dam and the spillway apron, bending moments will not be transmitted between the two structures. For this reason, the vertical loads will be independently tabulated for the dam and the spillway. The horizontal loads, however, will be resisted by both structures together.

DETERMINATION OF LOADS:

UPLIFT

Uplift can be determined using a hand drawn flow net or a 2 dimensional finite element solution. This example is not intended to demonstrate this procedure. The uplift head values shown below will be assumed for this example.

	Total Head (Ft)	Horizontal Location (Ft)	Force (kips)	At (Ft)
POINT A	32.52	0	80.87	1.00
U.S. SHEETPILE	32.28	2		
D.S. SHEETPILE	17.52	2		
			1109.20	31.69
POINT B	9.41	68		
		TOTAL->	1190.07	29.60
POINT B	9.41	68		
			297.19	82.59
POINT C	5.47	100		

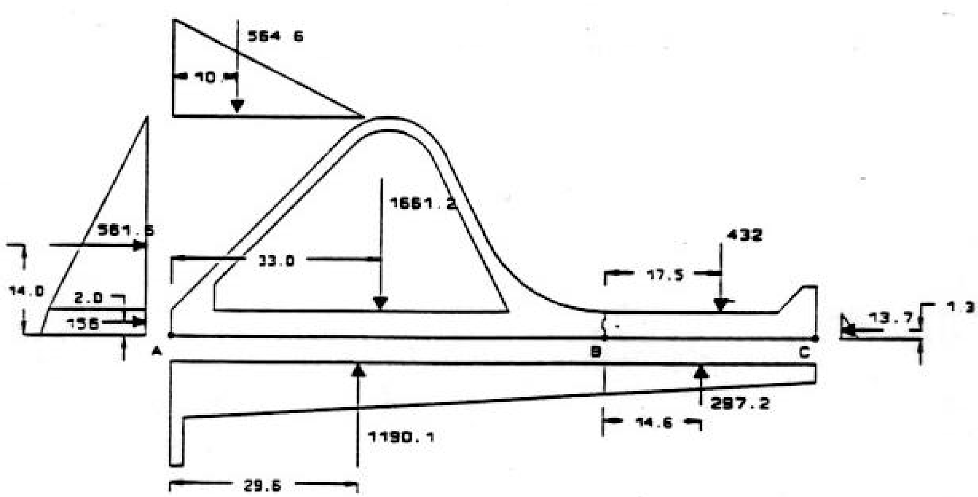

Figure 10-3.6 Force diagram

Now that the uplift has been determined, the other forces and moments can be summed to determine the resultants acting on each pile group.

Note below that the horizontal reservoir force has been broken up into 2 portions; the 30 feet above the reservoir bottom and the 4 feet acting on the vertical face of the dam below the reservoir bottom. This is necessary because the head loss due to vertical flow in the soil causes the head at point A to be slightly less than the full 34 feet of reservoir head. Note also that active lateral soil pressure is applied to the dam and passive to the spillway. These are not the forces present at service load, but they will be the forces present at failure; the failure condition is what we referenced to the safety factor.

	Fx (Kips)	@ (ft)	Fy (Kips)	@ (ft)	M @ A (Kip-ft)
AMBURSEN DAM					
Structure Wt.			-1661.2	33.0	54819.6
Vert. reservoir			-564.6	10.1	5702.5
Horiz. reservoir	561.6	14.0			7862.4
	156.0	2.0			312.0
Active earth	3.6	1.33			4.8
Uplift			1190.1	29.6	-35227.0
TOTALS->	721.2		-1035.7		33474.3
VERT. RESULTANT @ R=	32.3				
DOWNSTREAM APRON					M @ B
Structure Wt.			-432.0	17.5	7560.0
Tailwater	-13.7	1.33		-18.2	
Passive earth	-32.4	1.33		43.2	
Uplift			297.2	14.6	-4339.1
TOTALS->	-46.1		<u>-134.8</u>		3159.5
VERT. RESULTANT @ R=	<u>23.4</u>				
Total External Forces on Dam and Apron	675.1		1170.5		

INDIVIDUAL PILE AXIAL FORCES

To determine the axial forces on the piles, the pile group section properties must be computed. The axial force on a pile is given by the following equation:

$$P_x = \frac{F_v}{N} + \frac{F_v(R-\overline{X})(X-\overline{X})}{I}$$

WHERE:

$$\overline{X} = \frac{\sum X}{N}\ ,\ I = \sum (X-\overline{X})^2$$

PILE GROUP SECTION PROPERTIES

AMBURSEN DAM

# of piles in row	Location X (ft)	$5*X$	$5*(X-\overline{X})^2$
5	5	25	4500
5	9	45	3380
5	13	65	2420
5	17	85	1620
5	21	105	980
5	25	125	500
5	29	145	180
5	33	165	20
5	37	185	20
5	41	205	180
5	45	225	500
5	49	245	980
5	53	265	1620
5	57	285	2420
5	61	305	3380
5	65	325	4500
N → 80		$\sum 5*X$ → 2800	27200 ← I

GROUP CENTROID $\overline{X} = 35$

Note that the sheetpile is ignored in the calculation of the pile group section properties. The sheetpile was also ignored in the calculation of forces. This is conservative. If sheetpile walls are attached the base slab sufficiently to develop the moment capacity of the pile, and if the piles themselves are of substantial section, the lateral load resistance of sheetpile walls can be significant.

Upstream row:

$$P_x = \frac{-1035.7}{80} + \frac{-1035.7(32.3-35)(5-35)}{27200} = -16\ KIPS$$

Downstream row:

$$P_x = \frac{-1035.7}{80} + \frac{-1035.7(32.3-35)(65-35)}{27200} = -9.9\ KIPS$$

Note that all forces are compressive. The magnitude of these pile loads is less important than the sign. If the pile is overloaded in compression, the structure may settle, but it is not likely to fail. If a pile is loaded in tension, the analysis should be re-done with the pile in tension removed. Pile tension should not be relied upon for stability.

DETERMINATION OF PILE ULTIMATE STRENGTHS:

The ultimate moment capacity of a timber pile is a function of the extreme fiber bending strength. For a 12-inch diameter pile with an extreme fiber bending strength (F_b) of 2500 psi, the ultimate moment capacity is:

$$M_t = SF_b$$

$$S = I/C = \left(\frac{\mathrm{B}D^4}{64}\right) / \frac{D}{2} = \frac{\mathrm{B}D^3}{32}$$

$$M_t = \frac{\mathrm{B}(12)^3}{32}\ x\ \frac{2500}{1000} = 424.1\ kip\text{-}inch = 35.3\ kip\text{-}ft$$

Timber piles are tapered. Assuming the diameter of the pile tip is about 9.5 inches, the maximum moment M_b at the pile tip is half of the ultimate moment at the pile top, or 17.6 ft-kips:

$$M_b = \frac{\mathrm{B}(9.5)^3}{32}\ x\ \frac{2500}{1000} = 210.4\ kip\text{-}inch = 17.6\ kip\text{-}ft$$

Piles in the dam are embedded 2 feet. Those in the apron are embedded 1 foot. It is necessary to determine if the full moment capacity of the pile top can be developed based upon those embedment lengths into the pile cap.

If the pile embedment is not sufficient to resist rotation (twisting) of pile in the pile cap socket, then stress on the outside of the pile can be assumed to reach ultimate crushing stress (σ_c) at the pile centerline, and other stresses can be assumed as shown in Section A-A, Figure 10-3.7.

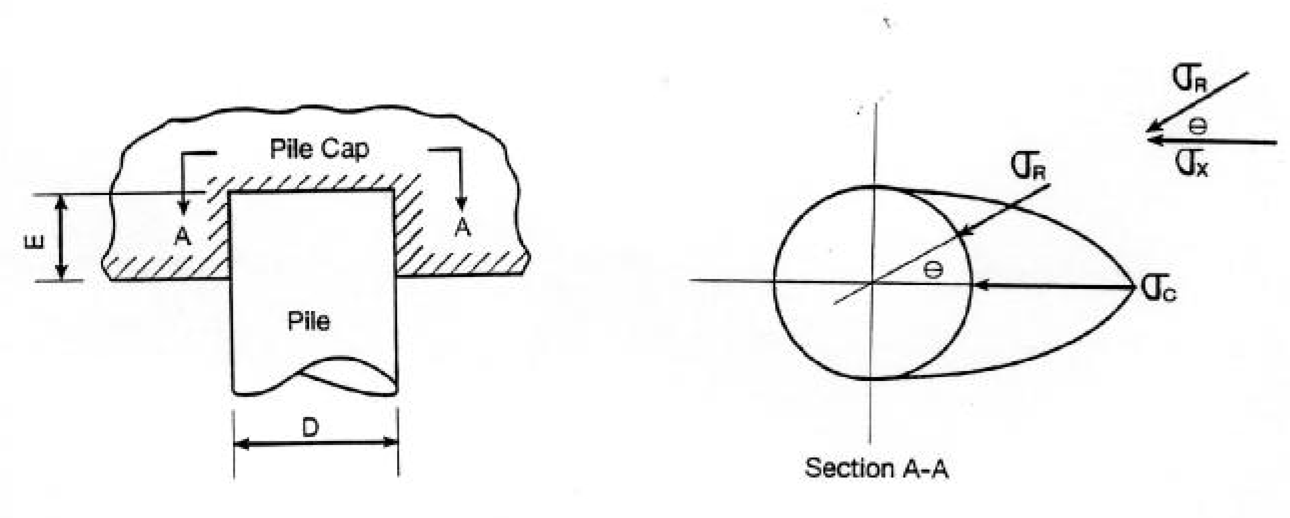

Figure 10-3.7 Pile Top Details

σ_c = Ultimate crushing strength
σ_R = Radial crushing strength
σ_x = Crushing strength normalized to x-direction

$$\boldsymbol{\sigma_R = \sigma_c \cos}$$
$$\boldsymbol{\sigma_x = \sigma_R \cos \quad = \sigma_c \cos^2}$$

The total force per unit length (vertical) available in the direction of movement (x) should be:

$$F = 2 \int_0^{B/2} F_c \cos^2 1 R d1$$

$$F = 2RF_c \left[\frac{1}{2} + \frac{\sin 2\ 1}{4} \right]_0^{B/2}$$

$$F = 2RF_c [B/4] = \frac{BRF_c}{2} = \frac{DF_c B}{4}$$

If it is assumed that rotation/twisting of pile in pile cap is about the midpoint of the embedment, then:

$$F_R = (F)(\frac{E}{2})$$

$$\sum M_o = 0$$

$$M = (F_R)(E/2) = \frac{FE^2}{4}$$

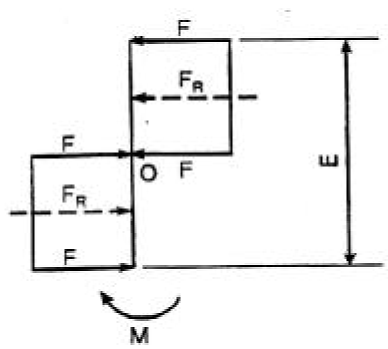

Figure 10-3.8 Force Diagram

$$Substituting: \; F = \frac{D F_c B}{4}$$

$$Results\ in: \; M = \frac{(D F_c B/4)E^2}{4} = \frac{D F^c B\ E^2}{16}$$

$$F_b = \frac{Mc}{I} = \frac{M}{S} \text{ Where } F_b \text{ is bending stress}$$

$$Section\ modulus\ (S)\ for\ round\ pile = \frac{BD^3}{32}$$

$$M = F_b\ S = \frac{F_b B D^3}{32}$$

If the bending strength is assumed to be three times the crushing strength:

$$F_c = \frac{F_b}{3}$$

Equating the moments results in:

$$\frac{F_b B D^3}{32} = \frac{F_c D B E^2}{16}$$

Substituting for σ_c results in:

$$\frac{F_b B\, D^3}{32} = \frac{(F_b/3) D B E^2}{16}$$

$$\frac{B D^2}{32} = \frac{B E^2}{48}$$

$$E = D\left[\frac{48B}{32B}\right]^{1/2} = \left(\sqrt{\frac{3}{2}}\right) D$$

$$E = 1.22D$$

Therefore, if E/D is greater than 1.22, then the full moment capacity of the pile top can be developed.

For a 12-inch diameter pile with an ultimate shear stress perpendicular to the pile axis of 500 psi, the ultimate shear strength is:

$$V = \left(\frac{500}{1000}\right) \frac{B D^2}{4} = 56.5\ kips$$

ELASTIC PILE ANALYSIS:

The lateral movement of the top of each pile should be the same as all piles are similarly embedded into the concrete. Since all the piles and the soils surrounding the piles are the same, the lateral resistance provided by each pile should also be the same provided they are all similarly fixed into the dam concrete. However, the piles in the downstream apron are embedded one foot into the concrete, so they may not be entirely "fixed." If so, each pile there should contribute less lateral resistance than each pile beneath the dam as fixed head pile provide more resistance than do pinned head or partially fixed head piles at the same amount of pile head movement.

Assuming the piles beneath the dam and apron are all fixed head piles, the lateral load carried by each pile should be:

$$V_o = (721.2 \text{ kips} - 46.1 \text{ kips})/(80 \text{ piles} + 40 \text{ piles})$$

$$V_o = 5.6 \text{ kips per pile}$$

Assuming the piles beneath the apron are not fixed and do not provide significant lateral resistance to horizontal dam forces, the lateral load carried by each pile beneath the dam would be:

$$V_o = (721.2 \text{ kips} - 46.1 \text{ kips})/(80 \text{ piles})$$

$$V_o = 8.4 \text{ kips per pile}$$

Soils around the piles are assumed to be sands with the following properties:

$$\gamma_m = 130 \text{ pcf}$$

$$\gamma_b = 67.6 \text{ pcf}$$

$$ø = 30^o$$

The properties of the timber piles used for the analysis are:

Length = 20 feet

Diameter at the top = 12 inches

Diameter at the tip = 9.5 inches

Average diameter (D_a) = 10.75 inches

Moment of Inertia (I) = (D_a^4)/64 = 665.6 in^4

Modulus of Elasticity (E) = 1.5(10^6) psi

EI = 0.998(10^9) pound - in^2

Using the program PILED/G (Geosoft 1987) and the above properties, the following were obtained:

Shear at Pile Top	Moment at Pile Top	Pile Top Movement
5.6 kips	14.5 kip-ft	0.2 inches
8.4 kips	24.7 kip-ft	0.4 inches

Theoretical factors of safety against failure in shear and bending are:

Factor of Safety Shear (FS_s) = (56.5 kips)/(8.4 to 5.6 kips)

$$FS_s = 6.7 \text{ to } 10.1$$

Factor of Safety Bending (FS_b) = (35.3 kip-ft)/(24.7 to 14.5 kip-ft)

$$FS_b = 1.4 \text{ to } 2.4$$

The lower theoretical factor of safety values so obtained assume only the piles beneath the dam resist lateral loads. Based on the relatively small lateral movement indicated by the analysis, regardless of the number of piles considered, it appears that the lateral resistance of each pile beneath the dam and apron should be nearly the same and near 5.6 kips per pile.

ULTIMATE PILE STRENGTH METHOD:

The ultimate soil strength for the sandy soils surrounding the piles should be:

$K = 3\ Kp\ \gamma\ D$

$Kp = (1+\sin ø)/(1-\sin ø) = (1+\sin 30^o)/(1-\sin 30^o) = 3.0$

$K = 3(3)(0.0676\ kips/ft^3)(1\ ft)$

$K = 0.608\ kips/ft^2$

Substituting into the appropriate single pile equations, as follows:

Short Pile:

$$V_u = \frac{KL^2}{2}$$

$$V_u = \frac{(0.608)(20)^2}{2} = 121.7\ kip$$

Medium Pile:

$$V_u = K\left[\left(\frac{3M_t}{2k} + \frac{L^2}{2}\right)^{\frac{2}{3}} - \frac{L^2}{2}\right]$$

$$= 0.608\left[\left(\frac{3\ x\ 35.3}{2\ x\ 0.608} + \frac{20^3}{2}\right)^{2/3} - \frac{20^2}{2}\right]$$

$$= 32.3\ kip$$

Long Pile:

$$V_u = [1.125k(M_t + M_b)^2]^{1/3}$$

$$= [1.125\ x\ 0.608(35.3 + 17.6)^2]^{1/3}$$

$$= 12.4\ kip$$

The piles in the downstream apron are embedded only 1 foot. It has previously been shown that an embedment of 1.22 pile diameters into the pile cap is required to consider the top of a round timber pile fixed. This means that the full moment capacity of the piles in the downstream apron cannot be counted on.

Referring to a previously developed relationship for round timber piles:

$$E = \sqrt{\frac{3}{2}}\, D$$

Which simplifies to:

$$\frac{2}{3}\left(\frac{E}{D}\right)^2 = 1.0$$

Using this relationship, the following equation can be used to determine the ultimate moment capacity at the top of a partially embedded pile. This equation is only applicable for round timber piles and assumes that the bending strength is equal to 3 times the crushing strength.

$$M+_T = \frac{2}{3}\left(\frac{E}{D}\right)^2 M_T$$

Where:

M'_T= Ultimate moment capacity at the top of a partially embedded pile.

E = Embedment distance into the pile cap

D = Diameter of pile

$$M+_T = 35.3\ kip\text{-}ft\left(\frac{2}{3}\right)\left(\frac{1}{1}\right)^2 = 23.5\ kip\text{-}ft$$

For these piles, the ultimate lateral capacity is 10.5 kip (based on the Long Pile formula and M_t = 23.5 kip-ft).

The total lateral resistance capacity for the dam and apron based on individual pile strengths is then:

$$80\ piles\ x\ 12.4\ kip/pile + 40\ piles\ x\ 10.5\ kips/pile = 1412\ kips$$

The sum total of the external lateral forces acting on the dam and apron is 675.1 kips, as previously determined under DETERMINATION OF LOADS. The overall theoretical factor of safety is:

$$\frac{\sum V_u}{\sum V_x} = \frac{1412}{675.1} = 2.1$$

10-4 Concrete Dams on Soil Foundations

Concrete dams constructed on soil foundations are relatively small structures that exert low bearing pressures on the foundation. Typically, such structures have been less than 50 feet in height with less than 20 feet of headwater-tailwater differential. Large structures on soil foundations are usually supported on piles as discussed in Chapter X, Section 10-3 of these Guidelines.

10-4.1 Forces

Forces for the analysis of concrete dams on soil foundations are generally the same as for gravity dams discussed in Chapter III, Section 3-2 of these Guidelines.

The resulting shear and moment from net pressure acting on the cutoff should be applied to the structure. For flexible steel sheet piles, the unbalanced load transferred to the structure may be negligible. For a continuous rigid cutoff, such as a concrete cutoff, the unbalanced load should be accounted for. If a cutoff is assumed to transfer the load to the dam, it must have sufficient strength.

10-4.2 Loading Combinations

Loading combinations for the analyses of concrete dams on soil foundations are generally the same as for gravity dams discussed in Chapter III, Section 3-3. In addition, for any new dam constructed on a soil foundation, the end of construction loading condition must be analyzed as discussed in Chapter IV, Section 4-6 of these Guidelines.

10-4.3 Analyses

Analyses for existing concrete dams on soil foundations typically use the gravity, and dynamic methods for concrete gravity dams discussed in Chapter III, Section 3-4 of these Guidelines. The cracked base type of analysis discussed in Section 3-4.6 is not applicable. Generally, the dam would be long in relation to height and transverse contraction joints neither keyed nor grouted in order to allow independent monolith movement, therefore the trial load analysis would not be appropriate. The foundation should be analyzed for static and seismic stability, seepage and erosion as discussed in Chapter III, Section 3-5.6.2 and liquefaction potential as discussed in Chapter IV, Section 4-5.6.

Analyses for new dams should consider the use of the finite element method in order to adequately model the dam-foundation interaction to account for foundation settlement.

10-4.3.1 Bearing Capacity

Foundation bearing capacity must be checked since the concrete dam may impose fairly high stresses on the foundation soil. The potential presence of weak soil beneath the dam should be carefully investigated. It may be necessary to enlarge the base of the dam to reduce stresses and improve the foundation bearing capacity.

10-4.3.2 Sliding

Passive resistance of the toe of the dam and foundation soil should be taken into account if the toe area is amply protected from scour. For existing dams where the downstream toe is inundated, adequate investigations should be made to evaluate existing scour. This is extremely important because if the soil can be eroded, it cannot be relied upon for passive resistance. If the scour protection is being continually displaced, heavier protection is required.

Sliding analysis should use the limit equilibrium approach as described in USACE ETL 1110-2-256 (1981). Special care should be taken to identify the presence of any thin weak lenses in the soil foundation such as clay layers in alluvium.

10-4.3.3 Deformation

Foundation settlement of existing dams is usually complete within a few years after construction. For existing dams, visual examination and surveys of the dam structure should be conducted to identify continuing movement and excessive or unequal settlement of the dam which would adversely impact the water tightness of the structure or the integrity of the dam and foundation seepage control facilities.

For new dams, verifying the overall stability includes an estimation of the absolute and differential settlements due to the soil foundation in order to make sure that deformation is compatible with the proper behavior of structures and with the deformation capacities of the rigid concrete dam. Adequate field studies are required to quantify the foundation deformability characteristics. Consideration should be given to the proper placement of keyed/unkeyed joints and waterstops in the rigid concrete dam to control differential settlement and seepage. A 50-foot spacing of contraction joints is usually sufficient. Where foundation conditions are such that undesirable differential settlement or displacement between adjacent blocks can occur, shear keys should be formed in the contraction joints. These may be formed vertically, horizontally, or in a combination of both, depending on the direction of the expected displacement. Leakage through the contraction joints is controlled by imbedding waterstops across the joints (USBR 1976).

10-4.4 Acceptance Criteria

Acceptance Criteria should be consistent with those given in Chapter III and Chapter IV of these Guidelines. The minimum sliding factor of safety for the worst static case should be 1.5.

The design must include adequate scour control measures if water passes over the crest, in order to prevent undermining the structure. Scour may be controlled by a concrete stilling basin apron for a short distance below the dam, the end of the apron being protected by an extension of rip-rap. The lower end of the apron is often further protected by a vertical diaphragm of concrete or sheet-piling, which serves to retain the foundation under the apron if the rip-rap is washed away. A vertical diaphragm is often also used at the toe of the dam to provide for a possible failure of the apron.

The size and extent of the riprap required in the exit area depends upon the effectiveness of the stilling basin, tailwater depth in the exit and configuration of the exit area. The maximum size is dependant on velocity and turbulence of water exiting the spillway apron. General guidelines for riprap design are given in USACE (1987). Model and computed results suggest a D_{50} size of 1.5 to 2 feet for velocities of 15 ft/sec and 3 to 4 feet for velocities of 30 ft/sec. Thickness for placement in the dry should be 1.5 D_{100} (max) or 2.0 D_{50} (max), whichever is greater. Thickness for placement underwater should be increased 50 percent. A bedding layer must be designed according to established filter criteria and placed under the riprap protection.

10-4.5 Material Properties

The construction materials for concrete dams on soil foundations are generally the same as for concrete gravity dams discussed in Chapter III, Section 3-6, of these Guidelines.

The geotechnical and subsurface investigations at the site must be adequate to determine the suitability of the foundation. FERC requirements are given in Chapter V of these Guidelines. Emphasis should be to determine the in-situ shear strength of the foundation soils and the presence of any weak layers.

Shear strength of the interface between concrete and soil can be assumed equal to the soil shear strength if the concrete has been cast directly against soil. If the concrete has not been cast directly against soil, such as in the case of the vertical surface of a retaining wall, the shear strength of the interface may be assumed to be 2/3 of the soil shear strength.

10-4.6 References

USACE, (U.S. Army Corps of Engineers), "Engineering and Design, Gravity Dam Design," EM-1110-2-2200, September 1958.

USACE, "Hydraulic Design of Flood Control Channels," EM1110-2-1601, July 1991.

USACE, "Hydraulic Design of Navigation Dams," EM-1110-2-1605, May 1987.

USACE, "Hydraulic Design of Spillways," EM-1110-2-1603, March 1965.

USACE, "Retaining and Flood Walls," EM-1110-2-2502, September 1989.

USBR, "Design of Small Dams," 1987.

USBR, "Embankment Dams," Design Standard No. 13.

10-5 Timber Dams

Timber dams include all dams that rely on timber for structural support and include timber buttress dams, timber crib dams, and embankment dams with timber cribbing used as reinforcement to steepen the slopes. Timber faced rockfill dams that rely on the timber only to limit seepage, or provide erosion protection, and embankments that have wooden trestles buried in them do not rely on the structural integrity of the wood for stability and are not considered to be timber dams.

Many of the early dams in the United States were built with timber. These were generally small dams that were designed empirically. An early text by Wegeman (1905) discusses the following six types of timber dams.

Brushwood Dams. Alternating courses of brushwood and gravel. Three- to five-foot thick courses of saplings and trees (with branches upstream) are sunk by placing stone and gravel on them. The dam was finished by covering the slopes with planking or rip rap.

Log Dams. Horizontal courses of logs with several inches of saplings, brush, stone, and earth placed between them. The butt ends of the logs were place downstream so that the dam had a triangular shape in cross section. The entire structure was covered with stone and earth.

Pile Dams. Consisted of one to three rows of timber piles driven vertically across the river with logs and brushwood placed horizontally between or against the piles.

Plank Dams. Formed by placing 2-inch by 12-inch by 12-foot planks to form vertical arches, convex upstream. Each course of planks was spiked or nailed to those beneath. On rock foundations, a single arch was used with the lower planks anchored to the foundation by iron bolts. On soil foundations, two arches were used with the space between filled with earth, gravel, or stone.

Timber Crib Dams. Consisted of square timber cribs filled with stone or gravel. The cribs were constructed by placing layers of timbers spaced 6 to 10 feet apart at right angles to the previous layer. The structure was faced with wooden planks spiked into the timber cribs. If placed on rock foundations, the bottom course of logs was fastened to the foundation by iron bolts.

Timber Buttress Dams. Deck and frame timber dams. Timber buttresses supporting a wooden plank face. These dams were typically spiked together. If founded on rock they were bolted to the rock with iron bolts. If placed on soil foundations, wooden sheetpiling was driven vertically from the heel (upstream toe) to form a seepage cutoff.

Few significant timber dams were built after the 1930s. Of the six designs discussed by Wegeman, only the timber crib and the timber buttress dam designs were durable enough for the structures to survive to the present. Many have been rehabilitated or modified to improve their integrity. Typical problems include deterioration of timber, undermining, excessive seepage, and wash out of rockfill. Modifications have included removal and replacement of the timber, concrete capping, and buttressing. Remnants of the other types may still be found in some dams.

Figures 10-5.1 and 10-5.2 show typical cross sections through timber crib and timber buttress dams. Creager, Justin, and Hinds (1945) discuss the timber crib dam as follows:

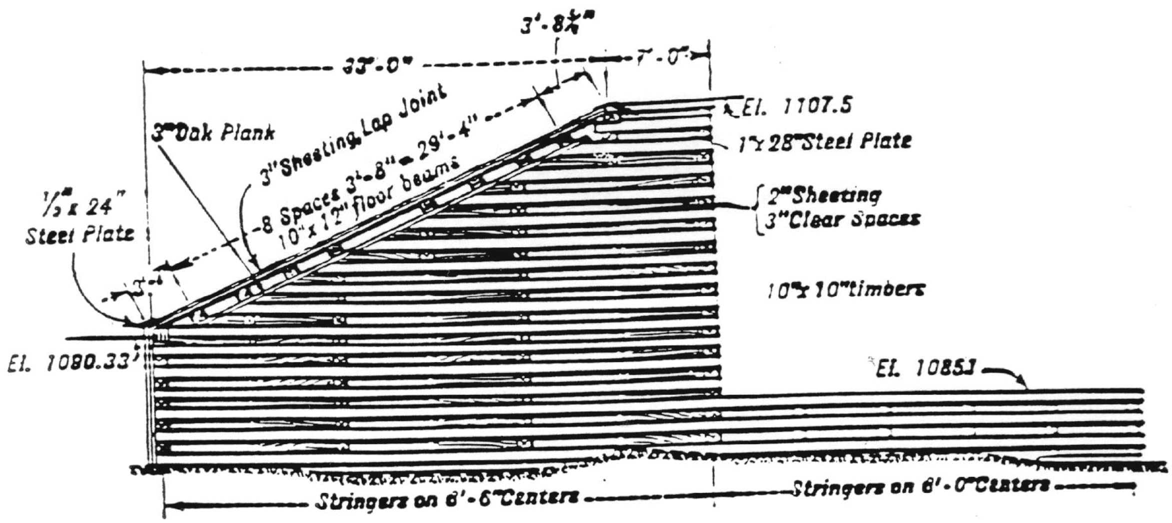

Figure 10-5.1 Typical Timber Crib Dam (source: Creager, Justin, Hinds 1945)

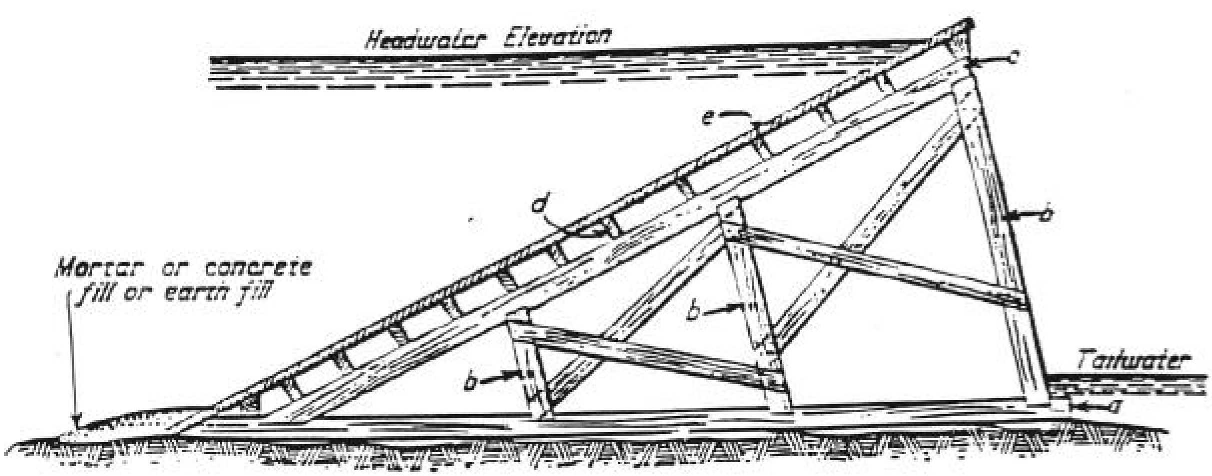

Figure 10-5.2 Typical Timber Buttress Dam (source: Creager, Justin, Hinds 1945)

> In this type of timber dam, cribs of round or squared timbers are drift-bolted together, filled with rock fragments or boulders, and topped by a plank deck. The timbers are usually spaced about 8 feet centers both ways. The bottom timbers of the cribs are often pinned to the rock foundation if the site is not submerged.

As much as 25 percent of a timber crib dam was wood. The logs were flattened at the ends to keep the sides of the cribs vertical and were spiked together with 3/4 inch square iron or steel bolts. Hardwood spikes were sometimes used in place of iron spikes.

On rock foundations, the upstream face was often sloped to improve stability by taking advantage of the weight of the water on the sloping deck. The downstream face was typically vertical and a timber apron extended downstream to prevent undermining.

On soil foundations, the section is usually reversed, with the upstream face vertical and the downstream face sloping and frequently stepped to limit erosion and undermining caused by overflowing water. The dams were typically wider than high to control underseepage. Often, wooden sheetpiling was driven vertically from the heel (upstream toe) to form a seepage cutoff.

Some timber crib dams were constructed in deep water by constructing the timber cribs on land, floating them into position, and sinking by filling with rockfill.

Creager, Justin, and Hinds (1945) discuss the timber buttress dam as follows:

> It is generally built of squared timbers and planks and is not rock-filled. For its stability it depends on the weight of the water on its deck and the anchorage of the sills to the foundation. .... The deck makes an angle of 30 degrees or less with the horizontal.
>
> The sill, *a*, (Figure 10-5.2) is first fastened to the ledge rock by wedge bolts or anchor bolts, preferably grouted in. The struts, *b*, are then framed to the sills and held in place by cross-bracing and batten blocks. The wales, *c*, are then placed, the entire structure being thoroughly drift-pinned together. These bents are placed from 6 to 12 feet apart, according to the height of the dam and the size of the timbers used. Across the bents are placed the studs, *d*, to which the lagging, *e*, is nailed. The lagging should be either tongued and grooved or lapped and should not be less than 2-inch stuff.

Less than 120 timber crib or timber buttress dams are currently under the jurisdiction of the FERC and nearly all of these are small, low hazard potential dams. Less than a dozen are significant or high hazard potential dams.

Timber cribbing was commonly used for steepening slopes around the turn of the century. Accurate figures on the number of significant and high hazard potential dams that used this type of construction are not available. Though timber is still commercially available and it may be appropriate in some situations such as for corrosive environments or bulkhead type cofferdams, new timber dams are generally not considered to be acceptable. The rest of this section is concerned only with existing dams.

10-5.1 Forces

Forces for the analysis of timber crib and buttress dams are generally the same as for concrete gravity dams discussed in Chapters III of these Guidelines. Forces for the analysis of timber crib supported slopes are generally the same as for retaining walls.

Uplift pressures for timber crib dams may be different from that commonly assumed for concrete gravity dams, because of the relative permeability of the dam. The location of the phreatic surface should be determined as would be done for an embankment dam. Alternatively, buoyant weight of the wood and rockfill could be used.

Uplift pressures for timber buttress dams should be the same as for buttress dams discussed in Section 10-2. Many timber crib structures have been grouted or capped with concrete and may therefore act similarly to a mass concrete structure with respect to uplift. In such cases, uplift must be considered.

10-5.2 Loading Combinations

Loading combinations for the analysis of timber crib and buttress dams are generally the same as for concrete gravity dams discussed in Chapter III of these Guidelines. Loading combinations for the analysis of timber supported slopes are generally the same as for embankment dams discussed in Chapter IV of these Guidelines.

10-5.3 Analyses

Methods of analyses of timber crib and buttress dams are generally the same as for concrete gravity dams discussed in Chapter III of these Guidelines. Methods of analyses of timber supported slopes are generally the same as for embankment dams discussed in Chapter IV Guidelines.

Low hazard potential timber dams and timber supported slopes whose failure would not jeopardize the safety of the dam do not require extensive evaluation. A cursory evaluation combined with vigilant surveillance for signs of distress should be sufficient for these structures.

Significant and high hazard potential timber dams warrant detailed stability analyses. The global stability of timber buttress and timber crib dams should be evaluated similar to concrete gravity dams, except that cracked base analyses are not appropriate.

Stresses at connections and within timber members of timber crib and buttress dams and timber crib supported slopes may be estimated with conventional structural analysis, mechanics of materials, and appropriate simplifying assumptions.

10-5.4 Acceptance Criteria

Acceptance criteria for timber crib and buttress dams are generally the same as for concrete gravity dams discussed in Chapter III of these Guidelines. Acceptance criteria for timber crib supported slopes are generally the same as for embankment dams discussed in Chapter IV of these Guidelines.

Connections between timbers should meet criteria in the National Design Specification for Wood Construction (American Forest and Paper Association 1991).

10-5.5 Material Properties

The strength of timber is highly dependent on the amount of deterioration that has occurred as well as the type of wood. For existing significant and high hazard potential timber dams, field inspections and investigations are extremely important to evaluate the extent of deterioration. Field investigations may also be appropriate to determine the species of wood, dimensions, and spacing of timbers. Field investigations that are appropriate may range from simple inspection by divers to exposing timbers in the fill. Whenever possible, a complete internal inspection of timber buttress dams is essential. Field investigations are also appropriate to determine the type, construction details, and condition of the connections between timbers.

10-5.5.1 Timber Deterioration

Although timber may last indefinitely when permanently submerged in fresh water or surrounded by saturated soil, untreated timber in air, unsaturated soil, partially saturated soil, or brackish water is susceptible to damage from decay, insects, and wood borers.

Reservoir fluctuations can result in wetting and drying cycles that are extremely detrimental to wood. Also, at times, deterioration of wood may not be visible.

Decay is caused by fungi or bacteria. Decay from fungi can completely destroy wood (dry rot). Wood that is exposed to the atmosphere is usually too dry to support fungus growth. Wood that is below water, below the phreatic surface, or embedded in clayey material is usually too wet to support fungus growth. Decay typically occurs in wood that is partially saturated. In some cases, wood submerged in water may be attacked by bacteria resulting a slow decay process (American Institute of Timber Construction 1985).

Damage from termites is the most common type of damage from insects. They are found throughout the United States, except for the north central states. They can completely destroy timber above the water level. Other types of insects that can damage wood include powder-post beetles and carpenter ants. Damage from these insects is usually minor (American Institute of Timber Construction 1985).

Marine borers inhabit salt or brackish waters and can severely damage wood below water level within a matter of months or years (American Institute of Timber Construction 1985). However, few if any hydroelectric projects under FERC jurisdiction are located in salt or brackish water.

10-5.5.2 Timber

Wood exhibits a wide range of engineering properties between species and even within species because of the differences in growth. Physical and mechanical properties of many species of wood are summarized in the Establishing Clear Wood Strength Values (ASTM 1981) and the Timber Construction Manual (American Institute of Timber Construction 1985).

Allowable stresses in several types of timber are given in Design of Pile Foundations (USACE 1991). Methods of establishing design stresses for timber piles and poles may be adapted for timber members in dams (ASTM 1974 and 1979).

For initial evaluations, estimates of engineering properties of wood may be developed knowing the species of wood, its condition, and tables of typical strengths. If unsatisfactory results are obtained, material properties should be developed from laboratory tests of a representative number of specimens (ASTM 1983a and 1984).

10-5.5.3 Rockfill

Testing of rockfill materials is often difficult and expensive because the large rockfill particles require large scale equipment and tests. It is usually sufficient to assume material properties based on tests on similar materials. Rockfill shear strength properties are summarized by Leps (1970).

10-5.6 References

American Forest and Paper Association, "National Design Standard for Wood Construction," Washington, DC, 1991.

American Institute of Timber Construction (Englewood, Colorado), Timber Construction Manual, 3rd Edition, John Wiley & Sons, New York, 1985.

ASTM (American Society for Testing Materials), "Standard Specification and Methods for Establishing Design Stresses for Round Timber Construction Poles," D-3200, Philadelphia, Pennsylvania, 1974.

ASTM, "Method for Establishing Design Stresses for Round Timber Piles," D-2899, Philadelphia, Pennsylvania, 1979.

ASTM, "Establishing Clear Wood Strength Values," D-2555, Philadelphia, Pennsylvania, 1981.

ASTM, "Small Clear Specimens of Timber, Methods of Testing," D-143, Philadelphia, Pennsylvania, 1983a.

ASTM, "Specific Gravity of Wood and Wood-base Materials," D-2395, Philadelphia, Pennsylvania, 1983b.

ASTM, "Static Tests of Timbers in Structural Sizes," D-198, Philadelphia, Pennsylvania, 1984.

Creager, P., Justin, J., and Hinds, J., Engineering for Dams, John Wiley & Sons, New York, 1945.

Leps, T.M., "Review of Shearing Strength of Rockfill," Journal of the Soil Mechanics and Foundations Division, American Society of Civil Engineers, July, 1970.

USACE (U.S. Army Corps of Engineers), "Design of Pile Foundations," EM 1110-2-2906, January 15, 1991.

Wegeman, E., "Design and Construction of Dams," 4th Edition, John Wiley & Sons, New York, 1905.

10-6 Inflatable Dams

An inflatable dam consists of a sealed, inflatable, rubber-coated fabric tube anchored to a concrete foundation constructed across a watercourse as shown on Figure 10-6.1. It is raised by inflating with air, water, or a combination of the two. When it is inflated to full design height, it impounds water and acts like any other fixed dam in this respect. However, it is capable of being completely deflated to allow maximum run-off during a storm, thereby reducing upstream flooding, and to allow passage of sediment, debris, and ice.

The first inflatable dam was installed in 1957 in the United States and in 1964 in Japan. There are no known United States standards for inflatable dams. As a result, the Japanese standards have been referred to for this section (LDTRC 1983).

Typical dimensions of an inflatable dam are generally less than 10 feet in height and 200 feet in length. However, heights up to 20 feet and unlimited lengths are possible. An inflatable dam has a service life of approximately 30 years and is adaptable for various installation. Typical uses are for diversion structures, check structures for flood control, overflow weirs, flashboard and gate replacement, sluice gates, control gates, and barriers for erosion.

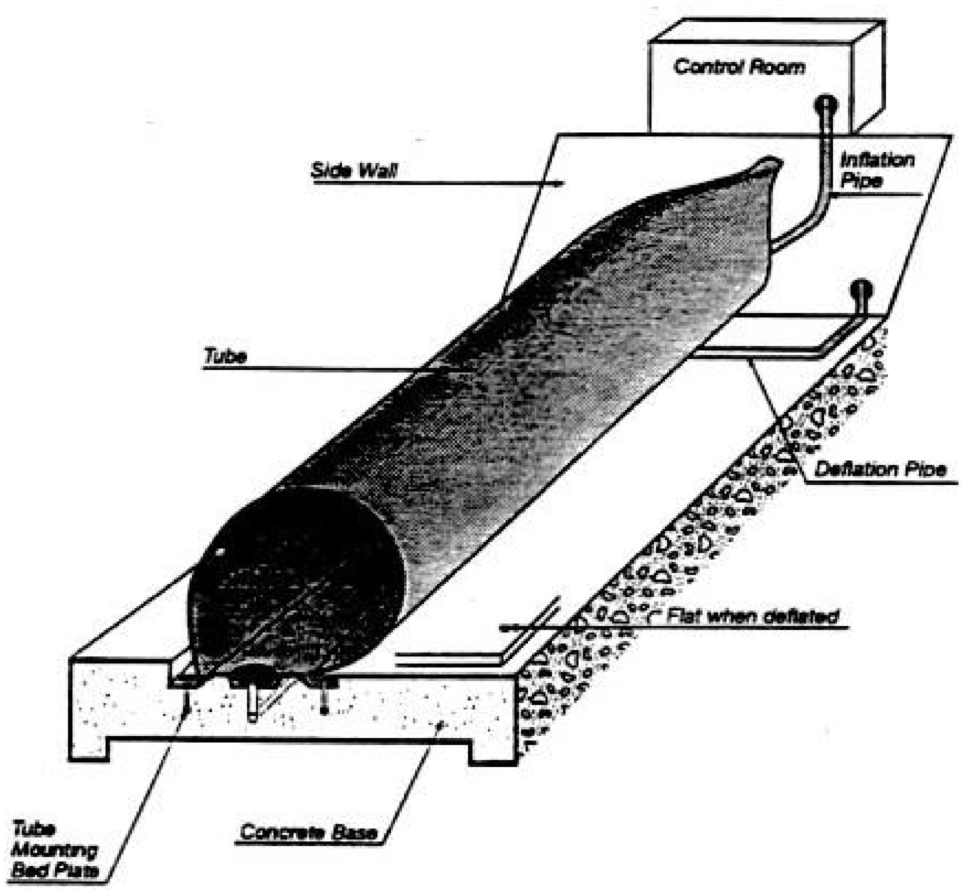

Figure 10-6.1 Inflatable Dam

The more important issues to be considered in their use are:

- Maximum overflow depth - if water overflow depth exceeds a certain limit, overflow pulsation may cause detrimental vibration in the tube.

- V notch effect - when air is used for inflation, a V notch forms during deflation of the tube resulting in flow concentration and difficulty in controlling a desired water level and flow rate. The location of the V notch should be controlled, otherwise it can happen anywhere and even move around. Since all the flow is concentrated in the V, adequate erosion protection must be planned for the area.

- Water level change and tube deformation - change of upstream and downstream water levels deform the tube and, consequently, change the dam height.

- Wave effects - tension fluctuation is produced by waves and may result in fatigue of the tube and fittings. Because of this, and the stress fluctuation caused by inflating and deflating, fatigue resistance must be guaranteed by the manufacturer.

- Flattening - under the conditions of a small upstream to downstream water level difference and small flow velocity, the tube may not totally flatten, thus increasing possible wear and/or damage to the tube.

- Prevention of damage to the tube - in a river with many rolling stones or flowing debris, the tube and fittings are vulnerable to wear and damage.

- Effect of silt - if a significant amount of silt accumulates on the tube while deflated, the tube may be difficult to inflate.

- Repair - when an inflatable dam is continuously submerged, repair in the wet is difficult to impossible.

- Vandalism is often a problem at inflatable dams. People tend to shoot at them. Compressors must have sufficient capacity to maintain the dam in an inflated state even with some holes in it.

10-6.1 Forces

Forces on an inflatable dam are generally the same as for concrete gravity dams and embankment dams, as discussed in Chapters III and IV of these Guidelines. Only exceptions, clarifications, and additions are discussed below.

10-6.1.1 Dead Loads

The effect of sediment buildup against an inflatable dam located at riverbed level may be significant and should be evaluated.

10-6.1.2 External Hydrostatic Pressure

The effect of dynamic forces caused by water flowing over the dam may require consideration in special cases. During deflation it is possible for the water pressure to pinch off sections of the dam trapping air in the middle making them impossible to fully deflate.

10-6.1.3 Ice Pressure

Ice pressure is normally not of great importance in the analysis of inflatable dams. Inflatable dams naturally pass ice (i.e., the ice pressure pushes the crest down slightly so it can pass). The excellent performance of the inflatable dam under ice conditions is due to its ability to deflect and absorb the thermal expansion of ice and the impact of ice flows. The tube also presents a sloped front to the ice, thus allowing it to ride up and over it, similar to the design of a sloped pier.

10-6.1.4 Temperature Effects

To consider a load caused by temperature change, a ±15°C temperature change and a coefficient of thermal expansion of 1×10^{-5} per °C should be adopted (LDTC 1983).

10-6.2 Loading Combinations

Loading combinations are generally the same as for concrete gravity dams, as discussed in Chapter III of these Guidelines.

10-6.3 Analyses

After the design parameters are established the tube suppliers generally perform the design and analysis of the superstructure, which includes the tube and tube attachment fittings. The substructure, which includes the piers, side walls, tube mounting base plate, aprons, and revetment, is generally analyzed by the inflatable dam designer. The stability of the substructure must be evaluated as with any other gravity structure. The manufacturer may provide the force on the anchoring strip for the dam, but if not, all vertical and horizontal forces must be determined as with any other structure.

10-6.3.1 Superstructure Analysis

Tube. The tube shape and tension are determined based on external pressure, internal pressure, and weir height.

Tube Attachment Fittings. Design of the tube attachment fittings should account for the tension in the tube's rubberized fabric and nut tightening forces. The fittings should be constructed so that the force in the tube's rubberized fabric is evenly held.

10-6.3.2 Substructure Analysis

The substructure includes a concrete base slab, side walls, tube mounting bed plate, and could include interior piers. The substructure stability should be analyzed by following the applicable methods presented in Chapter III (Gravity Dams), Chapter X, Section 10-3 (Pile Foundations) and Section 10-4 (Soil Foundations) of these Guidelines. Metal parts should be corrosion protected.

Tube Mounting Bed Plate. The width of the tube mounting bed plate required is dependent upon the height of the dam and the inflation medium used. The bed plate needs to be wide enough to allow sufficient room for the dam to lie flat when deflated. The tube mounting bed plate is designed to support the upper load and ensure tube water and air tightness. Stress of the tube mounting bed plate is calculated as a beam on an elastic floor or a cantilever fixed on the pier or side wall, depending on how it is constructed.

10-6.4 Acceptance Criteria

10-6.4.1 Pool Level

The tube does not deflate evenly, that is, the crest of the dam does not remain straight and parallel in relation to the foundation as it lowers. Discharging water rushing over a deflating dam increases the pressure acting on it, which tends to push it down in a particular place, creating a depression shaped like a "V" notch as the dam deflates to around 70-90% (depending on its height and length) of its normal operating height. The V-notch has the effect of concentrating water discharge at the point where this depression in the tube occurs, which makes water level adjustment difficult. To facilitate the start of deflation, a "V" notch (1 vertical to 10 or 20 horizontal) is formed by the foundation pad. Deflation then starts over this area.

10-6.4.2 Inflating Medium

Tube inflating medium should be selected from among water, air, and co-use of both considering the following:

- Natural conditions of the site (ambient temperature, foundation quality, fill water availability.

- Purpose and operation policy (required function, operation frequency).

- Maintenance and control.

- Economics.

10-6.4.3 Superstructure

The tube should be designed to satisfy the following requirements:

- Necessary dam height for all combinations of design water levels and design flow rates.

- Air and/or water tightness.

- Easily and adequately raised, and completely flattened in deflation.

- Sufficiently durable (service life of at least 30 years)

- No harmful vibrations.

- Convenient for maintenance.

Factors of Safety. The LDTRC (1983) recommends a factor of safety of 8 or more, for usual loading, be used for the tension in the tube after rubberization and adhesion. The attachment fittings should use a factor of safety of 3 or more. For earthquake design, the usual factors of safety can be reduced by 2/3.

Vibration. Due to the flexible structure of the inflatable dam, vibration takes place in the tube when the overflow and downstream water depth become greater. Generally, the higher the internal pressure, the more difficult it is for the tube to vibrate. In the case of the water-filled type, the tendency is that the larger the downstream water depth, the more easily the tube vibrates. The maximum recommended overflow depths from the viewpoint of vibration are (JIID 1989):

Air Inflation Type $h = 0.2H$

Water Inflation Type

downstream is exposed jet flow $h = 0.5H$

downstream water level higher	h = 0.4H, where (P/H 2.5 - 3.0)
	h = 0.3H, where (P/H 2.0 - 2.5)
	h = 0.2H, where (P/H 1.5 - 2.0)

where
- h = overflow water depth (ft)
- H = tube height at time of overflow (ft)
- P = Tube bottom internal pressure (water pressure in ft)

If the expected maximum overflow depth exceeds the above-mentioned maximum, vibration is possible. A spoiler or deflector (fin) should be provided to separate the nappe from the tube and reduce anticipated vibration.

External Damage. Adequate acceptance criteria should be established for external damage. Inflatable dams are susceptible to damage by abrading, cutting, and puncturing. Moving rocks, floating debris (such as trees), and vandalism are all sources for damage. The inflatable dam lies flat on its foundation (producing no obstruction) when deflated, which allows it to get out of the way of heavy flood-swept debris. When deflated, debris may land on the tube causing damage. To prevent such damage, polyethylene-based foam blocks may be placed inside the tube to act as cushioning against water borne debris. The amount of cushioning varies depending on the site conditions. The minimum and maximum amount of cushioning are 1.6 inch and 14.4 inches, respectively (Sumitomo 1991/1992).

Inflatable dams are highly resistant to wear and puncture. However, with considerable effort they can be cut and can suffer bullet damage. Knife cuts are more of a concern than bullet holes because the damage is larger and more difficult to repair. If the tube is punctured or cut it will usually not cause the dam to deflate since there is a sufficient air or water supply to maintain inflation until the damage can be repaired.

To deal with the threat of vandalism in high-risk areas, one manufacturer has developed a tube employing ceramic chips embedded in its outer lamination to provide protection from knife cuts.

10-6.4.4 Substructure

The substructure of the inflatable dam should be designed to be safe for expected loads, provide watertightness as required, and have adequate durability.

The acceptance criteria applicable to the substructure, such as sliding, settlement, bearing capacity and seepage are discussed in Chapter III and Chapter X, Section 10-3 (pile foundations), and Section 10-4 (soil foundations) of these Guidelines.

The foundation support should be selected considering the loading, effect of water flow, ground conditions, construction work, environmental conditions, safety, and economy. The foundation must not deform to a point where it impacts the inflation/deflation and pipes embedded in the tube mounting bed plate.

10-6.5 Material Properties

Inflatable dams have relatively simple foundation and structural requirements. A bed plate holding a rail and bolt assembly anchors the dam body with clamps and nuts. Inflation and deflation involves piping that leads from the dam to an intake/exhaust valve and pump system.

10-6.5.1 Site Investigation

The foundation investigations should follow the recommendations of Chapter V of these Guidelines. Bends in the river channel should be avoided. The plan configuration of the dam should be straight and normal to the flow.

10-6.5.2 Tube

The body of an inflatable dam must withstand tension, be air and watertight, be resistant to ozone to prevent deterioration from sun exposure, be abrasion resistant to minimize wear from rock transported in the stream, and be resistant to low and high temperatures.

Fabric. The fabric maintains, as a tension member, the tensile force caused by the internal pressure of the tube and the external water pressure. The fabric is normally a rubber and nylon laminated sheet. The stress bearing component is usually nylon. Depending on the load and environmental requirements, there are various fabrics available in terms of weaves, cord counts and weights.

A synthetic rubber is usually incorporated to provide air and watertightness and to protect the fabric. The rubber used should be designed with weatherability, ozone resistance, water tightness, and abrasion resistance taken into account. An Ethylene Propylene Diene Monomer (EPDM) based rubber or Chloroprene Rubber (CR) base are commonly used.

A number of fabric plies are laminated into a sheet to handle stress requirements. Strength is achieved by the lamination of fabric plies, and wear and weather resistance by the thickness. There are no known United States standards for inflatable dams. Appendix Table 1 in JIID (1989) lists properties of the tube.

Most repair is not difficult, rarely necessary, and can, depending on the position of the damage and river flow, normally be done without deflating the dam. In the case of minor damage (less than about 0.4 inch), it can be repaired using the rubber plug repair method that is used to repair the tubeless type tires. On the other hand, if the damage is relatively large (about 0.4 inch or more), a patch type repair as used for repairing a conveyor belt, etc. is carried out. In this instance, the repair has to be carried out with the bag deflated and in the dry (LDTRC 1983).

Dimensions. There is no limit to the length of an inflatable dam span from the standpoint of stress due to the fact that stress is evenly distributed along the entire length of the anchor line rather than concentrated at selected points. The length of a inflatable dam generally does not exceed 300 to 400 feet due to the economics of shipping and handling during installation. For longer dams concrete piers are used between each dam section.
The fabrics used for dam construction are capable of withstanding forces in dams up to about 20 feet in height. Under special circumstances (considerable downstream head against the dam), the total height of the dam may be safely increased.

Anchoring System. The clamping system acts as an anchor for the inflatable dam while also creating an air and watertight seal. This type of system provides for simple, fast, and assured installation at the dam site. The fixing method of the tube may include a single clamping line system (upstream side) or a dual clamping line system (upstream and downstream sides). In cases where there is a downstream water level or wave forces from the downstream are significant, it may be necessary to provide a dual clamping line system.

The fitting system consists of an embedded plate, clamping plate, and anchor bolts. Galvanized steel is generally used. Stainless steel is used if salt water is expected.

10-6.5.3 Inflation/Deflation System

Inflation and/or deflation of the dam can be simple or complex, depending on the requirements of the structure.

Safety apparatus that will prevent overinflation and ensure deflation should be provided. Adequate provisions must be made to ensure that the inflation/deflation system is operable at all times, particularly during adverse weather conditions. The system and backup power must be operated at least once each year, either during regular project operation or on a test basis.

The piping connecting the tube with the operation chamber includes the inflation medium supply and exhaust piping, the internal pressure sensing piping, and the drain piping. The piping connecting the operation chamber with the upstream and downstream sides of the tube includes the upstream water level detection pipe and the water exhaust pipe (water type only). The piping should be designed with sufficient capacity and durability based on the application and place of installation.

The size of the operating room depends on the inflation/deflation system chosen, the size of the dam, and the manufacturer of that system but is generally about 110 square feet. The operating room should be designed for safe and easy access and dry conditions, be equipped with lighting and ventilation, and display an instruction board of operating procedures in a suitable place.

Inflation System. The inflation system supplies inflating media to the tube. An engine- or motor-driven blower is used for air type tubes while an engine- or motor-driven pump and ancillary devices such as valves are used for water type tubes. The type, number, and capacity of units composing the power equipment should be selected taking dam size, design inflation time, control reliance, operation frequency, and cost into consideration. The types of valves and their operating forces should be selected based on the type of power equipment and the size of the dam.

The time needed for inflation is determined by the capacity of the pump or blower, diameter of feeding and discharging pipes, inner capacity, inner pressure, and loss rate of the tube. The inflation time required is normally within 10 minutes to 1 hour.

Deflation System. The deflation system removes the inflation medium from the tube. In addition to manual exhaust, an automatic deflation device that is linked with the upstream water level is normally provided.

Automatic deflation systems include mechanical float and bucket types, and electrical types. The float and bucket types introduce the river water into the operation chamber through an upstream water level detector pipe, and mechanically opens the valves by way of a buoyancy float or the weight of water. The electrical type detects the upstream water level by way of a water level indicator and opens electric valves, which is often jointly used with the float or bucket types, as a backup.

The deflation time differs depending on the size of dam, piping size and length, and reservoir capacity. The time for deflating is determined by the upstream and downstream water levels and internal pressure. Since these conditions are not constant, the deflating speed is not constant. Deflation time should be in accordance with the design criteria, generally from 10 minutes to about 1 hour in many cases. Normally the system is designed so that the deflation time can be adjusted by changing the opening of the discharge port.

10-6.6 References

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10-7 Stone Masonry Dams

Stone masonry dams are constructed of closely placed large stones with the spaces between the stones filled with mortar. (Dry-laid-stone dams are not considered here.) Stone masonry dams were common in the United States before about 1915.

Most stone masonry dams were made of unshaped quarried stone blocks of irregular sizes known as "rubble masonry." The size of the stone blocks varied, depending on the nature of the local rock, from several cubic feet to several cubic yards and weighed from several hundred pounds to about 5 tons. Stone masonry dams were constructed without regular lifts or vertical contraction joints. The stone blocks were fit together as tightly as possible, but often had large gaps between. Small rock chips or "spalls" were often placed with the mortar between the larger blocks. The mortar was composed of varying proportions of sand and cement, and was either packed into place by hand or grouted in place. Roughly 50 percent of a stone masonry dam was stone blocks, 25 percent spalls, and 25 percent mortar.

Many stone masonry dams were faced with ashlar masonry - stone blocks hewn into rectangular shapes of uniform sizes and fit tightly together. Typically every third or fourth stone know as a "header" was longer than adjacent stones and extended horizontally into the dam to interlock the facing with the rest of the dam.

Stone masonry dams are distinguished from cyclopean concrete dams in which large stone "plums" are bedded in concrete with the spaces between the stones filled with concrete. Typically about 25 percent of the dam is composed of large stones.

10-7.1 Forces

Forces for the analysis of stone masonry dams are generally the same as for concrete gravity dams, which are discussed in Chapter III, Section 3-2 of these Guidelines or the chapter on arch dams. Only exceptions, clarifications, and additions are discussed below.

For gravity stone masonry dams, uplift should be considered as it would be for concrete gravity dams. The argument is sometimes made that the uplift pressures would be relieved through unmortared or cracked mortared joints. Although there may be some merit it this argument, reduced uplift should not be used unless it is verified by piezometers.

10-7.2 Loading Combinations

Refer to Chapter III.

10-7.3 Analyses

Both gravity and arch dams were built of stone masonry. Stone masonry gravity dams were designed to avoid tension, but uplift was not considered. They should be evaluated similar to concrete gravity dams discussed in Chapter III of these Guidelines. Stone masonry arch dams were designed before analytical tools to evaluate arch stresses were developed. They should be evaluated the same as arch dams.

Stability should be evaluated for all kinematically possible failure surfaces through the dam. Although the principal of avoiding potentially weak horizontal planes through stone masonry dams by interlocking stones and avoiding horizontal courses was generally understood (Wegeman 1905), the possibility that they may exist should be evaluated. This is especially important for the analysis of flashboards on the upper courses of stone masonry dams. For flashboard installations, it may be necessary to tie the upper course of stones used for mounting flashboard pins to deeper courses within the structure.

By review of any construction photographs or drawings and data from similar dams, an assessment should be made of the extent of the design provisions included in the dam to prevent sliding, if any. Based on this assessment, conservative assumptions should be developed for evaluating the sliding resistance of various sliding planes being investigated.

10-7.4 Acceptance Criteria

Refer to Chapter III.

10-7.5 Material Properties

The unit weight of stone masonry dams can vary considerably and depends on the type and percentage of rock. Creager (1917) gives typical weights of various types of stone masonry dams. They vary from 130 pcf for sandstone rubble masonry to 165 pcf for granite ashlar masonry.

Creager (1945) gives unit weight, compressive strength, modulus of elasticity, Poissons ratio, shear strength, and coefficient of thermal expansion for a variety of building stones.

Generally the weakest material in a stone masonry dam is the mortar between the stone blocks. In many stone masonry dams, the mortar between the stone blocks has deteriorated resulting in leakage and occasionally deformation. Often the thin layer of mortar is cracked due to the different properties of the stone and the mortar. Unless there is a preponderance of evidence otherwise, the mortar should be assumed to be cracked and have no cohesion.

Rehabilitation of stone masonry dams may require use of stone that is similar in appearance to the original stone for aesthetic reasons. Care should be taken to select stone which will limit water absorption so that freeze-thaw damage can be minimized. Chinking repairs must be done with very hard stone.

10-7.6 References

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Veltrop J., "Concrete Arch Dams," Section 3 of <u>Development of Dam Engineering in the United States</u>, edited by E. Kollgaard and W. Chadwick, Pergamon Press, Elmsford, New York, 1988.

Wegeman, E., "Design and Construction of Dams," 4th edition, John Wiley & Sons, New York, 1905.

10-8 Water Retaining Power Plant Structures

This section covers the evaluation of a hydroelectric powerplant structure when the powerhouse and headworks form a part of the dam and retain water against them. The design is covered in detail in USACE (1993).

10-8.1 Forces

Forces are generally the same as for concrete gravity dams as discussed in Chapter III of these Guidelines. Only exceptions, clarifications, and additions are discussed below.

10-8.1.1 Dead Loads

Dead loads include the weight of the structure itself, including the walls, floors, partitions, roofs, and all other permanent construction and fixed equipment. The approximate unit weights of materials commonly used in construction can be found in AISC (1989 and 1991) and ANSI (1989).

10-8.1.2 External Hydrostatic Loads

The pressure of water in the penstocks should be included as hydraulic thrust resulting from wicket gate closure, depending upon the assumed conditions.

Since it is sometimes impracticable to protect the powerhouse against flooding at maximum tailwater elevation, a level should be selected above which flooding and equalization of interior and exterior water loads will occur.

10-8.1.3 Internal Hydrostatic Loads (Uplift)

Refer to Chapter III, Section 3-2.4, of these Guidelines for a detailed discussion on uplift forces.

It is nearly always advisable to reduce the uplift pressures on the draft-tube floor by means of a drainage system. When "floating" or relatively flexible floor slabs are used, they are not considered in the stability analysis, either as contributing weight or resisting uplift. When the floor slabs must take part of the foundation load, as is sometimes the case when the foundation is soil or poor rock, uplift should be assumed and the slab made an integral part of the draft-tube structure.

10-8.1.4 Compaction Residual Stresses

Compaction of backfill behind an unyielding wall tends to increase horizontal pressures beyond at-rest values. Estimation of these compaction residual stresses can be done as prescribed by EM-1110-2-2502, “Retaining and Flood Walls,” USACE 1989, Section 3-17, “Earth Pressures Due to Compaction.”

10-8.2 Loading Combinations

Loading combinations are generally the same as for concrete gravity dams, as discussed in Chapter III of these Guidelines. Case I and IA should be evaluated for normal loading conditions.

Case I Usual - Head gates closed, normal power pool, minimum tailwater, draft tube and spiral case open to tailwater.

Case IA Usual - Headgates closed, normal power pool, minimum tailwater, draft tube and spiral case empty. For a multi-unit plant, not all units should be assumed dewatered (50% would be acceptable).

Load cases IIA - Ice and III - Earthquake should be evaluated for both Cases I and IA.

Where only partial installation is to be made under the initial construction program, consideration should be given to the temporary loading conditions as well as those anticipated for the completed structures.

10-8.3 Analyses

Stability analyses are generally the same as for concrete gravity dams discussed in Chapter III of these Guidelines. Selection of the type of analysis should be governed by the design stage, the type and configuration of the structure considered, and the type of foundation.

Refer to Chapter III, Section 3-5.6.2 and Chapter X, Section 10-5 of these Guidelines, for analyses used for structures on soil foundations. If the foundation at the selected site is entirely soil, or is a combination of soil and rock, special consideration should be given to the possibility of unequal settlement.

Refer to Chapter X, Section 10-4 of these Guidelines for analyses used for structures on pile foundations.

10-8.3.1 Flotation

The structure should be adequately stable with respect to buoyant forces. The flotation safety factor, SF_f, is defined as:

$$SF_f = \frac{W_s + W_c + S}{U - W_g}$$

where: W_s Weight of the structure, including weights of fixed equipment and soil above the top surface of the structure.

W_c Weight of the water contained within the structure that is controlled by a mechanical operator (i.e., a gate, valve, or pump).

S Any surcharge loads (such as take-off towers or other structures).

U Uplift forces acting on the base of the structure.

W_g Weight of surcharge water above top surface of the structure that is totally controlled by gravity flow.

Vertical resistance mobilized by friction along the exterior faces of the structure should be generally neglected (USACE 1987).

The weight of generating machinery should be included in Ws, unless there is reason to believe that it will be removed and that it makes a significant contribution to the weight of the structure. Estimates of the weight of the embedded and rotating part of the generating machinery could be obtained from the equipment manufacturers for the unit ratings and specific data.

10-8.4 Acceptance Criteria

Refer to Chapter III, Section 3-5, of these Guidelines for a discussion of acceptance criteria. Only clarifications, exceptions, or additions are discussed below.

10-8.4.1 Flotation Stability

Concrete hydraulic structures should be designed to have the following minimum flotation safety factors (USACE 1987):

Loading Conditions	Minimum Safety Factor
Case I - Normal Operation	1.5

Case IA - Scheduled Maintenance [structure dewatered with normal tailwater(normal water pool)]	1.3
Extreme Maintenance [structure dewatered with maximum tailwater(max. water pool)]	1.1
Case II - Unusual Operation	1.3
Case IV - Construction	1.3

If the powerhouse has multiple units, the maintenance condition shall consist of 1 of the units de-watered. It shall be assumed that the machinery remains in the powerhouse, and therefore its weight shall be counted.

10-8.5 Material Properties

Refer to Chapter III, Section 3-6, of these Guidelines for a discussion of material properties.

General guidance on foundation properties may be found in Chapter III, Section 3-6.4 of these Guidelines.

10-8.6 References

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AISC, "Manual of Steel Construction Load and Resistance Factor Design First Edition," M015L, 1991.

ANSI (American National Standards Institute), "Building Code Requirements for Minimum Design Loads in Building and Other Structures," A58.1, 1989.

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10-9 Cellular Sheet Pile Structures

Cellular sheet pile structures consist of gravity retaining structures constructed of interlocking steel sheet piling forming adjacent cells that are filled with soil. They can be built in the wet, thus eliminating the need for dewatering. Cellular sheet pile structures are usually used as temporary cofferdams during construction. Occasionally they have been used as permanent retaining walls, fixed crest dams and spillway weirs. The FERC general policy is not to approve cellular sheet pile structures as permanent replacement dams.

The design of cellular sheet pile structures is discussed in several references (Belz 1970; Cummings 1957; LaCroix, Esrig, and Luscher 1970; Swatek 1966 and 1970; USACE 1989b; NAVFAC 1982; USS 1974). Planning, design, and construction of a temporary structure must be accomplished by the same procedures and with the same high level of engineering expertise as those required for permanent structures in order to protect personnel, equipment, and completed work.

There are three general types of cellular sheet pile structures, each depending on the weight and strength of the cell fill for its stability. The three common cell configurations and arrangements are the circular cell, diaphragm cell, and cloverleaf cell as shown in Figure 10-9.1.

<u>Circular Cells</u>. Consist of a series of individual large diameter circles connected by arcs of smaller diameter. These arcs generally intercept the circles at a point making an angle of 30 to 45 degrees with the longitudinal axis of the cofferdam. The primary advantage of circular cells is that each cell is self-supporting and independent of the adjacent cells. The circular cell can also be filled as soon as it is constructed, and it is easier to form by means of templates.

<u>Diaphragm Cells</u>. Comprised of a series of circular arcs connected by 120 degree crosswalls (diaphragms). The radius of the arc is often made equal to the cell width so that there is equal tension in the arc and diaphragm.

The stress at the joint of a diaphragm cell is smaller than that at the joint of a circular cell of an equal design. The diaphragm cell will distort excessively unless the various units are filled essentially simultaneously with not over 5 feet of differential soil height in adjacent cells. Unlike circular cells, diaphragm cells are not independently stable and failure of one cell could lead to failure of the entire cofferdam.

<u>Cloverleaf Cell</u>. Consists of four arc walls, within each of the four quadrants, formed by two straight diaphragm walls normal to each other, and intersecting at the center of the cell. Adjacent cells are connected by short arc walls and are proportioned so that the intersection of arcs and diaphragms form three angles of 120 degrees. The cloverleaf is used when a large cell width is required for stability against a high head of water. This type has the advantage of stability over the individual cells, but has the disadvantage of being difficult to form by means of templates. An additional drawback is the requirement that the separate compartments be filled so that differential soil height does not exceed 5 feet.

10-9.1 Forces

Forces for the analysis of cellular sheet pile structures are generally the same as for concrete gravity dams discussed in Chapter III of these Guidelines. Only exceptions, clarifications, and additions are discussed below.

10-9.1.1 Dead Loads

Dead loads should include the weight of the cell fill and the sheet pile shell.

10-9.1.2 Internal Hydrostatic Loads

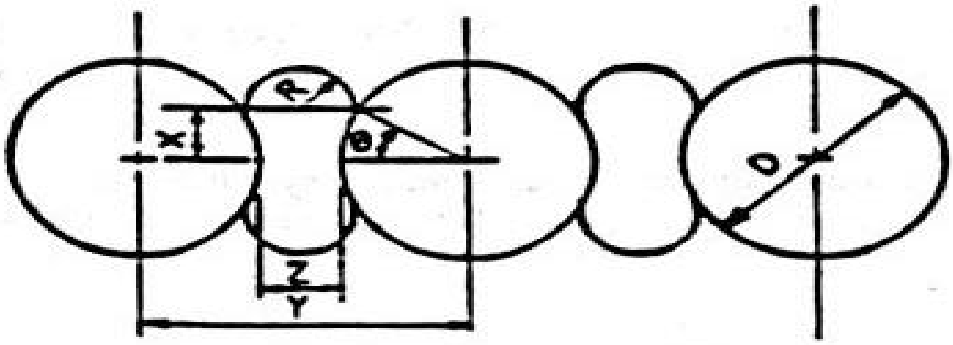
The slope of the saturation line is dependent upon the type of fill, the presence of a berm, and any positive measures taken to control the saturation surface in the cell or the berm such as weep holes in the cell or drains and pumped wells in the berm. The saturation level within the cell fill is perhaps the single most important consideration in the design of the cells; therefore, its location must be estimated with extreme care.

The location of the saturation line in a cell is usually estimated using empirical relationships based on the type of cell fill:

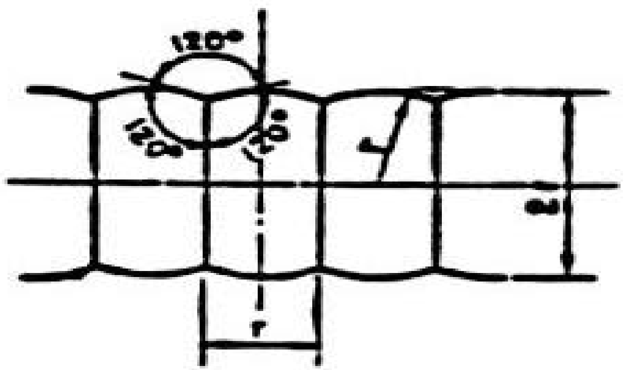
Slope	Fill
1H:1V	Free-draining coarse-grained
2H:1V	Silty coarse-grained
3H:1V	Fine-grained

These recommendations are conservative for most applications. Each design should be evaluated for conditions that would tend to raise the saturation line. If both the quality of the cell fill and the assurance of proper inspection cannot be guaranteed during construction, full saturation of the cell should be considered for design purposes. Some conditions that require evaluation are:

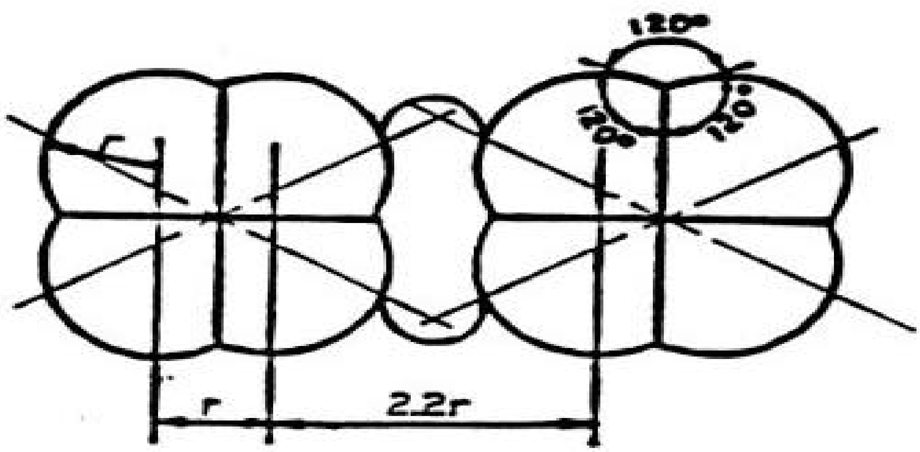
- possible leakage from pipelines crossing the cells;
- waves overtopping the outboard (upstream) sheet piles;
- excessive leakage through the outboard piles;



a. Plan circular cell



b. Plan arc and diaphragm cell



c. Plan clover leaf cell

Ref.: USACE 1989

Figure 10-9.1 Typical Arrangement of Circular, Diaphragm, and Cloverleaf Cells

10-9.1.3 External Earth Pressures

For computing the external earth pressures, reference is made to EM-1110-2-2503, USACE (1989b).

10-9.1.4 Berm Pressures

The passive force developed by a berm should be determined by a wedge analysis that accounts for the intersection of the failure wedge with the back slope of the berm. The Coulomb method of analysis or a Culmann graphical solution can be used when appropriate. The resistance provided by the berm should be limited to a value consistent with the berm reaction resulting from a sliding analysis.

10-9.1.5 Earthquake Forces

The seismic earth pressure and hydrodynamic pressures should be considered in the design and analysis. In addition to these loads, a reduction in strength of the foundation, cell fill, or berm can also simultaneously occur during an earthquake. Structures founded on loose, saturated, cohesionless materials or cohesive soils that contain lenses of loose saturated, cohesionless soil can lose much of their foundation support when subjected to earthquake loading. Similarly, the cell fill or the berm can also liquefy, increasing the lateral loading against the cell.

Further discussion of earthquake forces is given in Chapter III, Section 3-2.6 of these Guidelines.

10-9.1.6 Surcharge

Generally, the effect of surcharge from equipment working on the top of the cell is not significant because the horizontal earth pressure resulting from equipment is greatest at a shallow depth below the top of the cell where the interlock tension is low.

10-9.2 Loading Combinations

Loading combinations for cellular sheet pile structures used as dams are generally the same as for concrete gravity dams discussed in Chapter III of these Guidelines. Only exceptions, clarifications, and additions are discussed below.

10-9.2.1 Case I Usual Loading Combination

A cofferdam structure should be evaluated for drawdown (dewatering). The pool level inside the cofferdam is some specified distance below the pool level outside the cofferdam; the cell fill saturation line varies uniformly between the outside pool level and some specified distance above the pool level inside the cofferdam. This condition is checked to determine the maximum rate of dewatering. This condition can be critical for stability and interlock stress. Since the cell fill saturation level is critical, the actual saturation level must be monitored in the field during dewatering to verify the assumed conditions.

10-9.2.2 Case II Unusual Loading Combination

Cellular sheet pile structures are often used where they can be overtopped during flooding. Loading should consider the outside pool at the top of the cell with the cell fill saturation line assumed to slope from the top loaded face of the cell to the unloaded side of the cell. Flood gates should be provided to allow the interior of cofferdams to be flooded before the cells are overtopped.

10-9.2.3 Case IV End of Construction Combination

Forces acting upon a cofferdam can change significantly during construction and the stability should be evaluated at various stages of construction. For example, overburden may be present on the inboard side when it is initially dewatered; however, the overburden may subsequently be excavated, thus perhaps adversely affecting the stability of the cofferdam. For cellular fixed weir structures with flow over the weir, permanent upstream and downstream rock berms extending the full height of the cells are usually constructed for stability and scour prevention. Stability should be evaluated for the case where the cell is filled before the berms are placed and after the berms are placed. Maximum interlock stresses will probably occur in the construction condition when the cells are filled and before the berms are built.

Where the structure serves as a construction cofferdam, evaluate balanced pools on both the inside and outside of the cofferdam. For determination of maximum interlock stress, cell fill is assumed to be completely saturated to the top of the cell unless positive measures are taken to prevent fill saturation.

10-9.3 Analyses

The stability of a sheet pile cell results from the composite action of the soil fill and the interlocking steel piling. Because of this composite action, cells cannot be classified as a traditional concrete gravity monolith or a flexible earth embankment.

Analyses should evaluate the external stability (sliding, overturning, rotation, bearing capacity, settlement, seepage, and scour) and internal stability (pile interlock tension, tilting, pullout, penetration). A summary is provided below. A more detailed discussion is given in USACE (1989b).

The equivalent width, B, of a sheet pile cellular structure is used for design purposes. It is defined as the width of an equivalent rectangular section having a section modulus equal to that of the actual structure and may be expressed as A/2L, where A is the area of a main cell plus one connecting cell and 2L is the center to center distance between the main cells.

10-9.3.1 External Cell Stability

10-9.3.1.1 Sliding

Sliding along the base or deep seated sliding below the base of most sheet pile cellular structures can be adequately assessed using a limit equilibrium approach. A two-dimensional analysis with an assumed plane failure surface is usually sufficient. A more detailed analysis should be done if unique, three-dimensional geometric features and loads critically affect the sliding stability of a specific structure.

Considerations regarding displacements are excluded from the limit equilibrium approach. The relative rigidity of different foundation materials and the sheet pile cellular structure may influence the results of the sliding stability analysis. Such complex structure-foundation systems may require a more intensive sliding investigation than a limit equilibrium approach. The effects of strain compatibility along the assumed failure surface may be included by interpreting data from in situ tests, laboratory tests, and finite element analyses.

The possibility of a deep-seated failure along any weak seams below a cellular structure should be evaluated. Weak seams may exist as clay seams in alluvium or between competent rock strata in sedimentary rock formations. Seams of pervious sand within a clay deposit may permit the development of excess hydrostatic pressure, which reduces the effective stress and subsequently reduces the shearing resistance. A weak seam may also appear after excavation due to the removal of overburden pressure, which results in a drop in the shear strength of clay shale to its residual strength.

If sufficient space is available, a soil berm may be constructed on the unloaded side to increase the stability against sliding on the base. The berm will also serve to lengthen the path of seepage and decrease the upward seepage gradients on the unloaded side. However, the berm will require a larger cofferdam enclosure and an increase in the overall length of the cofferdam, which will increase construction and maintenance costs. Also, an inside berm inhibits inspection of the inside piling for driving damage and makes cell drainage maintenance more difficult. In order for a berm to function as designed, the berm must be constantly maintained and protected against erosion and the degree of saturation must be consistent with design assumptions.

10-9.3.1.2 Overturning

A soil-filled cellular structure is not a rigid gravity structure that could fail by overturning about the toe of the inboard side. Before overturning could occur, the structure must fail from causes such as pullout of the sheet piles at the heel and subsequent loss of cell fill. Nevertheless, a gravity-block analysis may serve as the starting point for determining the required cell diameter. Considering that the cell fill cannot resist tension, the cell should be proportioned so that the resultant of all forces falls within the middle one-third of the equivalent rectangular base. This type of analysis will also serve to determine the foundation pressures.

10-9.3.1.3 Rotation

Hansen's method considers cellular structures to act as rigid bodies (Hansen 1953; Oveson 1962). Stability, as determined by Hansen's method, is directly related to the engineering properties of the cell fill and the foundation, and properly considers the saturation level within the cell as well as seepage forces beneath the cell. Design should not be based on the Hansen method; rather, it should be emphasized as a sensitivity check only.

10-9.3.1.4 Bearing Capacity

The bearing capacity of granular soils is generally good if the penetration of the sheet piles into the overburden is adequate to control seepage of water underneath the cell base. Clay should be stiff to hard for a good bearing capacity. However, even on relatively soft soils, cellular structures have been successfully constructed using sand or rockfill berms (Cummings 1957). The bearing capacity of both cohesive and granular soils supporting cellular structures can be determined by Terzaghi's method of analysis. However, the failure planes assumed for the development of the Terzaghi bearing capacity factors do not appear to be as realistic as those developed specifically for cellular structures by Hansen. Hence, for bearing capacity investigation, the Hansen method of analysis should also be used (USACE 1989b).

The bearing capacity of rock is not readily determined by laboratory tests on specimens and mathematical analysis, since it is greatly dependent on the influence of nonhomogeneity and geologic defects on the behavior of rock under load. To allow for the possibility of unsound rock, a higher factor of safety is generally adopted to determine allowable bearing pressure.

10-9.3.1.5 Settlement

Two types of settlement can occur within a cellular sheet pile structure: settlement of the cell fill and settlement of the sheet piles. In some cases, settlement can also be caused by dewatering the cofferdam area. The impact of such settlement on the structure design and performance should be evaluated.

The settlement of cell fill occurs under the self load of the fill placed within the cell. For granular fill, generally a majority of the settlement will have been accomplished soon after the fill placement. Hence, the postconstruction settlement of granular cell fill under its own weight is, generally, insignificant. Any volume decrease of the cell fill due to settlement can always be compensated by placing additional fill in the cell before any other load is applied to the cell. The settlement of cell fill may also result from seismic liquefaction of uncompacted sand.

A cellular structure underlain by compressible soils below its base will undergo settlement due to the weights of the cell and berm fills. If the compressible soils below the structure continue to consolidate after the reservoir loads have been applied, the reservoir loads can create a moment on the structure producing an unequal distribution of pressure at the base of the cell resulting in differential settlement. Dewatering of a cofferdam may cause drawdown of water levels within soil layers located below nearby existing structures or utility lines, potentially resulting in undesirable settlement.

10-9.3.1.6 Seepage

Foundation underseepage is generally not a problem for structures built on clay or good quality rock foundations. Problems almost always are confined to coarse-grained soil such as gravel and sand and sometimes silty materials. The most serious conditions occur where undetected pervious seams exist in the foundation. Fill material should be of sufficient coarseness to prevent material loss through interlocks.

Major problems associated with seepage below a sheet pile cellular structure are listed as follows.

- Piping, boils, or heave of the soil mass in front of the toe may occur if the exit gradient exceeds the critical hydraulic gradient. Boils and heave will considerably lower the bearing capacity of the soil, potentially resulting in toe failure of the cell. Piping causes loss of materials underneath the cell foundation and may cause excessive settlement and eventual sinking of the cell.

- Upward seepage forces at the toe may excessively reduce the passive resistance of the soil. This loss of lateral resistance may cause sliding or overturning failure of the cell.

- Seepage forces acting on the soils at the inboard face of the cell may excessively increase the hoop stress in the sheet piles. This may increase the possibility of interlock failure of the sheet piles and result in the loss of cell fill.

Cofferdams on sand are often designed using a trial sheet pile penetration of two-thirds of the upstream water head. A flow net analysis is most often used to estimate the seepage forces. If the exit gradient at the toe of the structure is large, a loaded filter or a wide-base berm should be considered. Seepage pressures should be estimated as discussed in Chapter III, Section 3-2.4.4.

10-9.3.1.7 Scour

Scour has contributed to a number of cellular structure failures. By removing the lateral earth support, the interlock stresses increase. Continued scour exposes sheet piles that have not penetrated to rock or were driven out of interlock and result in loss of cell fill and subsequent failure.

Damage by scour should be prevented either by protecting the outside of the cell with riprap or by carrying the sheets to a greater depth. Deflectors designed to streamline flow are effective in minimizing scour along the face of the structure.

10-9.3.2 Internal Cell Stability

10-9.3.2.1 Interlock Tension

A cell must be stable against bursting pressure; that is, the pressure exerted against the sheets by the fill inside the cell must not exceed the allowable interlock tension. The interlock tension developed in a cellular structure is a function of the internal cell pressure and is proportional to the radius of the cell. It is common practice to calculate the interlock tension in the main cell sheet piling based solely on the internal pressure. The coefficient of internal pressure is dependent upon the type of cell fill material and the method of placement. Table 4-2 in USACE (1989) recommends a coefficient in the range of 1.2 Ka to 1.6 Ka where Ka is the active coefficient of horizontal earth pressure. The location of the maximum horizontal pressure is dependent on cell restraint and is shown in Figure 4-16 of USACE (1989). Further changes in the depth of overburden, removal of berms, saturation level in the cell fill, and rate of dewatering must be anticipated when determining the maximum interlock tension.

The interlock tension at the connections between the main cells and the connecting arcs is increased due to the pull of the connecting arcs. For critical structures, special analyses such as finite elements should be used to determine interlock tension at the connections.

Several methods to reduce interlock tension are given in USACE (1989).

10-9.3.2.2 Tilting

Tilting of cells is resisted by both the vertical and horizontal shear resistance of the soil in the cell, to which the frictional resistance of the steel sheet piling is added. Vertical shear resistance is determined by the theory developed by Terzaghi (1945). The horizontal shear resistance is determined by the theory proposed by Cummings (1957). Both of these methods of analyses should be used independently to determine the adequacy of the cell to resist tilting. Additionally, tilting resistance of cells founded on overburden should be investigated by the theory proposed by Schroeder and Maitland (1979). Refer to USACE (1989) for further discussion.

10-9.3.2.3 Pullout of Outboard Sheeting

The penetration of sheetpiling is generally determined by the need to control seepage by increasing the flow path. However, the penetration must also be adequate to insure stability with respect to pullout of the outboard sheeting due to tilting. The calculated overturning moments are applied to the sheet piles, which are assumed to act as a rigid shell. Resistance to pullout is computed as the frictional or cohesive forces acting on the embedded length of piling. Typical values of steel sheet piling against various soils are given in Table 4-3 of USACE (1989).

10-9.3.2.4 Penetration of Inboard Sheets

The penetration of the sheet piles on the inboard side must be sufficient to prevent any further penetration under loading. The factor of safety against sheet pile penetration is defined as the ratio of the shear resistance on both sides of the embedded portion of the piles on the unloaded side to the internal downward shear force on the unloaded side.

10-9.3.3 Dynamic Analysis

When seismic excitation is a design consideration, the structure should be sited, if practicable, on competent rock. Such siting will automatically preclude the possibility of strength reduction beneath the structure under earthquake conditions. For structures situated on sand or clay, a refined analysis to evaluate the effect of cyclic loading on strength is necessary.

The sliding stability of a sheet pile cellular structure for an earthquake-induced base motion should be checked by assuming that the specified horizontal and vertical earthquake acceleration act in the most unfavorable direction. The earthquake-induced forces on the structure and foundation wedges can then be determined by a rigid body analysis.

A general discussion of seismic design of cellular sheet pile structures is presented by Chakrabarti et al. (1978).

10-9.3.4 Finite Element Analysis

The application of finite element model (FEM) analysis to date has been to develop its state of the art to the point where it can be used to refine existing design techniques and to analyze potential failure modes that cannot be checked by other methods. All studies so far have been made by researchers or engineers who are extremely familiar with the FEM techniques using specialized FEM programs for soil and structure modeling. The FEM analysis does not yet lend itself to application to typical design engineers working with currently available general-use programs. USACE (1989) provides further discussion on the use of FEM.

10-9.4 Acceptance Criteria

10-9.4.1 Factors of Safety

Considering the temporary application of these structures, the recommended minimum factors of safety for various potential failure modes are listed in Table 10-9.1.

TABLE 10-9.1
RECOMMENDED MINIMUM FACTORS OF SAFETY

	Loading Combination		
Failure Mode	Usual	Unusual	Extreme
Sliding	1.5	1.5	1.3
Overturning (gravity block)[1]	Inside Kern	Inside Kern	Inside Base
Rotation (Hansen)[1]	1.5	1.25	1.1
Bearing capacity Sand Clay	 2.0 3.0	 2.0 3.0	 1.3 1.5
Interlock tension[2]	2.0	1.5	1.3
Vertical shear resistance (Terzaghi)	1.5	1.25	1.1
Horizontal shear resistance (Cummings)	1.5	1.25	1.1
Vertical shear resistance (Shroeder-Maitland)[1]	1.5	1.25	1.1
Pullout of outboard sheets[1]	1.5	1.25	1.1
Penetration of inboard sheets[1]	1.5	1.25	1.1
Piping	2.0	2.0	2.0

[1]Design should not be based on these modes of failure, but rather these analyses should be employed as sensitivity checks only.
[2]The factor of safety against interlock tension failure should be applied to the interlock strength value guaranteed by the manufacturer for the particular grade of steel. The guaranteed value for used piling should be reduced as necessary depending upon the condition of the piling.

10-9.4.2 Cell Deformation

The cell is a yielding structure that can adapt to some deformation. When a cell is initially filled, it bulges out under the imposed lateral stresses, which causes take-up of slack in the interlocks and strain in the sheet piles. The maximum bulging usually occurs at a depth approximately 2/3H below the top of the cell (where H = cell height above dredge line) and measures on the order of 3 to 6 inches. Additional deformation is created in circular cells by the pull of the adjacent cells when they are filled. Under the application of the horizontal load, high cellular cofferdams may deflect at the top anywhere from 3 to 18 inches. The above order of movements are not expected to lead to rupturing of interlocks or excessive bending at connections (LaCroix 1970).

10-9.5 Material Properties

The material strength assumptions made during the design process must be verified by the site investigations and attained in the field during the construction process. Specific details concerning geotechnical investigations are contained in Chapter V of these Guidelines. The following general guidance shall apply.

10-9.5.1 Foundation Properties

A detailed foundation investigation should be performed, because the foundation conditions significantly impact the cost and degree of difficulty in construction and eventual integrity of the cellular structure. Continuous soil sampling or rock coring should extend to at least 15 to 20 feet below the design base level of the cell.

The program must be refined through analysis of the geologic details to provide specific and reliable information on the character of the overburden:

- typical obstacles such as boulders in the overburden that may cause difficulty in driving the cell sheets and lead to interpretations of a false top of rock;
- the depth to and configuration of the top of rock;
- the depth and character of bedrock weathering and discontinuities;
- the physical properties of the foundation materials;
- the elevation and fluctuation limits of the ground water;

- potential foundation problems and their treatment, such as leakage and stability.

A reliable estimate of water inflow as well as an accurate determination of the elevation and fluctuation of the groundwater table are primary concerns in the design and construction. One method of obtaining information is the field pumping test, which may be performed to determine the permeability of the foundation materials.

10-9.5.2 Cell Fill

The performance of the sheet pile structure is directly related to the drainage characteristics of the cell fill. Clean, coarse-grained, free-draining granular (soils with less than about 5 percent of the particles by weight passing the No. 200 sieve and 15 percent passing the No. 100 sieve) soils are preferred for cell fill because they lower the seepage line. This improves the cell performance by reducing the sheet pile interlock force, increasing the lateral sliding resistance, and increasing the internal shear resistance.

10-9.5.3 Steel Sheet Piling

New piling in good condition should be considered for major structures. Steel for sheet piling should conform to the requirements of the following American Society for Testing and Materials (ASTM) standards:

A328	Steel Sheet Piling
A572	High-Strength Low-Alloy Columbium Vanadium Steels of Structural Quality
A690	High-Strength Low-Alloy Steel H-Piles and Sheet Piling for Use in Marine Environments

A328 is the basic sheet piling and is satisfactory for most installations. A572 specifies high-strength sheet piling and is applicable for use in large diameter (>70 feet) cells where high interlock strength is required. A690 steel sheet piling provides greater corrosion resistance than other steels and should be considered for use in permanent structures in corrosive environments.

Cold-formed steel sheet piling is also available, although this piling has limited applicability. Presently, there is no ASTM specification covering this piling.

An extruded wye, using A572, Grade 50 steel, is available on a limited basis. These wyes have a small cross section and are extremely flexible, thus creating handling and driving difficulties. As a result of this characteristic, together with their limited availability, the use of these extruded wyes is not recommended.

Since tees and wyes are subjected to high local bending stresses at the connection, strong ductile connections are essential. Welded connections do not always meet this requirement because neither the steel nor the fabrication procedure is controlled for weldability. Therefore all fabricated tees, wyes, and crosspieces shall utilize riveted connections. In addition, the piling section from which such connections are fabricated shall have a minimum web thickness of one-half inch. Only straight web pile sections shall be used for cells as the hoop-tension forces would tend to straighten arch webs, thus creating high bending stresses (USACE 1989b).

When cofferdams are used as permanent structures, especially in polluted brackish water, or seawater, severe corrosion occurs from the top of the splash zone to a point just below mean low water level. In these areas, protective coating, corrosion resistant steel, and/or cathodic protection should be used.

10-9.6 Instrumentation

The kinds of instruments selected will depend on the purpose, project conditions, and the variables that will be monitored. The following is a brief description of the more common instruments used in a program to monitor steel sheet pile structures:

- Observation wells are mainly used to measure unconfined ground-water levels and are monitored directly by a probe or tape. They may be installed to monitor ground-water levels in the cell fill, backfill materials, and stabilizing berms.

- Piezometers are used to monitor pore pressures in the cell fill and foundation, in the stabilizing berms, and in the backfill material.

- Inclinometers can be used to monitor horizontal deformation within the cell fill, along the length of a sheet pile section, in the cell foundation, and within the stabilizing berm.

- Earth pressure measuring devices are designed to measure the total stress at a point in an earth mass or to measure the total stress or contact stress against the face of a structural element.

- Strain gages are used to observe the interlock tension within sheet pile members.

- Precise measurement systems are used to detect horizontal and vertical surface displacement by making precise measurements of lengths, angles, and alignments between reference monuments and selected points on the structure.

A more detailed discussion of instrumentation is provided in Chapter IX of these Guidelines.

10-9.7 Construction Considerations

The safety and performance of cellular sheet pile structures are very sensitive to site conditions and construction practices. Great care must be taken to ensure that the effects resulting from all potential construction and inservice site conditions, and construction techniques are properly anticipated, considered, and accounted for in the design. In addition, construction progress must be closely monitored by design personnel in order to evaluate or verify design assumptions and to recognize any changed conditions that might require a design modification.

- All handling holes in the sheet piling on the loaded side of the structure should be plugged. This is necessary to prevent an objectionable amount of water from entering the cell or loss of cell fill.
- Sheet piling should not be driven through overburden containing boulders. Extremely dense overburden should be excavated to a depth such that it can be penetrated without damaging the piling. Although dependent on the nature of overburden, 30 feet is generally accepted as a maximum depth to drive through overburden.

- Setting sheet piling on bare rock should be avoided wherever possible since support from the overburden is beneficial in helping maintain the desired cell configuration.

- Wherever cells and fill are placed against sloped or stepped faces of existing concrete, care should be taken to seal the contact between the sheet piles and concrete to prevent infiltration of water that could saturate the fill or cause piping.

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10-10 Mechanically Stabilized Earth Dams

Mechanically Stabilized Earth (MSE) walls and slopes are cost-effective soil retaining structures that can tolerate much larger settlements than reinforced concrete walls. By placing tensile reinforcing elements in the soil, the strength of the soil can be improved significantly such that a vertical face of the soil/reinforcement system is self supporting.

Use of a facing system to prevent soil raveling between the reinforcing elements allows very steep slopes and vertical walls to be safely constructed.

MSE is defined as any wall or slope supporting system in which reinforcing elements are placed in a soil mass to improve its mechanical properties. Reinforcing elements is a generic term that encompasses all man-made elements incorporated in the soil to improve its behavior. Examples are steel strips, geotextile sheets, and steel or polymeric grids. The term reinforcement is used only for those elements where stress transfer occurs continuously along the reinforcement element. Other inclusions may act simply as tendons between the wall face and an anchorage element.

Facing is a component of the reinforced soil system used to prevent the soil from raveling out between the rows of reinforcement. Common facings include precast concrete panels, metal sheets and plates, gabions, welded wire mesh, shotcrete, wood lagging and panels, and wrapped sheets of geosynthetics.

Geosynthetics is a generic term that encompasses flexible synthetic materials used in geotechnical engineering such as geotextiles, geomembranes, geonets, and polymer grids (also known as geogrids).

Suitability. MSE is an attractive design alternative when space is limited. MSE has been used to raise dam heights, steepen slopes, and replace traditional sloped earth embankments with vertical faces. In a few cases MSE walls have been used for spillways however, this is discouraged.

MSE is a relatively new technology, having been used in dams in the United States since about the 1980's. The use of MSE in design of new dams and the repair of existing dams is growing rapidly, as is the development of new and different types and uses of reinforcement. MSE has been used in few projects with installed lifetimes exceeding 30 years. Since the service life of dams should be considered indefinite, prudence demands that the most durable materials be used. Experience with MSE is short and the long-term performance of these products is still pretty much unknown.

The potential for catastrophic damage and loss of life in the event of a dam failure suggests a cautious approach to use of MSE in dams. MSE should only be used:

- where it is not critical to the long-term performance of the dam, and generally where it can be readily exposed, repaired, or replaced if necessary;

- in a configuration where it does not serve the sole defense against dam failure. Particular attention must be given to the potential for a domino type failure, such as one facing unit dislodging, resulting in failure of the adjacent area.

Types. Table 10-10.1 provides a summary of many of the current systems by proprietary name, reinforcement type, and facing system. MSE systems can be described by the reinforcement geometry, the stress-transfer mechanism, the reinforcement material, and the extensibility of the reinforcement material as shown in Table 10-10.2.

TABLE 10-10.1
SUMMARY OF REINFORCEMENT AND FACE PANEL DETAILS FOR VARIOUS MSE SYSTEMS

SYSTEM NAME	REINFORCEMENT DETAIL	TYPICAL FACE PANEL DETAIL[1]
Reinforced Earth: The Reinforced Earth Company 2010 Corporate Ridge McLean, VA 22102	Galvanized Ribbed Steel Strips: 0.16 in (4mm) thick, 2 in (50 mm) wide. Epoxy-coated strips also available.	Facing panels are cruciform shaped precast concrete 4.9 ft x 4.9 ft x 5.5 in (1.5 m x 1.5 m x 14 cm). Half-size panels used at top and bottom.
VSL Retained Earth: VSL Corporation 101 Albright Way Los Gatos, CA 95030	Rectangular grid of W11 or W20 plain steel bars, 24 in x 6 in (61 cm x 15 cm) grid. Each mesh may have 4, 5 or 6 longitudinal bars. Epoxy-coated meshes also available.	Precast concrete panel. Hexagon-shaped, (59-1/2 in high, 68-3/8 in wide between apex points, 6.5 in thick (1.5 m x 1.75 m x 16.5 cm).
Mechanically Stabilized Embankment: Dept. Of Transportation Div. Of Engineering Services 5900 Folsom Blvd., P.O. Box 19128 Sacramento, CA 95819	Rectangular grid, nine 3/8 in (9.5 mm) diameter plain steel bars on 24 in x 6 in (61 cm x 15 cm) grid. Two bar mats per panel (connected to the panel at four points).	Precast concrete; rectangular 12.5 ft (3.81 m) long, 2 ft (61 cm) high and 8 in (20 cm) thick.
Georgia Stabilized Embankment: Dept. Of Transportation State of Georgia No. 2 Capitol Square Atlanta, GA 30334-1002	Rectangular grid of five 3/8 in diameter (9.5 mm) plain steel bars on 24 in x 6 in (61 cm x 15 cm) grid 4 bar mats per panel.	Precast concrete panel; rectangular 6 ft (1.83 m) wide, 4 ft. (1.22 m) high with offsets for interlocking.
Hilfiker Retaining Wall: Hilfiker Retaining Walls PO Drawer L Eureka, CA 95501	Welded wire mesh, 2 in x 6 in grid (5 cm x 15 cm) of W4.5 x W3.5 (.24 in x .21 in diameter), W7 x W3.5 (.3 in x .21 in), W9.5 x W4 (.34 in x .23 in), and W12 x W5 (.39 in x .25 in) in 8-ft-wide mats.	Welded wire mesh, wraps around with additional backing mat and 1.4 in (6.35 mm) wire screen at the soil face (with geotextile or shotcrete, if desired).
Reinforced Soil Embankment: The Hilfiker Company 3900 Broadway Eureka, CA 95501	6 in x 24 in (15 cm x 61 cm) welded wire mesh: W9.5 to W20 - .34 in to .505 in (8.8 mm to 12.8 mm) diameter.	Precast concrete unit 12 ft 6 in (3.8 m) long, 2 ft (61 cm) high. Cast-in-place concrete facing also used.
Websol: Soil Structures International, Ltd. 58 Highgate High St. London N65HX England	5.3 in (135 mm) wide Paraweb: made from high-tenacity polyester fibers by Imperial ChemicaL Industries.	T-shaped precast concrete panel 34.4 sq ft ($3.2m^2$) area, 6.3 in (160 mm) thick.
York Method: Transport and Road Research Laboratory Crowthorne Berkshire, England	Galvanized mild steel or stainless steel or glass fiber reinforced plastic or Paraweb or Terram.	Hexagonal: glass fiber reinforced cement; 24 in (61 cm) across the flat; 9 in (23 cm) deep.
Anda Augmented Soils: Anda Augmented Soils Ltd. Oaklands House Solarton Road, Farnborough Hants GU14 7QL England	Fibretain straps (pultruded fiberglass reinforced plastic strip, developed by Pilkington Brothers, 1.6, 3.1 or 6.3 in wide, .08, .1 or .16 in thick (40, 80 or 160 mm wide, 2, 2.5 or 4 mm thick).	Precast concrete crib units with 12-in (30 cm)-high headers 4 ft (1.2 m) apart.
Tensar Geogrid System: The Tensar Corporation 1210 Citizens Parkway Morrow, GA 30260	Non-metallic polymeric grid mat made from high-density polyethylene or polypropylene.	Non-metallic polymeric grid mat (wrap around of the soil reinforcement grid with shotcrete finish, if desired), precast concrete units.
Miragrid System: Mirafi, Inc. PO Box 240967 Charlotte, NC 28224	Non-metallic polymeric grid made of polyester multifilament yarns coated with latex acrylic.	Precast concrete units or grid wrap around soil.
Maccaferri Terramesh System: Maccaferri Gabions, Inc. 43A Governor Lane Blvd. Williamsport, MD 21795	Continuous sheets of galvanized double-twisted woven wire mesh with PVC coating.	Rock-filled gabion baskets laced to reinforcement.

[1]Many other facing types, as compared to those listed, are possible with any specific system.
Ref.: FHWA 1990

TABLE 10-10.2
COMPARISON OF MSE SYSTEMS

REINFORCEMENT TYPE		SLOPE GEOMETRY (30° 60° 90°)	RECOMMENDED SOIL TYPE[1] (Clay .002, Silt .02, Sand 2, Gravel 2mm)	STRESS TRANSFER MECHANISM: Surface Friction	STRESS TRANSFER MECHANISM: Passive Resistance	REINFORCEMENT MATERIAL: Metal	REINFORCEMENT MATERIAL: Non-Metal	EXTENSIBILITY: Extensible	EXTENSIBILITY: Inextensible	PROPRIETY SYSTEM / PRODUCT NAMES
STRIP	Smooth	├─┤	├─┤	●		●			●	Reinforced Earth
							●	├─┤	├─┤	
	Ribbed	├─┤	├─┤	●		●			●	Reinforced Earth
							●	●		Paraweb
GRID		├─┤	├─┤		●	●			●	VSL, MSE, GAS, RSE, and Welded Wire Wall
								●		Maccaferri Gabion
							●	├─┤	├─┤	Tensar, Mirafi, Tenax, Reinforced Earth Grid, Conwed
SHEET		├─┤	├─┤	●			●	├─┤		Geotextiles
BENT ROD		├─┤	├─┤		●	●			●	Anchored Earth
FIBER		├─┤	├─┤	●		●	●	├─┤		

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[1] Based on stress transfer between soil reinforcement. Other criteria may preclude use of soils for specific applications.

Ref: (FHWA 1990).

Three types of reinforcement geometry can be considered:

1) Linear unidirectional - strips; steel, plastic, and fabric; rods, and cables.

2) Composite unidirectional - grid strips or bar mats.

3) Planar bidirectional - continuous sheets of geosynthetics, welded wire mesh, and woven wire mesh.

Stresses are transferred between soil and reinforcement by friction and/or passive resistance, depending on the reinforcement geometry:

- Friction - Stresses are transferred from soil to reinforcement by shear along the interface. This is the dominant mechanism with linear and planar reinforcements (strips, rods, cables, fabrics, geotextile sheets).

- Passive resistance - Stresses are transferred from soil to reinforcement by bearing between the transverse elements against the soil. This is the dominant mechanism for reinforcement containing a large number of transverse elements of composite reinforcements such as bar mats, grids, and wire mesh.

The performance and durability considerations for the two classes of reinforcement vary considerably. The distinction made between the characteristics of metallic and nonmetallic reinforcement are:

- Metallic reinforcements - Consist of mild steel or aluminum.

- Nonmetallic reinforcements - Generally polymeric materials consisting of polypropylene, polyethylene, or polyester polymers.

There are two classes of extensibility:

- Inextensible - The deformation of the reinforcement at failure is much less than the deformability of the soil (metallic reinforcements).

- Extensible - The deformation of the reinforcement at failure is comparable to or even greater than the deformability of the soil.

10-10.1 Forces

Forces for the analysis are generally the same as for concrete gravity dams and embankment dams as discussed in Chapter III and Chapter IV of these Guidelines. Only exceptions, clarifications and additions are discussed below.

MSE resists these forces under conditions somewhere between those of an embankment dam and those of a concrete gravity dam (ICOLD 1993).

10-10.1.1 Dead Loads

The dead loads considered should include the effective weight of the fill, reinforcement, and the facing.

10-10.1.2 External Hydrostatic Loads

The hydrodynamic forces on a MSE body subject to overtopping and on the lower portion of MSE body subject to high tailwater levels can be severe. Every element of these parts (backfill, facing units, reinforcements, attachments) has to be designed to withstand these forces. The actions of high hydrodynamic pressures, their rapid changes, the shocks of the floating bodies, the risk of fines being removed, and the alternate chemical action of water and oxygen must be evaluated in a realistic manner and adequate measures must be taken against such actions. Adequate secondary defensive design must be provided.

10-10.1.3 Earth Pressures

Earth pressures exerted on the MSE body by soil backfill should be calculated using the appropriate coefficient of earth pressure based on the type of reinforcement, the character of the backfill, along with the construction sequence. Refer to FHWA (1990) for further discussion.

10-10.1.4 Ice Pressures

If the MSE body is to be used as a spillway and overflow is expected during the winter, ice formation should be prevented, or the structure should be designed for external ice load. The potential for ice pressures in the backfill behind facing panels should be evaluated if the dam is exposed to severe frost.

10-10.2 Loading Combinations

Loading combinations for the analysis of MSE dams are the same as for embankment dams discussed in Chapter IV, Section 4-6.6 of these Guidelines.

10-10.3 Analyses

A great deal of progress has been made in developing analytical techniques for MSE in the past 10 years. However, the analytical techniques are relatively new, and are still being refined. Details at interfaces, junctions, and boundaries are often the starting point of failures due to factors such as strain incompatibility or edge effects and should be carefully evaluated. All appurtenances behind, in front of, under, mounted upon, or passing through the fill structure such as drainage structures, utilities, or other appurtenances should be accounted for in the stability design of the MSE structure.

Many MSE systems are patented or proprietary. Some companies provide services including design preparation of plans and specifications for the structure, supply of the manufactured wall components, and construction supervision.

The design of MSE systems is discussed in Mitchell and Villet (1987), Koerner (1990) and FHWA (1990). Design and Construction Guidelines — Reinforced Soil Structures (FHWA 1990) was developed to assist designers in determining the feasibility of using MSE systems for walls and embankment slopes, evaluating different alternative reinforcement systems, and performing preliminary design of simple systems. It provides a basis for evaluation and preliminary design of new MSE systems that may be proposed in the future. The design methods provided in the manual are not meant to replace private and proprietary system-specific design methods, but they should provide a basis for evaluating such designs. The various systems have different performance histories, and this sometimes creates difficulty in adequate technical evaluation. Methods for handling the matter of specifications and for obtaining the most cost competitive and technologically acceptable system are given in FHWA (1990).

The MSE structure should be designed to using three types of analysis: an analysis at working stresses, a limit equilibrium analysis, and a deformation (or displacement) analysis.

The analysis at working stresses is used to: 1) select reinforcement location and check that stresses in the stabilized soil mass are compatible with the properties of the soil and reinforcement, 2) evaluate local stability at the level of each reinforcement and predict progressive failure, and 3) estimate vertical and lateral displacements.

The limit equilibrium analysis is used to check the overall stability of the structure. Three types of stability must be considered, external, internal, and combined. The external stability involves the overall stability of the stabilized soil mass considered as a whole. The internal stability analysis consists of evaluating potential slip surfaces within the reinforced soil mass. In some cases, the critical slip surface is partially outside and partially inside the stabilized soil mass, and a combined external/internal stability analysis may be required.

The deformation analysis is used to evaluate the performance of the structure with respect to anticipated displacement. In addition, the influence of variations in the type and density of reinforcement on the performance of the structure can be evaluated. Deformation analyses are the most difficult and least certain of the three types of analysis. In many cases, they are done only approximately or it is simply assumed that adjusted factors of safety against external or internal stability failure will ensure that deformations will be within tolerable limits.

10-10.3.1 External Stability

External stability of an MSE structure should be evaluated for sliding resistance, overturning, bearing capacity, and seismic stability.

10-10.3.1.1 Sliding Resistance

The MSE body is subject to the lateral driving pressures on the upstream face that must be resisted by the frictional and cohesive resistance along the base or a weak layer near the base of the MSE body and the passive resisting pressures on the downstream face. Passive resistance should be neglected due to the potential for erosion and deterioration of the toe area. The shear strength of any facing system should also be neglected.

Three possible sliding planes must be evaluated: 1) foundation soil, 2) backfill soil, and 3) soil-reinforcing interface (sheet-type reinforcement).

Stability is determined using rotational or wedge analyses, as appropriate, which can be performed using a slope stability analysis. Compound failures, passing through both the unreinforced and reinforced zones, must be considered if complex conditions exist. In the case of a modified structure such as a retaining structure on the crest of a dam or steepened slopes, the modified loads on the embankment and any potentially higher reservoir water surface should be used to check overall deep-seated slope stability.

10-10.3.1.2 Overturning

Due to the flexibility of MSE structures, it is unlikely that a block overturning failure could occur. Nonetheless, an adequate factor of safety against this classical failure mode will limit excessive outward tilting and distortion of a suitably designed structure.

10-10.3.1.3 Bearing Capacity

An MSE body, especially one with near-vertical slopes, and any surcharge loads supported by it should be checked for bearing capacity failure of the foundation soil. The maximum vertical earth pressure and the effective vertical stress exerted by the MSE body on the foundation soil are calculated by traditional geotechnical methods. Due to the flexibility of the reinforced fill mass, the safety factor with regard to bearing capacity failure can be less than for more rigid structures.

10-10.3.1.4 Seismic Stability

Relatively large earthquake shaking could result in significant permanent lateral and vertical deformations. Various methods of analyses are available for evaluating the seismic stability of an MSE body — pseudostatic, simplified, and deformation. Refer to Chapter IV, Section 4-7 of these Guidelines, FHWA (1990), and Richardson (1978) for further discussion.

10-10.3.2 Internal Stability

Internal failure of an MSE structure can occur in two different ways:

- <u>Tension</u> - The tensile forces (and, in the case of rigid reinforcements, the shear forces) in the reinforcements become so large that the reinforcements elongate excessively or break, leading to large movements and possible collapse of the structure.

- <u>Pullout</u> - The tensile forces in the reinforcements become larger than the pullout resistance. This, in turn, increases the shear stresses in the surrounding soil, leading to large movements and possible collapse of the structure.

The most critical slip surface in an MSE wall is assumed to coincide with the maximum tensile forces line. The maximum tensile forces line has been assumed to be approximately bilinear in the case of inextensible reinforcements, approximately linear in the case of extensible reinforcements, and passes through the toe of the wall in both cases.

The most critical slip surface may cross one or more layers of the reinforcements. A slope stability analysis method that takes into account the effect of reinforcements intercepted by the slip surface must be used in these cases.

In general, any shape of slip surface can be considered: plane, circle, multilinear. MSE slope stability analysis methods have been published and computer programs are available that allow for limit equilibrium analysis of reinforced soil masses.

Finite element methods (FEMs) have been used to study the performance of geotextile-reinforced embankments in both analysis and design situations (Andrawes 1982, Rowe 1984). Although these sophisticated computer-based methods might not be routinely used for noncritical situations, they do give great insight into the behavior of the system. Finite element studies have been found to provide more realistic results than limit equilibrium analysis, which have been found to overpredict stresses, loads and movements (Jarrett 1988).

10-10.3.2.1 Tension Failure

The reinforcement is subjected to tensile forces and extension through the shear-transfer that develops between it and the fill. The location of the maximum tensile forces line is influenced by the extensibility of the reinforcement as well as the overall stiffness of the facing. With inextensible reinforcements the maximum tensile forces line can be modeled by a bilinear failure surface that is vertical in the upper part of the structure. The state of stress is assumed to be at rest at the top and decreases to the active state in the lower part of the structure. With extensible reinforcements the maximum tensile forces line coincides with the Coulomb or Rankine active failure plane, and the stresses in the fill correspond to the active earth pressure condition. The location of the maximum tensile forces line may also vary due to external factors such as the shape of the structure and surcharge conditions.

The maximum tensile forces line separates two areas in the MSE body: an active area in which the shear stress exerted by the ground on the reinforcement is directed toward the outside of the wall and a resistant area in which this stress is directed towards the inside. For each reinforcement layer the "adherence length" is defined as the reinforcement length located within the resistant area.

The lateral earth pressure to be resisted by the reinforcements should be calculated using the appropriate coefficient of earth pressure based on the type of reinforcement used times the vertical soil stress at each reinforcement layer. The vertical soil stress shall be calculated using the Meyerhof method. The seepage force should also be taken into account. The maximum tensile force must be checked to see if it is less than the allowable reinforcement strength (not including the extra thickness provided for corrosion).

At the connection of the reinforcements with the facing, check that tensile force is not greater than the allowable tensile strength of the connection. The connection strength will depend on the structural characteristics of the facing system used.

10-10.3.2.2 Pullout

The design of the soil reinforcement system requires an evaluation of the long-term pullout performance with respect to three basic criteria:

- Pullout capacity, i.e., the pullout resistance of each reinforcement should be adequate to resist the design working tensile force in the reinforcement with a specified factor of safety.

- Allowable displacement, i.e., the relative soil to reinforcement displacement required to mobilize the design tensile force should be smaller than the allowable displacement.

- Long-term displacement, i.e., the pullout load should be smaller than the critical creep load.

The pullout resistance of the reinforcement is mobilized through interface friction and/or passive soil resistance. The load transfer mechanisms mobilized by a specific reinforcement depends primarily upon its structural geometry. The soil-to-reinforcement relative movement required to mobilize the design tensile force depends mainly upon the load transfer mechanism, the extensibility of the reinforcement material and the soil type. The long-term pullout performance is predominantly controlled by the creep characteristics of the soil and the reinforcement material.

The pullout resistance of the reinforcement is defined by the ultimate tensile load required to generate outward sliding of the reinforcement through the reinforced soil mass. Several approaches and design equations have been developed and are currently used to estimate the pullout resistance by considering frictional resistance, passive resistance, or a combination of both. The design equations use different interaction parameters, and it is, therefore, difficult to compare the pullout performance of different reinforcements for a specific application. Refer to FHWA (1990) for further guidance.

10-10.3.2.3 Seismic Loading

The seismic event will lead to dynamic incremental loads in the strips. The distribution of the corresponding incremental tensile forces is different from that occurring in the static case. However, as long as the seismic intensity is not very high this will have very little influence on the position of the resultant line of maximum tensile forces. Under strong ground movements the line will tend to move away from the wall facing.

High MSE fills may not behave in a ductile manner near the base of the fill, because the reinforcements in this location may fail in tension, rather than by sliding inside the backfill.

The internal stability of the MSE body under earthquake loading conditions is analyzed according to procedures presented by Richardson (1978). These procedures determine the dynamic force magnitude in each reinforcing strip through computation of the amplitude and of the distribution of dynamic earth pressures acting on the walls. Sufficient reinforcement must be provided so that combined static and dynamic earth pressures do not cause tensile or soil-strip frictional failure of the reinforcement.

Design strip forces are obtained by combining the dynamic and static forces. If changes in reinforcing strip density or wall geometry are necessary to obtain a desired factor of safety, the design procedure must be repeated since this will change the stiffness of the wall. The magnitude and distribution of the dynamic forces and peak dynamic strain are functions of the wall stiffness.

10-10.3.2.4 Facing Strength

A comparison must be made between the horizontal loads that the facing will have to bear and the allowable loads for each type of facing (according to their nature, thickness, and number of fixing points).

10-10.3.3 Back-to-Back Wall Design

The back-to-back design must be considered in the case of a double-faced wall used to raise dam crests, which is actually two separate walls with parallel facings. This situation can lead to a modified value of backfill thrust which influences the external stability calculations.

For the first case, the overall base width is large enough so that each wall behaves and can be designed independently. In particular, there is no overlapping of the reinforcements. The active thrust may be mobilized without any inclination on the horizontal or the active thrust may be reduced, depending on the distance between the two walls.

For the second case, there is an overlapping of the reinforcements, so that the two walls interact. Consequently, the two walls are designed independently with the same internal local stability procedure, but assuming no active thrust from the backfill.

10-10.3.4 Deformation

Conventional settlement analysis for shallow foundations should be carried out to ensure that immediate, consolidation, and secondary settlement of the structure are less than the performance requirements of the project. An estimation of the total and differential settlements due to the foundation ground, MSE, facing, and any other contiguous structures must be made. This information is necessary in order to make sure that the expected settlements are compatible with the proper behavior of the reinforcement connections, deformation capacities of the facing, and the behavior of integral or adjacent structures.

There is no standard method to evaluate the overall lateral displacement of MSE. Loading of the MSE section and associated lateral deformation will primarily occur during construction with the exception of post construction surcharge loads. Post-construction movement could also occur due to settlement of the structure.

The major factors influencing lateral displacements during construction include compaction intensity, reinforcement to soil stiffness ratio (i.e., the area of reinforcement and deformability as compared to the modulus and area of the reinforced soil section), reinforcement length, slack in reinforcement-to-facing connections, and deformability of the facing system.

The total lateral displacement of simple structures on firm foundations that is anticipated during construction can be estimated from Figure 32 in FHWA (1990), based on the length of reinforcement (L) to height of the wall (H) ratio and the extensibility of the reinforcement. This figure was empirically developed using data from actual structures and computer simulation models. It provides a first order lateral deformation estimate that could be used to establish appropriate face batter and to evaluate anticipated horizontal alignments. It should be noted that as L/H decreases, the lateral deformation increases. This is important when determining the suitability of the final reinforcement length. For example, going from a length of 0.7H to 0.5H could essentially double the lateral deformation anticipated during construction.

For critical structures requiring precise tolerances, the lateral displacement of the wall has to be calculated more accurately. A finite element method of calculation is recommended for this analysis.

10-10.3.5 Seepage

A seepage analysis should be performed to determine the amount of seepage and the magnitude of the seepage forces. The simplified line-of-creep method, flownet analysis and numerical methods, may be used to perform the seepage analysis. The potential for the reinforcement creating a seepage path through the dam should be evaluated. Refer to Chapter III, Section 3-2.4.4 and Chapter IV, Section 4-5 for further discussion.

10-10.3.6 Computer Programs

The ideal method for MSE design is to use a conventional slope stability computer program that has been modified to account for the stabilizing effect of reinforcement. Such programs should account for reinforcement strength and pullout capacity, compute reinforced and unreinforced safety factors automatically, and have some searching routine to help locate critical surfaces. The ideal method would also include the confinement effects of the reinforcement on the strength of the soil in the vicinity of the reinforcement. Very few of these programs are publicly available, and those are usually limited to specific soil and reinforcement conditions.

Refer to Chapter III, Section 3-4.7 for further guidance on review of computer programs.

10-10.4 Acceptance Criteria

Adequate safety concerning overall stability, reinforcement tensile resistance, pullout, facing stability, seepage, deformation of all parts of the structure, and the durability of all materials must be demonstrated.

10-10.4.1 External/Internal Stability

Recommended minimum factors of safety are given in Table 10-10.3.

10-10.4.2 Steel Reinforcement

The allowable tensile stress in steel reinforcements and connections, at the end of service life shall conform to the following:

$F_t = 0.55F_y$ at reduced gross section[1]
$F_t = 0.50F_y$ at the net section at bolt hole
(applicable to bolted connections only)

Where F_t allowable tensile stress
F_y yield stress

An allowance must be made in the metal cross section area to account for the estimated corrosion loss.

10-10.4.3 Geosynthetic Reinforcement

The tensile properties of geosynthetics are affected by creep, construction damage, aging, temperature, and confining stress. Furthermore, characteristics of geosynthetic products manufactured with the same base polymer vary widely.

Polymeric reinforcement, although not susceptible to corrosion, may degrade due to physicochemical activity in the soil such as hydrolysis, oxidation, and environmental stress cracking. In addition, it is susceptible to construction damage. Degradation most commonly occurs from mechanical damage, loss of strength due to creep, and deterioration from exposure to ultraviolet light.

TABLE 10-10.3
RECOMMENDED MINIMUM FACTORS OF SAFETY

Parameter	Loading Combination: Usual	Loading Combination: Seismic
External Stability	Recommended minimum factors of safety should be in accordance with Chapter III and Chapter IV of these Guidelines.	
Deformation	Maximum allowable total and differential, based on performance requirements of the project.	
Internal Stability		
Pullout Resistance[1]		
Granular soils	1.5	≥1.1
Cohesive soils	2.0	≥1.1
Breakage Strength	Determine allowable tension in reinforcement. (Do not forget the reinforcement/facing connection tensile strength.)	
Durability	Take into account the design life in the determination of the allowable tension	

[1]minimum embedment length 3 ft.

Ideally the allowable tension should be determined by thorough consideration of allowable elongation, creep potential and possible strength degradation using a complex method that requires extensive long-term strength testing of the geosynthetic product. In the absence of sufficient test data a simplified expression may be used. The expression takes into account yield tensile strength, a creep reduction factor, a durability factor of safety, a construction damage factor of safety, and an overall factor of safety. The expression must be equal to or less than the long-term tension capacity of the geosynthetic at a selected design strain (usually 5% or less) (FHWA 1990).

The yield tensile strength is obtained from wide strip tensile strength tests (ASTM D4595). The creep reduction factor (CRF) is the creep limit strength obtained from the creep test results divided by the yield tensile strength. If the CRF value of the specific reinforcement is not available, the following recommendations are provided in FHWA (1990).

Polymer Type	Creep Reduction Factors[1]
Polyester	0.4
Polypropylene	0.2
Polyamide	0.35
Polyethylene	0.2

[1] Additional reduction should be made for applications in high-temperature environments (temperatures greater than 90NF in the region of the reinforcement, e.g. at facing connection, in hot climates).

The durability factor of safety is dependent on the susceptibility of the geosynthetic to attack by microorganisms and chemicals, thermal oxidation, and environmental stress cracking and can range from 1.1 to 2.0. In the absence of product specific durability information use 2.0.

The construction damage factor of safety can range from 1.1 to 3.0. In the absence of product specific construction damage tests use 3.0.

The overall factor of safety accounts for uncertainties in the geometry of the structure, fill properties, reinforcement properties, and externally applied loads. For permanent, vertically faced structures use a minimum of 1.5.

10-10.4.4 Backfill

Backfill properties should be in accordance with the requirements of the MSE system being used. The design and construction of the MSE system depends on the specific properties of the soil used as backfill, and various types of soil (sand, silt, clay, gravel) can be used. However, most MSE systems specify high-quality backfill in terms of durability, drainage, and friction consisting of well graded, granular materials. Typical ranges of soil types for MSE are summarized in Table 10-10.2. Many of the MSE systems depend on friction between the reinforcing elements and the soil. In such cases, generally a material with high friction characteristics is specified and required. Some MSE systems rely on passive pressure on reinforcing elements and in those cases, the quality of backfill is still critical. These requirements generally eliminate soils with high clay contents. Corrosive conditions may be present due to acid or marine environments, aggressive soils, presence of sulfates, etc. and require special measures.

10-10.4.5 Seepage Control

As a general rule, MSE is not to be used for dam imperviousness. If possible, the waterproofing system should be located upstream of the MSE and drainage, including a filter, should be provided between the impervious layer and the reinforced fill body.

When MSE materials are sufficiently permeable and self-filtering, they can serve as drains. When the backfill is not highly permeable, a drainage layer should be included behind the facing panels.

For particular applications such as a dam raising, the reinforced fill system might serve as a watertight barrier for the design flood. Although water will be against the wall only during remote events, the structure must be capable of providing the necessary protection against seepage that could affect the integrity of the structure or embankment. A viable means of providing a water barrier is to seal all joints on the upstream reinforced fill facing. This could be accomplished by applying a waterproof membrane over the joints on the inside face of the wall. Although the membrane should provide a positive water barrier, deformations of the facing during normal operations or unnoticed damage to the membrane during construction could disrupt its integrity. Assurance of a continuous membrane-to-facing seal would be difficult to control during construction. Therefore, as a defensive design measure, it is necessary to provide zoning of materials in the retaining wall system to further control any seepage not stopped by the membrane. The zoning of materials should provide:

- Sufficiently low permeability to restrict seepage flow to manageable quantities should disruption of the membrane occur.

- Sufficiently high free-draining capability to prevent the retention of moisture around reinforcing strips, and thereby, minimize the potential for corrosion of the strips during normal operations.

- Filter protection between adjacent zones to prevent the loss of material should seepage occur.

10-10.4.6 Corrosion/Deterioration

The service life and safety of MSE structures centers around embedding reinforcement in fill materials and protecting that reinforcement from corrosion or loss of material that could eventually compromise stability. The MSE should be designed with a service life comparable with that of the rest of the dam.

Practically all the metallic reinforcements used in construction of embankments and walls, whether they are strips, bar mats, or wire mesh, are made of galvanized mild steel. Woven meshes with PVC coatings also provide corrosion protection, provided the coating is not significantly damaged during construction. Epoxy coating can be used for corrosion protection, but it is also susceptible to construction damage, which can significantly reduce its effectiveness.

10-10.5 Material Properties

The material strength assumptions made during the design process must be verified by the site investigations and attained in the field during the construction process. Specific details concerning geologic investigations are contained in Chapter V of these Guidelines. The following general guidance shall apply.

10-10.5.1 Site Investigation

If the MSE is to be used in conjunction with an existing structure, the stability and condition of that structure must be investigated. If the MSE is to be a new structure, the investigation should be sufficient to define the major geologic and hydrologic conditions with emphasis on those that will affect design. In all cases a special emphasis must be placed on the site factors that could aggravate the corrosion or the deterioration of any type reinforcement. Important factors to be considered are the reservoir water quality, underground water quality, and fill quality.

10-10.5.2 Reinforcement

The following information on the reinforcement materials is needed for the design: geometric characteristics, strength and stiffness properties, durability, and soil reinforcement interaction properties. The two most commonly used reinforcement materials are steel and geosynthetics.

Two types of geometric characteristics can be considered:

- Strips, Bars, and Steel Grids: A layer of steel strips, bars or grids is characterized by the cross-sectional area, the thickness and perimeter of the reinforcement element and the center-to-center horizontal distance between elements. A layer of geosynthetic strips is characterized by the width of the strips and the center-to-center horizontal distance between them.

- Sheets and Geosynthetic Grids: A layer of sheet of geosynthetic grid is characterized by the width of the sheet or grid component and the center-to-center horizontal distance between the sheets or grid components. The cross-sectional area is not needed, since the strength of a sheet and a grid are expressed by a tensile force per unit width, rather than by a stress.

A variety of geosynthetics may be used as multilayer reinforcement to construct slopes that are steep or even vertical. A high strength/high-modulus geosynthetic is usually required (typically geogrids or multifilament woven geotextiles) for dam support. Three basic properties that need to be established for the use of a geosynthetic are the tensile resistance, elongation to rupture, and the tear resistance. The long-term durability of the geosynthetic must also be established. The geosynthetics and other new reinforcement materials have limited performance history compared to traditional dam construction materials. While they offer the potential for economic design, the need for safety and longevity in dam design is paramount.

10-10.5.3 Corrosion Protection

For metallic reinforcements the site factors that could aggravate their corrosion or deterioration are mainly a marine environment, acid water, soil with high aggressive salts contents, and pure water. When exposed to corrosive conditions galvanized steel reinforcements are normally used. Means to control the effect of subsequent corrosion include increasing the reinforcement thickness, finding another fill material, or shielding the reinforced fill from the aggressive seepage with a watertight membrane. Shielding has included:

- Providing cathodic protection so that the rate of metal loss for the embedded metalwork could be effectively controlled and monitored.

- Insulating the connection between the metal embedded in soil and the metal embedded in concrete to reduce the potential for corrosion.

- Using an epoxy coating on all reinforcing components embedded in soil or concrete to reduce the potential for corrosion.

The corrosivity of the backfill material must be evaluated because of the corrosion of buried metals. The corrosion of buried metals depends on the presence of dissolved salts in the soil, pH, and degree of saturation. Highest corrosion rates are produced by a high content of dissolved salts, a high chloride concentration, a high sulfate content, and acidic or alkaline pH conditions in the soil.

The polymeric formulation and resin additive package of the geosynthetic must be compatible with the chemistry of the backfill and the potential for the environment within the backfill to change with time. The backfill should be checked for such items as high and low pH, chlorides, organics, and oxidation agents such as soils that contain Fe_2SO_4, calcareous soils, and acid sulfate soils that may result in deterioration of the geosynthetic with time. Other possible detrimental environmental factors include chemical solvents, diesel, and other fuels, active slag fills, and industrial wastes. FHWA (1990) provides additional data on corrosion rates. Generally, site specific corrosion studies should be performed to determine the appropriate metal loss rates.

10-10.5.4 Facing

The facing is not subjected to much stress and consists of relatively light units, except if seepage forces may be present. The facing is connected to the reinforcement so that it becomes part of the structure. It must be flexible enough to adapt to the settlements and deformations of the structure and its foundation. The edges of the facing panels must be shaped so that they are properly secured to one another.

Large deformations may lead to opening of the contacts between facing elements resulting in a loss of backfill. When such deformations are expected, the design must provide for them without loss of fill. Natural materials or filter cloth (geotextiles) can be used to prevent the loss of material through the contacts between the facing elements.

No facing component is excluded from consideration but it is essential that the assembly be rapid so that fill placement occurs smoothly. The major facing types are: segmental precast concrete panels; case-in-place concrete, shotcrete or full height precast panels; semi-cylindrical metallic facings; welded wire grids; gabion facings; fabric facing; plastic grids; and postconstruction facings. The use of separate panels provide the flexibility to absorb differential movements, both vertically and horizontally, without undesirable cracking that could occur in a rigid structure. Metal facings have the disadvantage of shorter life because of corrosion unless provision is made to compensate for it. Facings using welded wire mesh or gabions have the disadvantages of an uneven surface, exposed backfill materials, more tendency for erosion of the retained fill, possible shorter life from corrosion of the wires, and more susceptibility to vandalism. The greatest advantages of such facings are low cost, ease of installation, design flexibility, and good drainage (depending on the type of backfill) that provides increased stability. The surface deterioration of fabric facing and plastic grids must be evaluated.

10-10.6 Construction Considerations

Careful inspection during construction is important. Special attention must be paid to the of joints, seams, anchorages, penetrations, and similar interface conditions as well as to the reinforcement.

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