Chapter 9
Analysis and Design Criteria

9-1. Stability Analysis

Each lock monolith must be designed to resist movement caused by applied loads. Such movement could include settlement, flotation, sliding, or overturning. Criteria for stability design is contained in EM 1110-2-2200 and in other guidance publications. If stability requirements cannot be satisfied, a pile foundation may be used. Criteria for design of pile foundations are contained in EM 1110-2-2906.

9-2. Structural Analysis

a. General. Analysis and design methods for U-frame lock monoliths are presented in ETL 1110-2-355. Many of these concepts are also applicable to other types of lock monoliths.

b. Two-dimensional (2-D) analysis. An analysis of a 2-D slice through a monolith can reliably represent the behavior of the monolith under the following conditions:

(1) When the cross-section geometry of the structure, the soil and water conditions, the support conditions, and the other loading effects are constant throughout an extended length of the monolith.

(2) When a 2-D slice, obtained by passing parallel planes perpendicular to the longitudinal axis of the monolith, typifies adjacent slices and is sufficiently remote from any discontinuities in geometry and loading (i.e., the slice is in a state of plane strain).

c. Two-dimensional frame analysis. Structural analysis of the lock is based on the assumption that the various slabs, walls, etc., of the structure interact as elements of a 2-D frame. The parts of the structure act as flexible members connected at their ends to joints. However, because of the thickness of some lock elements, the large joint regions reduce member flexibility. Representation of these rigid joint regions is discussed in ETL 1110-2-355.

d. Three-dimensional (3-D) analysis. If the lock monolith geometry and/or loading does not meet the above requirements for a 2-D frame analysis, a 3-D finite element computer model should be used to analyze the monolith. Guidance on modeling of structure for linear elastic finite element analysis is provided in other Corps documents.

e. Seismic. Earthquake-induced ground motion effects must be considered in the analysis and design of navigation lock structures. The structures must be designed for the inertial forces from the structure mass combined with hydrodynamic pressures generated by the water inside and outside the lock chamber and within the culverts. These forces should be combined with any dynamic soil pressures generated within the backfill. Linearly elastic procedures used in design include the response spectrum analysis and the time history analysis.

(1) Seismic coefficient method. Traditional design practice based on the seismic coefficient method failed to account for the dynamic response characteristics of the soil-structure water system. Locks designed by the seismic coefficient methods may not be adequately proportioned or reinforced to resist forces generated during a major earthquake. Therefore, this approach should be used only as a simple, preliminary means of checking a new design or an existing structure for seismic susceptibility. It should not be used as a final analysis procedure for controlling member proportions or for remedial design (with the exception of those cases where extensive results or comparisions of previously designed or evaluated structures are available).

(2) Response spectrum analysis. A response spectrum is a plot of the maximum response of a series of single-degree-of-freedom (SDOF) systems with varying periods or frequencies. A response spectrum analysis partially accounts for the dynamic structural properties of the system. The response spectrum analysis can be accomplished by either a finite element or frame analysis. Results from these procedures provide only the *absolute* maximum stresses and forces due to the methods of combining modal responses.

(3) Time history analysis. The exact time history of a response quantity can be produced using this technique; therefore, an exact sign dependent stress distribution can be found at any given time. However, a digitized design earthquake record for the site is needed, and a significant computing effort is required for the numerical integration of the differential equation of motion using small time steps.

f. NISA. A nonlinear incremental structural analysis (NISA) should be done on massive concrete structures. This analysis should be performed if it will help achieve cost savings, develop more reliable designs for structures that have exhibited unsatisfactory behavior in the past, or predict behavior in structures for which a precedent has not been set. A NISA first requires that a time-dependent heat transfer analysis be performed. The results of the heat transfer analysis are then used in a time-dependent stress analysis that simulates the incremental construction of the structure and uses nonlinear properties for modulus of elasticity, creep, and shrinkage. For more information on performing a NISA, refer to ETL 1110-2-365.

g. *Load transfer between monoliths*. Lock monoliths should be designed to act independent of adjacent monoliths. Only when all other means fail should load transfer between monoliths be considered. However, when it is necessary to design adjacent foundations as interacting to resist large lateral loads, the monolith joint details must be designed to ensure proper load transfer. The primary area for load transfer should be the base slab and not the lock walls. Provisions should be included for keying and grouting the monolith joints between the base slabs of interacting monoliths. The wall joints should be detailed to accommodate monolith movements without significant load transfer in order to control localized cracking and spalling. Base slab displacements should also be extrapolated to the top of the monolith to make sure that the displaced structure does not make contact with the adjacent monolith.

h. *Effects of base slab offsets.* Many 3-D monoliths have vertical offsets in the base slab, such as a miter gate sill. When this type of monolith is analyzed, the base slab must be accurately modeled so that the proper stiffness relationships are obtained in the analytical model. Several methods are described in ETL 1110-2-355.

i. *Shear transfer between adjacent 2-D sections*. Often portions of 3-D monoliths are analyzed using a 2-D method. For example, the lock walls of a miter gate monolith may be analyzed independently as 2-D models. A 2-D section can be cut from the 3-D model, and the appropriate loads and reactions from the 3-D model applied. However, this 2-D "slice" will usually not be in equilibrium because it is a part of a 3-D monolith; thus shear transfer will occur between adjacent 2-D slices. These shear forces must be calculated (summation of loads and reactions) and applied at appropriate locations over the 2-D cross section so that the model is in equilibrium. Shear transfer requirements are discussed in ETL 1110-2-355.

j. *Articulated base slab*. In certain circumstances, the use of an articulated base slab may be practical to reduce concrete in the chamber monoliths. This reduction may be accomplished by placing a vertical joint in the base slab on each side near the face of the lock wall. This joint would be designed to transfer shear and axial forces but not moment. This process may reduce the concrete thickness and amount of reinforcing steel. This approach may be useful for monoliths that do not require unwatering such as approach monoliths.

9-3. Foundation Design and Soil/Structure Interaction

a. *Site selection*. The foundation conditions often influence the site selection for a lock project. Therefore, the foundation characteristics should be determined for each tentative site at an early stage of the investigation. These characteristics are usually determined by using available data and a minimum of foundation exploration. Sites chosen for further investigation should have foundation characteristics that would allow the lock structures to be constructed at a reasonable cost. The possible sites selected for study from a review of topography and hydraulics can thus be reduced to one or two after reviewing the site from a foundation and navigation standpoint. Final site selection requires extensive foundation exploration of the remaining sites under consideration.

b. *Foundation type.* Determining the type of foundation is probably the most critical aspect in the design of a lock. Since this decision will affect the project cost, the foundation type should be determined in the feasibility stage of the project. This analysis should involve the use of a thorough subsurface investigation and testing program to define the soil strengths and parameters. The criteria for selecting a soil or pile foundation are based on economic considerations and site-specific characteristics. Usually, a soil foundation is more economical if special measures (deeper excavation, elaborate pressure relief system, etc.) are not required. In addition, the structure on a soil foundation has to be able to satisfy stability requirements for sliding and overturning, as well as resisting flotation and earthquake forces. At some sites, liquefaction of the foundation during an earthquake becomes a determining factor in selecting the foundation type. Differential settlements between monoliths should also be considered in the foundation determination. If a soil foundation is not feasible or requires expensive special measures, then a pile foundation should be studied and compared to the cost of a soil foundation. The process for selecting a pile foundation should consider all reasonable types of pile and should select the most feasible solution based on the site geotechnical conditions, availability of material, and construction limitations. The quantities can be based on minimum spacing and approximate lateral and vertical capacities for one or two typical monoliths. The most cost-effective type of pile is thus determined for comparison to the soil foundation. Computer programs such as CPGA (rigid base) or CWFRAM (flexible base) or other finite elements are useful for designing pile foundations. The final decision between a soil and pile foundation is then based on a cost comparison using these refined pile quantities. Detailed design guidance for pile foundations is contained in EM 1110-2-2906.

c. Foundation pressures (compatible deformations). Foundation pressures depend on the type of foundation material, the nature of the loading, and the size and shape of the monolith. For gravity-type monoliths (due to their rigidity), a linear distribution of base pressure beneath the wall can be assumed. However, for U-frame-type monoliths and other structural monoliths with a flexible base, the distribution of base pressure should be based on a soil/structure interaction analysis.

d. Bearing strength of soils. The bearing strength of soils and methods for its determination based on field and laboratory test data are described in EM 1110-1-1905. Another good reference for the calculation of bearing capacities is the program documentation for the CASE computer program CBEAR.

e. Base pressures and settlement. For a gravity-type lock, settlement analyses can be performed by following the principles set forth in EM 1110-1-1904. For a U-frame-type lock, the computer program CWFRAM can be used to obtain base pressures and associated base slab deformations. The most difficult aspect of this type of analysis is selecting representative soil moduli to input into the program CWFRAM that would relate moduli and deflection. Currently, guidance on selecting representative soil moduli for use in this type of analysis is limited, so the user should work closely with the geotechnical engineers. These analyses should be verified by a finite element program such as Soil-Structure Interaction Program (SOILSTRUCT). This program can account for the incremental construction sequence of the lock. As the instrumentation of the Port Allen Lock and the Old River Lock U-frame locks showed, the construction sequence can significantly influence settlements and lock wall movements.

9-4. Reinforcing Design

a. General. Steel reinforcement should be designed and detailed as specified in EM 1110-2-2104. Because of the large wall and floor sections, reinforcement spacing should generally be set at 12 in. for ease of construction. However, in gravity walls, the requirements of EM 1110-2-2104 regarding the minimum steel do not apply.

b. Volume change induced cracking.

(1) Volume change in massive concrete occurs as cooling of the concrete takes place. The volume change can be minimized by reducing the heat generated by cement hydration. Reducing the heat is accomplished by replacing cement with pozzolan, cooling the aggregates in the mixture, and replacing some of the mixing water with ice. Also, limiting lift heights allows cooling to take place before the next lift is placed. Contraction joints are used to reduce tensile strains caused by cooling contraction and restraint at the foundation. Shrinkage (volume change caused by drying) is not considered a problem in mass concrete because drying only occurs at the outermost 6 to 12 in.

(2) In unreinforced mass concrete, once a crack is formed it can propagate throughout the structure. Heavy temperature reinforcement (number 9 bars at 12-in. spacings) can be used to prevent crack propagation and control crack widths (many small cracks rather than one large crack). Reinforcing for crack control may not be needed in massive sections (5 ft thick or more) because it is more economical to accomplish crack control using the measures described above. However, the presence of reinforcement provides a safety margin to prevent cracks at cold joints. In areas where volume change causes stress concentration, such as at the corners of the filling and emptying culverts, reinforcement should be provided to prevent cracks from propagating from the culvert to the outside face of the structure. In nonmassive concrete sections, temperature and shrinkage reinforcement is required to control cracking. Generally, small bars at close spacing provide the best control. However, for walls 2 ft thick or more, number 9 bars at 12-in. spacings are commonly used to ease construction while still providing the required steel percentage.

c. Recesses and openings. For reinforcing the corners of all lock wall recesses, diagonal bars should be used. In addition, reinforcing steel is sometimes used in the top of lock walls.