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ENGINEERING AND DESIGN

Structural Design and Evaluation of Outlet Works

ENGINEER MANUAL

AVAILABILITY

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DEPARTMENT OF THE ARMY
U.S. Army Corps of Engineers
Washington, DC 20314-1000

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Manual
No. 1110-2-2400

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Engineering and Design
STRUCTURAL DESIGN AND EVALUATION OF OUTLET WORKS

1. Purpose. This manual provides guidance for the planning and structural design and analysis of intake structures and other outlet works features used on U.S. Army Corps of Engineers projects for the purpose of flood control, water supply, water quality and temperature control, recreation, or hydropower.

2. Applicability. This manual applies to all HQUSACE elements and USACE commands having responsibilities for the design of civil works projects.

3. Distribution Statement. This publication is approved for public release; distribution is unlimited.

4. Scope of the Manual. This manual presents guidance for the planning and design of outlet works structures, with special emphasis on intake towers. Other outlet works structures covered include tunnels, cut-and-cover conduits, access bridges, gate structures, and approach and discharge channel structures. Appurtenant features covered include trashracks, gates, valves, and mechanical and electrical operating equipment. Chapter 2 presents general planning and design information; Chapters 3 and 4 provide structural and seismic design guidance; Chapter 5 describes trashracks, bulkheads, gates, valves, and operating equipment; and Chapter 6 covers access bridge design requirements and describes special detailing considerations. Appendices B through E cover the seismic design and evaluation of intake towers.

FOR THE COMMANDER:

5 Appendices
(See Table of Contents)

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Colonel, Corps of Engineers
Chief of Staff

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Chapter 1
Introduction

1-1. Purpose

This manual provides guidance for the planning and structural design and analysis of intake structures and other outlet works features used on U.S. Army Corps of Engineers projects for the purpose of flood control, water supply, water quality and temperature control, recreation, or hydropower.

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1-4. References

Required and related publications are listed in Appendix A.

1-5. Scope

a. Overview. This manual presents guidance for the planning and design of outlet works structures, with special emphasis on intake towers. Other outlet works structures covered include tunnels, cut-and-cover conduits, access bridges, gate structures, and approach and discharge channel structures. Appurtenant features covered include trashracks, gates, valves, and mechanical and electrical operating equipment. Chapter 2 presents general planning and design information; Chapters 3 and 4 provide structural and seismic design guidance; Chapter 5 describes trashracks, bulkheads, gates, valves, and operating equipment; and Chapter 6 covers access bridge design requirements and describes special detailing considerations. Appendices B through E cover the seismic design and evaluation of intake towers.

b. Guidance limitation.

(1) The procedures in this manual are intended for outlet works structures founded on rock. Outlet works structures should not be founded on soil unless piling is provided to support the structure in the event the soil supporting the structure is eroded. The design of pile-supported structures is covered in EM 1110-2-2906.

(2) Intake structures contained within or attached to concrete gravity dams are not addressed in this manual. Details of intake structures that are integral with a concrete gravity dam can be found in the U.S. Bureau of Reclamation handbook, "Design of Gravity Dams" (U.S. Bureau of Reclamation 1976).

Chapter 2
General Design Considerations

2-1. General

This chapter presents structural design considerations for outlet works structures used with embankment dams, or concrete dams with outlet works detached from the dam. A detached outlet works may be the most economical for a concrete dam when the dam is located in a narrow canyon with restricted space for the outlet features. The hydraulic design of outlet works is covered in EM 1110-2-1602. Intakes through concrete dams, normally called sluices, are covered in EM 1110-2-2200 and EM 1110-2-1602.

a. Outlet works. Outlet works consist of a combination of structures designed to control the release of water from the reservoir as required for project purposes or operation. The components of outlet works, starting from the upstream end, typically consist of an approach channel, an intake structure, a conduit or a tunnel, a control gate chamber (located in the intake structure, within the conduit, or at the downstream end of the conduit), an exit chute, an energy dissipater, and a discharge channel. Outlet works are frequently used to pass diversion flows during construction, regulate flood flows, aid in emptying the reservoir in an emergency condition, and permit reservoir lowering for inspections and special repairs. Typical arrangements of outlet works are shown in Figures 2-1, 2-2, and 2-3. The sizing of the outlet works should take into account the possibility of using it to reduce the size or frequency of spillway discharges. The necessity of emergency drawdown capability and low flow discharge capability should be considered during the outlet works planning phase. The selection of type and arrangement of outlet works structures should be based upon consideration of the costs of operation and maintenance likely to be incurred during the project life. Reliability under emergency flood conditions is a fundamental operational requirement of outlet works facilities.

b. Intake structures. The intake structure may serve several different functions in the outlet works system. Besides forming the entrance, it may include (1) a trashrack to block debris, (2) fish entrances, (3) multilevel ports or weirs for water temperature control, (4) temporary diversion openings, (5) water supply and irrigation intakes, (6) bulkhead or stoplogs for closure, and (7) control gates and devices.

c. Outlet tunnels or conduits. Water passage through or around the dam is provided through tunnels in rock abutments or through cut-and-cover conduits through the base of an embankment type dam.

(1) A tunnel in a rock abutment is preferred when it is more economical than other means and when topographic conditions permit its use. Narrow rock abutments, permitting a relatively short tunnel, are conditions preferable for tunnels. The selection of the number and size of tunnels depends on hydraulic requirements, the maximum size of tunnel for the rock conditions encountered, and overall economics. When a tunnel is used also for diversion, its sizing is often controlled by the diversion releases required during construction. Figure 2-1 shows a tunnel outlet works.

(2) Cut-and-cover conduits are economical for low- or moderate-head embankment dams because of lower construction costs and a shorter alignment. A typical cut-and-cover conduit is placed near streambed elevation, preferably outside the normal river channel to facilitate construction. It should be placed within the most competent portion of the foundation and, when practicable, on suitable rock to keep potential settlement to a minimum. High-head dams will require a thick concrete section to resist high embankment loads. Figure 2-2 shows a cut-and-cover outlet works.

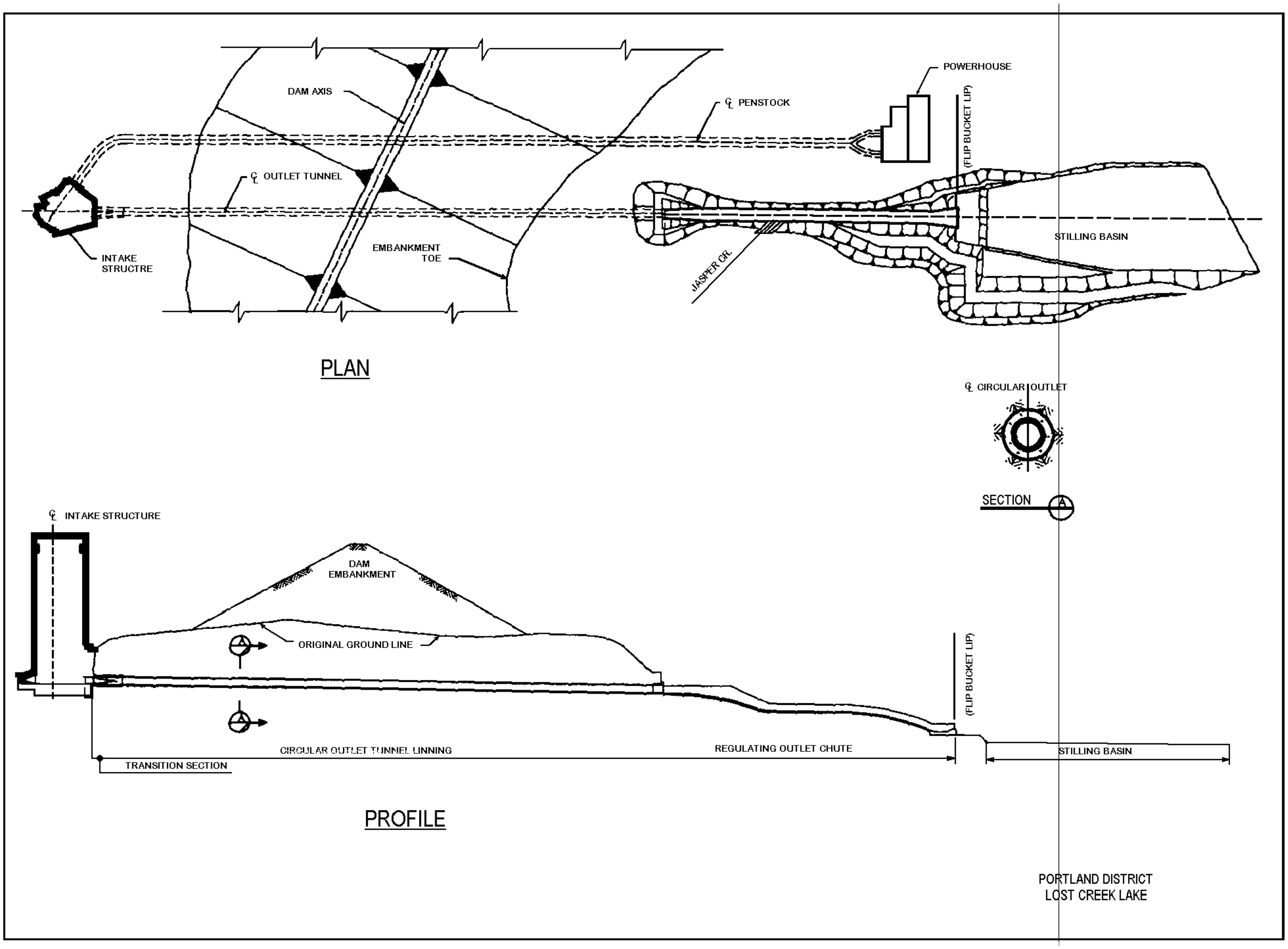


Figure 2-1. Outlet works through abutment, Lost Creek Lake, Oregon

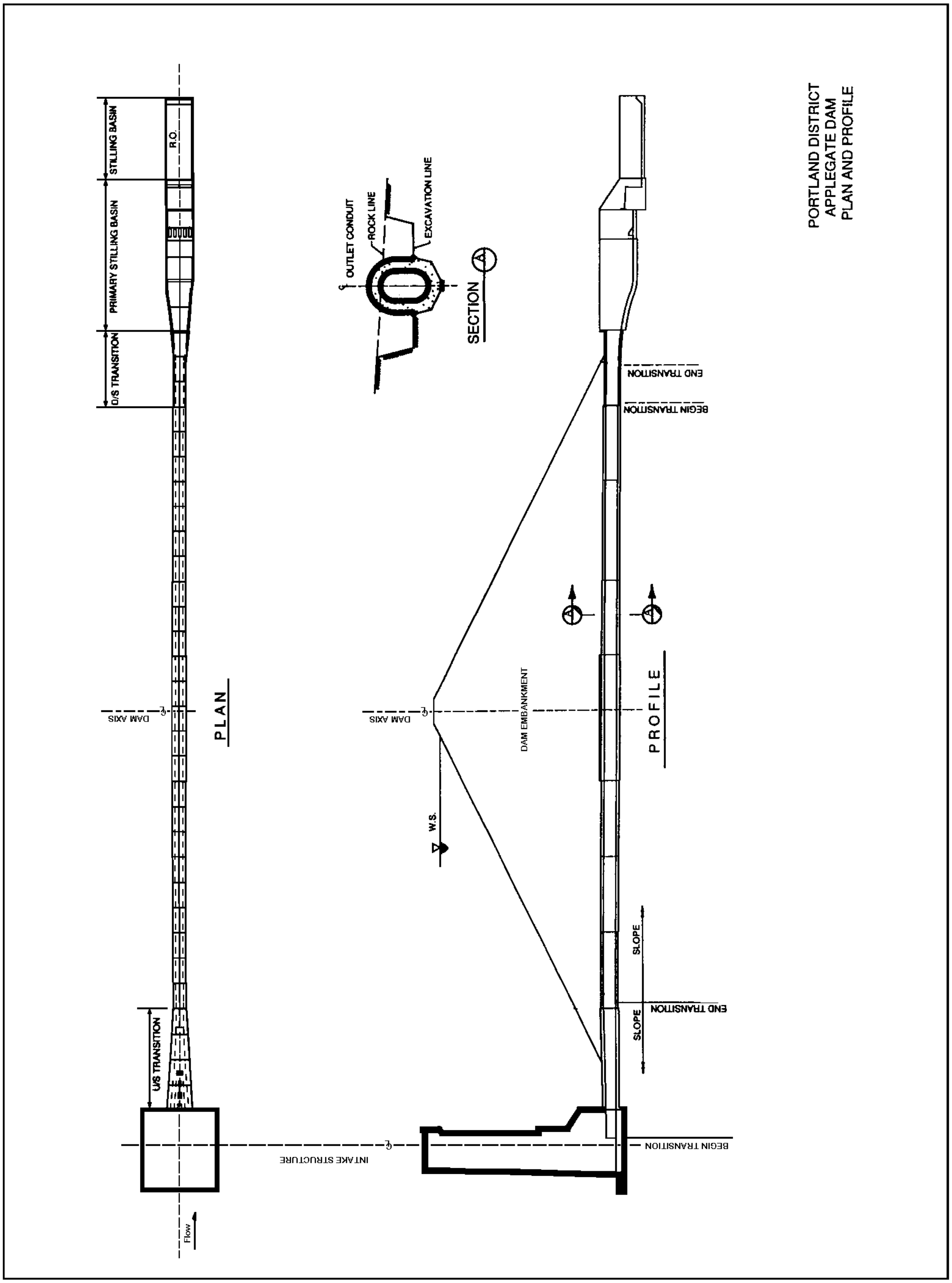


Figure 2-2. Outlet works under embankment dam, Applegate Lake, Oregon

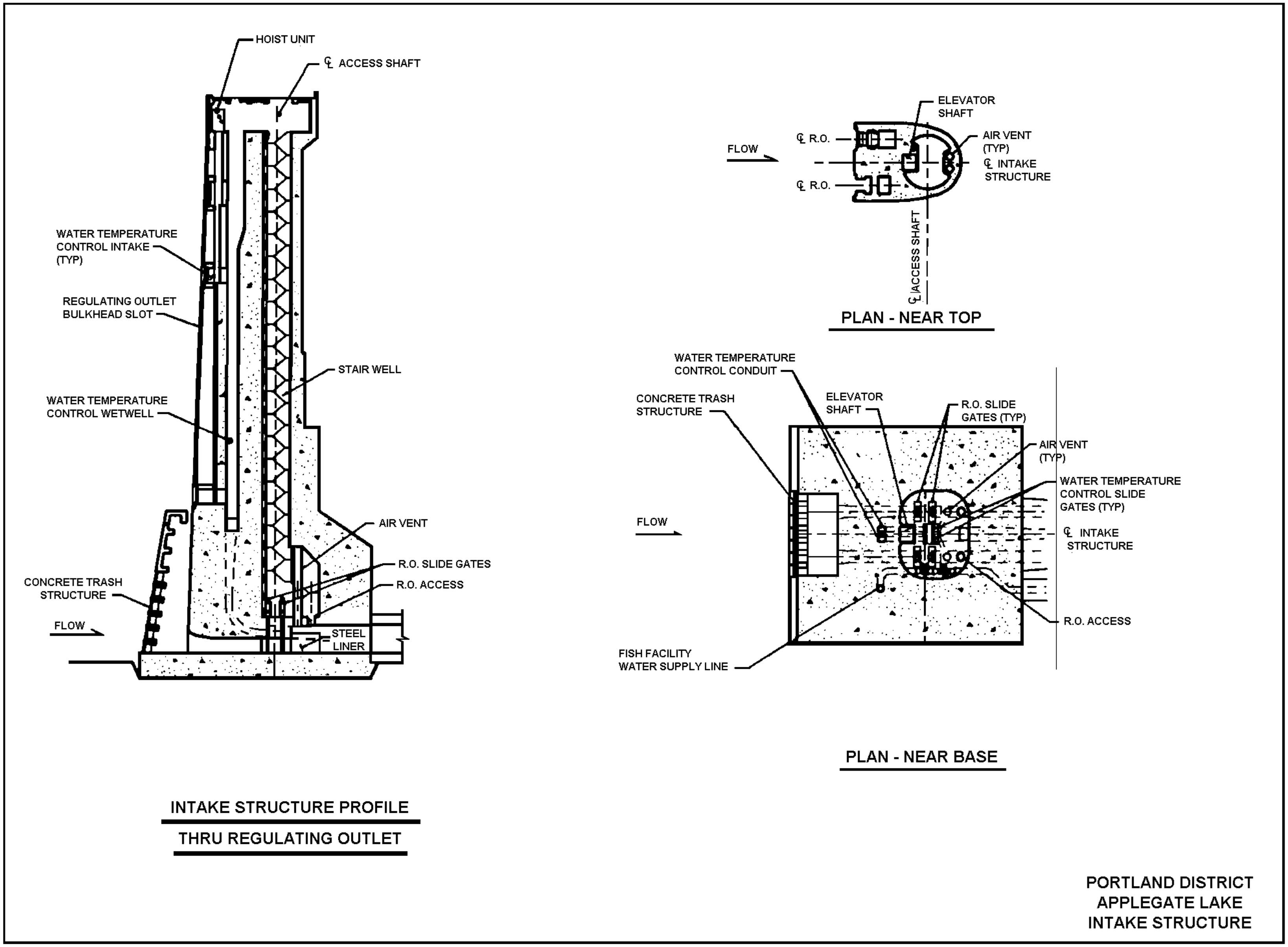


Figure 2-3. Vertical intake structure, Applegate Lake, Oregon

d. Control gating. Reservoir outlet works may be gated or ungated, depending on the project functions. Gated outlet works are needed for multiple-purpose reservoirs that provide storage for conservation, power, irrigation, etc. Gated outlet works are used also for single-purpose flood-control projects in which positive control of discharge regulation is required. Ungated outlets are adapted to some flood-control reservoirs, particularly small ones, where predetermined discharges (varying with the head) will meet the flood-control requirements. In general, the size of gates and conduits should be based upon diversion, evacuation, and operational requirements. A careful study and analysis should be made of the downstream channel capacity. The outlet works capacity should be adequate to discharge downstream channel capacity flows when the reservoir level is at the bottom of the flood-control pool. Consideration should be given to the time required to empty the reservoir for projects requiring emergency drawdown capability. The effect of rapid drawdown on materials at the reservoir margin must be considered. For gated structures, the number and size of gates should be such that one inoperative gate must not seriously jeopardize the flood-control function of the project. The use of two-gate passages is preferred for economy unless studies indicate the operating characteristics would be materially improved by the use of three-gate passages, or unless a wider structure is required for stability or structural reasons. The size of the gates should be based on an individual study for each project evaluating both the release capacity and downstream damage. The height of gates should generally be the same as the diameter of the tunnel or conduit in order to simplify the transition between the gated intake and the downstream tunnel or conduit. It is unlikely that large service gates will be able to regulate small flows with sufficient accuracy. This means, where low flow discharges are required, either a small gate within the service gate or a separate gate for small flows will be required. Usually, a separate gated system for low flow water and water supply regulation is the most reliable. In general, for medium height dams (heads between 80 and 120 ft) the most economical type of gate structure is one located at the upstream end of the outlet works consisting of a wet-well type structure with two gate passages, each passageway regulated by a hydraulically operated caterpillar or wheeled service gate with a single transferrable emergency gate operated by a traveling hoist located in the superstructure at the top of the intake tower. For high-head projects (heads over 120 ft) and for low-head projects (heads less than 80 ft), the most economical type of gate structure will often be different from that used for medium height dams. In any case, alternative studies will be required to verify that the type of outlet works structure selected is the most economical.

e. Exit chute and energy dissipater. Water flowing from the outlet works to the downstream river channel will be at high velocity. Energy-dissipating structures are commonly required to reduce flow velocities to levels that will not cause scouring, which can undermine tailrace structures and damage the downstream channel and riverbed. Energy-dissipating structures for outlet works may be similar to those found on spillways, such as hydraulic jump stilling basins or flip bucket plunge pools. Where releases are small, baffle block structures, riprap scour protection, or cut-off wall scour protection may be all that is required.

f. Approach and discharge channels. The purpose of the approach channel is to convey water into the intake structure. The discharge channel is designed to direct flows back to the river channel. The dimensions and alignment are determined by the hydraulic requirements and the stability of the excavated slopes.

2-2. Intake Structure Types

There are no standard designs for intake structures. Each design is unique and may take on many forms and variations. The intake structures discussed in this manual can be separated into two broad categories: free-standing and inclined. Selection of the appropriate type depends on a number of considerations including site conditions, economics, and effectiveness in meeting project requirements. Project requirements can include reservoir operating range, drawdown frequency, discharge range, trash conditions and required frequency of intake cleaning, reservoir ice conditions, water quality and temperature operating requirements, and environmental requirements such as fish passage. An intake structure may be submerged or may extend above the maximum reservoir water surface, depending on its function. Above-reservoir intake structures are necessary when gate controls are located on top of the structure, access to an internal gate control room is through the top of the structure, or when operations such as trash raking, stoplog or bulkhead installation, and fish screen cleaning are required from the structure deck. The submerged intake structure is primarily found at low-head, flood-control projects where submergence occurs only during flood periods or at projects where trash cleaning is not required. Submerged intake structures may also consist of simple submerged shafts and horizontal intakes equipped with a trash structure and bulkhead slots.

a. Free-standing structure. The most common type of intake structure is the vertical structure, generally referred to as a free-standing intake tower. It allows increased flexibility when locating the outlet works at the site. The vertical tower is usually more economical and easier to lay out than the inclined intake structure. Conduits and openings, operating equipment, and access features lend themselves more readily to arrangement in the vertical structure. A service bridge provides access to the top of the structure. Figure 2-3 shows a typical free-standing intake tower.

b. Inclined structure. For higher embankment dams in high seismic risk areas where a vertical structure may not be feasible, an inclined intake structure supported against the abutment is an alternative for consideration. An inclined structure has the advantage of increased stability over a vertical structure. In high-risk seismic locations and on steep abutment slopes, anchoring of the structure to the abutment to maintain stability and prevent liftoff should be investigated. Figure 2-4 depicts an inclined intake structure.

2-3. Functional Considerations for Outlet Works Facilities

The type and design of the outlet works facility will be greatly influenced by the project, its purposes, and structure-specific functions. An evaluation of each function is absolutely necessary in any design of the structure selected.

a. Flood control. Outlet works for flood-control projects generally require designs having large flow capacities and less regulation capabilities. Typically, the outlets are gated for flow regulation. However, the conduits may be uncontrolled (no gates) for reservoirs that are low or empty during non-flood periods.

b. Navigation. Projects releasing flows for downstream navigation usually involve lower discharge capacities than flood-control works. Requirements for close regulation of the flow and continuous releases are characteristic with outlets designed for navigation.

c. Irrigation. Gates and valves for irrigation require close regulation and lower discharge ranges than flood-control outlet controls. Releases may be discharged into a channel or conduit rather than into the original riverbed.

d. Water supply. Municipal water supply intakes are generally a secondary project function. Reliability and water quality are of prime importance in the design. Water intakes are located and controlled to ensure that the water is free of silt and algae, to obtain desired temperatures, and to allow intake cleaning.

e. Power. Power penstocks within intake structures should be located so as not to cause any undesirable entrance flow conditions such as eddies that might jeopardize turbine operation. Power intakes may require smaller trashrack openings to limit the size of debris that enters the penstock. Requirements for power intakes are covered in EM 1110-2-3001.

f. Low-flow requirements. Low-flow releases are required at some projects to meet environmental objectives, downstream water rights, water supply, etc. Smaller conduits and pipes separate from the main outlet conduits may be required to maintain minimum releases. Intake trashracks should be sized to prevent plugging of the smaller lines.

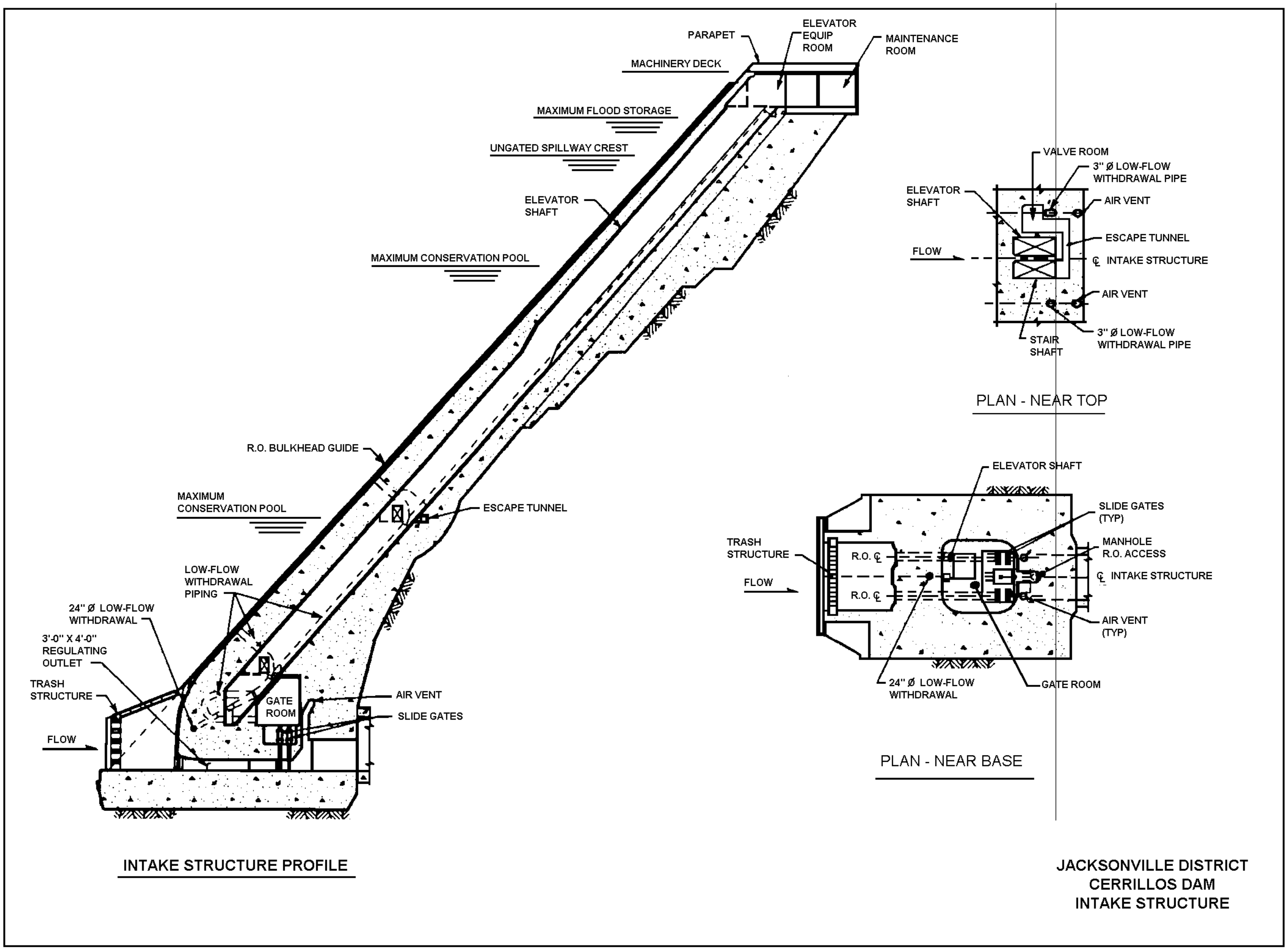


Figure 2-4. Inclined intake structure, Cerrillos Dam

g. Water temperature. Project requirements for fish and wildlife may involve intake features that control water temperature downstream of the reservoir. Multilevel ports and skimmer weirs in conjunction with a mixing wet well may be used to achieve this function. These features are incorporated separately or in combination with the other functions of the intake outlet works.

h. Sediment. Projects designed for sediment retention should be designed to pass flows as the sediment level rises in the reservoir and to prevent sediment from passing through and damaging or blocking both the entrance and the outlet works itself. Smaller releases are controlled by multilevel intakes that are closed by gating or stoplogs as the sediment level rises in the reservoir. For projects with flood flows that are released through the outlet works, a high-level intake that provides protection against large sediment buildup may be necessary.

i. Drawdown. ER 1110-2-50 requires that all new projects have low-level discharge facilities for drawdown of impoundments. Chapter 5 provides further discussion of this requirement.

j. Diversion. Outlet works may be used for total or partial diversion of the river during construction of the dam. The outlet works must be sized for both diversion and final project releases. A cut-and-cover outlet conduit that is used for diversion is typically located outside the normal river channel to facilitate ease of construction. Except for some high-head dams, it usually is economical to place an outlet tunnel near the river channel elevation, allowing its use for diversion. The scheduling and staging of the outlet works in conjunction with the dam construction require careful evaluation. This is particularly important when second-stage outlet works construction needs to be coordinated with dam closure and the ongoing diversion.

k. Outlet diversion with a high-level intake. An alternate plan used for some deep reservoirs includes a high-level intake structure at the head of an inclined tunnel connecting with a tunnel through the abutment near river level. After the river-level tunnel is used for diversion and construction is completed, the low-level tunnel is plugged at the upstream side of its intersection with the inclined tunnel and downstream portion of the low-level tunnel with a connecting curved transition forming the permanent outlet. Figure 2-5 shows an example of this plan. This plan may be economical and satisfactory under special conditions in which the reservoir is very deep and the site provides sound rock for supporting the intake structure and the complex tunnel construction. However, foundation investigations and comparative studies should be performed. The design will require low-level drawdown facilities as outlined in ER 1110-2-50. This plan should not be selected without full analysis of its practicability and economy.

2-4. Intake Tower Design Considerations

a. General. The outlet works is an integral part of a project that includes a dam and spillway. Its layout and configuration therefore should be associated with the planning and development of the complete project. In all cases, selection of the best overall plan for the outlet works should be made after careful comparative studies of alternative plans and consideration of the site conditions. Functional and service requirements, component interrelationships and compatibility, economy, safety, reliability, and repair and maintenance requirements should all be considered in the studies. Site conditions include topography, climate, foundation, geology, and seismicity. Hydrology and minimum flow requirements are important for determining the range of design releases for the outlet and diversion conditions. All operation and maintenance requirements must be identified in order that a safe, reliable, and economical outlet works will be designed. After all the purposes of the project are established and the functions and criteria for the outlet works have been clearly defined, the geometry and layout of the intake tower can proceed. Multiple alternatives should be developed and evaluated to determine the optimum plan.

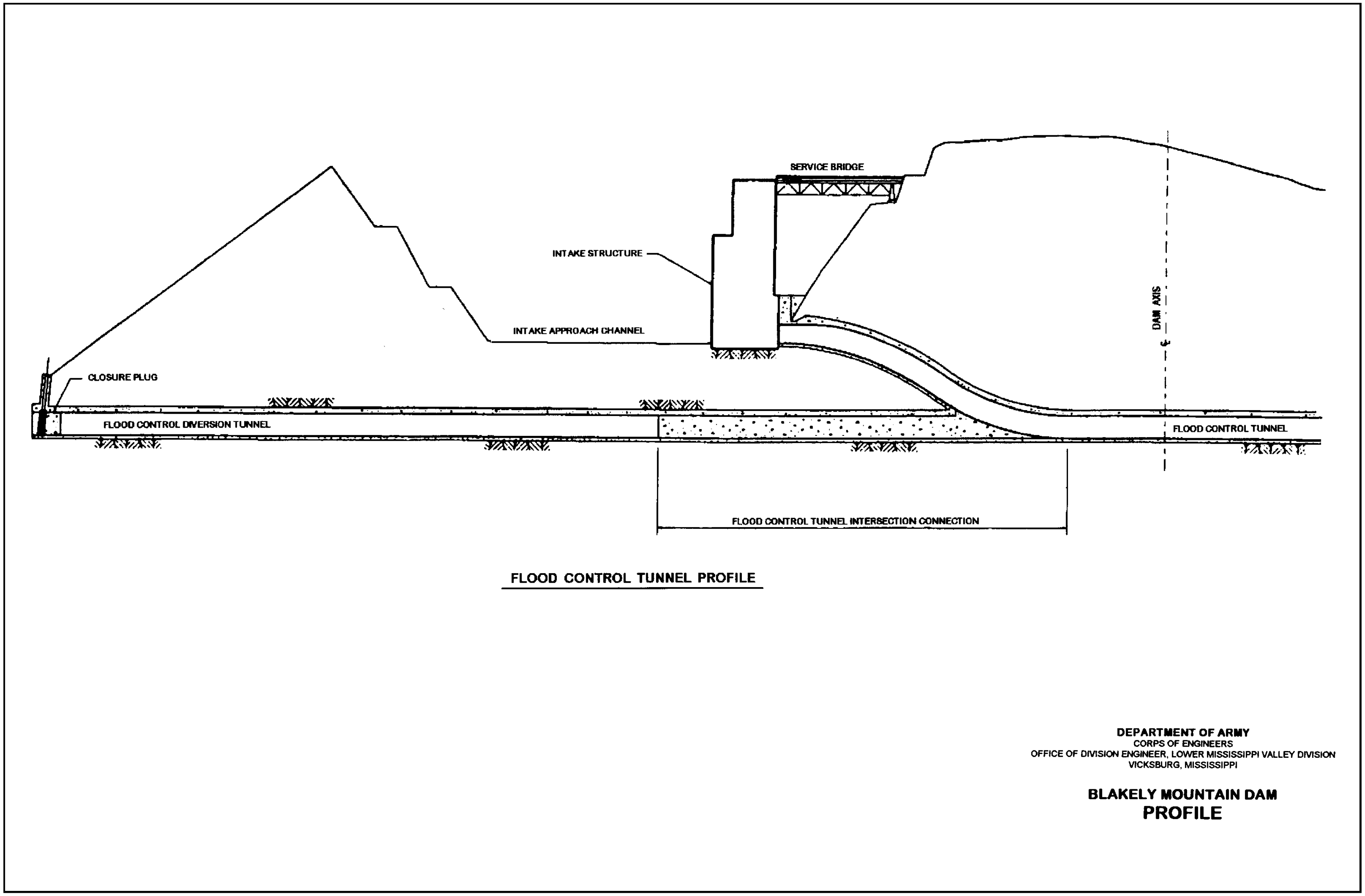


Figure 2-5. High-level intake, low-level diversion tunnel, Blakely Mountain Dam

b. Design considerations and criteria. The intake structure size and configuration are based on the inlets and collection wells size, control gate passages, inlet hydraulic configuration, exit passages, structural element size, and the space and clearance requirements for the mechanical and electrical equipment. The intake structure may take on numerous shapes and forms for each potential alternative. The final configuration will take the effort of a fully coordinated team of structural, mechanical, electrical, material, cost, and geotechnical engineers; a geologist; and hydrological and hydraulic engineers to ensure that all engineering and geological considerations are properly integrated into the overall design. When water quality is a consideration, reservoir hydrodynamics and environmental studies are important in the design process. Mathematical and/or physical hydraulic modeling may be required to support the hydraulic and environmental design.

c. Structural design considerations. After the design layout has been selected, the structural engineer should evaluate the structural integrity. One primary responsibility of the structural engineer is to ensure that the structural design meets the objective of carrying design loads from the top of the structure to the foundation. Wet wells with multilevel ports and weirs must be checked for structural stability, materials, sizing of members, connection details, reinforcement layout, and geometrical compatibility with adjacent features. The objectives of effective structural planning are to maintain symmetry, minimize torsional effects, provide direct vertical paths for lateral forces, and provide a proper foundation. A continuous load path or paths with adequate strength and stiffness that will transfer all forces from the point of application to the final point of resistance must be provided.

d. Siting. Selection of the location for an intake structure depends on multiple factors, including foundation approach conditions, alignment of conduit, and tunnel access. For an embankment dam, the structure is often located adjacent to a hillside into which the outlet works pass. For a cut-and-cover-conduit, the structure is generally located at the upstream toe of the embankment dam. The location and configuration of the structure will also depend on whether the structure will be vertical or inclined. For a vertical intake structure, the location will be determined largely by the design layout of the rest of the outlet works and the dam. However, adequate information about the foundation conditions should be obtained prior to the final site selection to ensure that the foundation will be suitable. For an inclined structure, the selection of the location of the base and the inclination angle are determined primarily by the ground profile and properties of the foundation materials present, e.g., depth of rock weathering, bearing strength, etc.

e. Water intakes. The water intakes are basically positioned with respect to the range of reservoir levels and to meet particular operating functions (project purposes). Low-level reservoir evacuation, sediment deposition levels, and minimum power generation are other criteria for consideration. Intake layout should consider the design of transitions, branches, heads, etc. A trash structure is required for most intakes to protect gates, valves, or turbines. The sizes of the trashrack openings are governed by the minimum waterways opening and the gate size. Further details on the trashracks and their design are provided in Chapter 5.

f. Locations of control gates. The control gates at U.S. Army Corps of Engineers projects are usually located in the intake structure, in a shaft or gate chamber near the extended axis of the dam, in the abutment under or within the dam section, or in one or both of the abutments. Under special conditions, the control gates may be located at the downstream end of a pressure tunnel. The principal advantages and disadvantages of alternative locations for the control gates are discussed in the following paragraphs. Control gates along the outlet and at the downstream end are discussed only briefly. The discussions are applicable to outlet works without diversion and to those with diversion.

(1) Control gates at upstream end. When the gates are placed at the upstream end of the outlet works, the emergency gates, bulkheads, service gates, and trash structures are all combined into a single structure. Closing of the gates allows for inspection and repair of the entire tunnel and for readily removing accumulated trash, sediment, or other deposits. A combined structure is particularly advantageous in cases in which

the permanent conservation or maximum power pool elevation is well above the river level. When other locations are selected for the gate structure, a separate structure for trash removal and bulkhead closures would generally be more expensive. With the control gates at the upstream end, the internal hydrostatic pressure in the tunnel is less than the weight of the overlying rock, earth, and water at all points along the conduit/tunnel profile. A disadvantage with upstream control gates is the cost of extending the structure above the pool and requiring an access bridge. The upstream gate location is preferred when the cost is nearly the same or even slightly higher than that for other gate locations. The upstream closure of the outlet conduits with the control gates provides additional dam safety. The types and limited sizes of gates or valves suitable for use under high heads sometimes require multiple gates in wide intake structures and long transitions between the gates and the outlet tunnel or conduit.

(2) Control gates near dam axis. Placing gates in a gate chamber or shaft in the abutments provides increased protection at high-seismicity sites. Disadvantages include the requirement of a separate bulkhead closure at the upstream end to provide the capability of dewatering the upstream portion of the tunnel for inspection, access for operation and maintenance, and designing for high earth loads where the gate is located in an embankment dam. Tunnels and conduits upstream of the gate chamber will need to be designed for full internal hydrostatic heads.

(3) Control gates at downstream end. Full internal hydrostatic pressure over the entire outlet tunnel requires a steel lining inside the concrete lining to prevent unsafe hydrocharging through cracks in the concrete liner of the surrounding rock near the downstream end. The length of the steel lining will depend on the geologic conditions and the depth of rock over the tunnel. This requirement generally makes this alternative less economical. The downstream location may be favorable in special conditions such as high-head projects and short outlet tunnels. As with the location along the dam axis, upstream bulkhead closure is required.

g. Intake structure shape. There are numerous geometric shapes and combinations that can be developed in the design of an intake structure. The elevational profile can be uniform or tapered, and the plan can be rectangular, circular, or irregular. The following paragraphs discuss three basic geometric shapes.

(1) Rectangular. Rectangular intake structures are more functional for low-head reservoirs that are designed for large discharges. A rectangular shape provides for more efficient layout of entrances and openings, gates and operating equipment, and other features. Rectangular intake structures are usually more easily constructable and site-adaptable. Figure 2-6 shows a rectangular intake structure.

(2) Circular. Circular intake structures are structurally more efficient, providing economic savings, particularly in high-head projects. Hydraulic and access requirements can readily be adapted to the circular shape. The lower section of the structure may be rectangular to provide for arrangement of the water intakes, trashracks, and bulkheads.

(3) Irregular. An irregular design may result in the development of very complex and unusual shapes. Structures with wet and dry wells, high-level intakes, multiple wet wells, fish entrances, and other special features result in numerous different and unusual configurations. Figure 2-7 shows an irregular intake structure.

h. Wet well. Wet wells of intake structures are used to withdraw water from different levels of the reservoir and for mixing to achieve specified controlled temperature releases. Wet wells are also used for multi-port sediment retention where ports are closed with stoplogs as the sediment level rises in the reservoir. Wet wells can be sized as a single unit or several wells.

i. Dry wells. Dry wells, also referred to as access shafts, provide access to the gate room when located near the base of the intake structure. Features commonly incorporated into a dry well are elevator, stairs, air

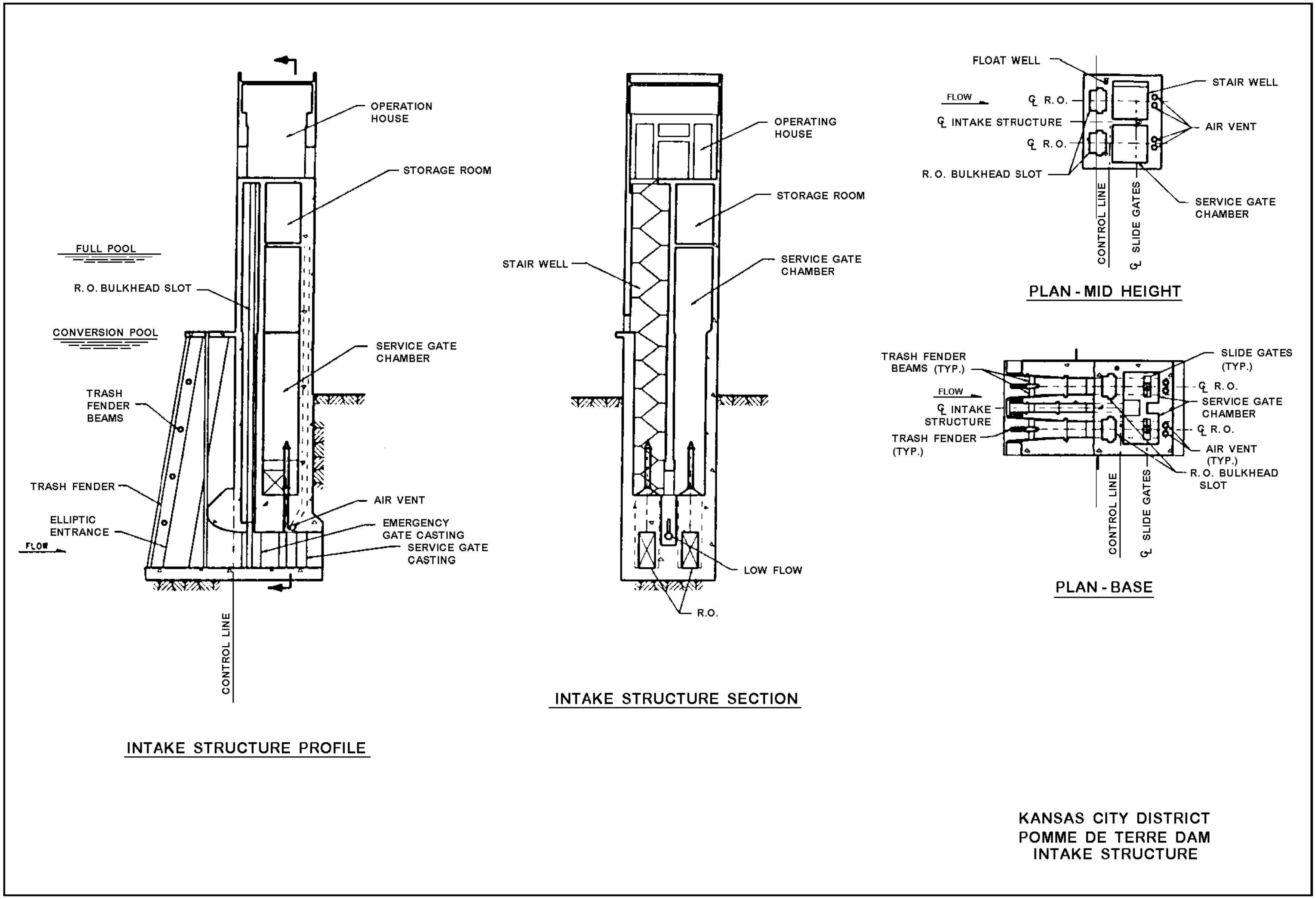


Figure 2-6. Rectangular intake structure, Pomme De Terre Dam

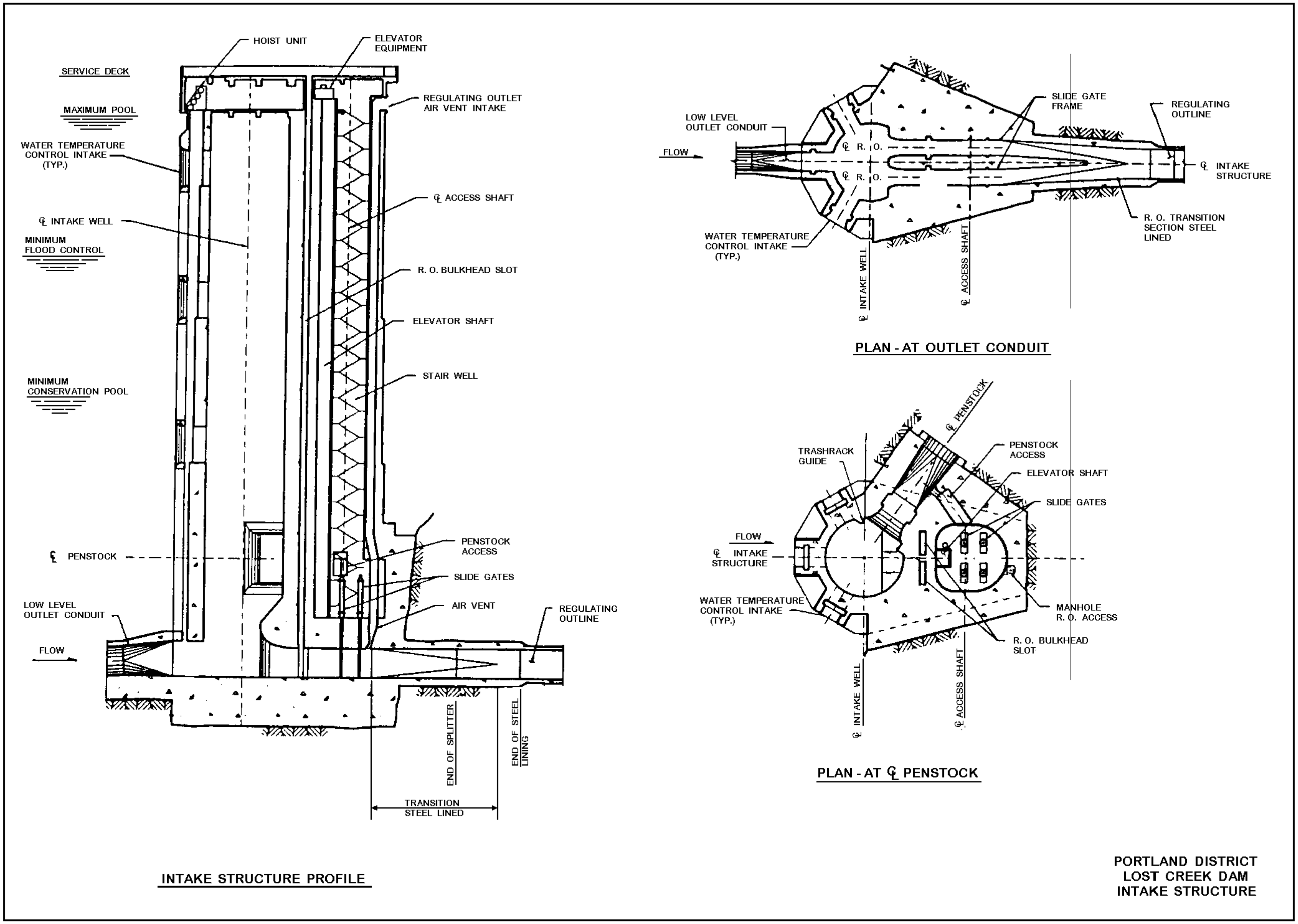


Figure 2-7. Irregular intake structure, Lost Creek Dam

vents, and an open shaft area for the removal and installation of equipment for repair and maintenance. These features should be arranged in the most efficient manner to minimize the dry well size. The open shaft area in the dry well should be sized to allow removal of gates and other gate room equipment.

j. Gate room. Gate room design with both service and emergency gates is outlined in this paragraph. When hydraulically operated slide gates are used, the service and/or emergency gates are usually located in a room in the base of the dry well. This arrangement is applicable to a single-conduit outlet works for a structure at the upstream end of the outlet works, a shaft near the dam axis, or a gate structure near the downstream end of the conduit. A circular-shaped gate room may be advantageous, especially for a high structure. When caterpillar or wheel service and emergency gates with downstream seals are used, the typical gate structure for a single-tunnel outlet works is of the wet well type (usually rectangular in horizontal section) with partition walls between the gate chambers dividing it into separate gate wells. The partition walls make possible the dewatering of any service gate by placing a single emergency gate in a guide upstream of the wall. This type of structure is used also for mechanically operated slide gates with gate stem extensions but is economical only for low-head structures. The water passages at the bottom of the structure are made with gradual transitions at the entrance and downstream from the gates. The intermediate and exterior walls of the passages are made the minimum thickness required to accommodate the gate frames and to withstand the hydraulic forces and structure loads. Minimum access provisions between the operating deck and base of the gate structure should consist of a ladder. In a dry-well structure, a stairway to the gate chamber is desirable. When the gate chamber is located more than 50 ft below the operating deck, an elevator of minimum size and capability is reasonable in addition to the stairway.

k. Basic architectural considerations.

(1) Functional design. Architectural design should focus on meeting operational, safety, and material requirements to include the following:

(a) Access/egress to/from the structure and movement within the facility shall comply with the latest edition of the NFPA 101, Life Safety Code. This includes design of such items as guardrails, walls, partitions, corridors, stairs, doors, etc. Ladders shall be designed in accordance with the requirements of EM 385-1-1, Appendix J. Compartmented storage or other specialized space features shall be incorporated in accordance with the latest edition of the International Code Counsel (ICC), International Building Code (IBC), and National Fire Protection Association (NFPA) requirements.

(b) System and materials should be selected based on durability, maintainability, economy, and aesthetics. Materials should conform to the Corps guide specification requirements and to nationally recognized industry standards. Selection should consider availability and future replacement to minimize first costs and costs associated with maintenance and repair. Exterior components should be selected and located where damage by vandals will be precluded.

(2) Aesthetic design. Locale must be considered in determining the exterior visual design of an intake tower. Geographical effects such as prevailing weather patterns should be considered in placement and protection of access points or other openings. The relationship of the tower to other existing or proposed structures should be considered and a compatible style or details developed. Remote sites with little opportunity for public access and limited visibility require less architectural treatment than sites where the tower can easily be viewed or visited by the public. When surface treatment of concrete is being considered, EM 1110-1-2009 should be followed.

2-5. Outlet Tunnels Design Considerations

a. General. The location of an outlet tunnel or tunnels is affected by topography, abutment foundation conditions, hydraulic requirements, and economy of construction. Except for some high-head dams, it usually is economical to place an outlet tunnel near stream elevation and use it for diversion during construction. A tunnel should be located far enough into the abutment to obtain adequate cover for the character of rock encountered, and the tunnel alignment must meet the hydraulic requirements. The most economical location to pass around or under the dam and meet the foregoing requirements should be selected. An alternate plan used for some deep reservoirs has a high-level intake at the head of an inclined tunnel connecting with a tunnel through the abutment near river level (see Figure 2-6). After serving its purpose of diversion during construction, the low-level tunnel is plugged at the upstream side of the intersection and the inclined tunnel and downstream portion of the low-level tunnel with a connecting curved transition from the permanent outlet (see Figure 2-5). This plan may be economical and satisfactory under special conditions where the reservoir is very deep, the minimum water surface is high, and rock foundation is adequate. Because of the difficult foundation and structural problems involved in this type of construction, thorough foundation investigations and comparative studies should be made and this plan should not be selected without full analysis of its practicability and economy. The advantage of the high-level intakes for a very deep reservoir is the lower intake structure and lower head on intake gates, resulting in a less expensive intake structure. Disadvantages of this type of outlet are: cost of the inclined tunnel, difficulty of obtaining an adequate foundation for the intake structure, and the difficulty of excavation for and the complexity of the transition structure at the junction of the inclined tunnel with the low-level tunnel. Particular care should be taken to obtain high quality concrete, smooth gradual curves, and good alignment at the junction. If the intake structure is located directly over the low-level tunnel, there should be an ample thickness of good, sound rock between the intake base and the tunnel to transmit the foundation loads to the main rock mass. Sometimes conditions may be favorable for locating the intake far enough to one side of the low-level tunnel to simplify the problem of supporting the intake structure. For some high dams, outlet works separate from diversion facilities and located much higher than the river channel may be suitable. In such cases, overall economy may result from use of separate high-level outlets and use of the downstream portion of the diversion tunnel as part of the spillway outlet.

b. Number and size of tunnels. The proper selection of number and size of outlet tunnels depends on hydraulic requirements, the maximum size of tunnel which it is practicable to drive in the rock encountered, operating flexibility, and economy of construction. In some instances it may be necessary to limit the size of tunnel in yielding rock to prevent excessive movement in the surrounding rock. Generally the larger the tunnel within practicable driving limits, the more economical it is per unit flow capacity. Also, the approach and discharge channels and the intake structure usually are more economical for fewer tunnels because the reduced width results in less channel excavation and a narrower intake structure. However, if it is necessary to use two parallel gates in each larger tunnel and only one in the smaller, the intake structure may not be reduced and economy may not be affected by use of larger tunnels. The most economical number of tunnels which will meet the other requirements should be used. A single tunnel is used when, as is frequently the case, the required diameter to handle the entire discharge is not over 20 ft to 25 ft. In order to provide reasonable operating flexibility, a total of not less than two service gates should be provided in the outlet works. In some cases, this requirement may influence the number of tunnels. Detailed design and construction procedures for tunnels are provided in EM 1110-2-2901, “Tunnels and Shafts in Rock.”

c. Approach and discharge tunnels. The dimensions and alignment of the excavated approach channel upstream from the river to the tunnel and of the discharge channel to convey the water back to the river downstream are determined by the hydraulic design requirements and the stable slopes for the material excavated.

2-6. Cut-and-Cover Conduit Design Considerations

a. General. A typical cut-and-cover conduit through an embankment dam is near stream bed level so that it will economically fulfill the hydraulic requirements for the outlet works and serve for diversion during construction. This conduit is placed on suitable rock whenever practicable, as good foundation is of paramount importance to keep settlement as small as possible. The most economical location which provides a good foundation is selected. When practicable the conduit is placed outside the normal river channel to facilitate construction. A location to one side of the river valley often provides the best foundation. Where the conduit sides are placed against rock, presplitting should be used to obtain close control of excavation. Water leaking from or onto the cut-and-cover conduit can lead to piping of embankment dam materials and jeopardize dam safety. Care must be taken when using cut-and-cover construction to assure that the conduit will remain watertight. The use of cut-and-cover construction where the conduit will be pressurized (gate structure near axis of dam or at downstream end) is unacceptable because leakage into the embankment under the higher internal water pressure would be dangerous should a rupture of the conduit occur.

b. Arrangement of multiple conduits. Multiple outlet conduits usually are placed close together so that the intake structure may be the minimum width that will satisfy the hydraulic and structural requirements. They should be planned to minimize cracking by keeping the conduit width to a minimum. This is accomplished by a single multiple-opening conduit structure with longitudinal contraction joints between conduits and generally not more than 50 ft apart. Detailed design and construction procedures for conduits are provided in EM 1110-2-2902, "Conduits, Culverts and Pipes."

c. Approach and discharge channels. The dimensions and alignment of the excavated approach channel upstream from the river to the tunnel and of the discharge channel to covey the water back to the river downstream are determined, in general, by the hydraulic requirements and the location of stable slopes for the material excavated. Concrete retaining walls are provided as needed just upstream from the intake and the trash rack to retain the upstream toe of the embankment on each side of the approach channel and similar retaining walls are provided at the downstream end of the outlet works, if required.

2-7. Energy-Dissipating Structures Design Considerations

Most often the type of energy-dissipating structure needed will be determined by hydraulic model studies. These studies should also be used to determine the loadings to be expected on the energy-dissipating structure during all flow conditions up to the Maximum Design Flood (MDF). Energy-dissipating structures may consist of (a) abrupt expansions in high pressure conduits, (b) hydraulic jump-type stilling basin structures, (c) flip bucket plunge pool structures, valves, and deflectors which spay jets into the air before entering a plunge pool structure, and (d) various baffle block energy-dissipating structures. The stability of the various energy-dissipating structures will be determined in accordance with EM 1110-2-2502. Information on spillway-type stilling basins can be found in EM 1110-2-2200. The structural design of energy-dissipating structures will be in accordance with the provisions of EM 1110-2-2104. The hydraulic design requirements of these structures can be found in EM 1110-2-1602.

2-8. Coordination Among Disciplines

The structural engineer should be involved in technical coordination of structural features during all project phases. Structural design activities should be coordinated with other functional elements of geotechnical, hydrology, hydraulic, mechanical, electrical, and architectural design, construction, operations, cost engineering, real estate, surveying, mapping, etc., to develop the design of the structural features. Technical coordination should be maintained with the technical staff of the local sponsor. Also, technical coordination with higher authority is encouraged to reach early agreement on unprecedented or complex problems.

Chapter 3
Outlet Works Design

3-1. General

a. Summary. This chapter presents discussion and guidance on the concrete and foundation properties generally considered in design, the loads and load conditions typically applied to outlet works intake towers, and various other information on stability analysis and structural design. Design and load conditions of the intake structure operating components and appurtenant features are discussed in Chapters 5 and 6.

b. Structural design. The structural design should be developed in conjunction with the hydraulic design to provide a stable, load-resistant structure which meets all functional and structural performance requirements. Structural elements of the intake structure should be sized and proportioned to resist the various load conditions that will be imposed during construction and operation. The structural design and geotechnical conditions at the site must be closely coordinated.

c. Stability. Initial layouts are followed by a stability analysis. If the structure fails to meet criteria, the layout is modified and reanalyzed. This process is repeated until an acceptable configuration which meets stability criteria for all load conditions is attained.

d. Strength, serviceability, and dam safety. Outlet works structures must be designed for strength and serviceability and to protect the project against potential latent deficiencies that could jeopardize dam safety.

(1) The structure strength must be sufficient to provide the required margin of safety for all possible loads and load conditions. Serviceability means the structure is watertight and resistant to potential damage from cavitation or abrasion erosion and is adequately protected against corrosion of reinforcing steel, embedded steel, and exposed steel components.

(2) The possibility of upstream ground slope instability due to floods or earthquakes should be taken into account during the design of the outlet works. During a major flood or earthquake, the ground slopes upstream of the project may become unstable causing landslides and the accumulation of large quantities of debris and sediment at the outlet works entrance. The configuration of the approach and approach walls should minimize the potential for sediment and debris accumulation which could block the outlet works entrance. Provisions should be made to allow for removal of debris and sediment that could accumulate at the outlet works entrance.

(3) Major floods and earthquakes can be catastrophic to lifelines. Roads, bridges, electrical power, and communications are necessary to respond quickly to an on-project emergency condition. The protection of project lifelines important to operation of the intake/outlet works should be provided for during design.

(4) Leakage from the outlet conduit or along the conduit/dam interface for cut-and-cover conduits can lead to slope instability at critical abutment areas and at the embankment dam. The intake structure/conduit layout and design should protect against any such deficiencies that might jeopardize dam safety.

3-2. Design Data

a. Concrete properties. Concrete properties normally used to design intake structures include the unit weight; compressive, tensile, and shear strengths (friction and cohesion); Young's modulus of elasticity; and Poisson's ratio. Mass concrete structures that require thermal studies to assess the effects of stresses induced by temperature changes in the concrete require thermal properties which include conductivity, thermal diffusivity, specific heat, and the coefficient of thermal expansion. In structures that are designed to pass sediment, trash, or other debris, special high-strength concretes such as silica fume concrete may be utilized to provide highly abrasion-resistant surfaces.

b. Foundation properties. The principal foundation properties for outlet works design are bearing capacity, deformation and elastic modulus, and shear strength. These properties determine suitability of the rock mass to support the structure placed on it and the resistance to sliding at the structure/foundation interface. Dynamic elastic modulus and shear strength are important factors in the seismic design of these structures. Also very important to stability analysis is whether any weak zones exist within the foundation where sliding failure could occur. Such zones may consist of shears, faults, or other discontinuities along which material of low strength such as clay gouge may be present in an otherwise sound rock mass. Significant voids such as those produced by dissolving of some limestone formations also influence foundation suitability. The foundation properties must be investigated early in the design phase of the project to determine design parameters and any remedial treatment that may be necessary during excavation and construction. The structural design and contract drawings should include provisions for unforeseen conditions which may require that the intake structure be founded at a lower elevation in order to reach competent rock.

3-3. Loads

The following forces are typical of those that may be required in the stability and stress analysis:

a. Dead load. The dead loads considered should include the weight of the structure and appurtenances such as gates, trashracks, bridges, and other attached features.

b. Hydrostatic. The hydrostatic loads against the structure should include internal and external pressures for all design operating conditions as well as for any construction conditions relevant to intake structures designed for diversion.

c. Uplift. Uplift pressure due to headwater exists along the structure-foundation interface, within the foundation below the base, and through any cross section within the structure. Uplift pressure is present within the cracks, pores, joints, and seams in the concrete and foundation material. Uplift at the base should be assumed to be 100 percent of the reservoir pressure over 100 percent of the base area. At internal planes, uplift shall be assumed to vary linearly from the hydrostatic head at the external surface of the intake structure to the hydrostatic head at any internal surface. Uplift pressures should be applied over 100 percent of the area of the plane and are assumed to remain unchanged during an earthquake.

d. Earth and silt. For structures within the dam embankment, earth pressures should be included in the design. Silt pressures are included where sediment buildup around the intake structure can be expected. The silt and earth material may or may not be submerged. The design should include evaluation of the site for liquefaction. The magnitudes of earth and silt pressures are discussed in EM 1110-2-2502.

e. Temperature. Temperature rises and cracking resulting from cement hydration are a concern only in intake structures with large massive concrete sections. Temperature rises caused by cement hydration can cause significant cracking. Cracking can be minimized and controlled through temperature control of mix, lift heights, insulation, and use of reinforcement steel. Temperature control measures are discussed in greater detail in Chapter 6, paragraph 6-4.

f. Earthquake forces. Chapter 4 presents criteria and guidance for determining earthquake forces.

g. Wind pressure. Wind loads shall be applied to the structure in accordance with American Society of Civil Engineers (ASCE) 7 (ASCE 1995). Wind loads will also be applied to the appurtenant structures as applicable.

h. Ice pressures. In locations where ice is a design consideration, the ice pressure should be applied in combination with operation and maintenance conditions. EM 1110-2-1612 furnishes guidance for magnitude and application of ice loads.

i. Debris and trash. Loadings on the structure resulting from debris and trash will depend on the nature of the trash burden in the reservoir, maximum hydraulic differential created by debris buildup on trashracks, the structure layout and operation, and other factors. Chapter 5 further discusses debris loading.

j. Wave pressure. Wave pressures will be determined from the fetch of the reservoir as related to the intake structure and the wind velocity and duration. The *Shore Protection Manual* (1984) provides information relating to wave forces.

k. Operation and maintenance loads. Cranes, trucks, stoplog and bulkhead installation and storage, and other maintenance equipment are loadings for consideration.

3-4. Load Conditions

a. General. The loading conditions provided in Table 3-1 generally apply for checking the stability and structural design of an intake structure. The loading cases for a specific intake structure should be chosen as applicable from the lists below. Specific operational and site conditions from construction through project life and structure configuration may require that the following conditions be modified or that additional analysis of conditions be made.

b. Basic loading conditions descriptions. Specific operational and site conditions from construction through project life and structure configuration may require that the stability loading conditions be modified, or that additional analysis of conditions be made. The loading conditions of Table 3-1 are described below.

(1) Loading Condition U1 - normal pool, all gates open.

- Dead load of structure.
- Reservoir at normal operating pool, annual mean maximum pool elevation with a 2-year return period.
- Earth load (if any).
- Ice loads, if applicable.
- Uplift.
- Water surface inside structure drawn down to hydraulic gradient with all gates fully open.
- Wave loads, if applicable.

Table 3-1
Intake Tower Loading Condition Classification

Load Case Number	Loading Description	Loading Condition Classification
U1	Normal Pool, All Gates Open	Usual
U1	Normal Pool, All Gates Open	Usual
U2	Normal Pool, One or More Gates Closed	Usual
U3	Normal Pool, All Gates Closed	Usual
U4	Normal Pool with Silt	Usual
U5	Minimum Pool	Usual
UN1	Infrequent Flood, All Gates Open	U/UN/E
UN2	Infrequent Flood, One or More Gates Closed	U/UN/E
UN3	Infrequent Flood, All Gates Closed	U/UN/E
UN4	Construction	Unusual
UN5	Diversion	Unusual
UN7	Maintenance Bulkheads in Place	Unusual
UN8	Operating Basis Earthquake	Unusual
E1	Maximum Design Earthquake	Extreme
E2	Maximum Design Flood	Extreme

Note: U = Usual; UN = Unusual; E = Extreme.

(2) Loading Condition U2 - normal pool, one or more gates closed.

- Dead load of structure.
- Reservoir at normal operating pool.
- One or more gates closed with other gates fully open and water surface drawn down to hydraulic gradient in remainder of structure in combinations that produce the most unstable conditions.
- Earth load (if any).
- Ice loads, if applicable.
- Uplift.
- Wet well full of water upstream from closed gate.
- Wave loads, if applicable.

(3) Loading Condition U3 - normal pool, all gates closed.

- Dead load of structure.
- Reservoir at normal operating pool.
- Earth load (if any).
- Uplift.
- Wave loads, if applicable.

(4) Loading Condition U4 - loading conditions U1 through U3 with silt (if any).

(5) Loading Condition U5 - minimum pool.

- Reservoir empty or at minimum pool.
- Dead load of structure.
- Earth load (if any).
- Ice loads, if applicable.
- Wind load in the direction that would produce the most severe foundation pressures.
- Uplift.
- Wave loads, if applicable.

(6) Loading Conditions UN1 through UN3. Loading conditions UN1 through UN3 are the same as U1 through U3 except the reservoir level is at the infrequent flood stage (i.e., water is at the top of the spillway gates for gated spillways or at the spillway crest for flood-control projects with ungated spillways).

(7) Loading Condition UN4 - construction.

- Reservoir empty.
- Dead load of structure (partially or fully completed).
- Earth load (if any).
- Heavy construction equipment required on or near the structure during construction.
- Wind load in the direction that would produce the most severe foundation pressures.

(8) Loading Condition UN5 - diversion.

- Reservoir at maximum elevation expected during diversion.
- Dead load of structure at diversion level completion.
- Earth load (if any).
- Heavy construction equipment required on or near the structure.
- Wind load in the direction that would produce the most severe foundation pressures.

(9) Loading Condition UN7 - maintenance bulkheads in place.

- Bulkheads in place, no water in structure downstream of bulkheads.
- Dead load of structure.

- Reservoir at maximum pool level at which bulkheads are used.
- Earth loads (if any).
- Uplift.

(10) Loading Condition UN8 - operating basis earthquake (OBE).

- OBE for the most critical of the conditions U1 through U5 with the reservoir at the elevation that is likely to exist coincident with the selected earthquake event.
- No ice.

(11) Loading Condition E1 - maximum design earthquake (MDE).

- MDE for the most critical of the conditions U1 through U5 with the reservoir at the elevation that is likely to exist coincident with the selected earthquake event.
- No ice.

(12) Loading Condition E2 - maximum design flood (MDF).

- Pool at probable maximum flood (PMF) elevation.
- All gates opened or closed, depending on project operating criteria.

3-5. Stability Analysis

a. General requirements. The basic requirements for the stability of an intake structure and other outlet works structures are described in EM 1110-2-2502. In summary:

(1) Outlet works structures should be safe against rotational instability at the base, at any plane below the base, and at any horizontal plane within the structure.

(2) Outlet works structures should be safe against sliding on any plane within the structure, at the base, or on any rock seam within the foundation.

(3) Outlet works structures should be safe against foundation bearing failures.

3-6. Structural Design

All parts of the intake structure should be designed to meet the strength and serviceability requirements outlined in EM 1110-2-2104. Load factors should be in accordance with EM 1110-2-2104 unless otherwise indicated (Refer to Appendix B for seismic loading condition strength requirements). The reinforced concrete elements of the structure should be designed to withstand the loading conditions given in paragraph 3-4. Additional loading conditions may be required to account for site-specific conditions not covered by paragraph 3-4.

Chapter 4
Outlet Works Seismic Criteria

4-1. General

a. Design and evaluation objectives. Outlet works located in seismic zones 1 through 4 as defined in ER 1110-2-1806 should be seismically evaluated or designed with regard to strength, stability, and serviceability. New intake structures located in seismic zone 0 need not be seismically analyzed unless local site conditions indicate possible response to a nearby seismic event. The criteria set forth in this chapter apply to the structural analysis for earthquake loading of outlet works in accordance with ER 1110-2-1806. The approach accounts for the dynamic characteristics of the tower-water-foundation system, and includes the interaction of the system with the ground motion, gravity forces, and appurtenant structures. The seismic analyses should proceed progressively to produce cost-effective, efficient, and reliable structural configurations and seismic details.

b. Coordination of seismic analysis and design. The design team should normally consist of a structural engineer, a materials engineer, a geotechnical engineer, and an engineering geologist. Such a team should ensure that the seismic analyses, designs, or evaluations are appropriate for the project, and include all aspects of material properties, foundation properties, analysis procedures, and design criteria. The design should be based on historical data, appropriate testing, and engineering judgment. It is essential that the structural engineer be involved in all phases of the material and foundation property assessments, the site seismicity assessment, and the assignment of design ground motions. The basic concepts involved in the seismicity studies must be understood by the structural engineer because he or she must use the results of such studies and therefore must ensure that the information provided is meaningful for structural design or retrofit. In an assessment of the proper level of input, establishing either the level of confidence or the upper and lower bounds will define the sensitive parameters and bracket the results. Such steps should help to ensure that a logical and reasonably conservative design/evaluation of the intake tower could be performed. Assessments should not be based on unnecessary compounding of conservatism because this will lead to an uneconomical design or unnecessary retrofit.

c. Structural stability analysis.

(1) Sliding stability. Preliminary seismic sliding stability can be estimated by the seismic coefficient method. A seismic coefficient equal to two-thirds the peak ground acceleration (PGA) should be used. Shear strength parameters required to perform the sliding analysis should be provided by the engineering geologist or geotechnical engineer. If appropriate testing has not been conducted, an estimated range of values should be provided. The following are minimum required safety factors for seismic sliding analysis:

Operating Basis Earthquake (OBE) = 1.7 for critical structures, and 1.3 for other structures
Maximum Design Earthquake (MDE) = 1.3 for critical structures, and 1.1 for other structures

The computer program CSLIDE may be used to perform the sliding analysis. When rigid body sliding analysis indicates a factor of safety is less than required, a better estimate of safety factor can be made using dynamic analysis or a permanent displacement analysis, if necessary. In the permanent displacement approach the structure is permitted to slide along its base and the accumulated displacement during the ground shaking should be limited to specified allowable values. The analysis method is described in paragraph 2-9*a*(2) of EM 1110-2-6050. The computer program CSLIP may be used.

(2) Rotational stability. For OBE loading conditions, the location of all forces acting on the base of the structure must be such that 75 percent of the base of the structure is in compression. For MDE loading conditions, the resultant location must be within the base. If the resultant location is outside the base, a

dynamic rotational stability analysis should be performed. Rocking occurs when the foundation does not liquefy and

$$S_A > g(b/h) \tag{4-1}$$

where

S_A = spectral acceleration of the first model

g = gravitational acceleration

b = one half of the base width

h = vertical distance from the base to the center of gravity

If rocking occurs, the tower may not be rotationally unstable during the OBE or MDE. Scaling effects, as discussed in Appendix E, show that if S_A is just sufficient to rotate the block through a critical angle α_{cr}, the block will not overturn unless the spectral displacement S_d exceeds one half of the base width. So if a tall slender block is given an initial velocity S_v, which is just sufficient to rock the block to the point of impending static instability, and for small angles (less than 20 degrees).

$$\alpha_{cr} = \frac{S_v}{\sqrt{gr}} \tag{4-2a}$$

and

$$S_d < b \tag{4-2b}$$

where

α_{cr} = the critical angle of rotation from the initial upright position of static equilibrium to the final tipping position of impending rotational instability, radians

S_v = the response spectral velocity for the OBE or MDE ground motion, cm per second

g = gravitational acceleration

r = radial distance from the edge of the base to the center of gravity of the tower, cm

S_d = the response spectral displacement for the OBE or MDE ground motion, cm

The example of a rotational stability of an intake tower is given in Appendix E.

(3) Bearing pressure. Bearing capacity is analyzed according to paragraph 2-6 of EM 1110-2-2200.

4-2. Seismic Evaluation and Design of Tunnels and Cut-and-Cover Conduits

Seismic evaluations of underground structures, where complex structure-foundation interaction is involved, are often based on the performance of similar structures during major earthquakes rather than on the results of numerical analysis. Underground structures are generally less sensitive to seismic effects than surface structures. Information about the performance of underground structures is relatively scarce, largely because of the rarity of failures of underground structures during earthquakes. Underground structures, unless very rigid, deform to accommodate earthquake ground displacements due to traveling seismic waves, ground settlement, lateral spreading, liquefaction-induced deformations, and fault movements. Inertial forces due to ground shaking, as well as any deformational effects due to traveling seismic waves, generally are not significant for underground structures. The ground displacement effects due to lateral spreading of an embankment can be significant, but the effect on cut-and-cover conduits can be difficult to predict. Conduits for outlet works must be subjected to significant ground distortions or permanent displacements due to lateral spreading of the dam embankment or crossing an active fault before they would be expected to experience damage that would lead to failure. A concrete shell will be subjected to compression and extension at points on the exterior and interior of the lining. The exterior extension is of no consequence. In the event that tension cracks appear on the interior surface, they will close again after a fraction of a second. Such cracks do not usually extend through the thickness of the concrete and cannot, in themselves, form a failure mechanism. Information on seismic effects on tunnels can be found in EM 1110-2-2901 along with a simplified method for analyzing tunnels in rock for seismic effects.

4-3. Seismic Evaluation or Design of Intake Structure Bridge

Bridges to intake towers will interact with the tower during an earthquake. The type of interaction will depend on bearing restraint conditions. Most restraint conditions cause a nonlinear response between the bridge and the tower. This nonlinear response is difficult to incorporate into a model that includes both the tower and bridge. Guidance used for modeling the response between bridge superstructures and their supports is provided in Federal Highway Administration (1995). This information is also applicable to modeling bridge-tower interaction. An alternate approach is to assign the bridge mass to the tower, piers, and abutment based on a tributary mass approach. In most cases this approach provides reasonable results since the stiffness of the bridge system usually has little influence on tower behavior. Additional guidance on the seismic analysis methods for bridges and their supports is provided in American Association of State Highway and Transportation Officials (1989).

4-4. Seismic Evaluation or Design of Mechanical and Electrical Equipment

Mechanical and electrical equipment should be seismically protected in accordance with the provision of Tri-Service manual TI 809-04 "Seismic Design for Buildings."

4.5. Seismic Evaluation or Design of Basin Walls (BW)

a. Seismic performance. Case studies of damage to or failure of basin walls induced by ground motions during earthquakes clearly show the need for including appropriate provisions in the design and detailing of walls located in zones of moderate and high seismic activity. Damage to channel walls in the 1971 San Fernando earthquake is described in Clough and Fraguszy (1977). Damage was typically associated with backfill settlement and inward tilting of the walls toward the channel with the center of rotation at the connection between the wall and the channel slab. A few monoliths collapsed into the channel, but most damaged monoliths moved only enough to yield the reinforcement and severely crack the concrete. Severe damage to the walls or a basin induced by lateral backfill pressures may impair capability to release water from the reservoir, and therefore basin walls should be considered a vital feature in the overall seismic design process for outlet works and embankment dams.

b. Progressive analysis. Design of basins depends on the site characteristics, foundation conditions, basin width, and discharge capacity of the outlet works. Basin walls may be a gravity or cantilever structure that may be laterally restrained by the basin slab, or a U-frame structure supported on sound rock, highly overconsolidated soil, or piles. Considering the number of potential design parameters and the complex dynamic nature of the backfill-wall-foundation interaction during earthquakes, it is clear that the seismic design or evaluation of basin walls necessitates many simplifying assumptions. The investigation usually begins with a stability analysis using seismic coefficients and a simplified wedge model. When a rigid body sliding analysis indicates the safety factor is less than required, a permanent displacement analysis should be performed as described in paragraph 4-1*d*(1). In special cases, a dynamic soil-structure interaction (SSI) analysis of the basin walls may be performed using a finite element model. Deciding to proceed to the next level of investigation depends on if the seismic condition is controlling the design or evaluation, and if the relative cost of obtaining information from a more realistic analysis is a better investment than building reserve capacity into the wall system.

c. Basin walls with backfill. The time-history response of walls with backfill is complicated because in addition to the interaction with the foundation rock and water, the wall is also subjected to the dynamic soil pressures induced by ground shaking. Dynamic backfill pressures are related to the relative movement between the soil and the basin wall and the stiffness of the backfill. Behavior of the basin walls may be controlled by a rocking and/or translational response to earthquake shaking. The type of response will correlate to different distributions of backfill pressure acting against the wall. The appropriate method for analyzing the backfill pressure may be categorized according to the expected movement of the backfill and wall during seismic events.

(1) Backfill yields. The relative motion of the wall and backfill material may be sufficiently large to induce a limit or failure state in the soil. This condition may be modeled by the Mononobe-Okabe method (Mononobe and Matuo 1929; Okabe 1924), in which a wedge of soil bounded by the wall and an assumed failure plane are considered to move as a rigid body with the same ground acceleration. The dynamic soil pressures using this approach are described in Chapter 3, Section IV, paragraph 3-26*b* of EM 1110-2-2502.

(2) Backfill does not yield. For sufficiently low intensity ground motions the backfill material may respond within the range of linear elastic deformations. Under this condition the shear strength of the soil is mobilized at a low level so there are no nonlinear deformations in the backfill. The dynamic soil pressures and associated forces in the backfill may be analyzed as an elastic response using Wood's method as described in Ebeling and Morrison (1992).

(3) Backfill partially yields. The intermediate condition in which the backfill soil undergoes limited nonlinear deformations corresponds to the shear strength of the soil being partially mobilized. The dynamic backfill pressures may be idealized as a semi-infinite uniform soil layer using a constant-parameter, single-degree-of-freedom (SDOF) model (Veletsos and Younan 1994) or a frequency-independent, lumped-parameter, multiple-degree-of-freedom (MDOF) system (Wolf 1995). The dynamic pressures for an irregular backfill may be analyzed using a SSI model such as FLUSH (Lysmer et al. 1975). The wall is usually modeled with two-dimensional (2-D) elements. The foundation rock is represented by 2-D plane-strain elements with an appropriate modulus, Poisson's ratio, and unit weight. Transmitting boundaries in the form of dashpots are introduced at the sides of the foundation rock to account for the material nonlinear behavior with depth. The shear modulus and soil damping vary with the level of shearing strain, and this nonlinear behavior is usually approximated by an equivalent linear method. The boundary conditions for the backfill may also be represented by dashpots. Hydrodynamic pressures exerted on the wall are computed using the Westergaard formula.

d. Simplified wedge method. A seismic coefficient method may be used to estimate the backfill and wall inertial forces as described in Chapter 3, Section IV, paragraph 3-26*c* of EM 1110-2-2502. Theoretically, a wall may behave as a rigid body that is fully constrained along its base and sides by the ground, so all parts of the wall may be uniformly affected by accelerations, which are identical to the time-history of the ground motions. Therefore it would be appropriate to use a seismic coefficient equal to the peak ground acceleration for stability analysis of short, stiff walls. However, field and test data show that most walls do not behave as a rigid body, but respond as a deformable body subjected to effective ground motions. Thus the magnitude of the accelerations in a deformable wall may be different from those at the ground surface, depending on the natural period and damping characteristics of the wall and the shaking characteristics of the ground motions. Furthermore the maximum acceleration will affect the wall only for a short interval of time, and the inertia forces will not be equivalent to those of an equal static force that would act for an unlimited time, so the deformations resulting from the maximum acceleration will be smaller. Design or evaluation of basin walls for zero relative displacement under peak ground accelerations is unrealistic, so the seismic stability analysis should be based on a seismic coefficient that recognizes that an acceptably small amount of lateral displacement will likely occur during a major earthquake. Experience has shown that a seismic coefficient equal to two-thirds of the peak ground acceleration is a reasonable estimate for basin walls. For partially yielding backfill, the strength mobilization factor should be equal to the reciprocal of the minimum required sliding safety factor for that load case.

4-6. Seismic Evaluation or Design of Intake Towers

a. Critical classifications for intake towers. According to ER 1110-2-1806, critical features of civil works projects are the engineering structures, natural site conditions, or operating equipment and utilities at high hazard projects whose failure during or immediately following an earthquake could result in loss of life. A critical intake tower is as described based on its capability to lower the reservoir. Damage to or failure of an intake tower located at a high-hazard project may result in a reduced ability to lower the pool following an earthquake. Lowering of the pool may be necessary to relieve pressure head on an embankment dam possibly damaged by earthquake ground motions, or to inspect and repair an embankment dam. In cases where the loss of capacity to lower the pool will result in downstream fatalities, the tower is a critical project feature. If these conditions do not jeopardize lives, the tower is not critical.

b. Design earthquakes and performance requirements. There are two design earthquakes used in accordance with ER 1110-2-1806:

(1) Operational basis earthquake (OBE). The OBE is the level of ground motion for which the structure is expected to remain functional with little or no damage. The OBE is defined as a ground motion having a 50 percent probability of exceedance during the service life of 100 years (a 144-year return period). The associated performance level is the requirement that the structure will function within the elastic range with little or no damage and without interruption of function. In a site-specific study the OBE is determined by a probabilistic seismic hazard analysis (PSHA). The PSHA is described in EM 1110-2-6050.

(2) Maximum design earthquake (MDE). The MDE is the maximum level of ground motion for which the structure is designed or evaluated. The tower may be damaged but retains its integrity. The earthquake performance evaluation of an intake tower for the MDE is based on linear elastic analyses. The evaluation may require postelastic analyses with considerable judgment and interpretation of the results and should be done in consultation with CECW-E.

(a) For critical structures the MDE is set equal to the maximum credible earthquake (MCE). The MCE is defined as the largest earthquake that can reasonably be generated by a specific source on the basis of seismological and geological evidence (ER 1110-2-1806). A site-specific MCE is determined by a deterministic seismic hazard analysis (DSHA). The conditions defining an intake tower as critical are described in *a* above.

(b) For other than critical structures the MDE is selected as a lesser earthquake than the MCE, which provides for an economical design meeting specified safety standards. Because the purpose of the MDE is to protect against economic losses from damage or loss of service, alternative choices of return period for the MDE may be made on the basis of economic considerations. Ordinarily, the MDE is defined for intake towers as a ground motion having a 10 percent probability of exceedance during the service life of 100 years. This results in a return period of about 1,000 years. For structures a site-specific MDE is determined by a PSHA.

(3) Damping. Damping is a naturally occurring dissipation of energy within the structure. Typical causes of damping are internal friction and hysteretic material behavior. Although damping is mainly a material and structural system-related phenomenon, it is generally incorporated within the response spectra used for analyses. Five percent of critical damping is commonly used for the analysis of intake towers for the MDE and OBE.

(4) Effective stiffness. Postyield response is appropriate when performing seismic design or evaluation for the MDE. Based on Dove (1998), during postyield response the tower will crack at the base and will form a plastic hinge, which will decrease stiffness. The reduced moment of inertia is designated as the effective moment of inertia I_E and is a function of the ratio of the nominal moment capacity M_N to the cracking moment M_{CR}. For intake structures, base the effective moment of inertia on the following relationship:

$$\frac{I_E}{I_g} = 0.8 - 0.9\left[\frac{M_N}{M_{CR}} - 1\right] \tag{4-3}$$

The ratio of I_E/I_G where I_G is the gross moment of inertia (uncracked) should not be greater than 0.8, nor less than 0.35 for walls reinforced with 40 grade steel, nor less than 0.25 for walls reinforced with 60 grade steel. For the OBE the tower should be uncracked at the base; therefore, the effective stiffness is equal to the gross stiffness.

(5) Performance requirements. The performance requirements for intake structures subjected to MDE and OBE demands are provided in *e* below along with the strength requirements for both events.

(6) Earthquake directional components. Earthquake ground motion can be defined for three individual components: a principal horizontal component, a second horizontal component perpendicular to the principal horizontal component, and a vertical component. The three earthquake components should be statistically independent; therefore, the maximum structural responses due to each component will not occur at the same time. It is the designer's responsibility to coordinate with geotechnical engineers on the ground motion components required as described in EM 1110-2-6050. Methods for combining the structural responses from the two horizontal components are described in *e*(1)(d) and (e) below.

c. General design and analysis. The load conditions consist of the usual loads coupled with loads due to a horizontal component of earthquake ground motion. The vertical component of the ground motion has little influence on the overall response of a free-standing tower. Usual loads include the dead weight of the tower, hydrostatic loads due to normal pools, routine operating loads, normal debris loads, and sediment loads. The effects of the design earthquake event should be combined only with other loads that are most likely to occur coincident with normal pool conditions.

(1) Loads. The seismic analysis should account for the dynamic characteristics of the tower-water-foundation system by using the simplified two-mode, lumped-mass method, or by using finite element models. The two-mode, lumped-mass method was developed only for the preliminary design or evaluation of vertical intake structures that are surrounded by water and is not suitable for inclined towers. Finite element models should be used for the final design or evaluation of towers in seismic zones 3 and 4, or in zones 2A or 2B if the seismic load case controls the preliminary design or evaluation.

(2) Analysis. For a vertical tower, the structure may be modeled as a cantilever lumped-mass system of beam elements, and the inertial effects due to the ground motion will induce bending and shear forces in the tower. The earthquake load is modeled by a design response spectrum, or by multiple time-histories for more refined analyses. The response spectrum modal analysis provides only the absolute maximum values of forces, shears, moments, and displacements. The analysis methods and procedures, advantages and disadvantages of response spectrum modal analysis, and procedure for combining modal values are

described in Appendix B. The two-mode approximation and computer solution methods of analysis for a free-standing intake tower is described in Appendix C. A refined method of computing hydrodynamic added mass is described in Appendix D.

d. Torsional effects.

(1) General. Because of the random nature of earthquake ground motions, the direction of seismic loading must be considered. One important consequence of the direction of loading is the potential introduction of torsion. Two categories of towers, regular and irregular, are used to distinguish those towers for which torsional effects are likely to be significant. In deciding whether torsion is significant, the structural engineer should consider all factors such as access bridge effects and local irregularities in the mass or stiffness on a case-by-case basis. A designer should take all reasonable steps to provide symmetry and uniformity without detracting from the functions required for hydraulic purposes.

(2) Regular towers. Regular towers are less subject to damage than irregular towers. Regular towers have a symmetric distribution of mass and stiffness about their principal axes and along the full height. Towers are symmetric in plan if the center of mass is relatively close to the center of rigidity. If these two points are within 10 percent of the tower width when measured perpendicular to the earthquake motion, then the tower is basically symmetric. Towers that are basically symmetrical in plan throughout their height can be designed or evaluated for the bending moments and shears from two-dimensional beam model response spectra or time-history analyses. Earthquakes generate ground motions that occur simultaneously in all directions. For regular towers, the directional uncertainty of the ground motions can be evaluated by performing separate seismic analyses for each principal direction and combining the results as described in *e*(1)(d) below.

(3) Irregular towers. Towers that do not meet the criteria of the preceding paragraph are irregular and should be designed or evaluated for torsion. There are two different approaches for including torsion. The first method uses two-dimensional cantilever beam analyses as performed for the regular towers. On this analysis a separate calculation of torsional effects is superimposed. The torsional moment acting on a cross section is calculated as the sum of all incremental torsional moments above the cross section. An incremental torsional moment is obtained by multiplying the lateral inertial force for a given vertical portion of the tower by its eccentricity from the center of rigidity. The eccentricity is the perpendicular distance from the line of action of the inertial force to the center of rigidity. The resulting summation of the incremental torsional moments above the cross section must then be distributed to the shear-resisting elements according to their relative stiffness and superimposed with the analysis results. This method is common to building evaluations where each wall or lateral force-resisting element of the structure is required to carry both direct shear and torsional shear in accordance with its relative stiffness. A second method for computing torsional effects is to use a three-dimensional structural model to represent the mass and stiffness of the structural system. Various directions of the earthquakes can be investigated to determine the directions that govern the design. In this approach the torsional effects are automatically included in the analysis.

e. Strength and service requirements.

(1) Seismic load cases and combinations.

(a) General. Seismic design for new towers and the evaluation of existing towers must demonstrate that the tower has adequate strength, ductility, and stability to resist the specified earthquake ground motions. Flexural ductility and the benefits of energy dissipation beyond yield should be considered in the design and evaluation process for the MDE, if it can be demonstrated that the tower is not vulnerable to brittle modes of failure. Flexural ductility is indirectly accounted for in the strength equations through the use of a moment reduction factor. Intake structures should have adequate capacity (strength) to resist the MDE demand by allowing for some energy dissipation through inelastic rotation. Intake structures must also have adequate capacity to perform elastically under OBE loading conditions. An example of seismic analysis for the determination of structural capacity is provided in Appendix C.

(b) Limitations. The provisions described herein for the seismic design and evaluation of intake towers are limited to free-standing intake towers with height-to-width aspect ratios of two or more.

(c) Strength requirements. The ultimate strength U or capacity of new and existing towers will be determined using the principles and procedures described in EM 1110-2-2104. Capacities are based on ultimate strength, or the nominal strength multiplied by a capacity reduction factor. The capacity reduction factor is 0.9 for bending and 0.85 for shear. Intake tower sections shall have the strength to resist load combinations involving dead load, live load, and earthquake load as prescribed by the following load factor equation:

For the MDE

$$U = D + L + 1.1E / R_M \tag{4-4}$$

For the OBE

$$U = 1.4\,(D + L) + 1.5E \tag{4-5}$$

where:

U = value of thrusts, shears, or moments due to the effects of dead load, live load, and earthquake

D = internal forces from self-weight

L = internal forces from live loads

E = internal forces from the MDE or OBE. Two orthogonal components must be considered (see the following paragraph).

R_M = moment reduction factor (use $R_M = 1$ for shear and thrust, $R_M = 2$ for moment)

(d) Multicomponent earthquake responses. Intake towers must be capable of resisting maximum earthquake ground motions occurring in any direction. The inertial forces obtained from an analysis generally represent the effects of the principal ground motion component. For a moment, thrust, shear, or force at a particular location, the direction of the earthquake components causing the maximum value needs to be determined. Since an investigation of all possible earthquake directions is difficult, the following alternative methods for estimating peak values should be used.

(e) Circular or rectangular towers. For circular ($\alpha = 0.4$) or rectangular ($\alpha = 0.3$) towers, the orthogonal combination method should be used to account for the directional uncertainty of the MDE or OBE earthquake motions and the simultaneous occurrences of earthquake forces in two perpendicular horizontal directions. This is accomplished by considering the two following combinations:

$$E = \pm[E_X + \propto E_Y] \tag{4-6}$$

$$E = \pm[\propto E_X + E_Y] \tag{4-7}$$

where:

E = peak positive or negative values of the forces, shears, moments, or thrust due to the alternating directional effects of the earthquake

E_X = effects resulting from the X-component of ground motion occurring in the direction of the major principal tower axis

E_Y = effects resulting from the Y-component of ground motion occurring in the direction of the minor principal tower axis.

(f) Unsymmetrical or inclined towers. The square root of sum of square (SRSS) method should be used for a preliminary evaluation of the multidirectional earthquake effects on towers that are irregular in plan and elevation. The force, shear, moment, or thrust at a particular location can be estimated from any set of orthogonal analyses by a SRSS combination.

$$E = \sqrt{E_X^2 + E_Y^2} \tag{4-8}$$

(2) Ductility.

(a) Inelastic behavior. Ductility is an indicator of inelastic deformation. Research performed by Dove (1998) shows that a reinforced concrete tower should have a displacement at failure that is equal to 150 to 350 percent of the displacement at yield. Based on a corresponding bending displacement ductility of 1.5 to 3.5, a tower should be designed or evaluated for seismic moments that are 70 to 40 percent, respectively, of the elastic moments obtained from a response spectrum or time-history analysis. Some load transfer mechanisms in reinforced concrete are brittle, and the strength may degrade after repeated cycles of loading beyond the yield level. The brittle failure modes are due to fracture of the reinforcement, an anchorage failure of the reinforcement, a splice failure (bond) of the reinforcement, a shear (diagonal tension) failure, a compressive spalling failure, and a sliding shear failure.

(b) Moment reduction factor (R_M). The moment reduction factor is the ratio of the elastic response moment to the applied seismic moment. A moment reduction factor of two should be used only when brittle failure modes cannot occur, and the structural engineer should design and detail the structure so that all the brittle failure modes are precluded. Properly detailing the reinforcement is important to assure adequate performance and to preserve the structural integrity of the tower during a seismic event. Adequate detailing of reinforcement should be provided in accordance with the provisions in (3) to (8) below. An example is in Appendix C.

(c) Confinement requirements. Some towers may require special confinement reinforcement and boundary elements. A moment reduction factor greater than two should be used only if appropriate measures are taken to confine the concrete in regions subjected to large compressive strains, and to improve the cyclic performance of splices and anchorages by providing heavy confinement reinforcement in plastic hinge regions. Plastic hinges or inelastic zones occur in regions with large moments. The confining reinforcement keeps the concrete from spalling and allows the tower to redistribute loads after cracking. Typical locations occur at the base of conduits and other openings, or at discontinuities due to abrupt changes in structural geometry. Corners should have diagonal reinforcement to control anticipated cracks. Evaluation of postelastic and moment reduction factors greater than two should be performed in consultation with and approved by CECW-EW.

(3) Fracture of reinforcement requirements.

(a) General. The tensile reinforcement can fracture suddenly if the reinforcement is less than 1 percent and cause a brittle failure. To prevent a brittle failure mechanism, the nominal moment capacity should equal or exceed the uncracked moment capacity by at least 20 percent. The cracking moment can be calculated using Equation 4-9:

$$M_{cr} = \left(\frac{I_g}{C}\right)\left(\frac{P}{A_G} + f_r\right) \tag{4-9}$$

where

C = the distance from the neutral axis to the extreme fiber

P = axial load on the tower (positive for tension)

A_g = gross section area (uncracked)

f_r = modulus of rupture, $0.62\sqrt{f_c'}$ (MPa units), 7.5 $\sqrt{f_c'}$ (psi units) where f_c' is the concrete compression strength

(b) Existing towers. A force-based or a displacement-based approach may be used to evaluate the minimum reinforcement requirement if the nominal moment capacity is less than 120 percent of the cracking moment.

(c) New towers. For all seismic designs, sufficient reinforcing steel should be provided to assure that the nominal moment capacity equals or exceeds 120 percent of the cracking moment (Equation 4-10):

$$M_N \geq 1.2 M_{cr} \tag{4-10}$$

(4) Anchorage failure mode.

(a) Existing towers. The flexural strength of the intake tower will deteriorate during a major earthquake if the vertical reinforcement is not adequately anchored. For straight bars, the anchorage lengths should be greater than required by Equation 4-11:

$$l_a = \frac{2.626 k_s d_b}{\sqrt{f_c'}\left(1 + 2.5\frac{c}{d_b}\right)} \quad \text{(KPa units)} \tag{4-11}$$

$$l_a = \frac{k_s d_b}{\sqrt{f_c'}\left(1 + 2.5\frac{2}{d_b}\right)} \quad \text{(psi units)}$$

where

l_a = minimum required effective anchorage length of longitudinal reinforcement

k_s = a constant for reinforcing steel with a yield stress of f_y

$$k_s = \frac{\left(f_y - 76845\right)}{33.1} \qquad \text{(KPa units)}$$

$$k_s = \frac{\left(f_y \text{ -} 11000\right)}{4.8} \qquad \text{(psi units)}$$

f_y = yield stress of the longitudinal reinforcement

d_b = nominal bar diameter

c = the lesser of the clear cover over the bars, or half the clear spacing between adjacent bars, in.

The term 2.5(c/d_b) introduces the effect of clear cover to the calculation of development length. The value of (c/d_b) shall not be greater than 2.5. For 90-degree standard hooks, the anchorage provided should be greater than that required by Equation 4-12. For anchorage with 90-degree standard hooks, the effective anchorage length in inches is

$$l_a = 1,200 d_b \frac{2.626 f_y}{60,000\sqrt{f_c'}} \qquad \text{(KPa units)}$$

$$l_a = 1,200 d_b \frac{f_y}{60,000\sqrt{f_c'}} \qquad \text{(psi units)} \qquad (4\text{-}12)$$

(b) New towers. The required effective anchorage length of longitudinal reinforcement should be equal to or greater than that provided by Equations 4-11 and 4-12. The minimum anchorage length, however, should not be less than 30 bar diameters for straight anchorage or less than 15 bar diameters for hooked anchorages.

(5) Splice failure of bending reinforcement.

(a) Existing towers. Splices in the flexural steel, if located in a plastic hinge region, may undergo strength deterioration during a major earthquake. When compressive strains in the concrete are less than 0.002 in./in., the splices will perform satisfactorily without transverse confinement steel. When concrete compressive strains exceed 0.002 in./in., transverse confinement steel should be provided at splice locations. The minimum area of transverse reinforcement A_{tr} necessary to prevent bond deterioration is:

$$A_{tr} = \frac{s f_y}{l_s f_{yt}} A_b \qquad (4\text{-}13)$$

where

A_{tr} = minimum area of transverse reinforcement

s = average spacing of transverse reinforcement over the splice length

l_s = splice length

f_{yt} = yield stress of the transverse reinforcement

A_b = area of the spliced bar

(b) New towers. Deterioration of bond and splice strengths of reinforcing bars is a critical potential failure mode for reinforced concrete structures. Transverse reinforcement provides the best protection against splice strength degradation. Perimeter transverse reinforcing steel in the amount equal to or greater than that indicated by Equation 4-13 should be provided at all splice locations where concrete compressive strains are expected to exceed 0.002 in./in. Transverse steel provided to resist shear and bending forces induced by hydrostatic or gravity loads may also be considered as effective transverse confinement steel and used to prevent splice failures. Perimeter transverse confinement reinforcement using smaller bars at close spacings is better than that obtained using larger bars at wide spacing.

(c) Splice length requirements. Splice performance will be greatly improved if splices are located away from potential plastic hinge regions and if lap splice locations are staggered (i.e., no more than half of the bars spliced at any horizontal plane). The lap splice length required for new and existing towers should not be less than 154 $d_b \div \sqrt{f'_c}$ MPa units (1,860 $d_b \div \sqrt{f'_c}$ psi units). Existing towers with lesser splice lengths may perform acceptably if splices are staggered. Lesser splice lengths should be evaluated according to their location and seismic demand.

(6) Shear (diagonal tension) failure. For existing and new towers, the capacity of the intake tower in shear shall be equal to or greater than the lesser of the full elastic demand placed on the tower by the design earthquake, or the shear corresponding to 1.5 times the shear associated with the nominal flexural strength of the tower. The capacity of the concrete in shear and the shear resistance available from the transverse reinforcing after deductions for other live load effects may be considered. The shear capacity of the tower includes contributions from both the concrete and the shear reinforcement. The total ultimate shear strength V_U is

$$V_U = \theta\,(V_C + V_S) = 0.85\,(V_C + V_S) \qquad (4\text{-}14)$$

where V_C is the contribution from the concrete:

$$V_c = 2\left[K + \frac{P}{13.8A_g}\right]0.083\sqrt{f'_{CA} \cdot A_E} \qquad \text{(KPa units)}$$

(4-15)

$$V_c = 2\left[K + \frac{P}{2{,}000A_g}\right]\sqrt{f'_{CA} \cdot A_E} \qquad \text{(psi units)}$$

where

$K = 1$ for $R_M = 1$; $K = 0.5$ for $R_M = 2$

P = axial load on the section

f'_{CA} = actual concrete compression strength (typically $f'_{CA} \geq 1.5 f'_C$)

$A_E = 0.8\ A_{gross}$ (A_{gross} = actual cross-section area)

and V_S is the contribution from the shear reinforcement as defined for circular towers as equal to:

(a) For circular towers:

$$V_s = \frac{\pi A_h (f_y)(0.8d)}{2s} \tag{4-16}$$

where

A_h = horizontal reinforcement cross-section area

d = outside diameter of the tower

f_y = stress in the transverse reinforcing due to unfactored hydrostatic loading and other non-earthquake loads

s = spacing of reinforcement

(b) For rectangular towers,

$$V_s = \frac{A_h (f_y)(0.8d)}{s} \tag{4-17}$$

where d is the tower dimension in the direction of the seismic shear force. Since this method uses an interaction equation to evaluate biaxial shear (see Appendix C), the earthquake forces must be in the directions of the principal axes.

(7) Compressive spalling failure. In existing and new towers, excessive compressive strains can cause spalling of the concrete cover and rapid degradation of the transverse confining reinforcement, which can lead to splice failures and buckling of the longitudinal reinforcing steel. However, the chance of compressive spalling failures is small because the compressive stresses and the reinforcing steel percentages are low in intake towers. Compressive spalling failures will not occur at ultimate load conditions if the concrete compressive strains are less than 0.4 percent, or if the location of the neutral axis is less than 15 percent of the effective depth to the centroid of the reinforcement. The latter condition can be expressed as

$$\frac{c}{d} \leq 0.15 \tag{4-18}$$

where

c = distance from extreme compression fiber to the neutral axis, in.

d = distance from extreme compression fiber to the centroid of tension reinforcement, in.

(8) Sliding shear failure investigation. In existing and new towers, the potential for sliding along a horizontal crack at all possible failure planes within the structure should be evaluated. The nominal sliding

shear strength can be used to determine if a sliding shear failure is possible. The nominal shear strength can be determined using Equation 4-19:

$$V_{SL} = P + 0.25 f_y A_{VF} \tag{4-19}$$

(Assuming coefficient of friction $\mu = 1$)

where

V_{SL} = nominal sliding shear strength

P = axial load on section

f_y = yield strength of reinforcement

A_{VF} = area of shear friction reinforcement (area of vertical reinforcement crossing the horizontal failure plane)

The sliding shear resistance in KN (kips) calculated by use of Equation 4-19 shall not be greater than

$$0.0008\sqrt{f_c'}\,A_g \text{ MPa} \quad (0.01\sqrt{f_c'}\,A_g \text{ psi})$$

where A_g is the gross area of concrete normal to the horizontal failure plane. It should be noted that sliding shear does not necessarily constitute a failure condition, since the permanent displacements that occur during sliding may not be sufficient to cause structural instability.

(9) Displacement-based analysis of existing rectangular and circular towers.

(a) Tower performance. Many existing intake towers are lightly reinforced structures (steel percentage less than 0.50 percent) that may have a nominal moment capacity less than 120 percent of the cracking moment. If the nominal moment capacity exceeds the cracking moment, then the tower will be able to form a plastic hinge at critical points of structural damage similar to the behavior of reinforced concrete bridge piers or building columns. Towers that do not meet this minimum requirement will have plastic hinge lengths that are much less than those in buildings or bridges, and these towers are more likely to fail by fracturing the longitudinal reinforcement. Recent analytical and experimental research that addressed the nonlinear seismic response and ductility of lightly reinforced towers has shown that many towers will have limited ductility corresponding to a highly localized failure of the reinforcement within a single crack (Dove 1998). The critical crack may form at the base of the tower or at other abrupt changes in structural stiffness. Typical rectangular or circular towers will have small plastic hinge lengths with a limited displacement capacity.

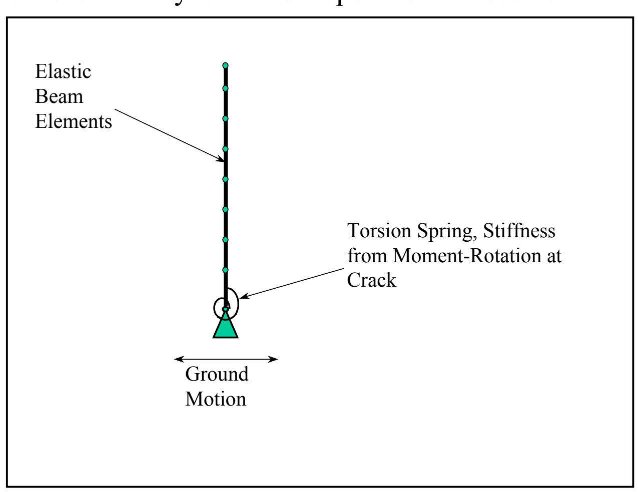


Figure 4-1. Tower model of displacement demand

(b) Displacement demand. A displacement-based dynamic analysis may be used to evaluate this localized failure mode, and to determine if the ultimate displacement capacity δ_U at the top of the tower (as shown in Figure 4-1) exceeds the displacement demand δ_D for the MDE. This technique is a modified response spectrum analysis that considers the earthquake-induced displacements of a tower, and uses effective stiffness properties (*b*(4) above) to account for the shift in the fundamental frequencies of the tower due to formation of plastic regions. The displacement demand is the maximum deflection calculated at the top of the tower from a response spectrum analysis using a linear spring stiffness, the beam element properties, and the added mass due to surrounding or contained water. The tower is modeled as a cantilever beam that is supported at the base on a rotational spring (Figure 4-1). The beam represents the elastic response of the tower above the crack, and the spring represents the nonlinear response of the cracked region. The nonlinear rotational stiffness of the spring may be estimated by using an appropriate reinforced concrete section analysis computer program to calculate the moment-curvature relationship for the cross-section of the tower, and then multiplying the curvature values by an appropriate plastic hinge length to plot the corresponding moment-rotation relationship. The moment-rotation relationship represents the stiffness of the rotational spring, and this relationship is often strongly bilinear. The following simplification is required to approximate the spring stiffness as a linear relationship in the elastic response spectrum analysis. First, calculate the area under the bilinear moment-rotation curve from the origin to the value of the maximum allowable rotation, and then generate a fictitious linear, moment-rotation curve that has the same area and maximum rotation. The fictitious spring stiffness should have the same total energy and maximum rotation as the bilinear curve, but the rotational stiffness and the frequency of spring response will lie between the real elastic and inelastic values.

(c) Displacement capacity. The displacement capacity at the top of the tower is related to the height, the length of the plastic hinge, and the fracture strain capacity (approximately 0.05 in./in.) of the reinforcement. The displacement capacity may be modeled as shown in Figure 4-2 and discussed below.

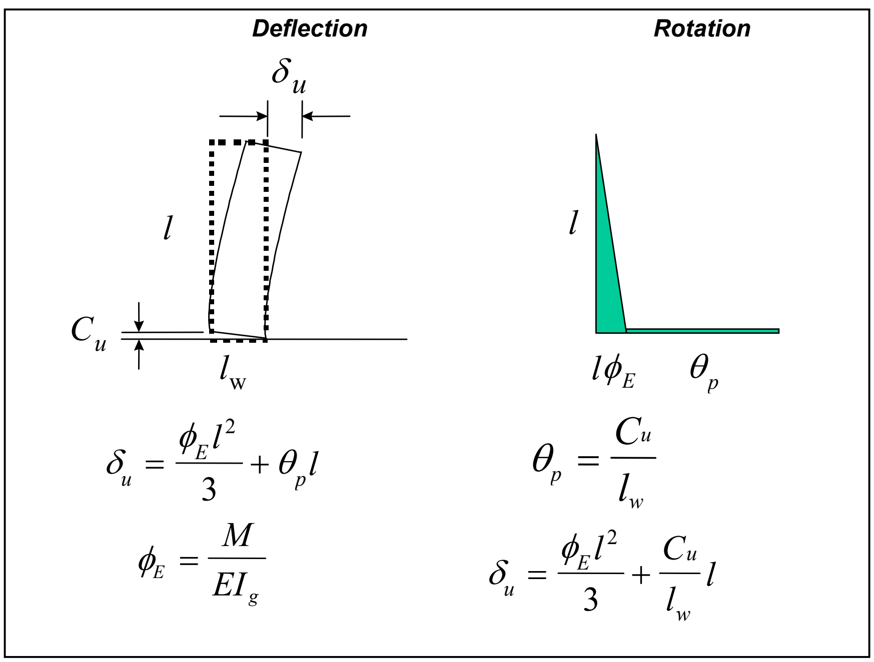


Figure 4-2. Tower model of displacement capacity

where

δ_u = the ultimate displacement capacity

ϕ_E = the elastic curvature at cracking (at the base of the tower)

l = the height of the tower above the crack

θ_p = the plastic rotation at failure

M = the moment at the base of the tower

C_u = the ultimate crack width at failure

l_w = the depth of the section

The ultimate lateral deflection is modeled as the sum of an elastic response of the tower above the cracked section and a rigid body rotation of the tower as the crack opens at the base. It is conservative to assume the tower rotates about a neutral axis of the cracked section that is coincident with the edge of the tower. Hence, the lateral rigid body deflection at the top of the tower varies directly with the crack width, and its maximum value varies as a ratio of the tower height to the tower width, times the ultimate crack width. The ultimate crack width at failure C_u is a function of the ultimate strain in the reinforcement and the strain penetration. Recent experiments have modeled the cyclic response of single bars for different concrete strengths, bar sizes, and bar strengths. Statistical analyses of these experiments indicate that for a single crack response, the crack widths are reliably predicted by the ultimate strain capacity of the bar and the bar diameter. An appropriate value of the ultimate crack width may be estimated using the following empirical equation:

$$C_u = 0.12 + 2.47\varepsilon_u + 0.312d_b \tag{4-20}$$

where

ε_u = ultimate strain at failure of the bar as measured over a standard 20.32-cm (8-in.) gage length

d_b = diameter of the reinforcing, cm

(d) Tower example. A displacement-based evaluation of the tower in Appendix C is based on a failure mechanism that may be characterized by a single crack at the base of the tower. The analysis is summarized below.

(1) Compute the moment-curvature relationship for the bottom section of the tower about the weak and strong axes, and include the deadweight of the tower. For an 18 percent ultimate strain, the ultimate crack width and the strain penetration length L_s may be estimated as 1.65 cm (0.65 in.) and approximately 10.0 cm (4.0 in.), respectively, from the previous empirical equation and the following equation:

$$L_s = C_u / \varepsilon_u \tag{4-21}$$

(2) Multiply the moment-curvature values by this strain penetration length to obtain the moment-rotation relationship (Figure 4-3). The values of the linear spring stiffness for bending about the weak and strong axes are estimated based on having equal total areas under the actual and fictitious moment-rotation curves. The rotational spring constants for the weak and strong axes are 2.107E+12 N-m/Radian and 3.561E+12 N-m/Radian, respectively.

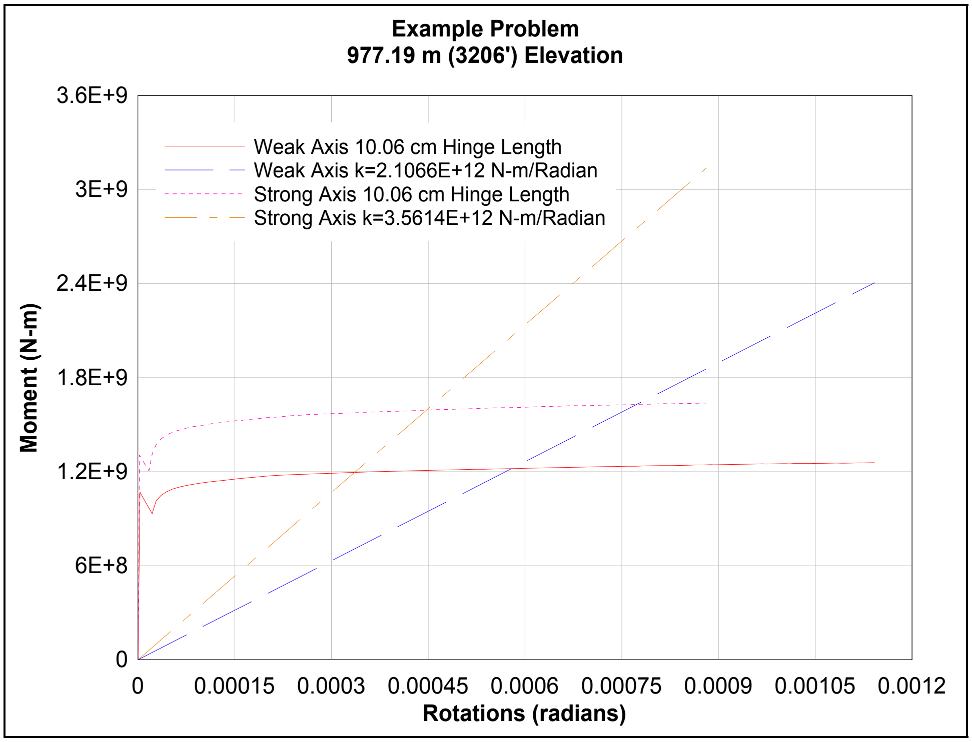


Figure 4-3. Moment-rotation relationship for the tower base

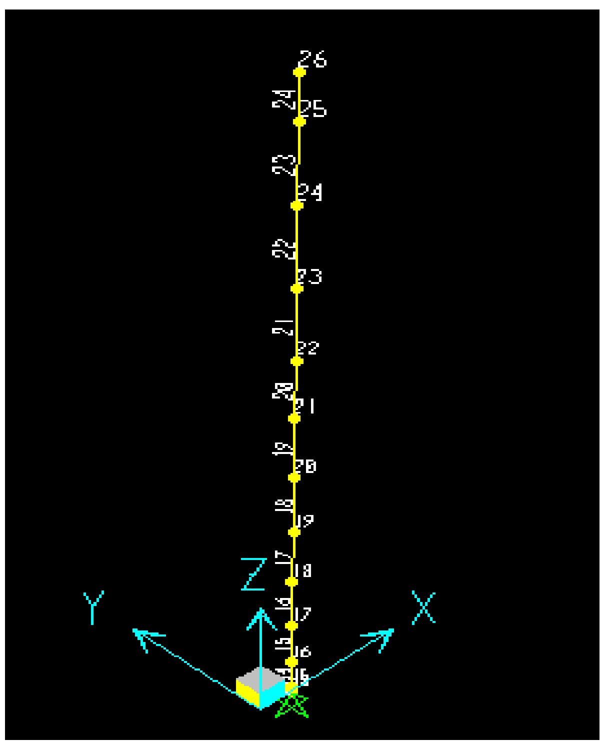


Figure 4-4. SAP 2000 intake tower model

(3) Perform a structural analysis for the MDE response spectrum with 5 percent damping (as shown in Figure C-3) using commercial software such as SAP 2000. This is similar to the analysis presented in Section C-8 except for the hinge supports and rotational springs provided at the base of the tower to model the properties of the cracked section (Figure 4-4). The same section properties and masses were used (as shown in Figures C-1 and C-5) to model the tower and water. The sectional properties of the tower were uniformly distributed, and the added masses $\left(m_a^o + m_a^i\right)$ were lumped at nodes only (as shown in Tables C-3 and C-4). The deflections calculated at the top of the tower are 9.6 cm (3.8 in.) and 10.1 cm (4.0 in.) for the strong and weak axes, respectively.

(4) The elastic curvature ϕ_E at the base is read from the moment-curvature relationship for the section. The ultimate base rotation and the ultimate displacement capacity are calculated using the following equations:

$$\theta_p = C_u / l_w \tag{4-22}$$

$$\delta_u = \frac{\phi_e l^2}{3} + \theta_p l \tag{4-23}$$

(5) The deflection capacities at the top of the tower are 10.7 cm (4.2 in.) and 14.2 cm (5.6 in.) about the strong and weak axes, respectively. Therefore the tower satisfies the performance requirements for the MDE.

4-7. Special Guidance for Inclined Intake Towers

a. Inclined and vertical towers supported along one side by the rock abutment will experience complicated bending, shear, and torsion under earthquake ground shaking. The ground motions excite the tower not only from the base but also along the entire abutment support. The three-dimensional geometry of supported towers combined with composite seismic input requires a three-dimensional finite element modeling of the tower for earthquake response analysis. Generally, supported towers should be discretized using a combination of three-dimensional solid and shell elements, as appropriate, or simply by three-dimensional solid elements. However, some components of the tower such as the trashrack can be modeled more appropriately using beam elements. The interaction with the foundation and abutment may be considered by including a portion of the foundation and abutment regions as part of the tower model. The foundation and abutment models should be developed to have dimensions comparable to those of the tower section. Competent rock sites may adequately be represented by a massless foundation model while foundation soil models are usually represented by equivalent springs, by a combination of springs and dashpots, or by finite element soil models. More detailed guidance on foundation-structure interaction is covered in EM 1110-2-6050 and EM 1110-2-6051.

b. The effect of hydrodynamic pressures on supported towers is represented by the equivalent added-mass concept. The added hydrodynamic mass, however, depends not only on the geometry of the tower but also on topography of the surrounding abutment-foundation region. If the effect of topography on the added mass is judged significant, the added mass should be computed using the finite element or boundary element procedures as discussed in EM 1110-2-6051.

c. The dynamic response of a supported tower is a complex combination of biaxial bending and torsion depending on the tower-abutment-foundation and tower-water interaction. This condition is best treated by three-dimensional response-spectrum (EM 1110-2-6050) and time-history analyses (EM 1110-2-6051). The analysis may start with the response-spectrum modal superposition followed by time-history procedure. The seismic input consists of three components of ground motions in the form of response spectra or acceleration time-histories applied at the fixed exterior nodes of the foundation model.

A response-spectrum analysis provides only maximum stresses and section forces. A time-history analysis provides both the maximum magnitudes and distributions of stresses and section forces at any time during a seismic event. In case of severe seismic events, both magnitudes and time variation of stresses and forces may be required to design or evaluate the structure and the anchorage system.

4-8. Remedial Strengthening of Existing Towers

The seismic analysis indicates an existing intake tower is in need of a seismic retrofit, or when nonlinear, ineleastic seismic analysis procedures are necessary to investigate performance of the structure. The need for remedial strengthening will be determined on a case-by-case basis after suitable ductility evaluation has been performed. Any evaluations of existing intake towers that indicate the need for remedial strengthening shall be done in consultations with CECW-EW.

Chapter 5
Gates, Valves, Bulkheads and Guides, Trashracks, and Operating Equipment

5-1. Functions and Requirements

Gates, valves, and bulkheads in intake structures serve a number of different functions related to the purposes of the project. Type, location, arrangement, layout, and design will depend on the various operational, maintenance and servicing, dam safety, and construction (if used for diversion) requirements. Further discussion and details on gates and valves are provided in *Advanced Dam Engineering for Design Construction and Rehabilitation* by Robert Jansen and EM 1110-2-1602, Chapter 3.

a. *Low-level drawdown intakes.* Low-level drawdown intakes are required on new projects to lower the reservoir below minimum pool elevation or to provide a dry reservoir behind the dam. The reasons for this capability may include provisions to draw down the reservoir for inspection after a seismic event, for periodic inspections and repair of features or structures normally submerged, to flush fine sediment accumulations in the reservoir, to provide flexibility for changes in reservoir regulation, and to provide no pool during low flows to meet environmental requirements. Whether these intakes are gated or bulkheaded will depend on economics, frequency of use, and the type of operation. The criteria for low-level drawdown are addressed in ER 1110-2-50.

b. *Control gates.* The control gates located in each gated passage to the outlet tunnel or conduit generally consist of an emergency gate followed by a service gate. Guides are provided just upstream of the gates for stoplogs or a bulkhead. The number of gates and passageways, size, types, and layout in relation to the structure will depend on the project purpose(s); the hydraulic design, operation, and flow regulation requirements; hydraulic head; and structural design, construction, and economic requirements. Hydraulic criteria for the design of gates are discussed in EM 1110-2-1602.

(1) Service gates. Service gates are used for flow regulation, and one gate is required for each water passage. For low- and medium-head conditions, the type of gate may be a tractor (mechanically or hydraulically operated), hydraulically operated slide gate, or hydraulically operated tainter gate. For high-head conditions, hydraulically operated slide gates are generally preferred for long periods of operation at partial gate openings.

(2) Emergency gates. Emergency gates should be located within the intake structure immediately upstream of the service gates. Emergency gates are required in reservoirs having water conservation, power, or a similar type pool to prevent loss of stored water if a service gate is inoperable (see EM 1110-2-1602, Chapter 3, Section III, Gate Passage Requirements for additional guidance on emergency gate requirements). Emergency gates must be designed to close under free flow at the maximum conservation or power pool. However, emergency gates are not required to regulate flow, as a service gate, because sufficient conservation or power releases can be made through the remaining facilities. Typically, emergency gates are of the same type as the service gate; however, for low- and medium-head conditions, a single transferable tractor gate installed by a traveling hoist at the operating deck near the top of the structure may be used. Emergency gates must be designed to withstand the maximum reservoir head at stresses not exceeding 33-1/3 percent of the basic stress.

c. *Water temperature and quality.* Projects providing temperature or water quality controls may have multi-level intake ports or telescoping weirs with a mixing wet well. The intakes may be gated or ungated. Flow regulation with control gates is required downstream of the wet well.

5-2. Gate Types

A wide variety of gates have been used in outlet works. The principal types of regulating gates on U.S. Army Corps of Engineers dams for heads of 100 ft or more are hydraulic slide gates, caterpillar gates with cable hoists, hydraulic caterpillar or wheel gates, and hydraulic tainter gates. The type selected depends on the purpose, operating characteristics, flow regulation requirements, maintenance and serviceability, and life-cycle cost.

a. Hydraulically operated slide gates. The hydraulically operated slide gate has a record of dependability and low maintenance and is capable of providing fine regulation. These gates have been designed for projects up to approximately a 500-ft head. Compared to the tractor or roller gates and tainter gates, hydraulically operated slide gates for high heads are smaller because of the much greater operating force per square foot of gate and the greater rigidity required to meet the operating requirements. The gate is housed in steel gate frames with attached conduit liners requiring close fabrication tolerances to provide a watertight unit. The assembly is embedded and anchored directly in the main concrete body. The seating or sealing surfaces of embedded gate frames must be accessible for inspection and maintenance. Cavitation problems have occurred in the gates' leaf bottom edges and the gate frame downstream of the gate under high-velocity flow conditions at small gate openings.

b. Roller- and wheel-mounted gates. Roller-mounted (tractor or caterpillar) gates have roller trains mounted on the gate parallel to the gate slot, and wheel-mounted gates use fixed wheels attached to the gate. Both gate types can be hydraulically operated or are suspended with cable hoists and hydraulically operated gates for low-flow regulation. The cable hoist gates are not as positive for low-flow regulation as other gates because of the elastic properties of the cables. These types of gates are used for heads up to about 200 ft. A gate frame and conduit liner are provided with the gate. The roller trains and wheels on the gates are vulnerable to seizing caused by corrosion and debris.

c. Tainter (radial) gates. Radial gates are simple, reliable, and generally less expensive than other gates of equal size. They are satisfactory for low-flow regulation. Because gate slots are not required, head loss is minimized. Top seals may limit heads to about 250 ft. Eccentric trunnion designs have been used to facilitate sealing. In some cases, vibration has resulted where bottom leaf seals and beams were not properly designed.

5-3. Bulkheads and Guides

A bulkhead or stoplog and guides should be provided at the entrance to each water passage for dewatering the service and emergency gates for maintenance except when they are excessively costly or clearly unfeasible and other means can be improvised. When emergency gates are not required, stoplog or bulkhead guides for maintenance should always be provided upstream of the service gates.

5-4. Trashracks

Trashracks are needed to prevent clogging and debris damage to outlet control gates and turbines. The trash-rack type and size of openings depend on the pool elevation, intake elevation, the size of the outlet conduit, the reservoir trash conditions, type of control device used, use of the water, and the need to exclude the trash.

a. Intakes for flow regulation. A simple trash structure, usually of reinforced concrete, with clear horizontal and vertical openings not more than two-thirds the gate width should be provided at the upstream end of the outlet works to catch trees and other large trash which may reach the entrance and be capable of blocking the gated passages (EM 1110-2-1602, Chapter 3, Section II). Large trash at the tunnel entrance occurs more often when the permanent pool is only slightly above the entrance than when the permanent pool is high above the entrance. Only in special cases in which trees and floating debris are absent from the reservoir and watershed should the trash structure be omitted.

(1) Metal trashracks fabricated of closely spaced bars are generally used for small conduits with control valves and water supply intakes that require screening of small debris.

(2) To facilitate trash removal immediately after a flood, the working platform at the top for raking and removal of trash usually should not be lower than the top elevation of the conservation or maximum power pool. The usual trash structure consists of upright beams, inclined slightly downstream from the vertical to facilitate raking, and horizontal beams.

(3) If the gate structure is located at the upstream end of the outlet tunnel, the trash structure is combined with it for economy.

(4) The area of the openings in the trash structure should limit the local net-area velocity to less than 10 to 15 ft/sec.

b. Power intakes. EM 1110-2-3001 discusses the requirements for power intake trashracks and the design loads and spacing of trashracks from the standpoint of hydraulic and structural requirements.

c. Floating trash and debris control facilities. Floating trash and debris control can be provided by a basic trash boom constructed of logs or floating pontoons. Trashracks would still be necessary to protect intakes from occasional water-logged trees or large logs that pass under the trash boom.

5-5. Load Cases

a. Gates, valves, bulkhead, and stoplogs. The gates, valves, bulkhead, and stoplog design loads should include dead loads, static live loads, dynamic live loads, temperature loads, and any unusual load which can potentially occur due to misoperation. Dead loads include self weight and the weight of the attached equipment. The static live load includes interior and exterior water surface pressures, silt pressures, temperatures, debris, ice, lifting and installation forces on components, and any other loading pertinent to the project operation. Dynamic loads include seismic, dynamic hydraulic conditions, load rejection for power intakes, vibration, fluctuating hydraulic forces, waves, and hydraulic drawdown forces for closure under flow conditions.

b. Trashracks. The trashrack structure should be designed to withstand hydraulic pressure based on velocity head loss with a minimum design head of 5 ft. External forces resulting from reservoir ice, debris clogging, and sediment buildup should be considered in the design at projects where those conditions apply.

5-6. Strength and Serviceability Requirements

Strength and serviceability requirements have an important bearing on the development and design of gates, valves, bulkhead and guides, trashracks, and operating equipment. The types of factors that should be considered include frequency and intensity of operation, cavitation and abrasion during operation, maximum deflection for operation, vibration and hydrodynamic loading, corrosion, and maintainability. Gates, bulkheads, stoplogs, trashracks, and other steel features should be designed in accordance with the strength and serviceability requirements presented in EM 1110-2-2105.

5-7. Gate Room Layout/Operating Equipment

a. Gate room layout. The gate room layout will depend on the mechanical equipment and electrical controls which will be located in the room. Adequate room size must be allowed for safe clearance and maintainability of operating equipment such as gate operators, valves, and electrical components. The room must also have adequate clearance for removal of the gates and valves. Vertical intake structures generally utilize a crane on the intake deck to disassemble and remove items requiring major maintenance. This work requires that hatches be provided to facilitate removal of components. Inclined intake structures may require a bridge crane in the gate room for disassembly of components. Lifting eyes should be strategically placed in the gate room along with provisions for monorail hoists, if necessary.

b. Operating equipment. Additional information about gates and valves is provided in EM 1110-2-1602, Hydraulic Design of Reservoir Outlet Works; EM 1110-2-2702, Design of Spillway Tainter Gates, and UFGS 11287A, Tainter Gates and Anchorages.

(1) Slide gates. Slide gates are typically hydraulic cylinder operated. The hydraulic power units can be package units. Two pump/motor combinations are used, with the hydraulic piping valved in such a way that one pump may be removed from service without rendering the system inoperative. A mechanical position indicator can be used to show the gate opening. This position indication is converted into an electronic signal for remote sensing and operation. Figure 5-1 shows a typical gate room layout with slide gates.

(2) Roller gates. Roller gates can be operated by the use of hydraulic cylinders or wire rope hoists. Position indication can be noted by use of an indicator rod or, in the case of hoists, by use of a revolution counter.

(3) Tainter gates. Tainter gates are typically hoist operated. A single motor/reducer drives a torque tube which drives a wire rope drum on each end of the tainter gate.

(4) Valves. Valves are generally electric motors operated from a pushbutton station, both locally at the valve and at a control in the gate room. Types of valves include ball valves, which act as a guard valve to fixed-cone valves in a minimum discharge type outlet, and gate valves, which are used in a main outlet conduit fill pipe.

(5) Controls and instrumentation. Controls and instrumentation for gate or valve positioning, emergency generator, and sump and utility pumps may be local, remote, or both. Piezometers, flow meters, and level gauges are discussed in EM 1110-2-4205, Hydroelectric Power Plants Mechanical Design.

c. Other equipment. Heating and ventilating of the gate room are generally achieved through the use of a ventilation fan, duct work, and unit heaters located strategically in the gate room. Minimum outdoor air quantities should be required in accordance with American Society of Heating, Refrigerating, and Air-Conditioning Engineers (ASHRAE) criteria. Special design features may be required in situations where the possibility of infiltration of gases from the decay of vegetation into the tower exists. Emergency power is usually provided by an emergency generator located at the top of the intake tower. However, in towers, emergency power may be provided by an emergency generator located in a remote building away from the intake tower. Sump pumps are normally required at the lowest point of the tower. When sump pumps are required, the discharge is pumped into the reservoir through an oil/water separator. Overhead cranes (bridge type or monorail) may be included to facilitate maintenance functions. The coverage envelope should be large enough to permit access to all gates and valves.

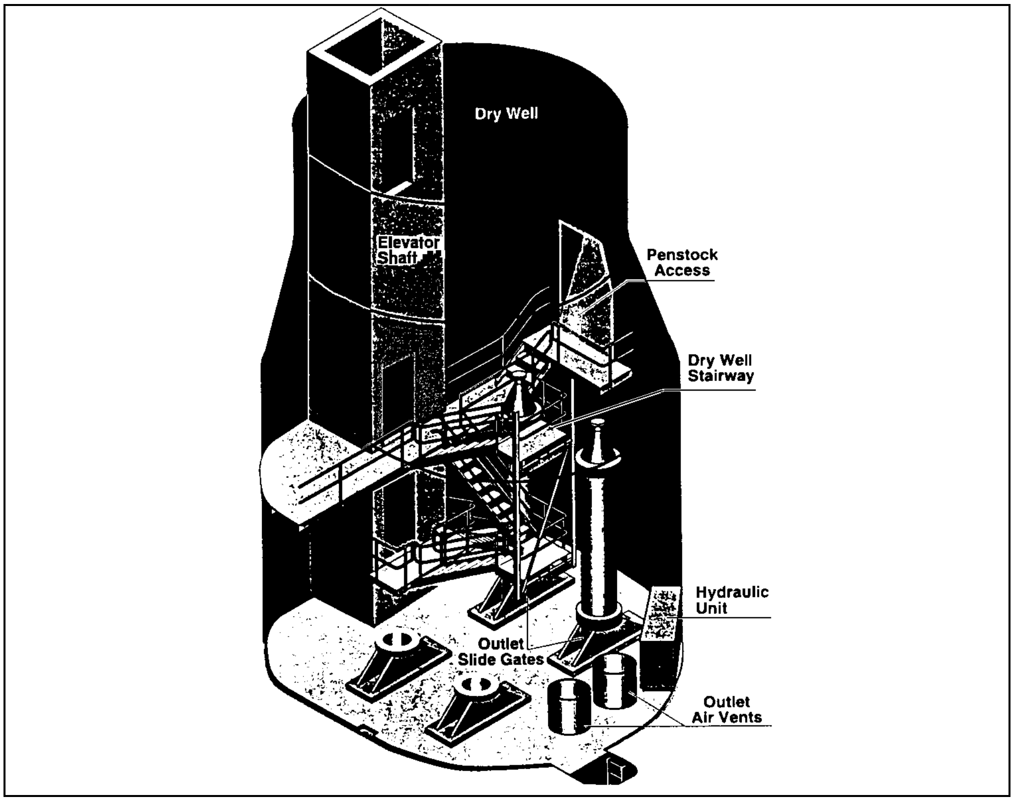


Figure 5-1. Gate room perspective

5-8. Elevator

Design and justification factors for elevators are discussed in EM 1110-2-4205, Hydroelectric Power Plants Mechanical Design. UFGS 14210A, Elevators, Electric, should be in accordance with American Society of Mechanical Engineers (ASME) A17.1, Safety Code for Elevators and Escalators.

Chapter 6
Special Detailing Requirements and Other Design Considerations

6-1. Introduction

After the general arrangement of the intake structure has been determined, access, openings, and operational and maintenance provisions should be developed to provide a functional project.

6-2. Access Bridge Requirements

a. General. Where necessary, a service bridge extends from the top of the dam or from the valley wall to the intake tower to provide access for the operation, repair, and maintenance of the gates, equipment, and appurtenances. A single-lane access bridge is sufficient. The bridge is designed for the AASHTO standard truck loading and for all possible off-highway loads due to cranes lifting or transporting gates or other equipment into or out of the intake tower. The designer should thoroughly investigate the methods used to transport and remove or install gates to make sure that all possible bridge loadings have been accounted for in the design. Generally, a single-span bridge is most economical due to the high costs of constructing intermediate supporting piers. The single spans may be prestressed girders, rolled steel beams, welded plate girders, or steel trusses. Prestressed concrete girders and rolled steel girders are economical for spans up to 110 ft. Welded plate composite steel girder bridges are commonly used for spans from 110 to 175 ft and have been used for spans up to 350 ft. However, steel trusses or intermediate piers should be considered when spans exceed 175 ft. The engineer should determine whether it is best to accommodate expansion at the abutment end or tower end of the bridge. Attaching the bridge at the tower with a fixed bearing prevents the tower and bridge from hammering during an earthquake. All bearings should have adequate seat widths and restraints to prevent loss of support during an earthquake. When the bridge is attached to the intake tower with a fixed bearing, the bridge and tower should be combined in a single analytical model to determine the combined effects of the bridge tower system when subjected to the design earthquake ground motions. The bridge superstructure will normally be capable of withstanding all the seismic inertial forces without difficulty. The objectives of the seismic analysis are to ensure that the bridge bearings and supports can accommodate the seismic inertial forces and displacements and that the supporting piers, abutments, and their foundations can accommodate the seismic inertial forces.

b. Load criteria. The bridge should be designed to withstand safely all group combinations of forces in AASHTO that are applicable. The bridge should be designed by load factor design methods using the appropriate load factors and strength reduction factors in AASHTO. The design for earthquake loadings should employ the response spectrum methodology with the earthquake demands in accordance with those established for the dam and intake tower. Impact may be omitted except when a mobile crane or gantry crane is used to pick up a gate from the bridge. When it is anticipated that construction equipment will have to use the bridge, the bridge shall be designed to accommodate construction load conditions.

6-3. Tower Service Deck Requirements

a. General. Simplicity and economy are the primary considerations in design of the intake tower service deck. Although the intake structure is essentially a reinforced concrete structure, economy may be affected and construction simplified by the use of steel beams in the operating and service decks, steel roof joists, etc.

b. Load criteria.

(1) The service deck at the top of the intake structure should be designed to withstand 250 lb/ft^2, or a loaded crane or truck, whichever controls. The removable hatch covers and openings should be protected by a structural steel frame. All edges exposed to traffic and heavy equipment should have armor angle protection.

(2) Intake structures with gate hoisting equipment on the operating deck are designed for the loads of the gate hoists with maximum gate operating loads. The floor slab between hoists should be designed for a uniform load of 250 lb/ft^2 unless a different load is warranted. The service deck should be designed for the concentrated dead load of a service gate in each service gate opening or the emergency gate in any emergency gate opening and supported by logs or blocking at the edge of the opening. The house above the operating deck is designed according to usual load and stress criteria for buildings unless a different load is warranted.

6-4. Concrete Temperature Control Measures

Cracking occurs when tensile stresses within the concrete exceed the tensile strain capacity of the concrete. In mass concrete, primary stresses result from volume changes that occur as the concrete cools from its initial peak temperature down to ambient air temperature. If not controlled or limited to acceptable levels, the tensile stresses will quickly exceed tensile strain capacity of the concrete and result in unacceptable cracking. Measures can be taken to prevent the tensile stresses from exceeding acceptable values only after the tensile strain capacity of the concrete has been determined. To achieve proper temperature control, the designer must determine various thermal properties for both the aggregates and the concrete. These properties include thermal conductivity, diffusivity, specific heat of hydration, coefficient of thermal expansion, etc. After the tensile strain capacity of the concrete and the amount of thermal contraction that can occur without causing cracking have been determined, specific measures can be taken to limit the initial peak temperature. One measure is to reduce concrete placement temperatures, and thereby lower final peak concrete temperatures. Other measures can be taken (individually or in combination) that will be successful in controlling temperature gains sufficiently to reduce or prevent cracking. Some of these measures are as follows: reducing Portland cement contents, use of fly ash as Portland cement replacement, use of chilled water or ice to reduce initial concrete placement temperature, provision of postcooling techniques such as using cooling coils, cooling aggregate stockpiles, using the largest size aggregate practical, placement of concrete in thinner lifts, and/or extending intervals between concrete placements. For additional discussion on crack control, see ACI 224R-90 Committee Report, and ETL 1110-2-365, "Nonlinear Incremental Structural Analysis of Massive Concrete Structures."

6-5. Air Vents

Circular or rectangular air vents are placed at the downstream side of the service gates to reduce cavitation damage and prevent erratic flow conditions. Vents are extended above pool elevations, preferably above the spillway design flood pool, and are arranged to open outside the structure far enough away from the service bridge or any platform where personnel could possibly be endangered by the strong inflow of air during outlet operation. When convenient and economical, the air vents from two or more gates may be brought together at a point above the pressure gradient into a larger vent. Vents may be steel pipes or may be formed in concrete. Appropriate air venting should also be provided for dewatering and filling operations downstream of valves, bulkheads, and stoplogs in water passages. EM 1110-2-1602, Chapter 3, provides pertinent information on how to size and determine the locations of air vents.

6-6. Abrasion- and Cavitation-Resistant Surfaces

a. General. Consideration should be given to all water surfaces in the structure that may be subject to abrasion resulting from sediment flow and potential cavitation caused by high velocities and surface irregularities. All operating conditions, including diversion, should be evaluated for potential adverse effects.

b. Provisions for abrasion and cavitation protection. For concrete surfaces, the use of stainless steel linings, special concrete, abrasion-resistant coatings, and special concrete specifications to provide a high quality surface may be necessary to attain the required protection. Special concretes may include special aggregate concrete and silica fume concrete. Areas in the concrete specifications that should be addressed include the use of steel forms, additional forming support, restrictions on form joint locations, tighter surface tolerances, placement sequencing, prohibiting positive projections into the flow, and required submittals for critical concrete operations.

c. Repair. Specific methods for concrete repairs, patching, and grinding for construction deficiencies should be clearly defined in the specifications. Typical repair methods may not be satisfactory to provide the desired long-term durability. Repairs in high-velocity areas may require excavation to a depth that will allow bonding of the patch to the near-face reinforcement.

6-7. Corrosion Control

Gates, liners, piping, and other ferrous equipment and materials shall be protected against corrosion by coating and/or cathodic protection. For further information refer to UFGS 09965A, Painting: Hydraulic Structures; UFGS 09971A, Metalizing: Hydraulic Structures; TM 5-811-7, Electrical Design, Cathodic Protection; and the U.S. Army Construction Engineering Research Laboratory paint lab, Champaign, IL.

6-8. Operations and Maintenance Considerations

An important operation and maintenance consideration for a newly constructed intake structure is to minimize staffing requirements. To accomplish this objective, a remote control system and automated surveillance systems should be considered. To reduce maintenance, materials should be selected with extended service life in mind. The materials selected should resist corrosion and abrasion and minimize periodic maintenance. Features needed for maintenance should be provided including the maintenance bulkheads and bulkheadslots required for the on-site maintenance of emergency and service gates.

6-9. Instrumentation

Structural behavior instrumentation programs are provided for intake structures to measure the structural integrity, check design assumptions, and monitor the behavior of the foundation and structure during construction and the various operating phases. The extent of instrumentation at projects will vary depending on particular site conditions, the size of the structure, and needs for monitoring critical sections. Instrumentation can be grouped into those that either directly or indirectly measure conditions related to the safety of the structure. Plumbness, alignment, uplift, and seismic instruments fall into the category of safety instruments. In the other group, the instruments measure quantities such as stress and strain, length change, pore pressure, leakage, and temperature change. EM 1110-2-4300 provides details and guidance on the planning of instrumentation programs, types of instruments, and the preparation, installation, and collection of data.

Appendix A
References

A-1. Required Publications

TI 809-04
Seismic Design for Buildings

TM 5-811-7
Electrical Design, Cathodic Protection

ER 1110-2-50
Low Level Discharge Facilities for Drawdown of Impoundments

ER 1110-2-110
Instrumentation for Safety Evaluations of Civil Works Projects

ER 1110-2-1806
Earthquake Design and Evaluation of Civil Works Projects

EM 385-1-1
Safety and Health Requirements

EM 1110-1-2009
Architectural Concrete

EM 1110-2-1602
Hydraulic Design of Reservoir Outlet Works

EM 1110-2-1612
Ice Engineering

EM 1110-2-2104
Strength Design for Reinforced-Concrete Hydraulic Structures

EM 1110-2-2105
Design of Hydraulic Steel Structures

EM 1110-2-2200
Gravity Dam Design

EM 1110-2-2502
Retaining and Flood Walls

EM 1110-2-2702
Design of Spillway Tainter Gates

EM 1110-2-2901
Tunnels and Shafts in Rock

EM 1110-2-2902
Conduits, Culverts, and Pipes

EM 1110-2-2906
Design of Pile Foundations

EM 1110-2-3001
Planning and Design of Hydroelectric Power Plant Structures

EM 1110-2-4205
Hydroelectric Power Plants Mechanical Design

EM 1110-2-4300
Instrumentation for Concrete Structures

EM 1110-2-6050
Response Spectra and Seismic Analysis for Concrete Hydraulic Structures

EM 1110-2-6051
Time-History Dynamic Analysis of Concrete Hydraulic Structures

ETL 1110-2-365
Nonlinear Incremental Structural Analysis of Massive Concrete Structures

UFGS 11287A
Tainter Gates and Anchorages

UFGS 13080
Seismic Protection for Miscellaneous Equipment

UFGS 09971A
Metallizing: Hydraulic Structures

UFGS 09965A
Painting: Hydraulic Structures

UFGS 14210A
Elevators, Electric

American Concrete Institute Committee 224 2001
American Concrete Institute Committee 224. 2001. "Control of Cracking in Concrete Structures," ACI 224R-89, Part 3, Farmington Hills, MI.

American Society of Civil Engineers 1995
American Society of Civil Engineers. 1995. "Minimum Design Loads for Buildings and Other Structures," ASCE 7-95 (Revision of ASCE 7-88), New York.

American Society of Mechanical Engineers 1996
American Society of Mechanical Engineers. ASME A17.1 Safety Code for Elevators and Escalators.

American Association of State Highway and Transportation Officials 1998
American Association of State Highway and Transportation Officials. 1998. "AASHTO LRFD Bridge Design Specifications," Washington, DC.

Bathe and Wilson 1976
Bathe, K. J., and Wilson, E. L. 1976. *Numerical Methods in Finite Element Analysis*, Prentice-Hall, Inc., Englewood Cliffs, NJ.

Ebeling 1992
Ebeling, R. M. 1992. "Introduction to the Computation of Response Spectrum for Earthquake Loading," Technical Report ITL-92-4, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Ebeling and Morrison 1992
Ebeling, R. E., and Morrison, E. E., Jr. 1992. "Seismic Design of Waterfront Retaining Structures," Technical Report ITL 92-11, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS

Federal Highway Administration 1995
Federal Highway Administration 1995. "Seismic Retrofitting Manual for Highway Bridges," Report No. FHWA-RD-94-052, I. G. Buckle and I. M. Friedland, ed., McLean, VA.

French, Ebeling, and Strom 1994
French, S. E., Ebeling, R. M., and Strom, R. 1994. "Dynamics of Distributed-Mass Intake Towers using the Rayleigh Method," 1994 Corps of Engineers SEC Paper.

Goyal and Chopra 1989
Goyal, A., and Chopra, A. K. 1989. "Earthquake Analysis and Response of Intake-Outlet Towers," Report No. UCB/EERC-89-04, Earthquake Engineering Research Center, University of California, Berkeley.

Hall and Radhakrishnan 1983
Hall, R. L., and Radhakrishnan, N. 1983. "Case Study of Six Major General-Purpose Finite Element Programs," Technical Report K-83-4, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Jansen 1988
Jansen, R. B. 1988. "Advanced Dam Engineering for Design, Construction, and Rehabilitation," Van Nostrand Reinhold, New York, NY.

Moehle 1992
Moehle, J. P. 1992. "Displacement-Based Design of RC Structures Subjected to Earthquakes," *Earthquake Spectra* 8(3), 403-428.

Mononobe and Matsuo 1929
Mononobe, N., and Matsuo, H. 1929. "On the Determination of Earth Pressures During Earthquakes," *Proceedings*, World Engineering Congress, 9.

Newmark and Hall 1982
Newmark, N. M., and Hall, W. J. 1982. "Earthquake Spectra and Design," Earthquake Engineering Research Institute, Berkeley, CA.

Okabe 1984
Okabe, S. 1984. *Introduction to Earthquake Engineering*. 2nd ed., University of Tokyo Press, 629 p.

Pauley 1980
Pauley, T. 1980. "Earthquake-Resisting Shearwalls--New Zealand Design Trends," *ACI Structural Journal,* May - June.

Priestley and Benzoni 1996
Priestley, M. J. N., and Benzoni, G. 1996. "Seismic Performance of Concrete Circular Columns with Low Longitudinal Reinforcement Ratios," *ACI Structural Journal,* July - August.

U.S. Bureau of Reclamation 1976
U.S. Bureau of Reclamation. 1976. "Design of Gravity Dams," Denver, CO.

Wilson, Suharwardy, and Habibullah 1995
Wilson, E. L., Suharwardy, I., and Habibullah, A., 1995. "A Clarification of the Orthogonal Effects in a Three-Dimensional Seismic Analysis," *Earthquake Spectral* 11(4).

Wood 1989
Wood, S. L. 1989. "Minimum Tensile Reinforcement Requirements in Walls," *ACE Structural Journal,* September - October, 582-591.

Wood 1990
Wood, S. L. 1990. "Shear Strength of Low-Rise Reinforced Concrete Walls," *ACE Structural Journal,* January - February, 99-107.

A-2. Related Publications

American Concrete Institute
American Concrete Institute. *Building Code Requirements for Reinforced Concrete (ACI 318)*, Redford Station, Detroit, MI.

Biggs 1964
Biggs, J. M. 1964. *Structural Dynamics,* McGraw-Hill, New York.

Clough and Fragaszy 1977
Clough, G. W., and Fragaszy, R. F. (1977). "A study of earth loadings on floodway retaining structures in the 1971 San Fernando Valley earthquake," *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, 1977, Vol III, pp 2455-2460.

Clough and Penzien 1993
Clough, R. W., and Penzien, J. 1993. *Dynamics of Structures,* McGraw-Hill, New York.

Der Kiureghian 1979
Der Kiureghian, A. 1979. "On Response of Structures to Stationary Excitation," Report No. EERC 79-32, Earthquake Engineering Research Center, University of California, Berkeley.

Dove 1998
Dove, R. C. 1998. "Performance of Lightly Reinforced Concrete Intake Towers Under Selected Loadings," Technical Report SL-98-1, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Hays 1980
Hays, W. W. 1980. “Procedures for Estimating Earthquake Ground Motions,” Geological Survey Professional Paper No. 114, U.S. Geological Survey, Washington, DC.

Housner 1963
Housner, G. W. 1963. “The Behavior of Inverted Pendulum Structures During Earthquakes,” *Bulletin of the Seismological Society of America* 53(2), 403-417.

Ishiyama 1980
Ishiyama, Y. 1980. “Review and Discussion on Overturning of Bodies by Earthquake Motions,” BRI Research Paper No. 85, Building Research Institute, Ministry of Construction.

Krinitzsky 1982
Krinitzsky, E. L. 1982. “Essentials for Specifying Earthquake Motion Engineering Design,” U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Liaw and Chopra 1973
Liaw, C. Y., and Chopra, A. K. 1973. “Earthquake Response of Axisymmetric Tower Structures Surrounded by Water,” Report No. 73-25, Earthquake Engineering Research Center, University of California, Berkeley.

Lukose, Gergely, and White 1982
Lukose, K., Gergely, P., and White, R. N. 1982. “Behavior of Reinforced Concrete Lapped Splices for Inelastic Cyclic Loading,” *ACI Structural Journal*, September - October, 355-365.

Lysmer et al. 1975
Lysmer, J., Udaka, T., Tsai, C.-F., and Seed, H. B. 1975. “FLUSH – A Computer Program for Approximate 3-D Analysis of Soil-Structure Interaction Problems,” Report No. EERC 75-30, Earthquake Engineering Research Center, University of California, Berkeley.

Meek 1978
Meek, J. W. 1978. “Dynamic Response of Tipping Core Building,” *Earthquake Engineering and Structural Dynamics* 6(5), 437-454.

Mlakar and Jones 1982
Mlakar, P. F., and Jones, P. S. 1982. “Seismic Analysis of Intake Towers,” Technical Report SL-82-8, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Montes and Rosenblueth 1968
Montes, R., and Rosenblueth, E. 1968. “Cortantes y Momentos Sismicos en Chimeneas,” Segundo Congreso Nacional de Ingenieria Sismica, Veracruz, Mexico.

National Earthquake Hazards Reduction Program 1992
National Earthquake Hazards Reduction Program. 1992. “Recommended Provisions for the Development of Seismic Regulations for New Buildings,” Report Nos. FEMA 222 and 223, Federal Emergency Management Agency, Publications Division, Washington, DC.

Orangun, Jirsa, and Breen 1977
Orangun, C. O., Jirsa, J. O., and Breen, J. E. 1977. “A Reevaluation of Test Data on Development Length and Splices,” *ACI Journal,* March, 114-122.

Paz 1991
Paz, M. 1991. *Structural Dynamics: Theory and Computation,* Van Nostrand Reinhold, New York.

Priestley and Park 1987
Priestley, M. N. J., and Park, R. 1987. "Strength and Ductility of Concrete Bridge Columns Under Seismic Loading," *ACI Structural Journal*, January - February.

Quest Structures 2000
Quest Structures. 2000. "ITAP – Intake Tower Analysis Program; Finite Element Program for Analysis of Free Standing Intake Tower," Quest Structures, Orinda, CA.

Rea, Liaw, and Chopra 1975
Rea, D., Liaw, C.-Y., and Chopra, A. K. 1975. "Dynamic Properties of San Bernardino Intake Tower," Report EERC 75-7, Earthquake Engineering Research Center, University of California, Berkeley.

Rosenbloeth and Contreas 1977
Rosenbloeth, E., and Contreas, H. 1977. "Approximate Design for Multicomponent Earthquakes," *ASCE Journal of the Engineering Mechanics Division,* October.

Seed, Ugas, and Lysmer 1974
Seed, H. B., Ugas, C., and Lysmer, J. 1974. "Site-Dependent Spectra for Earthquake-Resistant Design," *Bulletin of the Seismological Society of America* 66(1).

Seismic Design Handbook 1989
Seismic Design Handbook, Van Nostrand Reinhold, Farzad Naeim, ed., New York, NY.

***Shore Protection Manual* 1984**
Shore Protection Manual. 1984. 4th ed., 2 Vol, U.S. Army Engineer Waterways Experiment Station, U.S. Government Printing Office, Washington, DC.

Veletsos and Younan 1994
Veletsos, A. S., and Younan, A. H. 1994. "Dynamic Soil Pressures on Rigid Vertical Walls," *Earthquake Engineering and Structural Dynamic*, Vol 23, pp 275-301.

Wolf 1995
Wolf, J. P. 1995. "Discussion on paper "Dynamic Soil Pressures on Rigid Vertical Walls," by A. S. Veletsos and A. H. Younan, *Earthquake and Structural Dynamic*, Vol 24, pp 1287-1291.

Appendix B
Seismic Analysis for Preliminary Design or Screening Evaluation of Free-Standing Intake Towers

B-1. Introduction

a. General. Guidance for the preliminary design or screening evaluation of intake towers is based on linear elastic response spectra modal techniques, and a brief discussion of nonlinear dynamic analyses is also presented. The procedures presented here are adequate for the seismic analysis of intake towers that are square, rectangular, or circular in plan and bend about a elastic center line as a cantilever beam.

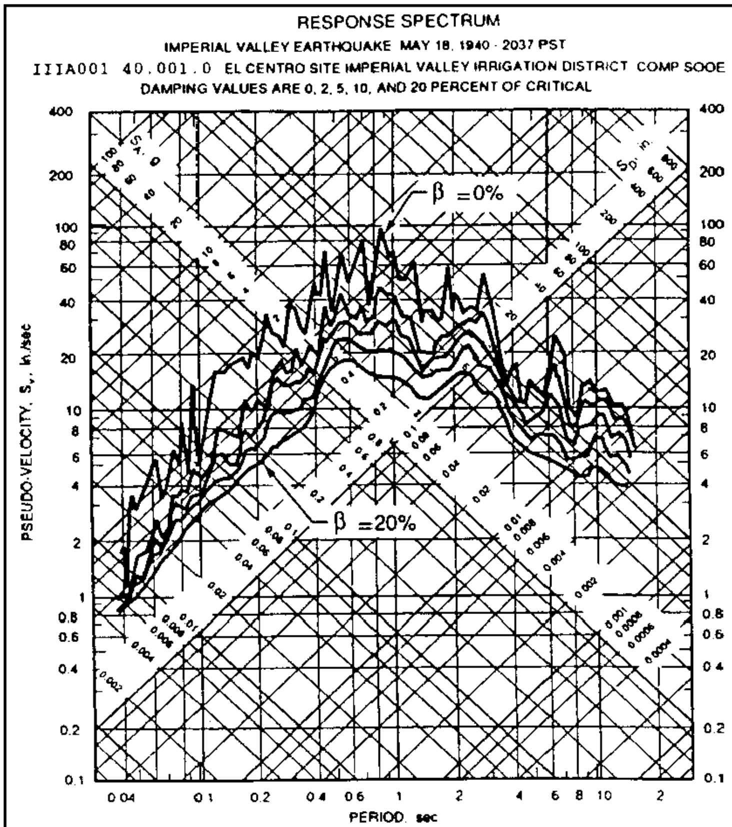
b. Standard spectra. Standard spectra are commonly used for preliminary design when site-specific response spectra are not available. A Newmark and Hall Standard Response Spectrum is illustrated in Figure B-1. ER 1110-2-1806 and EM 1110-2-6050 also provide guidance on the use of standard response spectra, and an example of the standard acceleration response spectra is shown in Figure B-2.

c. Smoothed response spectra. Actual site response spectra generated from specific earthquake accelerograms tend to be very jagged, as indicated in Figure B-2a, and are not necessarily good design tools. Therefore a modified form of the response spectrum, developed by averaging the response values from many response spectra for several different earthquakes and then smoothing the resulting peaks and valleys, is used for analysis. Equal hazard spectra developed by probabilistic methods are also used for analysis. More information about response spectra used for the design or evaluation of hydraulic structures is in EM 1110-2-6050.

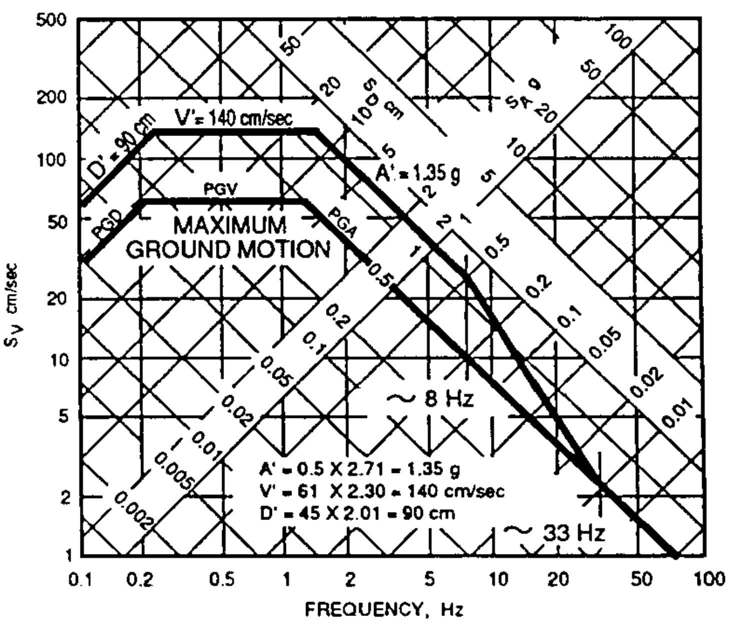
d. Linear elastic response spectrum modal analyses (RSMA). Response spectra may be used for the analysis of multiple-degree-of-freedom (MDOF) systems by entering the spectrum with any one of the natural periods of the MDOF system. In such circumstances, the spectrum will yield only the response for the single vibration mode having that natural period. It is necessary in such an approach to find the individual responses for several modes (and their natural periods), then combine them to find the total response. The response spectrum provide the maximum displacement (and pseudo-quantities) for each natural period, even though each maximum displacement occurs at a different time. Using direct superposition to combine the displacements would accumulate all maximum displacements as if they were occurring at the same instant. Such a solution would grossly distort the structural response. In addition, the displacements bear no algebraic signs. Only the absolute value of the displacement is known. The direct sum, without algebraic signs, would further distort the response. Two rational means of combining several responses from the response spectrum are the square-root-of-the-sum-of-the-squares (SRSS) method and the complete-quadratic combination (CQC) method. Both methods attempt to find the least upper bound for the behavioral response.

(1) SRSS method.

(a) SRSS is an approximate method for combining modal responses. In the SRSS method, the squares of a specific response are summed (e.g., displacement, drift, story/base shear, story/base overturning moments, elemental forces, etc.). The square root of this sum is taken to be the combined effect. It is important to note that the quantities being combined (e.g., story drift, base shear, etc.) are those for each individual mode.



a. Actual Response Spectra for a Recorded Ground Motion



b. Newmark and Hall Smooth-Shaped Response Spectra

Figure B-1. Typical tripartite plot and smooth-shaped response spectrum (Newmark and Hall 1982)

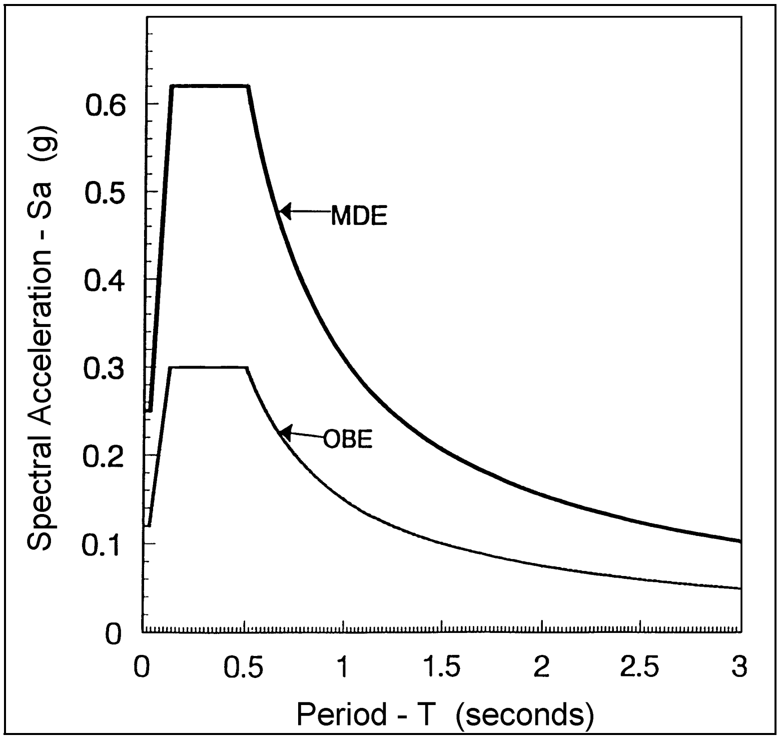


Figure B-2. Standard acceleration response spectra

(b) The governing equation of the SRSS method can be written mathematically as

$$u_i = \sqrt{\sum_{j=1}^{n} u_{ij}^2} \tag{B-1}$$

where

u_i = approximate maximum response for the i^{th} component of the behavioral response

n = number of modes to be used in the analysis

u_{ij} = i^{th} component of the j^{th} modal behavioral response vector

The SRSS method is conservative most of the time. It tends to be unconservative when the modal frequencies are closely spaced.

(2) CQC method. CQC is a modal combination method based on the use of cross-modal coefficients. The cross-modal coefficients reflect the duration and frequency content of the seismic event as well as the modal frequencies and damping ratios of the structure.

(a) Mathematically, the governing equation of the CQC method can be written as:

$$u_i = \sqrt{\sum_{j=1}^{n}\sum_{k=1}^{n} u_{ij}\rho_{jk}u_{ik}} \tag{B-2}$$

where

u_{ik} = i^{th} component of the k^{th} modal response vector

ρ_{jk} = cross-modal coefficient for the j^{th} and k^{th} modes

As indicated in *d* above, the behavioral responses are calculated for each mode prior to their superposition in CQC. If the duration of the earthquake is long compared with the periods of the structure and if the earthquake spectrum is smooth over a wide range of frequencies, the cross-modal coefficient can be approximated as:

$$\rho_{jk} = \frac{8\xi^2(1+r)r^{1.5}}{(1+r^2)^2 + 4\xi^2 r(1+r)^2} \tag{B-3}$$

where r is the reference level and ξ is the modal damping ratio and

$$r = \frac{T_k}{T_j} \tag{B-4}$$

where

T_k = period of the system for mode k

T_j = period of the system for mode j

Note that if the frequencies are well separated, the off-diagonal terms approach zero and the CQC method reverts to the SRSS method.

B-2. RSMA of Intake Tower

A response spectrum modal analysis of an intake tower can be performed the using an approximate two-mode model, a beam model, or a finite element model.

a. Approximate two-mode model. The approximate two-mode model is a manual procedure for the preliminary analysis of intake towers. The two-mode method includes bending deformations only, which provides sufficient accuracy for preliminary analysis of intake towers that are regular in plan and elevation and are supported on rigid foundation. If a sufficient number of lumped masses are used to represent the distributed mass system, any error introduced by the lumped mass approximation will be negligible. A minimum of six lumped masses should be used for modeling an intake tower. A two-mode, added-mass analysis can be performed on a spreadsheet. The following six-step procedure applies to the analysis of a free-standing stepped tower, such as that shown in Figure B-3. An example of the two-mode approximate method is presented in Appendix C:

(1) Determine the mass per unit length.

(a) An added mass concept is used to simulate the hydrodynamic effects of the mass of water in the tower and the water surrounding the tower. Establish all discontinuities in size, stiffness, mass, and added mass. For each piecewise continuous segment, determine the flexural moment of inertia I of the cross section about both the xx- and yy-axes, the mass of the structure per unit length m_d, the added mass of water per unit length outside the tower m_a^o, the added mass of water per unit length inside the tower m_a^i

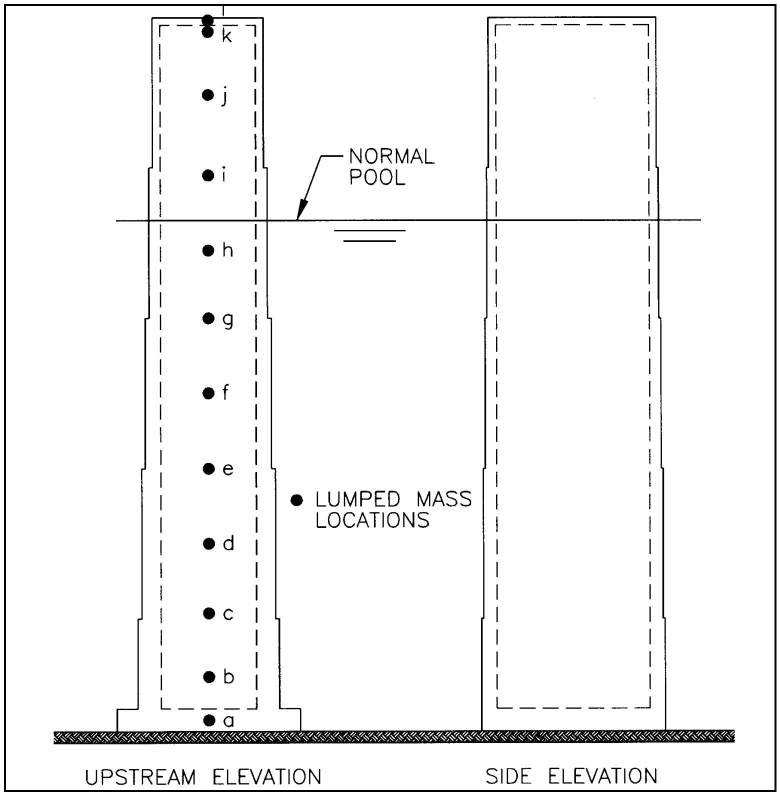


Figure B-3. Tower geometry

and the total mass per unit length $m(z)$ of the tower at any height z/L, where L is the total height of the tower. The total mass per unit length of the tower is the sum of the individual masses:

$$m(z) = m_d(z) + m_a^o(z) + m_a^i(z) \tag{B-5}$$

where

z = coordinate measured along the height of the tower

$m_d(z)$ = actual mass of the tower

$m_a^o(z)$ = hydrodynamic added mass due to the surrounding water

$m_a^i(z)$ = hydrodynamic added mass due to theinternal water

(b) The hydrodynamic added mass $m_a^o(z)$ can be found for the two-mode case from Figure B-4 and the mass $m_a^i(z)$ from Figure B-5. The use of these figures to obtain the added hydrodynamic mass is illustrated in Appendix C. A refined procedure for determining more accurately the added hydrodynamic mass is described in Appendix D. The total mass M_n of any segment n is then computed as

$$M_n = m(z)(\Delta L) \tag{B-6}$$

where ΔL is the length of the segment n.

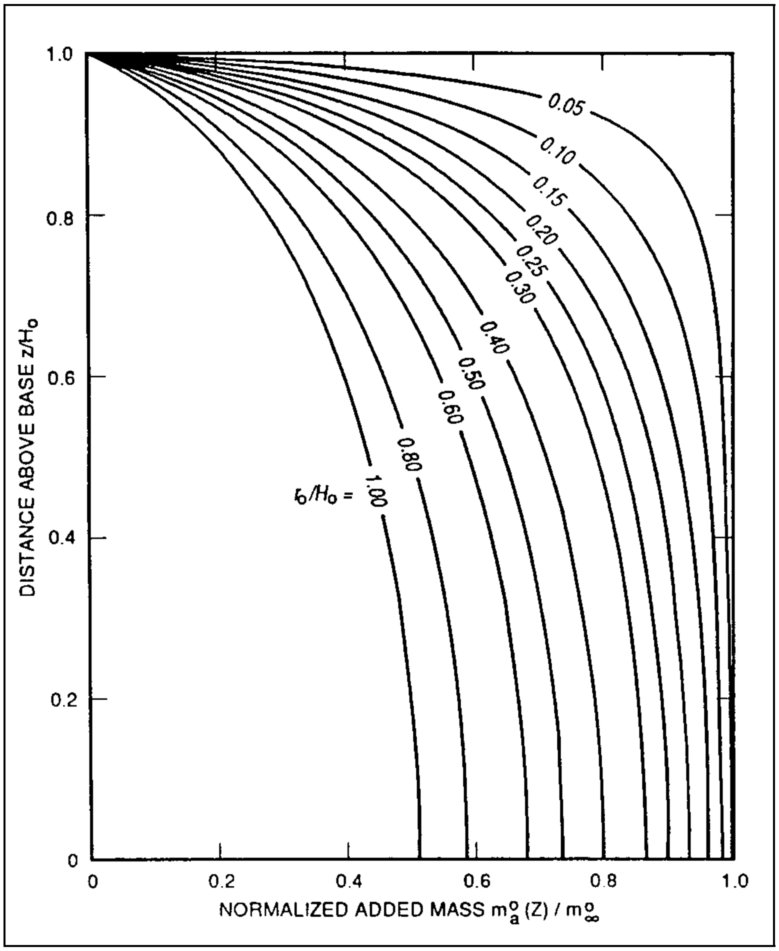


Figure B-4. Hydrodynamic added mass m_a^o from the two-mode approximate analysis procedure

(2) Determine the stiffness per unit length. The shape function φ in a distributed-mass analysis corresponds to the mode shape in a lumped-mass analysis. The shape functions for distributed-mass towers are given in Tables B-1 and B-2 for various amounts of taper. The displacement of the *normalized* shape functions at the tenth points are given directly in the tables. (The normalized shape function has a displacement of 1 at the top of the tower.) If the structural mass is the only mass (no added masses), the period can also be computed from the coefficients given in the tables. Note that if Tables B-1 and B-2 are used, the ratio of moments of inertia for bending about the xx-axis will not likely be the same as that for bending about the yy-axis. There will, therefore, be two values of the natural period T (one for each shape function) in each of the x- and y-directions. The generalized stiffness k^* may be computed from the coefficients given in Tables B-1 and B-2. The value of k^* will be needed in subsequent calculations and should be determined at this point.

(3) Determine the normalization ratio (L_n/m *). The normalization ratio L_n/m^* is computed directly and recorded. Physically, it is the ratio of the displacement (or acceleration) of the actual shape function to the displacement (or acceleration) of the normalized shape function. The normalization factor L_n is given by

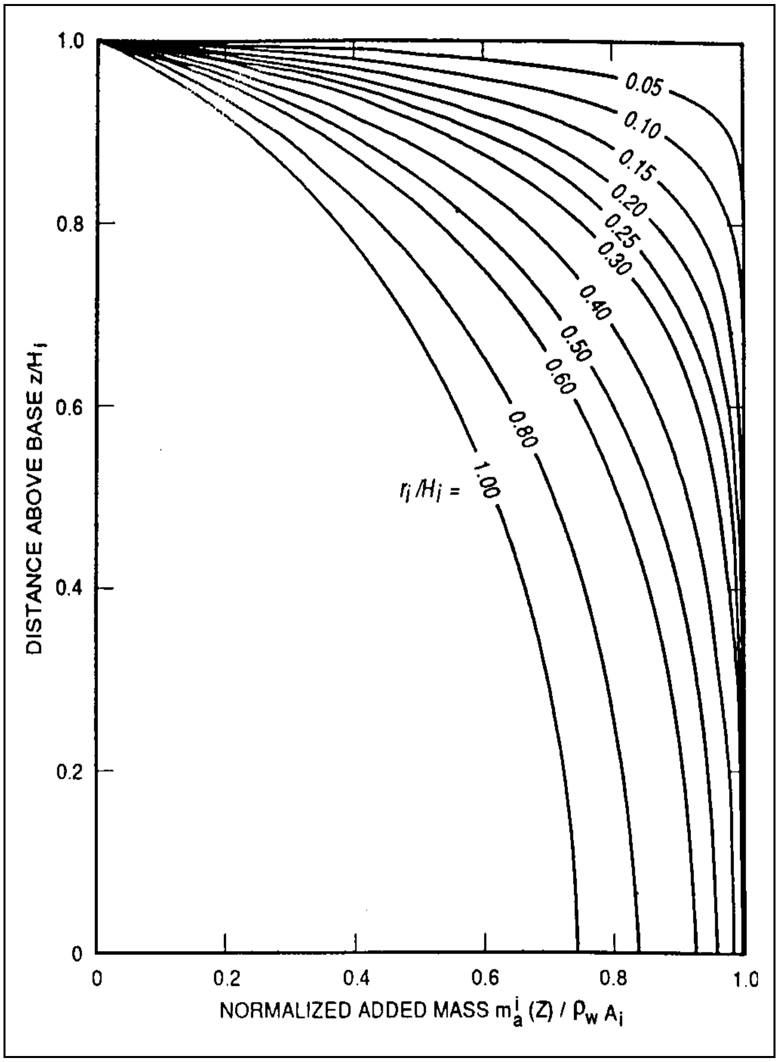


Figure B-5. Hydrodynamic added mass m_a^i from the two-mode analysis procedure

$$L_n \sum_0^L M_n \phi_n \qquad \text{(B-7)}$$

and the generalized mass m^* is given by

$$m^* = \sum_0^L M_n \phi_n^2 \qquad \text{(B-8)}$$

where M_n is found in step 1, and ϕ_n is the distance from the center line of the tower to the center of the deflected segment n.

(4) Determine the natural period of vibration T in seconds, the natural frequency ω in radians per second, and the mode shapes for the first two shape functions in both directions:

$$T = 2\pi\sqrt{\frac{m*}{k^*}} \tag{B-9}$$

and

$$\omega = \sqrt{\frac{k^*}{m^*}} \tag{B-10}$$

(5) Determine the spectral pseudo-acceleration S_A from the response spectrum. For the spectral pseudo-acceleration S_A, compute the pseudo-displacement S_D, where

$$S_D = \frac{S_A}{\omega^2} \tag{B-11}$$

Physically, the spectral pseudo-displacement S_D times the normalization value 1 is the normalized displacement at the top of the tower.

(6) Determine the lateral displacement, and the inertial forces, shears, and moments at the center of each segment.

(a) The actual lateral displacement of any mass M_n is given by

$$Y_n = (L_n/m^*)S_D\phi_n \tag{B-12}$$

(b) The inertial force acting on any mass M_n is computed as

$$F_n = M_n a_n = (L_n/m^*)M_n S_A \phi_n \tag{B-13}$$

where a_n is the actual acceleration of the mass M_n.

(c) The shear at the center of any segment is the sum of all inertial forces above that level:

$$V_n = \sum_{n+1}^{L} F_{n+1} \tag{B-14}$$

(d) The moment at the center of any segment n is the sum of moments above that level:

$$M_n = M_{n+1} + V_{n+1} (Z_{n+1} - Z_n)L \tag{B-15}$$

where Z_m is the n[th] coordinate measured along the height of the tower

b. Computer analysis using beam elements.

(1) Typically, the output from a beam modal analysis is in terms of moments, shears, and thrusts that can be directly used to design or evaluate reinforced concrete sections. Beam or finite element modals can be used with response spectra modal analyses as deemed appropriate by the structural engineer.

(2) Alternatively, the structure may be modeled using beam elements, as per the two-mode approximate method, and analyzed using a structural analysis computer program with dynamic analysis capability.

Computer programs are capable of analyzing as many modes of vibration as there are degrees of freedom, although for regular towers only the first two or three modes of vibration need usually be considered. Bending, shear, and torsional stiffness can be considered in a computer analysis. Some computer programs can also model structure-foundation interaction and structure-reservoir interaction. The use of beam elements is satisfactory for towers that are regular in plan and elevation, but it may not be appropriate for irregular towers. The added-mass concept described in paragraph B-2*a*(1) can be used to account for the hydrodynamic effects of the internal and surrounding water. The refined procedure of Appendix D for computing hydrodynamic added mass remains valid for beam element analyses.

c. Computer analysis using finite elements. The finite element response spectrum approach can be used for intake towers that cannot be adequately modeled with beam elements (e.g., irregular towers). RSMA of intake towers can also be performed by using finite elements such as plate, shell, and three-dimensional solid elements for either preliminary or final design using commercially available software with dynamic analysis capability (SAP, ANSYS, ADINA, GTSTRUDL, STAAD, etc.). Dynamic analysis performed by commercially available software can include the effects of both flexural and shear deformation and can consider the effects of any number of modes of vibration.

(1) The primary advantage of this approach lies in its ability to evaluate three-dimensional systems, to include torsional effects, and to pinpoint areas of localized stress concentrations. The finite element method warrants consideration when the tower is irregular in plan and/or elevation (e.g., torsion, vertically soft areas, major openings, etc.); or when soil-structure interaction is important (e.g., embedment greater than one third of the total tower height, siltation surrounding the tower, nonrigid foundation, etc.). Multiple analyses are often used to bracket their effects on the response of the tower.

(2) Two problems exist when finite elements are used to model the foundation of a structure subjected to seismic input.

(a) First, the sides and base of the elements do not allow the wave motion to permeate the boundaries (base and sides). The waves are, therefore, reflected back into the finite element grid, which distorts the results.

(b) Second, it is inappropriate to excite the base of the foundation model with an earthquake motion recorded at the ground surface. Several computer programs have a feature called a transmitting boundary that alleviates this problem. Otherwise, the foundation can be modeled as massless. This is equivalent to using massless springs, and the motion at the base of the foundation is equivalent to the motions at the surface. The size of the foundation model should be no less than three times the longest base dimension of the structure in width and two times the longest base dimension in depth. The model size and influence can be checked statically by increasing the size of the foundation modal and checking the difference in the static stresses at the base of the structure. The correct dimensions are found once the difference in the stresses becomes negligible. Most commercial finite element programs in structural analysis can perform a response spectrum analysis, but the resulting analysis provides only the *absolute maximum* stresses. Therefore, actual stress distributions are not provided by the analysis. The structural engineer must make some assumptions regarding the nature and shape of the distributions. Using these assumptions, the engineer then converts the results into the moments, shears, and axial loads required for the ultimate strength design (see Section 2-8 of EM 1110-2-6050).

(3) The procedure for a finite element response spectrum analysis by finite elements is as follows:

(a) Determine the added mass. The added-mass concept as explained in paragraph B-2*a*(1) remains valid in the finite element approach. The total added mass is distributed to the elements in a way that approximates the actual location and effects of the water.

(b) Generate the structural mesh. The finite element mesh should be designed to represent adequately the structural configuration and stiffness. The elements (beam, plates, and/or solids) should be chosen and sized to represent both the shear and bending effects of the seismic event adequately.

(c) Generate the soil/foundation mesh. If the foundation of the tower is very stiff relative to the tower, or if the effect of a more flexible foundation causes a smaller pseudo-acceleration to be chosen from the design spectra, the base may be modeled as rigid, which will produce conservative results. The more flexible the base, the larger the period; therefore, it is conservative to use a rigid base when the period is on the descending portion of the acceleration response spectrum curve. If the foundation cannot be modeled as rigid, it can be modeled with the use of springs or finite elements. Springs have often been used to model soil-structure interaction (Hall and Radhakrishnan 1983). The difficulty lies in choosing spring constants that reflect the vertical, horizontal, and rotational stiffnesses of the foundation with adequate accuracy. These constants are dependent on the size of the structure, the design loads, the loading history, etc., all of which are quite difficult to quantify. The spring constants are usually given upper and lower bounds and are used in multiple analysis to bracket their effects on the response of the tower.

Table B-1
Displacements for Step-Tapered Towers for the First Shape Function

First Shape Function										
I_{BASE} / I_{TOP}	1	2	3	4	5	6	7	8	9	10
With or without added mass, Coeff. of k^*	3.091	5.394	7.391	9.194	10.86	12.41	13.88	15.27	16.60	17.87
When there is no added mass:										
Coeff. of m^*	0.250	0.253	0.254	0.254	0.254	0.254	0.254	0.254	0.254	0.254
L_n/m^*	1.566	1.633	1.677	1.710	1.737	1.760	1.781	1.799	1.815	1.830
Coeff. of T	1.787	1.360	1.164	1.045	0.962	0.900	0.851	0.811	0.777	0.749
Value of The Shape Function at The Tenth Points (ϕ)										
1.0L	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
0.9L	0.862	0.856	0.853	0.849	0.847	0.845	0.843	0.841	0.840	0.838
0.8L	0.725	0.714	0.706	0.701	0.696	0.692	0.689	0.685	0.683	0.680
0.7L	0.591	0.575	0.565	0.557	0.551	0.545	0.540	0.536	0.533	0.529
0.6L	0.461	0.443	0.431	0.422	0.415	0.409	0.404	0.400	0.396	0.392
0.5L	0.340	0.321	0.309	0.301	0.294	0.288	0.283	0.279	0.275	0.272
0.4L	0.230	0.214	0.204	0.197	0.191	0.186	0.182	0.179	0.176	0.173
0.3L	0.136	0.125	0.118	0.113	0.109	0.106	0.103	0.101	0.098	0.097
0.2L	0.064	0.058	0.054	0.051	0.049	0.048	0.046	0.045	0.044	0.043
0.1L	0.017	0.015	0.014	0.014	0.013	0.013	0.012	0.012	0.012	0.011
0.0L	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

Stiffness k^* = coeff. $\times \dfrac{EI_{TOP}}{L^3}$ for all values of $\dfrac{I_{BASE}}{L_{TOP}}$

With no added mass: m_{TOP} = mass of top step
Mass m^* = coeff. $\times\ m_{TOP}\ L$

$$\text{Period } T = 2\pi\sqrt{\frac{m^*}{k^*}}$$

L = overall height of tower

E = modulus of elasticity

I_{TOP} = moment of inertia of the top step

When an added mass exists, see paragraph B-2*a*(4) for the calculation of period *T*.

Table B-2
Displacements for Step-Tapered Towers for the Second Shape Function

Second Shape Function										
i_{base} / i_{top}	1	2	3	4	5	6	7	8	9	10
with or without added mass, coeff. of *k**	121.4	137.4	147.3	154.7	160.6	165.6	169.9	173.7	177.1	180.3
when there is no added mass:										
coeff. of *m**	0.250	0.216	0.198	0.187	0.180	0.174	0.169	0.165	0.162	0.159
l_n/m^*	-0.868	-0.939	-0.976	-0.999	-1.015	-1.027	-1.036	-1.043	-1.048	-1.053
coeff. of *t*	0.285	0.249	0.231	0.219	0.210	0.204	0.198	0.194	0.190	0.187
Value Of The Shape Function At The Tenth Points (ϕ)										
1.0l	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
0.9l	0.524	0.545	0.556	0.563	0.568	0.572	0.574	0.577	0.579	0.580
0.8l	0.070	0.117	0.141	0.156	0.167	0.176	0.183	0.188	0.193	0.197
0.7l	-0.317	-0.241	-0.201	-0.176	-0.157	-0.143	-0.131	-0.121	-0.113	-0.106
0.6l	-0.589	-0.483	-0.428	-0.392	-0.366	-0.346	-0.329	-0.316	-0.304	-0.294
0.5l	-0.714	-0.585	-0.520	-0.477	-0.446	-0.422	-0.402	-0.386	-0.372	-0.360
0.4l	-0.683	-0.552	-0.486	-0.443	-0.413	-0.389	-0.370	-0.354	-0.341	-0.329
0.3l	-0.526	-0.416	-0.361	-0.326	-0.301	-0.282	-0.267	-0.255	-0.244	-0.235
0.2l	-0.301	-0.233	-0.200	-0.179	-0.165	-0.153	-0.145	-0.137	-0.131	-0.126
0.1l	-0.093	-0.071	-0.060	-0.054	-0.049	-0.046	-0.043	-0.041	-0.039	-0.037
0.0l	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

Stiffness $k^* = \text{coeff.} \times \dfrac{EI_{TOP}}{l^3}$ for all values of $\dfrac{I_{BASE}}{I_{TOP}}$

with no added mass:
mass $m^* = \text{coeff.} \times m_{top} l$ *mtop* = mass of top step

$$\text{Period } T = 2\pi\sqrt{\frac{m^*}{k^*}}$$

l = overall height of tower

E = modulus of elasticity

I_{top} = moment of inertia of the top step

When an added mass exists, see paragraph B-2*a*(4) for the calculation of period *T*.

Appendix C
Two-Mode Approximate and Computer Solution Methods of Analysis for a Free-Standing Intake Tower

C-1. General

The example problem presented in this appendix illustrates the procedure used to determine the inertial forces, shears, and moments acting on an intake tower when it is subjected to a given design earthquake. Those forces are then used to illustrate the procedure used to select the reinforcement required for shear and bending resistance and the procedure used to check for potential brittle failure modes. The approximate lumped-mass method described in Appendix B is used to determine the inertial forces, shears, and moments for a maximum design earthquake (MDE) acting in the long (longitudinal) direction of the tower. Those results are compared with the results obtained from a structural analysis computer program with dynamic analysis capability. The structural analysis program was also used to obtain results for the MDE in the transverse direction, and to obtain results for the operational basis earthquake (OBE) in both the longitudinal and transverse directions. The results obtained from the structural analysis program are used to design the reinforcement for the tower and to investigate all potential modes of failure. To illustrate the process, reinforcing steel design and failure mode investigations were performed for the MDE and OBE. This example problem does not account for the presence of an access bridge.

C-2. Discretized Lumped-Mass Model

a. A cantilever tower having five steps in size throughout its height is chosen for the example. Figure C-1 is a sketch of such a tower. The tower has a 0.61-m- (2.0-ft-) thick concrete slab at the top and a heavy 1.83-m- (6.0-ft-) thick slab at its base. The unit weight of concrete (γ_{conc}) is equal to 2,402.7 kg/m^3 (150 lb/ft^3).

b. The tower is discretized into 13 lumped masses, as shown in Figure C-2. The masses are lumped at the two ends and middle of each step, except for the top and bottom slabs where the masses are lumped at the top and bottom of the slabs.

c. The tower is investigated for forces, shears, and moments resulting from earthquakes in each principal direction. The results are combined in accordance with the provisions of Chapter 4 and Appendix B to account for multidirectional ground motion effects.

C-3. Design Earthquake

The tower is located in seismic zone 2B in Oregon. Since no site-specific earthquake has been developed for this particular tower, a standard design response spectrum for a rock site is used. Standard spectra are intended to be used only for preliminary structural evaluations when site-specific response spectra are not yet available for the site. The standard design response spectrum representing the MDE, developed using the spectral acceleration maps of ER 1110-2-1806, is shown in Figure C-3. The peak ground accelerations for the MDE and OBE are 0.25g and 0.12g, respectively.

C-4. Inside and Outside Added Hydrodynamic Masses

a. *Approximate method*. The hydrodynamic masses are approximated using the following simplifying assumptions:

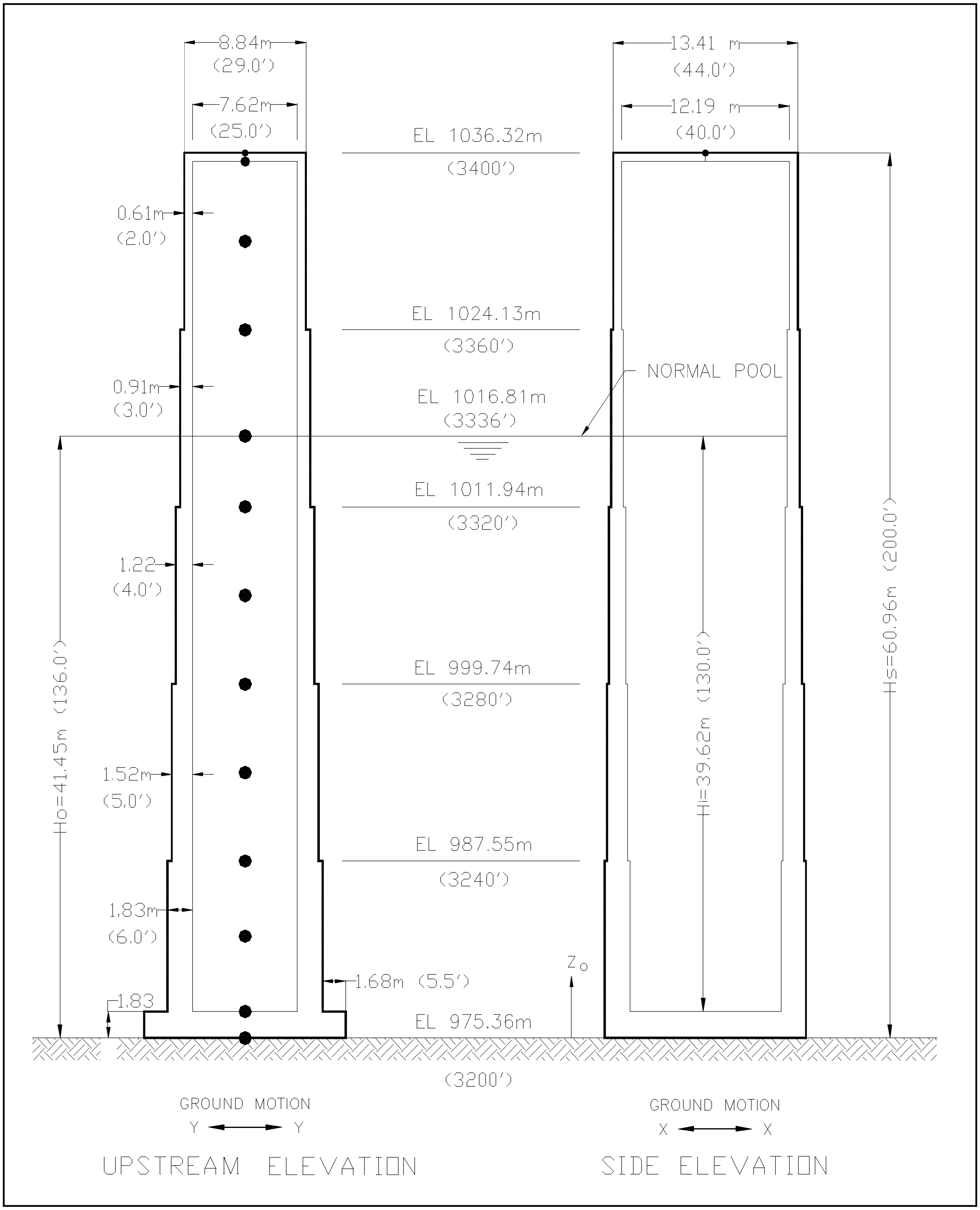


Figure C-1. Tower geometry

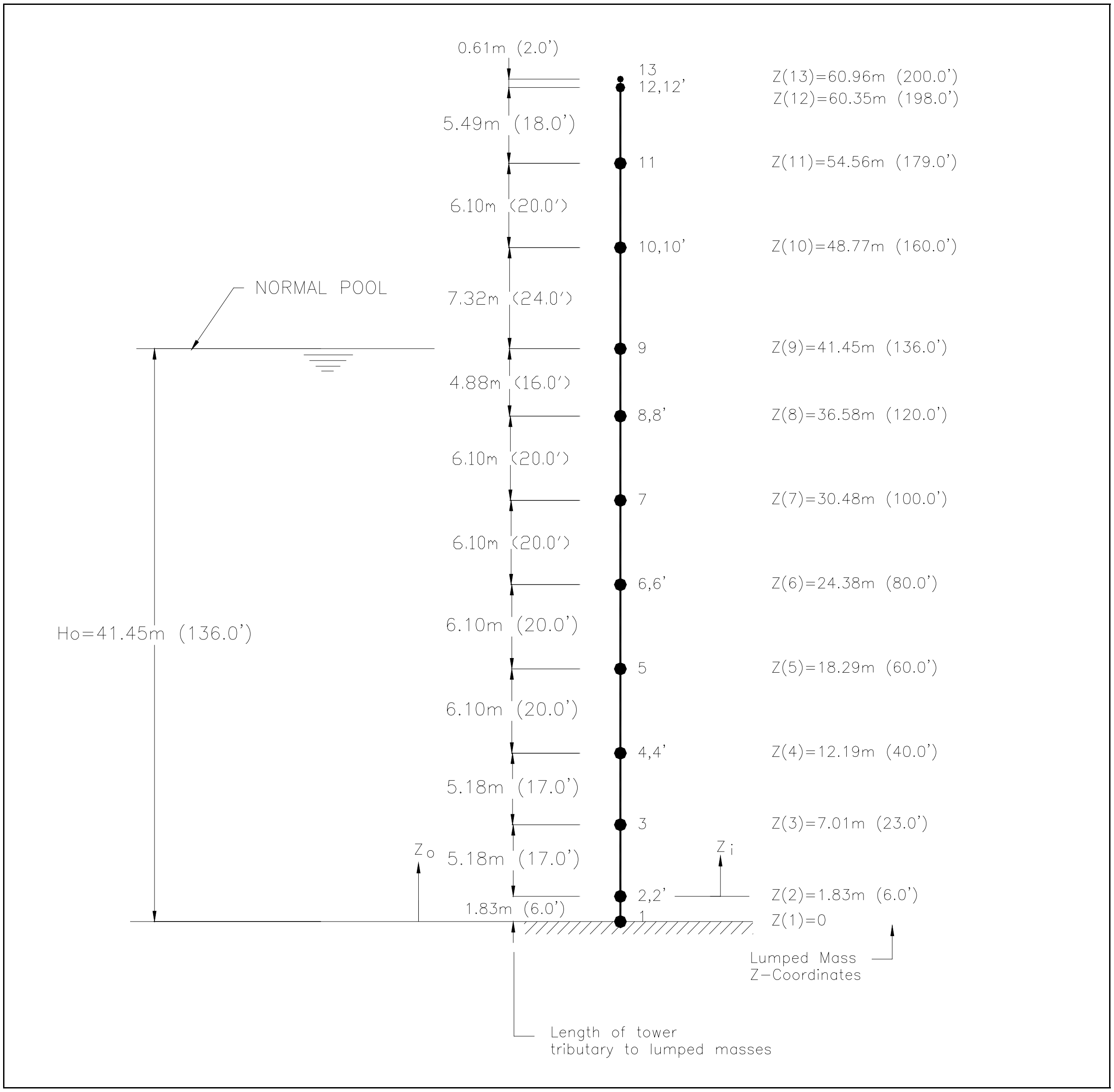


Figure C-2. Tower lumped-mass beam element idealization

(1) Constant average section dimensions such as that between elevations 999.74 m and 1011.94 m are assumed for the entire submerged height of H_o (Figure C-2). Dimensions for these typical sections are shown in Figure C-5.

(2) The area of the tower average section both inside (A_i) and outside (A_o) is converted directly to equivalent circular areas (πr_i^2 and πr_o^2) for the added-mass calculations.

(3) The normalized hydrodynamic added mass $m_\infty^o / \rho_w A_o$ for the infinitely long tower associated with the outside water is obtained from Figure C-6 for the width to depth ratio (a_o/b_o) of the average cross-section.

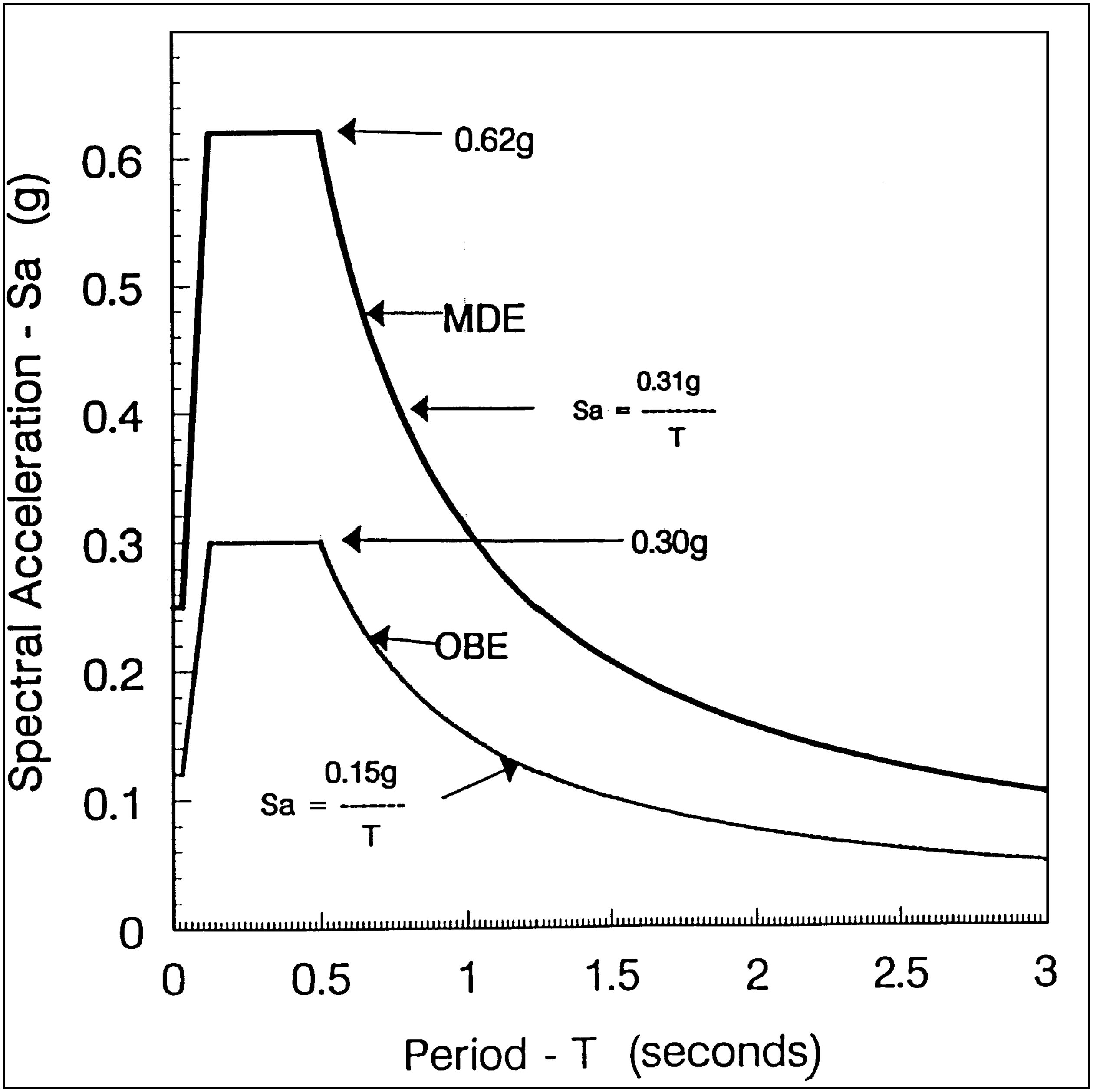


Figure C-3. Standard response spectra for example problem

(4) The normalized added mass is multiplied by $\rho_w A_o$ to obtain the absolute added mass m_∞^o, where ρ_w is the water mass density and A_o is the outside area of the average section.

(5) Knowing r_o/H_o and m_∞^o, the outside added mass m_a^o at any distance z above the base is obtained from Figure C-7. The inside added mass is obtained similarly from Figure C-8, except that no added mass for the infinitely long tower is involved. Tables C-1 and C-2 provide summary calculations for the approximate added hydrodynamic masses for the outside and inside water in transverse direction, respectively.

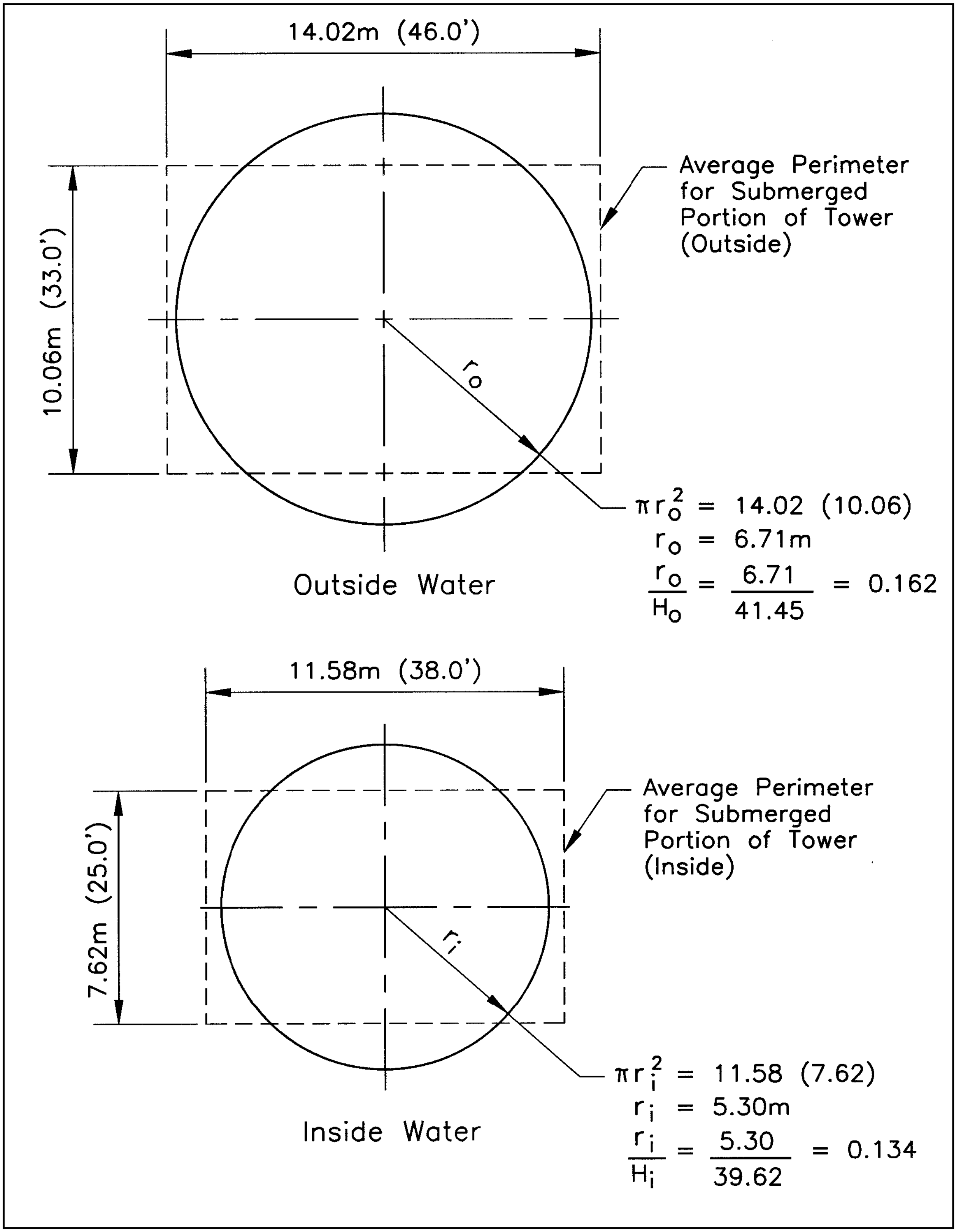


Figure C-4. Added hydrodynamic mass, outside and inside water, circular areas equivalent to average tower dimensions

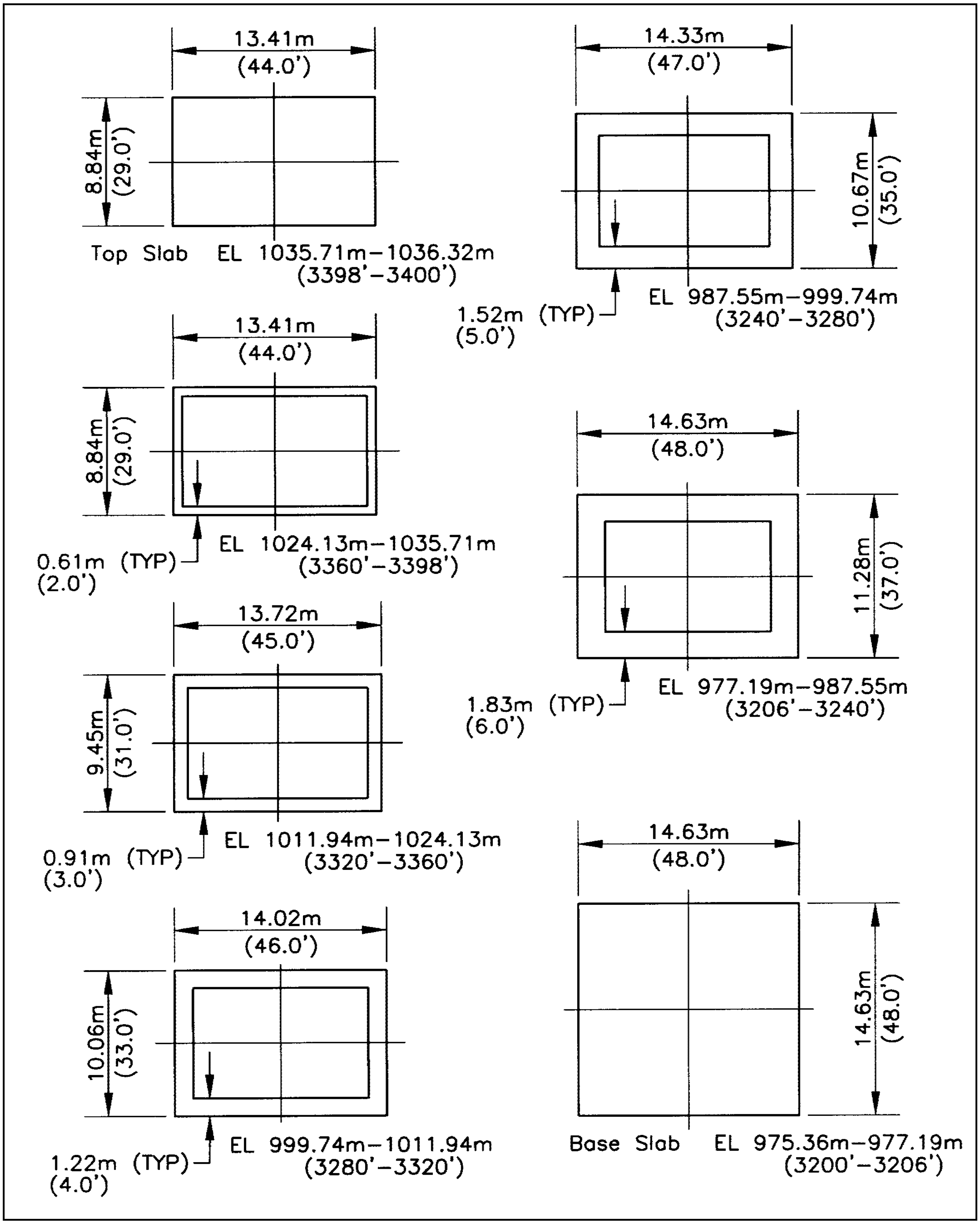


Figure C-5. Tower typical sections and dimensions

a_o/b_o X	$\frac{m_\infty^o}{\rho_w A_o}$ Y
1/5	0.311
1/4	0.377
1/3	0.480
1/2	0.667
2/3	0.853
1	1.186
3/2	1.661
2	2.136
3	3.038
4	3.896
5	4.752

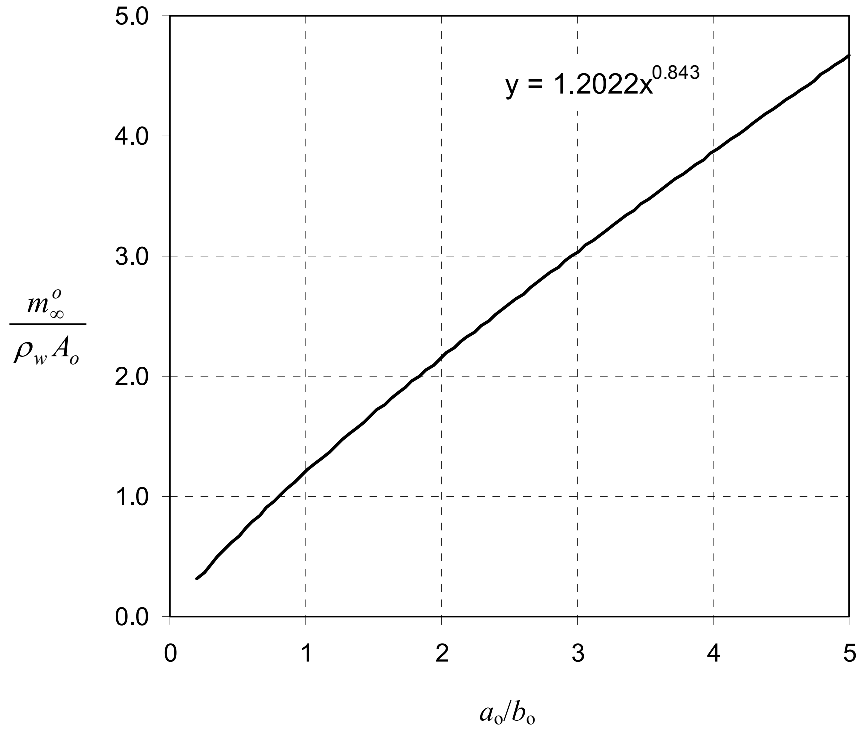


Figure C-6. Added mass m_∞^o for infinitely long tower associated with outside water

b. Refined method.

(1) Refined added-mass analysis is carried out by converting each uniform section of the tower, first into an “equivalent” uniform elliptical section, and then into a corresponding “equivalent” circular section. The calculation steps are generally similar to those described in C-4*a*, except that they are performed separately for each uniform section of the tower and involve conversion to both elliptic and circular sections, as described in Appendix D. Tables C-3 and C-4 provide a summary of the refined added-mass calculations for the outside and inside water, respectively. Figure C-9 provides a comparison between the total (inside + outside) refined and approximate added masses. The results show a close agreement between the refined and approximate added masses for the example tower. The approximate added mass values are therefore used in all succeeding calculations.

(2) Appendix D provides a step-by-step procedure for computation of the refined added-mass of water applied to longitudinal excitation. Note that Tables C-3 and C-4 were developed using the same procedure. However, since locations of lumped masses in this example somewhat differ from those selected in Appendix D (compare Figures C-1 and C-2 with similar figures in Appendix D), the unit added-mass values in Tables C-3 and C-4 do not match the corresponding values for transverse excitation given in the last two columns of Table D-7.

C-5. Structural Mass and Moments of Inertia

a. The area, mass, and gross moments of inertia at each level of discontinuity are now computed, with the results presented in Table C-5. Dimensions are taken from Figure C-5. I_{yy} is the larger moment of inertia for bending in the longer (longitudinal) direction, and I_{xx} is the smaller moment of inertia for bending in the shorter (transverse) direction. Mass m_o is mass/unit length.

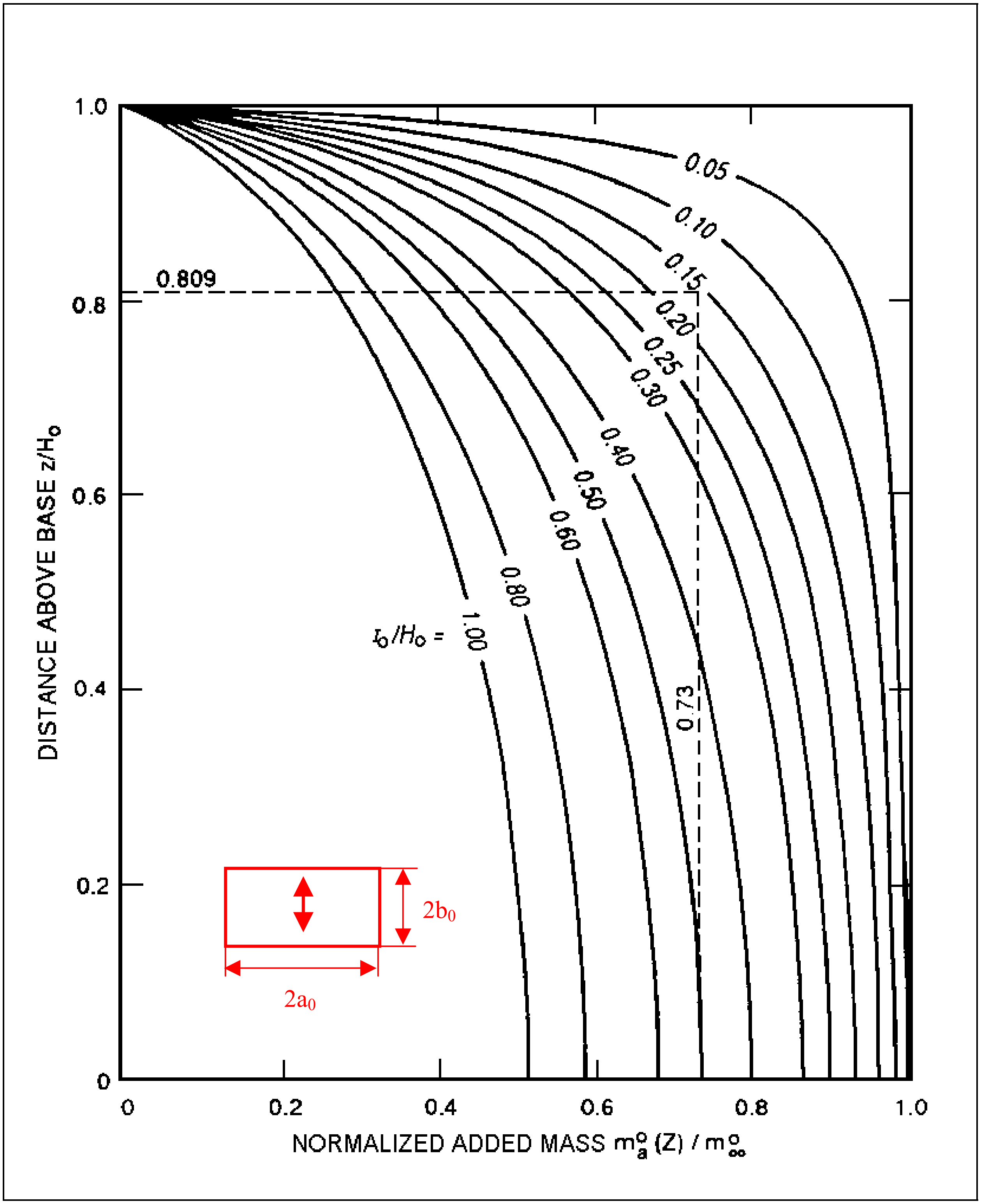


Figure C-7. Normalized outside hydrodynamic added mass

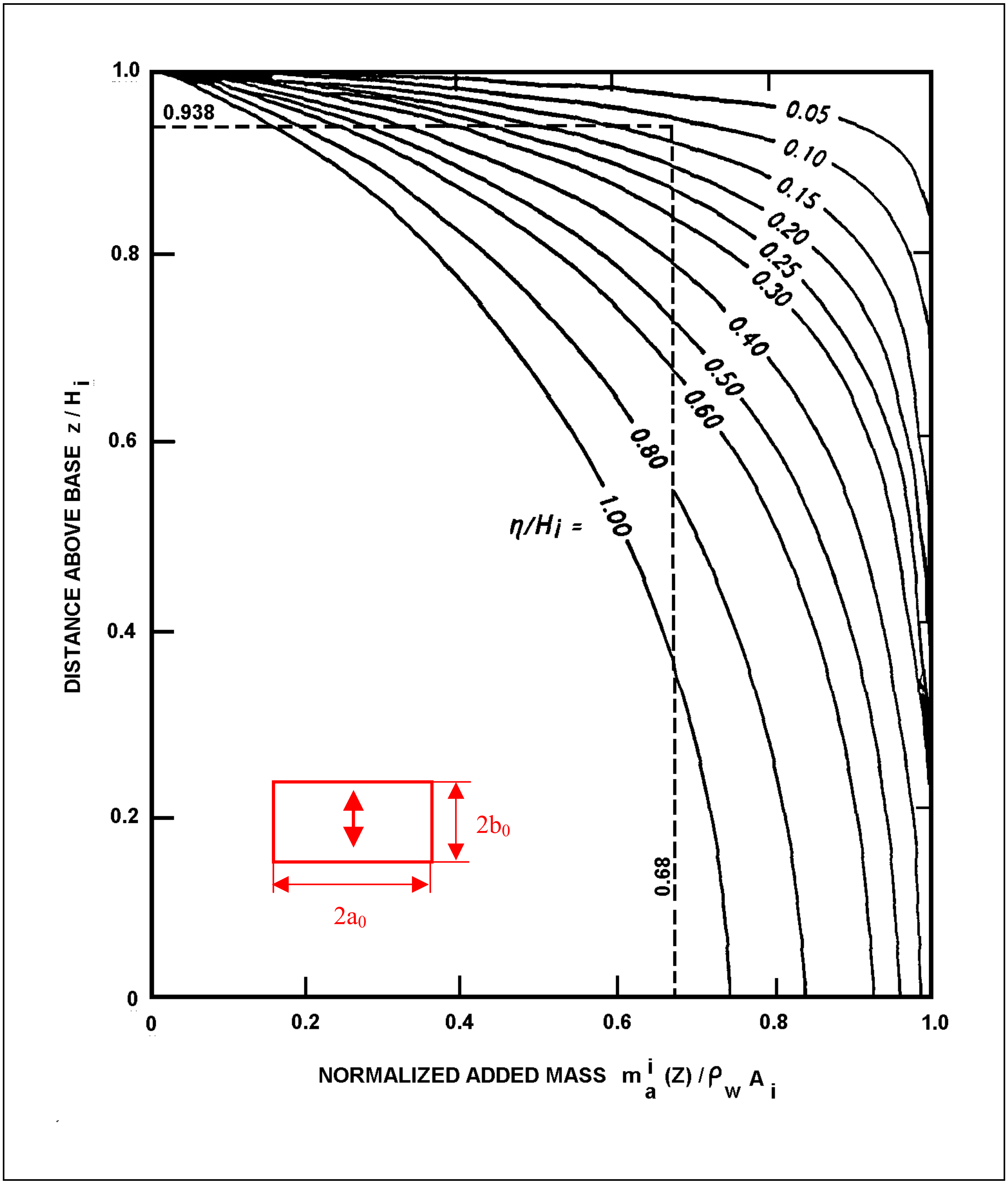


Figure C-8. Normalized inside hydrodynamic added mass

Table C-1
Approximate Added Hydrodynamic Mass Calculations, Outside Water (Transverse Direction)

Node #	Distance from the Base z m (ft)	z / H_0	Elem #	Depth $2a_0$ m (ft)	Width $2b_0$ m (ft)	a_0/b_0	r_0 m (ft)	r_0/H_0	$\frac{m_a^o}{m_\infty^a}$	$\rho_w A_0$ kN-s^2/m^2 (k-s^2/ft^2)	$\frac{m_\infty^a}{\rho_w A_o}$	m_∞^a kN-s^2/m^2 (k-s^2/ft^2)	Length m (ft)	Distributed Hydrodynamic Added Mass m_a^o kN-s^2/m^2 (k-s^2/ft^2)	Lumped Hydrodynamic Added Mass m_a^o kN-s2/m2 (k-s^2/ft^2)
9	41.45 (136)	1.000													327.777 (22.47)
			8	14.02 (46)	10.06 (33)	1.394	6.700 (21.98)	0.366	0.610	141.027 (2.94)	1.563	220.407 (4.60)	4.88 (16)	134.423 (2.81)	
8	36.58 (120)	0.882													867.719 (59.47)
			7	14.02 (46)	10.06 (33)	1.394	6.700 (21.98)	0.366	0.804	141.027 (2.94)	1.563	220.407 (4.60)	6.10 (20)	177.147 (3.70)	
7	30.48 (100)	0.735													1132.322 (77.61)
			6	14.02 (46)	10.06 (33)	1.394	6.700 (21.98)	0.366	0.882	141.027 (2.94)	1.563	220.407 (4.60)	6.10 (20)	194.350 (4.06)	
6	24.38 (80)	0.588													1210.027 (82.94)
			5	14.02 (46)	10.06 (33)	1.394	6.700 (21.98)	0.366	0.919	141.027 (2.94)	1.563	220.407 (4.60)	6.10 (20)	202.641 (4.23)	
5	18.29 (60)	0.441													1248.282 (85.56)
			4	14.02 (46)	10.06 (33)	1.394	6.700 (21.98)	0.366	0.939	141.027 (2.94)	1.563	220.407 (4.60)	6.10 (20)	206.901 (4.32)	
4	12.19 (40)	0.294													1171.452 (80.29)
			3	14.02 (46)	10.06 (33)	1.394	6.700 (21.98)	0.366	0.947	141.027 (2.94)	1.563	220.407 (4.60)	5.18 (17)	208.746 (4.36)	
3	7.01 (23)	0.169													1083.679 (74.28)
			2	14.02 (46)	10.06 (33)	1.394	6.700 (21.98)	0.366	0.951	141.027 (2.94)	1.563	220.407 (4.60)	5.18 (17)	209.534 (4.38)	
2	1.83 (6)	0.044													734.484 (50.34)
			1	14.02 (46)	10.06 (33)	1.394	6.700 (21.98)	0.366	0.951	141.027 (2.94)	1.563	220.407 (4.60)	1.83 (6)	209.578 (4.38)	
1	0.00 (0)	0.000													191.638 (13.14)

Table C-2
Approximate Added Hydrodynamic Mass Calculations, Inside Water (Transverse Direction)

Node #	Distance from the Base z m (ft)	z / H_i	Elem #	Depth $2a_i$ m (ft)	Width $2b_i$ m (ft)	a_i/b_i	r_i m (ft)	r_i/H_i	$\frac{m_a^i}{m_\infty^i}$	m_∞^i kN-s^2/m^2 (k-s^2/ft^2)	Length m (ft)	Distributed Hydrodynamic Added Mass m_a^i kN-s^2/m^2 (k-s^2/ft^2)	Lumped Hydrodynamic Added Mass m_a^i kN-s2/m2 (k-s^2/ft^2)
9	41.45 (136)	1.046											151.735 (10.40)
			8	11.58 (38)	7.62 (25)	1.520	5.300 (17.39)	0.134	0.705	88.258 (1.84)	4.88 (16)	62.227 (1.30)	
8	36.58 (120)	0.923											410.954 (28.17)
			7	11.58 (38)	7.62 (25)	1.520	5.300 (17.39)	0.134	0.964	88.258 (1.84)	6.10 (20)	85.045 (1.78)	
7	30.48 (100)	0.769											526.681 (36.10)
			6	11.58 (38)	7.62 (25)	1.520	5.300 (17.39)	0.134	0.994	88.258 (1.84)	6.10 (20)	87.750 (1.83)	
6	24.38 (80)	0.615											535.931 (36.73)
			5	11.58 (38)	7.62 (25)	1.520	5.300 (17.39)	0.134	0.998	88.258 (1.84)	6.10 (20)	88.080 (1.84)	
5	18.29 (60)	0.462											537.039 (36.81)
			4	11.58 (38)	7.62 (25)	1.520	5.300 (17.39)	0.134	0.998	88.258 (1.84)	6.10 (20)	88.114 (1.84)	
4	12.19 (40)	0.308											496.888 (34.06)
			3	11.58 (38)	7.62 (25)	1.520	5.300 (17.39)	0.134	0.999	88.258 (1.84)	5.18 (17)	88.126 (1.84)	
3	7.01 (23)	0.177											456.620 (31.30)
			2	11.58 (38)	7.62 (25)	1.520	5.300 (17.39)	0.134	0.998	88.258 (1.84)	5.18 (17)	88.121 (1.84)	
2	1.83 (6)	0.046											228.303 (15.65)
			1	0.00 (0)	0.00 (0)	1.000	0.000 (0.00)	0.000	0.000	0.000 (0.00)	1.83 (6)		
1	0.00 (0)	0.000											

Table C-3
Refined Added Hydrodynamic Mass Calculations, Outside Water (Transverse Direction)

Node #	Distance from the Base z m (ft)	z / H_0	Elem #	Depth $2a_0$ m (ft)	Width $2b_0$ m (ft)	a_0/b_0	$\tilde{a}_o$ m (ft)	$\tilde{b}_o$ m (ft)	$\frac{\tilde{a}_o}{H_o}$	r_0/H_0	r_0 m (ft)	$\frac{m_a^o}{m_\infty^o}$	$\rho_w A_0$ kN-s²/m² (k-s²/ft²)	$\frac{m_\infty^o}{\rho_w A_o}$	m_∞^o kN-s²/m² (k-s²/ft²)	Length m (ft)	Distributed Hydrodynamic Added Mass m_a^o kN-s²/m² (k-s²/ft²)	Lumped Hydrodynamic Added Mass m_a^o kN-s2/m2 (k-s ²/ft²)
9	41.45 (136)	1.000																310.283 (21.27)
			8	13.72 (45)	9.45 (31)	1.452	7.738 (25.39)	5.331 (17.49)	0.423	0.372	6.798 (22.30)	0.607	129.600 (2.70)	1.618	209.653 (4.37)	4.88 (16)	127.249 (2.66)	
8	36.58 (120)	0.882																843.909 (57.84)
			7	14.02 (46)	10.06 (33)	1.394	7.910 (25.95)	5.675 (18.62)	0.433	0.385	7.044 (23.11)	0.794	141.027 (2.94)	1.563	220.407 (4.60)	6.10 (20)	175.074 (3.66)	
7	30.48 (100)	0.735																1121.204 (76.85)
			6	14.02 (46)	10.06 (33)	1.394	7.910 (25.95)	5.675 (18.62)	0.433	0.385	7.044 (23.11)	0.875	141.027 (2.94)	1.563	220.407 (4.60)	6.10 (20)	192.775 (4.03)	
6	24.38 (80)	0.588																1229.038 (84.24)
			5	14.33 (47)	10.67 (35)	1.343	8.082 (26.52)	6.019 (19.75)	0.442	0.399	7.290 (23.92)	0.909	152.826 (3.19)	1.514	231.414 (4.83)	6.10 (20)	210.453 (4.40)	
5	18.29 (60)	0.441																1297.771 (88.95)
			4	14.33 (47)	10.67 (35)	1.343	8.082 (26.52)	6.019 (19.75)	0.442	0.399	7.290 (23.92)	0.930	152.826 (3.19)	1.514	231.414 (4.83)	6.10 (20)	215.325 (4.50)	
4	12.19 (40)	0.294																1244.985 (85.33)
			3	14.63 (48)	11.28 (37)	1.297	8.254 (27.08)	6.363 (20.88)	0.451	0.412	7.534 (24.72)	0.936	164.996 (3.44)	1.471	242.675 (5.06)	5.18 (17)	227.217 (4.75)	
3	7.01 (23)	0.169																1179.899 (80.87)
			2	14.63 (48)	11.28 (37)	1.297	8.254 (27.08)	6.363 (20.88)	0.451	0.412	7.534 (24.72)	0.940	164.996 (3.44)	1.471	242.675 (5.06)	5.18 (17)	228.202 (4.77)	
2	1.83 (6)	0.044																806.719 (55.29)
			1	14.63 (48)	14.63 (48)	1.000	8.254 (27.08)	8.254 (27.08)	0.451	0.451	8.248 (27.06)	0.929	214.049 (4.46)	1.185	253.562 (5.29)	1.83 (6)	235.666 (4.92)	
1	0.00 (0)	0.000																215.493 (14.77)

Table C-4
Refined Added Hydrodynamic Mass Calculations, Inside Water (Transverse Direction)

Node #	Distance from the Base z m (ft)	z / H_i	Elem #	Depth $2a_i$ m (ft)	Width $2b_i$ m (ft)	a_i/b_i	$\tilde{a}_i$ m (ft)	$\tilde{b}_i$ m (ft)	$\frac{\tilde{a}_i}{H_i}$	r_i/H_i	r_i m (ft)	$\frac{m_a^i}{m_\infty^i}$	m_∞^i kN-s²/m² (k-s²/ft²)	Length m (ft)	Distributed Hydrodynamic Added Mass m_i kN-s²/m² (k-s²/ft²)	Lumped Hydrodynamic Added Mass m_i kN-s2/m2 (k-s²/ft²)
9	41.45 (136)	1.046														169.505 (11.62)
			8	11.89 (39)	7.62 (25)	1.560	6.707 (22.00)	4.299 (14.10)	0.169	0.110	4.339 (14.24)	0.767	90.580 (1.89)	4.88 (16)	69.515 (1.45)	
8	36.58 (120)	0.923														433.422 (29.71)
			7	11.58 (38)	7.62 (25)	1.520	6.535 (21.44)	4.299 (14.10)	0.165	0.110	4.346 (14.26)	0.981	88.258 (1.84)	6.10 (20)	86.587 (1.81)	
7	30.48 (100)	0.769														532.166 (36.48)
			6	11.58 (38)	7.62 (25)	1.520	6.535 (21.44)	4.299 (14.10)	0.165	0.110	4.346 (14.26)	0.997	88.258 (1.84)	6.10 (20)	88.008 (1.84)	
6	24.38 (80)	0.615														529.744 (36.31)
			5	11.28 (37)	7.62 (25)	1.480	6.363 (20.88)	4.299 (14.10)	0.161	0.110	4.352 (14.28)	0.998	85.935 (1.79)	6.10 (20)	85.792 (1.79)	
5	18.29 (60)	0.462														523.033 (35.85)
			4	11.28 (37)	7.62 (25)	1.480	6.363 (20.88)	4.299 (14.10)	0.161	0.110	4.352 (14.28)	0.999	85.935 (1.79)	6.10 (20)	85.807 (1.79)	
4	12.19 (40)	0.308														477.819 (32.75)
			3	10.97 (36)	7.62 (25)	1.440	6.191 (20.31)	4.299 (14.10)	0.156	0.110	4.357 (14.29)	0.998	83.613 (1.74)	5.18 (17)	83.480 (1.74)	
3	7.01 (23)	0.177														432.590 (29.65)
			2	10.97 (36)	7.62 (25)	1.440	6.191 (20.31)	4.299 (14.10)	0.156	0.110	4.357 (14.29)	0.999	83.613 (1.74)	5.18 (17)	83.491 (1.74)	
2	1.83 (6)	0.046														216.295 (14.83)
			1	0.00 (0)	0.00 (0)	1.000	0.000 (0.00)	0.000 (0.00)	0.000	0.000	0.000 (0.00)	0.000	0.000 (0.00)	1.83 (6)		
1	0.00 (0)	0.000														

C-6. Total Lumped-Mass Values

a. As indicated in Figure C-2, the lumped masses 1 through 13 are positioned at convenient points along the height of the tower. The magnitude of each of the lumped masses can be found by summing the mass of the structure with that of the inside and outside added hydrodynamic masses. The computed results are shown in Table C-6 for the longitudinal direction and in Table C-7 for the transverse direction.

b. Calculations of the total mass for each direction follow the same procedure, except that the values of the hydrodynamic added masses are different. In Tables C-6 and C-7, the structural mass m_0 is the same for each direction. However, the added hydrodynamic masses for the transverse direction are larger than for the longitudinal direction. This is because for the earthquake motions in the transverse direction, the wider face of the tower is accelerated into the surrounding water. Therefore, the added masses are commensurately higher.

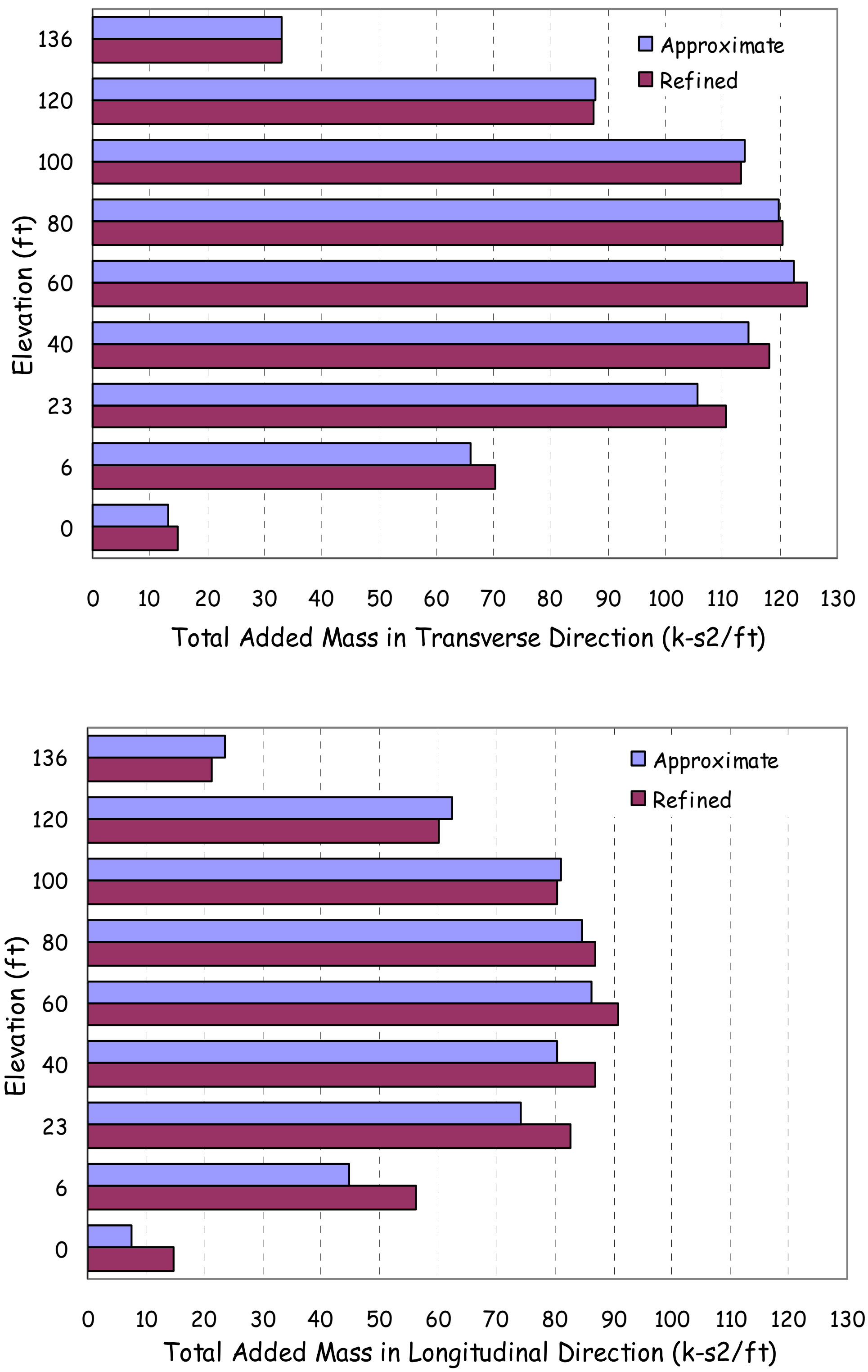


Figure C-9. Comparison of approximate and refined total added hydrodynamic masses

Table C-5
Tower Section Properties and Mass

Node #	Distance from the Base m (ft)	Elem #	Length m (ft)	Section Area m^2 (ft^2)	Ixx m^4 (ft^4)	Iyy m^4 (ft^4)	Distributed Mass Due to Self Weight kN-s^2/m^2 (k-s^2/ft^2)	Mass - m_0 Due to Self Weight kN-s^2/m^2 (k-s^2/ft)
13	60.96 (200)							86.72 (5.94)
		12	0.61 (2)	118.54 (1276)	772 (89,426)	1,777 (205,861)	284.53 (5.94)	
12	60.35 (198)							264.93 (18.16)
		11	5.79 (19)	25.64 (276)	322 (37,343)	626 (72,528)	61.54 (1.29)	
11	54.56 (179)							356.41 (24.43)
		10	5.79 (19)	25.64 (276)	322 (37,343)	626 (72,528)	61.54 (1.29)	
10	48.77 (160)							520.75 (35.69)
		9	7.32 (24)	39.02 (420)	526 (60,935)	965 (111,825)	93.65 (1.96)	
9	41.45 (136)							570.91 (39.13)
		8	4.88 (16)	39.02 (420)	526 (60,935)	965 (111,825)	93.65 (1.96)	
8	36.58 (120)							614.41 (42.11)
		7	6.10 (20)	52.77 (568)	762 (88,279)	1,324 (153,357)	126.65 (2.65)	
7	30.48 (100)							772.09 (52.92)
		6	6.10 (20)	52.77 (568)	762 (88,279)	1,324 (153,357)	126.65 (2.65)	
6	24.38 (80)							875.39 (60.00)
		5	6.10 (20)	66.89 (720)	1,034 (119,750)	1,703 (197,290)	160.55 (3.35)	
5	18.29 (60)							978.70 (67.08)
		4	6.10 (20)	66.89 (720)	1,034 (119,750)	1,703 (197,290)	160.55 (3.35)	
4	12.19 (40)							995.42 (68.23)
		3	5.18 (17)	81.38 (876)	1,344 (155,737)	2,104 (243,792)	195.33 (4.08)	
3	7.01 (23)							1012.14 (69.37)
		2	5.18 (17)	81.38 (876)	1,344 (155,737)	2,104 (243,792)	195.33 (4.08)	
2	1.83 (6)							975.85 (66.89)
		1	1.83 (6)	214.05 (2304)	3,818 (442,368)	3,818 (442,368)	513.75 (10.73)	
1	0.00 (0)							469.78 (32.20)

C-7. Inertial Forces by the Approximate Lumped-Mass Method (Longitudinal Direction)

a. The physical constants computed in paragraphs C-5 and C-6 are now used to compute the dynamic response of the tower to earthquake excitation. The dynamic analysis follows five steps:

(1) Using the moments of inertia, determine the normalized displacements of the first and second shape functions.

(2) Using the normalized displacements and the physical constants, determine the first and second natural periods of vibration and the mass participation factors. Compute natural periods using the effective moments of inertia. For the MDE excitation, the effective moment of inertia is taken equal to 80 percent of the gross moments of inertia. For the OBE excitation, the effective moment of inertia is taken the same as the gross values.

(3) Using these natural periods and the standard response spectrum, determine the first and second spectral accelerations.

Table C-6
Total Mass for Earthquake Motions in Longitudinal Direction

Node #	Distance from the Base m (ft)	Mass - m_0 Due to Self Weight $kN\text{-}s^2/m$ ($k\text{-}s^2/ft$)	Inside Hydrodynamic Added Mass Longitudinal Dir. $kN\text{-}s^2/m$ ($k\text{-}s^2/ft$)	Outside Hydrodynamic Added Mass Longitudinal Dir. $kN\text{-}s^2/m$ ($k\text{-}s^2/ft$)	Total Lumped Mass Longitudinal Dir. $kN\text{-}s^2/m$ ($k\text{-}s^2/ft$)
13	60.96 (200)	86.72 (5.94)			86.72 (5.94)
12	60.35 (198)	264.93 (18.16)			264.93 (18.16)
11	54.56 (179)	356.41 (24.43)			356.41 (24.43)
10	48.77 (160)	520.75 (35.69)			520.75 (35.69)
9	41.45 (136)	570.91 (39.13)	138.27 (9.48)	173.33 (11.88)	882.51 (60.49)
8	36.58 (120)	614.41 (42.11)	389.32 (26.68)	487.26 (33.40)	1490.98 (102.19)
7	30.48 (100)	772.09 (52.92)	516.45 (35.40)	657.68 (45.08)	1946.22 (133.40)
6	24.38 (80)	875.39 (60.00)	526.45 (36.08)	741.40 (50.82)	2143.24 (146.90)
5	18.29 (60)	978.70 (67.08)	522.49 (35.81)	803.83 (55.10)	2305.02 (157.99)
4	12.19 (40)	995.42 (68.23)	477.73 (32.74)	791.49 (54.25)	2264.64 (155.22)
3	7.01 (23)	1012.14 (69.37)	432.58 (29.65)	772.18 (52.93)	2216.90 (151.95)
2	1.83 (6)	975.85 (66.89)	216.30 (14.83)	602.36 (41.29)	1794.50 (123.00)
1	0.00 (0)	469.78 (32.20)		215.49 (14.77)	685.27 (46.97)

(4) Using the normalization ratios and the spectral acceleration of each lumped mass, determine the inertia forces at each lumped mass, as well as the corresponding shears and moments at those points with computations including both shape functions.

(5) Combine the two sets of forces, shears, and moments to obtain the final design values.

b. For the example tower, the foregoing steps are presented in sequence in the following discussions.

(1) The first step in the approximate analysis is to determine the first and second shape functions and their normalized displacements. Figure C-10 shows plots of the first and second shape functions interpolated from the normalized shape function values given in Tables B-1 and B-2. The ratio of the moment of inertia of the base to that of the top step I_{BASE}/I_{TOP} is computed using the earlier calculations for moment of inertia. The section just above the base is used for the base moment of inertia, and the section just below the top slab is used for the top moment of inertia. Using the moments of inertia for the stiff bottom and top slabs would introduce additional inaccuracies because the bottom and top slabs are not representative of the stiffness of the rest of the tower. Also, it should be remembered that the stepped taper assumed in creating the tables may or may not agree exactly with the actual stepped taper. For these reasons, the shape functions are considered approximate. The following calculations illustrate the use of the two-mode approximate method for the longitudinal MDE excitation. Similar calculations are required for the transverse MDE and for both the longitudinal and transverse OBE excitations.

$$\frac{I_{BASE}}{I_{TOP}} = \frac{2096.61}{623.74} = 3.361 \tag{C-1}$$

Values of the interpolated normalized displacements are listed in Figure C-10 for the appropriate values of I_{BASE}/I_{TOP}.

Table C-7
Total Mass for Earthquake Motions in Transverse Direction

Node #	Distance from the Base m (ft)	Mass - m_0 Due to Self Weight kN-s²/m (k-s²/ft)	Inside Hydrodynamic Added Mass Transverse Dir. kN-s²/m (k-s²/ft)	Outside Hydrodynamic Added Mass Transverse Dir. kN-s²/m (k-s²/ft)	Total Lumped Mass Transverse Dir. kN-s²/m (k-s²/ft)
13	60.96 (200)	86.72 (5.94)			86.72 (5.94)
12	60.35 (198)	264.93 (18.16)			264.93 (18.16)
11	54.56 (179)	356.41 (24.43)			356.41 (24.43)
10	48.77 (160)	520.75 (35.69)			520.75 (35.69)
9	41.45 (136)	570.91 (39.13)	169.51 (11.62)	310.28 (21.27)	1050.70 (72.02)
8	36.58 (120)	614.41 (42.11)	433.42 (29.71)	843.91 (57.84)	1891.74 (129.66)
7	30.48 (100)	772.09 (52.92)	532.17 (36.48)	1121.20 (76.85)	2425.46 (166.24)
6	24.38 (80)	875.39 (60.00)	529.74 (36.31)	1229.04 (84.24)	2634.18 (180.55)
5	18.29 (60)	978.70 (67.08)	523.03 (35.85)	1297.77 (88.95)	2799.50 (191.88)
4	12.19 (40)	995.42 (68.23)	477.82 (32.75)	1244.98 (85.33)	2718.22 (186.31)
3	7.01 (23)	1012.14 (69.37)	432.59 (29.65)	1179.90 (80.87)	2624.63 (179.89)
2	1.83 (6)	975.85 (66.89)	216.30 (14.83)	806.72 (55.29)	1998.86 (137.00)
1	0.00 (0)	469.78 (32.20)		215.49 (14.77)	685.27 (46.97)

(2) The second step in the analysis is to determine the first and second natural periods of vibration. If there were no added mass of water associated with the tower, the natural periods could be obtained directly from Figure C-11. Nevertheless, it is good practice to compute these periods for reference and comparison. Figure C-11 is entered with a parameter:

$$\sqrt{\frac{m_{\mathrm{TOP}}\mathrm{L}^4}{\mathrm{EI}_{\mathrm{TOP}}}} = \sqrt{\frac{61.54 \times 60.96^4}{21{,}525{,}000 \times 626 \times 0.8}} = 0.281 \qquad \text{(C-2)}$$

where

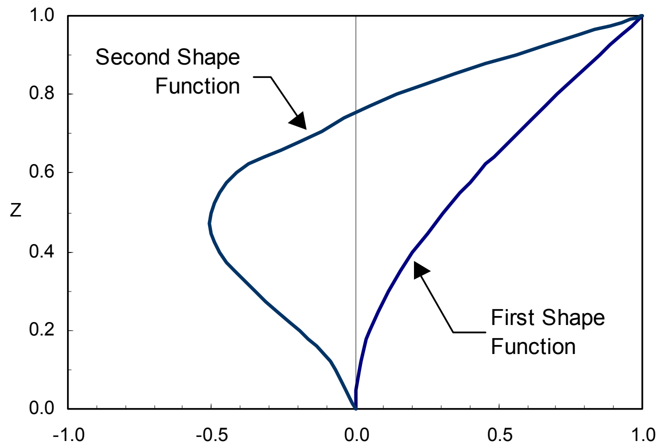
m_{top} = mass of top step

L = overall height of tower

E = modulus of elasticity

For this value of the parameter and $(I_{\mathrm{BASE}}/I_{\mathrm{TOP}}) = 3.361$, Figure C-11 results in a first period of $T_1 = 0.32$ sec, and a second period of $T_2 = 0.06$ sec for a case with no added mass.

(a) Since the example tower carries added mass, the actual calculations for the natural periods must be made using calculated values for the stiffness k^* and the actual effective mass m^*. The stiffness k^* is independent of mass and is determined from the coefficients given in Tables B-1 and B-2. The coefficients of k^* are interpolated for a value of $(I_{\mathrm{BASE}}/I_{\mathrm{TOP}}) = 3.361$.

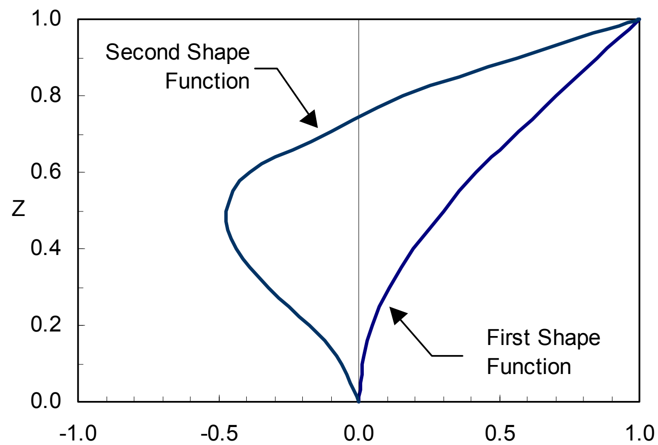


Displacement ϕ

$$I_{BASE}/I_{TOP} = 3.361$$

Longitudinal Directon

Z	$\phi 1$	$\phi 2$
1.00	1.000	1.000
0.99	0.985	0.956
0.90	0.852	0.559
0.80	0.704	0.146
0.68	0.538	-0.191
0.60	0.428	-0.415
0.50	0.306	-0.504
0.40	0.201	-0.470
0.30	0.116	-0.348
0.20	0.053	-0.192
0.12	0.022	-0.085
0.03	0.006	-0.021
0.00	0.000	0.000



Displacement ϕ

$$I_{BASE}/I_{TOP} = 4.170$$

Transverse Direction

Z	$\phi 1$	$\phi 2$
1.00	1.000	1.000
0.99	0.985	0.956
0.90	0.849	0.564
0.80	0.700	0.158
0.68	0.533	-0.170
0.60	0.421	-0.388
0.50	0.300	-0.472
0.40	0.196	-0.438
0.30	0.112	-0.322
0.20	0.051	-0.177
0.12	0.021	-0.078
0.03	0.005	-0.020
0.00	0.000	0.000

Figure C-10. Approximate shape functions

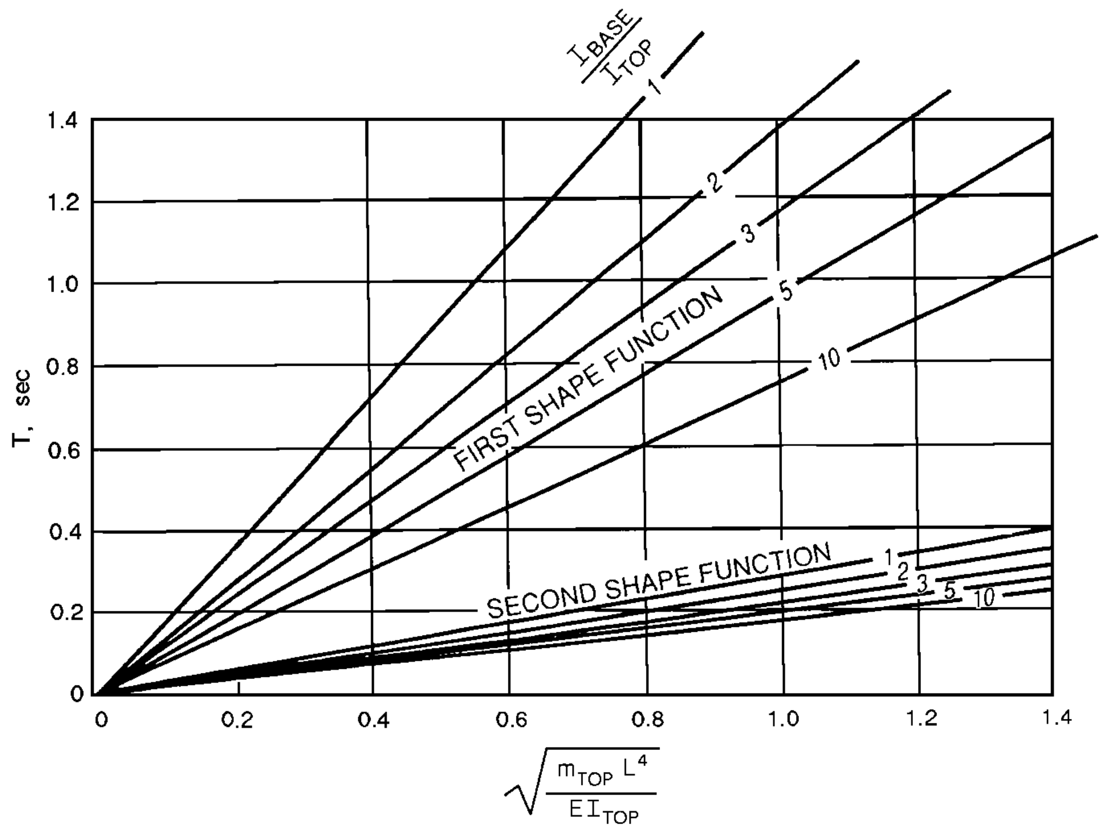


Figure C-11. Approximate natural periods of intake towers having no added mass

- For the first shape function:

$$k^* = 8.042 \frac{EI_{\text{TOP}}}{L^3}$$

$$= \frac{8.042 \times 21{,}525{,}000 \times 626 \times 0.8}{60.69^3}$$

$$= 387{,}811 \text{ kN/m} \, (26{,}566 \text{ kips/ft})$$

- For the second shape function:

$$k^* = 149.97 \frac{EI_{\text{TOP}}}{L^3} = 7{,}232{,}034 \text{ kN/m} \, (495{,}413 \text{ kips/ft})$$

The effective masses m^* and the normalization ratio L_n/m^* are computed using the total lumped mass $M(z)$ at each discretized point.

(b) The values of L_n and m* are defined mathematically as

$$L_n = \Sigma M(z)\phi(z)$$

$$m^* = \Sigma M(z)\phi(z)^2$$

where z is the height of the lumped mass $M(z)$ above the base and $\phi(z)$ is shape function, as defined in Figure C-10. Tables C-8 and C-9 provide the values of L_n and m^* for the two shape functions. Values of $\phi(z)$ are obtained from the values given in Figure C-10.

Table C-8
***L_n*, *M^**, Forces, Shears, and Moments for Longitudinal MDE Excitation, First Shape Function**

Node #	Height h_j m (ft)	Mode Shape ϕ_{j1}	Lumped Mass m_j kN-s²/m (k-s²/ft)	ϕ_{j1} X m_j kN-s²/m (k-s²/ft)	ϕ_{j1}^2X m_j kN-s²/m (k-s²/ft)	Lateral Displacement ux_{j1} mm (in)	Elastic Force fx_{j1} kN (kips)	Shear Vx_{j1} kN (kips)	h_j X $f_{j,1}$ kN-m (k-ft)	Moment My_{j1} kN-m (k-ft)
13	60.96 (200)	1.000	86.72 (5.94)	86.724 (5.94)	86.724 (5.94)	56.656 (2.23)	1,114 (251)	1,114 (251)	67926.486 (50152.41)	0 (0)
12	60.35 (198)	0.985	264.93 (18.16)	261.008 (17.89)	257.145 (17.62)	55.817 (2.20)	3,354 (755)	4,468 (1,005)	202390.398 (149431.63)	679 (502)
11	54.56 (179)	0.852	356.41 (24.43)	303.662 (20.81)	258.720 (17.73)	48.271 (1.90)	3,902 (878)	8,369 (1,884)	212869.669 (157168.83)	26,554 (19,605)
10	48.77 (160)	0.704	520.75 (35.69)	366.608 (25.13)	258.092 (17.69)	39.886 (1.57)	4,710 (1060)	13,080 (2,944)	229717.068 (169607.83)	75,023 (55,392)
9	41.45 (136)	0.538	911.98 (62.51)	491.008 (33.65)	264.359 (18.12)	30.503 (1.20)	6,309 (1420)	19,389 (4,363)	261516.277 (193086.25)	170,705 (126,037)
8	36.58 (120)	0.428	1526.57 (104.63)	653.371 (44.78)	279.643 (19.17)	24.249 (0.96)	8,395 (1889)	27,784 (6,253)	307051.911 (226706.74)	265,260 (195,850)
7	30.48 (100)	0.306	1952.82 (133.85)	597.562 (40.96)	182.854 (12.53)	17.337 (0.68)	7,678 (1728)	35,461 (7,980)	234020.543 (172785.23)	434,628 (320,901)
6	24.38 (80)	0.201	2110.27 (144.64)	424.164 (29.07)	85.257 (5.84)	11.388 (0.45)	5,450 (1226)	40,911 (9,207)	132890.686 (98117.66)	650,801 (480,508)
5	18.29 (60)	0.116	2236.77 (153.31)	259.466 (17.78)	30.098 (2.06)	6.572 (0.26)	3,334 (750)	44,245 (9,957)	60968.020 (45014.74)	900,197 (664,645)
4	12.19 (40)	0.053	2168.97 (148.66)	114.956 (7.88)	6.093 (0.42)	3.003 (0.12)	1,477 (332)	45,722 (10,289)	18007.814 (13295.77)	1,169,915 (863,787)
3	7.01 (23)	0.022	2094.71 (143.57)	46.084 (3.16)	1.014 (0.07)	1.246 (0.05)	592 (133)	46,314 (10,423)	4150.930 (3064.77)	1,406,829 (1,038,709)
2	1.83 (6)	0.006	1628.41 (111.61)	8.956 (0.61)	0.049 (0.00)	0.312 (0.01)	115 (26)	46,429 (10,449)	210.450 (155.38)	1,646,810 (1,215,895)
1	0.00 (0)	0.000	580.47 (39.79)	0.000 (0.00)	0.000 (0.00)	0.000 (0.00)	0 (0)	46,429 (10,449)	0.000 (0.00)	1,731,720 (1,278,587)

NOTES:
$L_1 = \Sigma\phi_{j1} X m_j =$ 247.676 (k-s²/ft) 3613.567 (kN-s²/m)
$M_1 = \Sigma\phi_{j1}^2 X m_j =$ 117.208 (k-s²/ft) 1710.047 (kN-s²/m)
$L_1 / M_1 =$ 2.113
$\omega_1 = 2\pi / T_1 =$ 15.059 rad/sec ($T_1 =$ 0.42 sec)
S_a(MDE) = 0.620 g = 19.96 ft/sec² 6.080 m/sec²

Table C-9
***L_n*, *m**, Forces, Shears, and Moments for Longitudinal MDE Excitation, Second Shape Function**

Node #	Height h_j m (ft)	Mode Shape ϕ_{j2}	Lumped Mass m_j kN-s²/m (k-s²/ft)	ϕ_{j2} X m_j kN-s²/m (k-s²/ft)	ϕ_{j2}^2X m_j kN-s²/m (k-s²/ft)	Displacement ux_{j2} mm (in)	Elastic Force fx_{j2} kN (kips)	Shear Vx_{j2} kN (kips)	h_j X $f_{j,2}$ MDE kN-m (k-ft)	Moment My_{j2} kN-m (k-ft)
13	60.96 (200)	1.000	86.72 (5.94)	86.724 (5.94)	86.724 (5.94)	-3.006 -(0.12)	-908 -(204)	-908 -(204)	-55359.368 -(40873.68)	0 (0)
12	60.35 (198)	0.956	264.93 (18.16)	253.246 (17.36)	242.077 (16.59)	-2.874 -(0.11)	-2,652 -(597)	-3,560 -(801)	-160040.515 -(118163.29)	-554 -(409)
11	54.56 (179)	0.559	356.41 (24.43)	199.233 (13.66)	111.371 (7.63)	-1.680 -(0.07)	-2,086 -(470)	-5,646 -(1,271)	-113825.083 -(84040.88)	-21,170 -(15,631)
10	48.77 (160)	0.146	520.75 (35.69)	76.030 (5.21)	11.100 (0.76)	-0.439 -(0.02)	-796 -(179)	-6,442 -(1,450)	-38826.249 -(28666.72)	-53,869 -(39,773)
9	41.45 (136)	-0.191	911.98 (62.51)	-173.823 -(11.91)	33.131 (2.27)	0.573 (0.02)	1,820 (410)	-4,622 -(1,040)	75451.616 (55708.46)	-100,996 -(74,569)
8	36.58 (120)	-0.415	1526.57 (104.63)	-633.525 -(43.42)	262.913 (18.02)	1.248 (0.05)	6,634 (1493)	2,012 (453)	242643.192 (179151.62)	-123,538 -(91,212)
7	30.48 (100)	-0.504	1952.82 (133.85)	-984.220 -(67.46)	496.047 (34.00)	1.515 (0.06)	10,306 (2319)	12,318 (2,772)	314134.091 (231935.75)	-111,274 -(82,157)
6	24.38 (80)	-0.470	2110.27 (144.64)	-991.826 -(67.98)	466.158 (31.95)	1.413 (0.06)	10,386 (2337)	22,704 (5,109)	253249.340 (186982.49)	-36,184 -(26,716)
5	18.29 (60)	-0.348	2236.77 (153.31)	-778.397 -(53.35)	270.882 (18.57)	1.046 (0.04)	8,151 (1834)	30,855 (6,944)	149064.876 (110059.60)	102,219 (75,472)
4	12.19 (40)	-0.192	2168.97 (148.66)	-416.443 -(28.54)	79.957 (5.48)	0.577 (0.02)	4,361 (981)	35,216 (7,925)	53166.531 (39254.64)	290,310 (214,346)
3	7.01 (23)	-0.085	2094.71 (143.57)	-178.050 -(12.20)	15.134 (1.04)	0.256 (0.01)	1,864 (420)	37,080 (8,345)	13070.542 (9650.42)	472,783 (349,072)
2	1.83 (6)	-0.021	1628.41 (111.61)	-34.604 -(2.37)	0.735 (0.05)	0.064 (0.00)	362 (82)	37,442 (8,426)	662.669 (489.27)	664,917 (490,931)
1	0.00 (0)	0.000	580.47 (39.79)	0.000 (0.00)	0.000 (0.00)	0.000 (0.00)	0 (0)	37,442 (8,426)	0.000 (0.00)	733,392 (541,488)

NOTES:

$L_2 = \Sigma\phi_{j2} X m_j =$ -245.078 (k-s²/ft) -3575.655 (kN-s²/m)

$M_2 = \Sigma\phi_{j2}^2 X m_j =$ 142.306 (k-s²/ft) 2076.230 (kN-s²/m)

$L_2 / M_2 =$ -1.722

$\omega_2 = 2\pi / T_2 =$ 59.019 rad/sec ($T_2 =$ 0.106 sec.)

S_a(MDE) = 0.620 g = 19.96 ft/sec² 6.080 m/sec²

(c) The natural periods are calculated using the values of k^* and m^* defined above, where :

$$T = 2\pi\sqrt{m^* / k^*}$$

- First Shape Function

$$T_1 = 2\pi\sqrt{\frac{1{,}710.047}{387{,}811}}$$
$$T_1 = 0.42 \text{ sec}$$
$$\omega_1 = 14.96\ rad / \text{sec}$$

- Second Shape Function

$$T_2 = 2\pi\sqrt{\frac{2{,}076.23}{7{,}232{,}034}}$$
$$T_2 = 0.106 \text{ sec}$$
$$\omega_2 = 59.275\ rad / \text{sec}$$

where ω is the natural frequency.

(d) The mass participation factors are similarly calculated using the sums defined above.

- First Shape Function

$$\frac{L_1}{m^*} = \frac{3{,}631.567}{1{,}710.047}$$
$$= 2.113$$

- Second Shape Function

$$\frac{L_2}{m^*} = \frac{-3{,}575.655}{2{,}076.230}$$
$$= -1.722$$

(3) The third step in the analysis is to determine the first and second spectral accelerations S_A. Given the two natural periods calculated in the preceding section the corresponding spectral accelerations are determined using the standard response spectrum of Figure C-3.

- First Shape Function

$T_1 = 0.42$
$S_A = 0.62\text{g} = 6.08 \text{ m/sec}^2$

- Second Shape Function

$T_2 = 0.106$
$S_A = 0.62\text{g} = 6.08 \text{ m/sec}^2$

The absolute displacement of the top mass in the longitudinal direction is computed as

$$\Delta_{TOP} = \frac{L_n}{m^*} \times \frac{S_A}{\omega^2}$$

- First Shape Function

Δ_{TOP} = 57.40 mm (2.26 in)

- Second Shape Function

Δ_{TOP} = 2.98 mm (0.12 in)

(4) The fourth step in the analysis is to determine the forces, shears, and moments at the level of each lumped mass. The forces acting on each mass consist of longitudinal and transverse forces from each of the two shape functions, producing four sets of forces, shears, and moments. Tables C-8 and C-9 summarize the computations of these values for the longitudinal direction. The lateral force F_n acting on any mass n is computed by:

$$F_n = (L_n/m^*)\, S_{An}\, M_n\, \phi_n$$

The shear at any level n is the sum of forces F_n above that level. The moment at any level n is given by

$$V_n = \sum_{i=n}^{L} F_i$$

$$M_n = M_{n+1} + V_{n+1} \times \Delta h$$

where Δh = height of mass M_{n+1} - height of mass M_n

(5) The fifth step in the analysis is to combine the shears and moments for the two shape functions into a single resultant. In this example, the combination is computed using the square-root-of-the-sum-of-the-squares (SRSS) method. The absolute displacement of the top mass, for the combined first and second shape functions is

Δ_{TOP} = 57.49 mm (2.27 in)

The base shears V_b and base moments M_b are, for the first and second shape functions combined

V_b = 59,646 kN (13,423 kips)
M_b = 1,880,616 kN-m (1,388,522 kip-ft)

C-8. Comparison of Results From Two-mode Approximate Method With Those From Intake Tower Analysis Program (ITAP)

a. Table C-10 compares results for the MDE longitudinal excitation from the two-mode approximate procedure with those obtained from the Intake Tower Analysis Program (Quest Structures 2000). The ITAP model of the example tower consisted of 12 beam elements with shear deformation, as shown in Figure C-2. The first 10 modes of vibration and 5 percent modal damping were used to compute the dynamic response. Some of the unique capabilities of the program ITAP developed by QUEST Structures for Structures Laboratory at the U.S. Army Engineer Research and Development Center include the following:

Table C-10
Approximate 2-Mode Method versus Intake Tower Analysis Program (ITAP) -- MDE Longitudinal Excitation

Item	2-Mode Approximate Method	Multi-mode Computer Solution
Period of first mode of vibration	0.42 sec	0.45 sec
Period of second mode of vibration	0.106 sec	0.134 sec
Top displacement	57.49 mm (2.27 in)	62.43 mm (2.46 in)
Base shear	59,646 kN (13,423 kips)	57,855 kN (13,020 kips)
Base moment	1,880,616 kN-m (1,388,522 kip-ft)	1,783,747 kN-m (1,317,000 kip-ft)

- Automatic computation of section properties for solid and hollow sections from limited input geometry data.
- Automatic evaluation of the inside and outside added-mass of water from input water levels.
- Static, response spectrum, and time history response analyses.
- Pre- and post-processor capabilities with graphical presentation of the model and results.

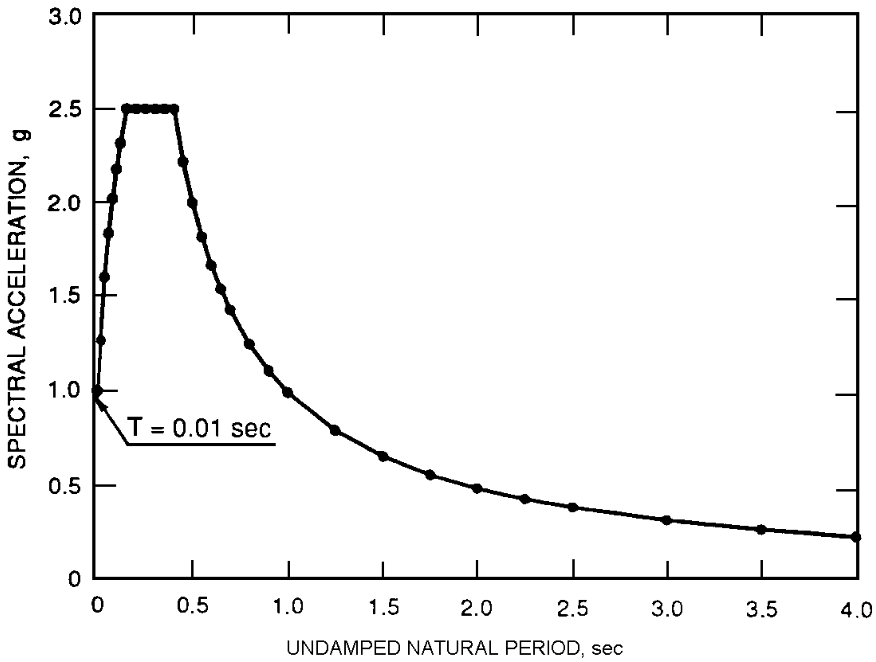
These unique capabilities listed above make ITAP a more cost-effective and efficient program. It runs with simple input data requiring the least amount of input data, yet automatically produces all pertinent results needed for performance evaluation of the tower. Generally, input for computer analysis includes joint designations and coordinates with member designations and properties. In addition, joint masses and the seismic input in the form of response spectrum (Figure C-12) or acceleration time-histories are needed for the evaluation of earthquake response. Input to ITAP for the longitudinal MDE Excitation is shown in Figure C-13.

b. The results from the two-mode approximate procedure are generally in good agreement with the results from the computer solution. The periods and displacements for the approximate method are smaller than those for the computer solution, but the base shear and bending moment are slightly higher (Table C-10). The complete results for the MDE and OBE excitations (Figures C-14 and C-15) show slightly higher base shear and moments for the approximate method than the computer solution, but displacements may be higher or lower. Overall, the two-mode approximation provides reasonable results for the example tower. In other situations, with significant irregularities, a computer solution should be considered for the final design.

c. The computer analysis results for both directions of earthquake ground motion and for both the MDE and OBE are presented in Tables C-11 and C-12. These results will be used to demonstrate the methods for calculating reinforcing steel requirements and for investigating all potential modes of failure.

C-9. Intake Tower Reinforced Concrete Design - General

a. According to ER 1110-2-1806, hydraulic structures should have adequate earthquake resistance to sustain the MDE without collapsing and to survive the OBE with only cosmetic damage. It is generally uneconomical to design intake towers to remain elastic during the MDE, so the structure must dissipate energy through postelastic deformation. The structure is therefore designed to undergo inelastic deformation under MDE but remain elastic under OBE, whichever controls.



SPECTRAL ACCELERATION VERSUS PERIOD (5% DAMPED)

T, sec	S_A, g (5% DAMPED)
0.002	1.0000
0.005	1.0000
0.008	1.0000
0.010	1.0000
0.020	1.2643
0.040	1.5985
0.060	1.8335
0.080	2.0210
0.100	2.1795
0.120	2.3182
0.150	2.5000
0.200	2.5000
0.250	2.5000
0.300	2.5000
0.350	2.5000
0.400	2.5000
0.450	2.2222
0.500	2.0000
0.550	1.8182
0.600	1.6667
0.650	1.5385
0.700	1.4286
0.800	1.2500
0.900	1.1111
1.000	1.0000
1.250	0.8000
1.500	0.6667
1.750	0.5714
2.000	0.5000
2.250	0.4444
2.500	0.4000
3.000	0.3333
3.500	0.2857
4.000	0.2500

Figure C-12. Response spectrum seismic input for computer analysis

b. Intake structures will undergo failure mechanisms similar to cantilever shear walls and cantilever bridge piers. The general procedures and design philosophy outlined in Pauley (1980) and Federal Highway Administration (1995) are usually applicable to the design of intake towers. The assumptions made in this manual are that intake tower walls, unless specially designed with ductility in mind, will be structures with limited ductility. Ductility can be enhanced with confinement reinforcement and specially reinforced boundary elements. However, the guidance provided herein pertains to structures without special ductility enhancement, which includes all existing intake towers designed and constructed by the Corps to date.

c. Structures with limited ductility will have displacement ductility ratios between 1.5 and 3.5. A displacement ductility of 2.5 will provide energy dissipation equivalent to a fully elastic structure designed for a moment reduction factor of two (Figure C-16). The structure, however, can undergo repeated cycles of inelastic displacement only if brittle modes of failure do not occur. The seismic design approach for intake towers is therefore to

(1) Permit limited inelastic bending displacements to occur under the MDE to reduce seismic bending moment demands and bending reinforcing steel requirements.

```
$ ITAP (Intake Tower Analysis Program, developed by QUEST Structures)
$ Intake Tower Example Problem
$ Earthquake in X-direction (longitudinal)
$ Maximum Design Earthquake (MDE)
$ Effective stiffness equal to 0.8 gross stiffness
$ Metric Units (kN-m)
$ Title Card
 TOWER WITH APPROXIMATE MASS
$
$ Master Control Card
$ NumElms NumMode lCases iShea WatDen  WatEo   WatEi  Gravity  Mass Sec
    12      20      1      1    9.807 41.4528 41.4528   9.807         4
$
$ Nodal Point and Element Data
$ Elm  Z-bot   Z-top   SecID MatID
  1   0.0      1.8288    1     1
  2   1.8288   7.0104    2     1
  3   7.0104   12.192    2     1
  4   12.192   18.288    3     1
  5   18.288   24.384    3     1
  6   24.384   30.48     4     1
  7   30.48    36.576    4     1
  8   36.576   41.4528   5     1
  9   41.4528  48.768    5     1
 10   48.768   54.5592   6     1
 11   54.5592  60.3504   6     1
 12   60.3504  60.96     7     1
$
$ Material Properties
$ iMatID       E          nu   weight density
    1     0.2153E+08     0.2     0.2400E+02
$
$ Section Properties
$ iSecID iSecType   a       b       c     d         shear coeffs
stiff.Red
    1       1    14.6304 14.6304              0.666651093 0.666651093  0.8
    2      11    14.6304 11.2776 10.9728 7.62 0.505525540 0.394891945  0.8
    3      11    14.3256 10.6680 11.2776 7.62 0.508635682 0.386206897  0.8
    4      11    14.0208 10.0584 11.5824 7.62 0.512310606 0.377518939  0.8
    5      11    13.7160 9.44880 11.8872 7.62 0.516349125 0.369078270  0.8
    6      11    13.4112 8.83920 12.1920 7.62 0.521940764 0.360950896  0.8
    7       1    13.4112 8.83920              0.666686351 0.666686351  0.8
$
$ Type of Load Case
$(0:eigen, 1:gravity, 2:hydrostatic, 3:static, 4:RSA, 5:TH)
                                                     4
$
$ Response Spectrum Analysis
$ Spectrum Heading
Maximum Design Earthquake (MDE) with PGA=0.62g
$
$  npts   FX     FY     FZ  0=Displ,  1=Acc. vs. period
    34   9.807  0.00  0.00            1
```

Figure C-13. Input to ITAP for the longitudinal MDE excitation (Continued)

```
$
$ Period      X-Spec
  0.0010      0.2517
  0.0020      0.2554
  0.0050      0.2666
  0.0080      0.2778
  0.0100      0.2852
  0.0200      0.3224
  0.0400      0.3968
  0.0600      0.4712
  0.0800      0.5456
  0.1000      0.6200
  0.1200      0.6200
  0.1500      0.6200
  0.2500      0.6200
  0.3000      0.6200
  0.3500      0.6200
  0.4000      0.6200
  0.4500      0.6200
  0.5000      0.6200
  0.5500      0.5636
  0.6000      0.5167
  0.6500      0.4769
  0.7000      0.4429
  0.8000      0.3875
  0.9000      0.3444
  1.0000      0.3100
  1.2500      0.2480
  1.5000      0.2067
  1.7500      0.1771
  2.0000      0.1550
  2.2500      0.1378
  2.5000      0.1240
  3.0000      0.1033
  3.5000      0.0886
  4.0000      0.0775
$
$ Load Combinations for various load cases
$ Number of Combinations
  0
$ Combination Factors
$ When several load cases are considered in a single computer run, they can be
$ combined by assigning a none-zero load factor here
```

Figure C-13. (Concluded)

(2) Prevent shear failure by providing strengths equal to the full elastic demand of the MDE.

(3) Check other potential brittle modes of failure.

d. A moment reduction factor R_M has been developed for intake tower sections to represent the acceptable ratio of seismic moment that could develop in the tower if the structure behaved entirely elastically under the ground motion associated with the MDE to the prescribed design seismic moment considered acceptable if some limited inelastic displacement is permitted.

e. The design calculations for the example intake tower illustrate the approach used for new towers without special ductility reinforcement. The design will investigate the conditions necessary to prevent the following failure mechanisms from occurring during the MDE:

Table C-11
Maximum Design Earthquake Results

Item	Longitudinal Direction	Transverse Direction
Period of first mode of vibration	0.45 sec	0.58 sec
Period of second mode of vibration	0.134 sec	0.18 sec
Top displacement	62.43 mm (2.46 in)	88.44 m (3.48 in)
Base shear	57,855 kN (13,020 kips)	57,233 kN (12,880 kips)
Base moment	1,783,747 kN-m (1,317,000 kip-ft)	1,925,959 kN-m (1,422,000 kip-ft)

Table C-12
Operational Basis Earthquake Results

Item	Longitudinal Direction	Transverse Direction
Period of first mode of vibration	0.406 sec	0.529 sec
Period of second mode of vibration	0.126 sec	0.165 sec
Top displacement	30.21 mm (1.19 in)	42.80 mm (1.69 in)
Base shear	27,995 kN (6,300 kips)	27,701 kN (6,234 kips)
Base moment	863,025 kN-m (637,200 kip-ft)	931,693 kN-m (687,900 kip-ft)

(1) Moment failure (flexure).

(2) Shear failure (diagonal tension).

(3) Sliding shear failure.

(4) Rebar splice failure.

(5) Rebar anchorage failure.

(6) Compressive spalling failure.

(7) Fracture of tensile reinforcement (minimum tensile reinforcement requirements).

f. The example problem inertia forces, shears, and moments from the ITAP lumped-mass computer solution are used as the basis for design. The section at the base of the tower, where maximum shears and moments occur, is used to illustrate the design process.

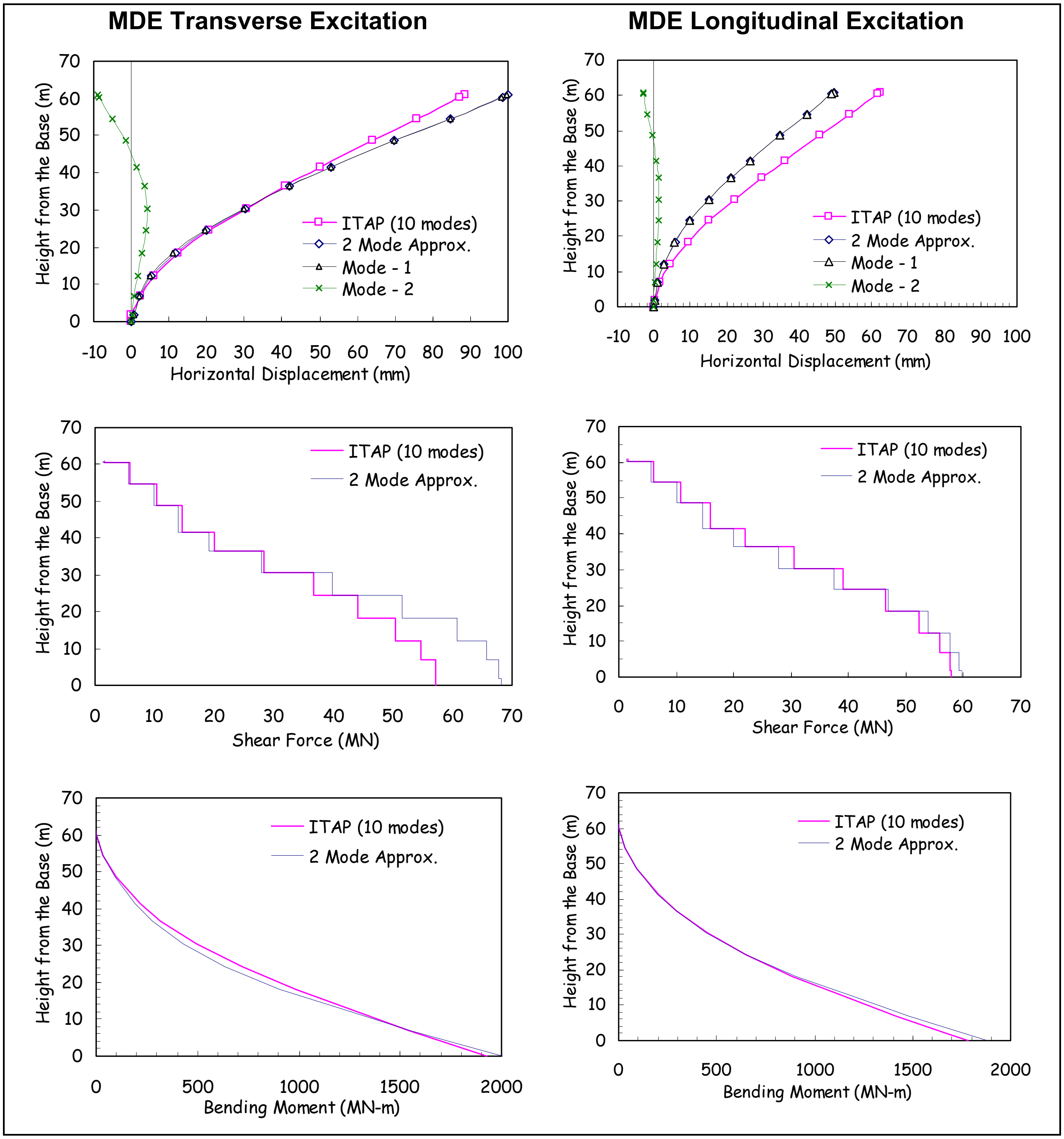


Figure C-14. Comparison of MDE results for the two-mode approximate procedure and computer dynamic analysis

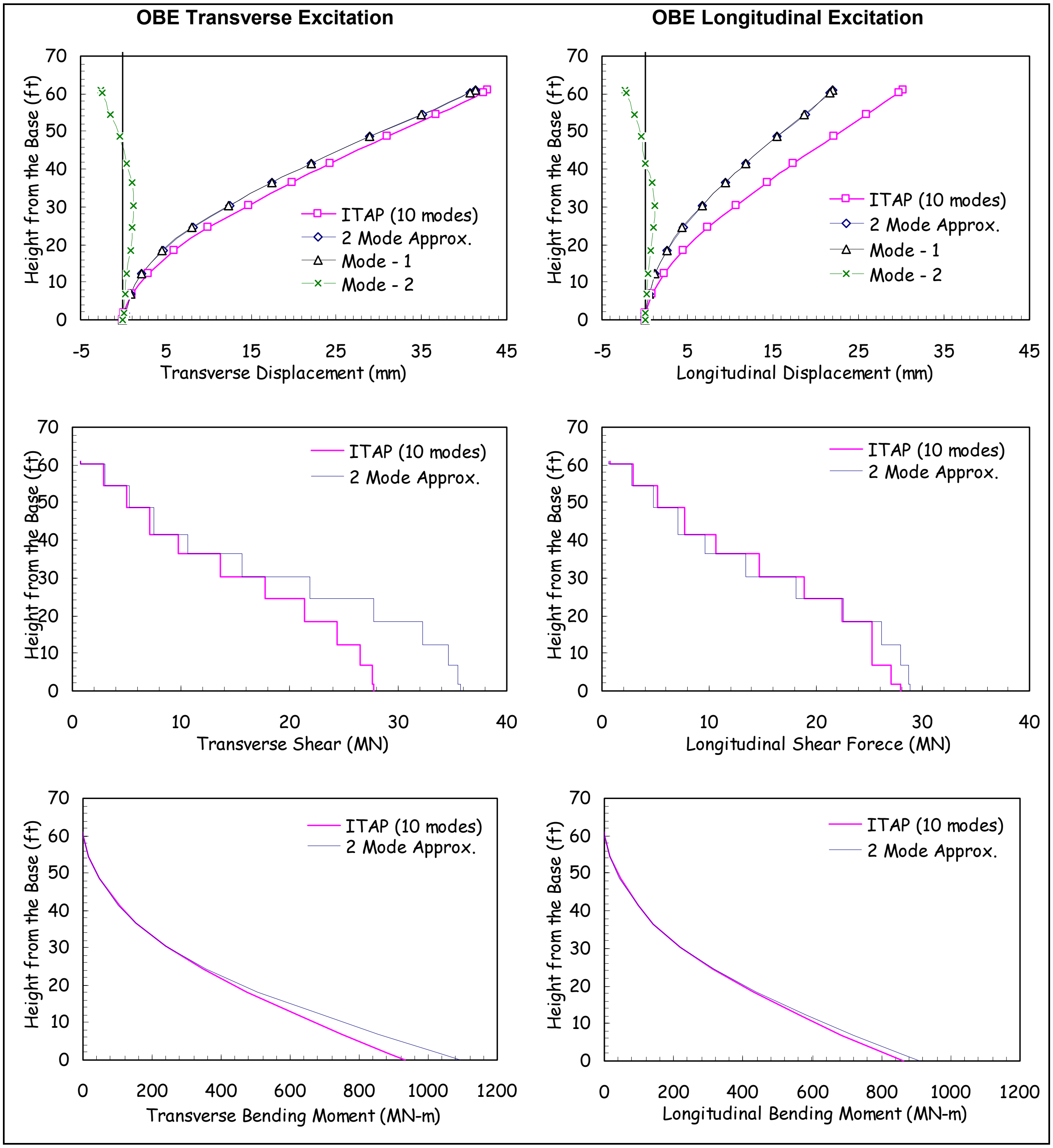
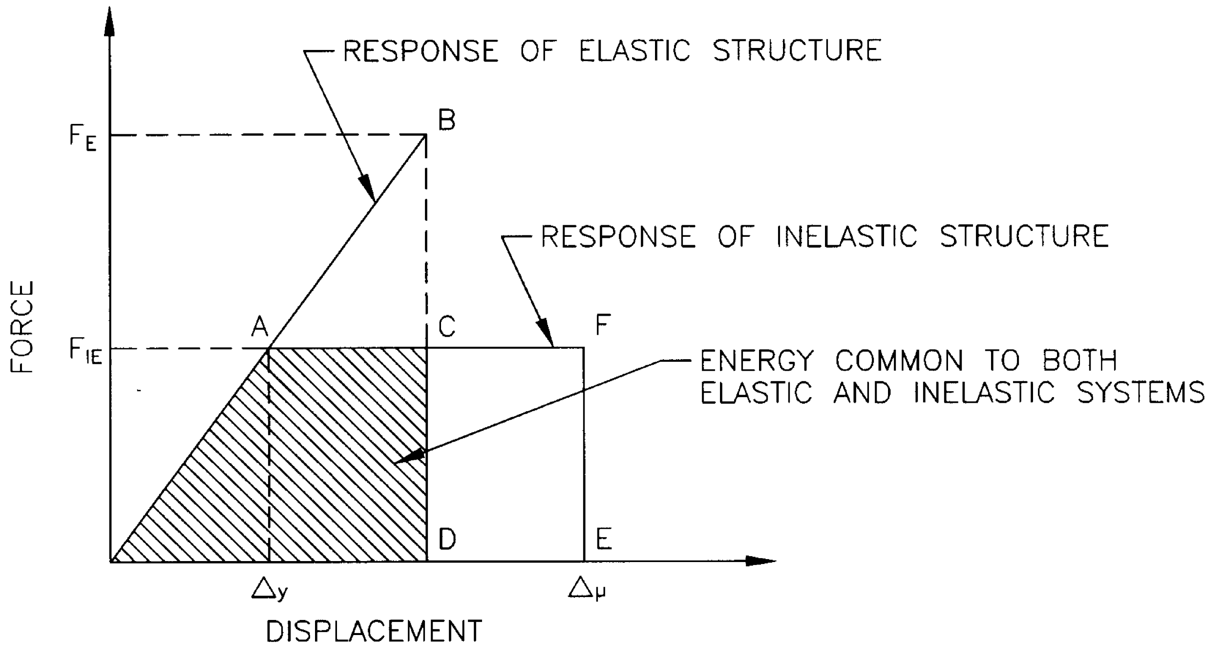


Figure C-15. Comparison of OBE results for the two-mode approximate procedure and computer dynamic analysis



μ = displacement ductility factor
R_M = moment reduction factor
F_E = force experienced by elastic structure
F_{IE} = force experienced by inelastic structure
Δy = displacement of inelastic structure at yield
$\Delta \mu$ = displacement of inelastic structure at ultimate

Assuming an equal maximum potential energy response for the elastic and inelastic structures, the area ABC must equal the area CDEF.

With ductility equal to $\Delta\mu/\Delta y$ and
the moment reduction factor equal to F_E/F_{IE}
then: $R_M = \sqrt{2\mu - 1}$
A μ factor of 2.5 will provide a R_M factor of 2.0

Figure C-16. Relationship between displacement ductility and moment reduction factor using equal maximum potential energy response

C-10. Procedures for Seismic Design of Intake Towers

a. In most cases intake towers are subjected to the gravity, hydrostatic, and earthquake loading. Since the tower represents a cantilever structure, the applied loads are balanced by reaction moment (M_u), shear (V_u), and axial force (P_u), as shown in Figure C-17. The basic objective of the design is to ensure that the safety and serviceability of the structure under these loads are maintained.

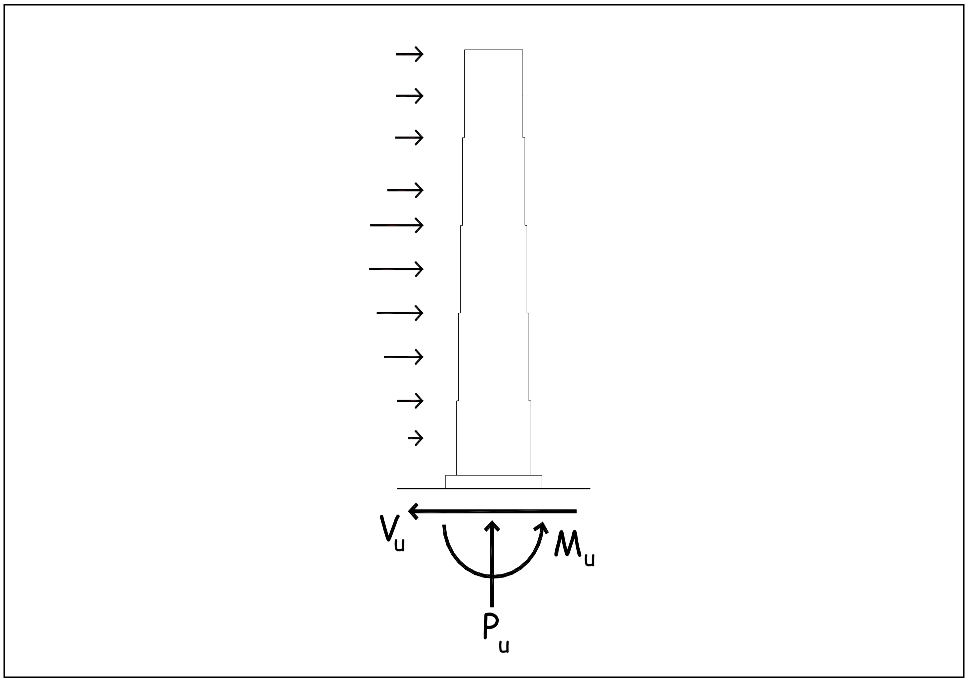


Figure C-17. Loads acting on tower during an earthquake

b. Design provisions outlined in this example are applicable to structures with capacities well below the balanced point of the axial-force bending-moment interaction curve (Figure C-18). Since the vertical load in towers is generally the result of self-weight, which is not very large, most intake towers will have capacities well below the balanced point.

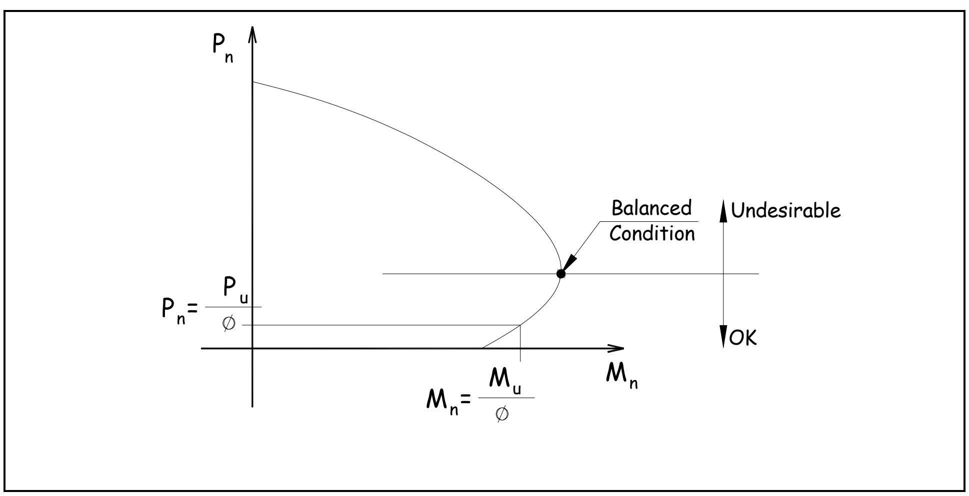


Figure C-18. Axial force and bending moment interaction diagram

c. The basic seismic design of the reinforced concrete intake towers should follow the following steps:

(1) Select desirable material properties (usually known or assumed).

(2) Select overall geometry of the tower cross sections (usually known before the design).

(3) Determine demands M_u, P_u, and V_u (from the analysis) and apply appropriate load factors.

(4) Determine the approximate amount of reinforcement steel.

(5) Perform shear design with $\phi = 0.85$ to obtain V_n.

(6) Perform bending design including vertical load to obtain M_n.

(7) Check ultimate-state failure modes and redo shear design if necessary:

- Tower will fail in bending. Since the bending is a ductile failure mode, the design is OK if the ductility requirement is met and the shear strength is not exceeded.

- Tower will fail in shear. Since the shear is a brittle failure, the failure under shear should be avoided. This is done by increasing shear strength of the tower by redoing shear design with $\phi = 0.60$.

(8) Perform sliding shear failure check.

(9) Check anchorage for the moment reinforcing bars at the base.

(10) Design reinforcing bar splices.

(11) Perform comprehensive spalling check.

(12) Perform minimum tensile reinforcement requirements check.

(13) Sketch final reinforcement arrangement and detailing.

(14) Repeat the design for other cross sections.

Note: since Microsoft Excel spreadsheet was used in developing this design example, the numbers given may not exactly match the numbers obtained by hand calculations. This is because hand calculations usually consider fewer decimal digits than the spreadsheet.

C-11. Material Properties

The material properties assumed in this example are summarized in Table C-13.

Table C-13
Assumed Material Properties

Parameter		Value	
		Non-SI Units	Metric Units
Re-bar Material Properties			
Modulus of Elasticity	(E_s)	29,000.00 ksi	199,947.95 MPa
Nominal Yield Strength	(f_y)	60.000 ksi	413.69 MPa
Strain Hardening		0.80 %	0.80 %
Steel Ultimate Stress		75.000 ksi	517.11 MPa
Ultimate Strain		5.00 %	5.00 %
Concrete Material Properties			
Modulus of Elasticity	(E_c)	3,122.00 ksi	21,525.43 MPa
Shear Modulus	(G)	1,300.83 ksi	8,986.93 MPa
Poisson's Ratio	(ν)	0.20	0.20
Compressive Strength	(f'_c)	3.00 ksi	20.68 MPa
Modulus of Rupture	(F_r)	0.41 ksi	2.82 MPa
Concrete Ultimate Strain	(e_c)	0.30 %	0.30 %

C-12. Section Properties

In order to perform a design, the overall dimensions of the structure should be known. In most cases geometry of the tower is known prior to the design process. Figure C-19 shows dimensions of the most critical or bottom section of the tower, where the largest bending moment, shear, and axial forces occur. The critical section can also be determined on the basis of the demand-capacity ratios, if approximate capacities of the sections are known.

C-13. Determination of Nominal Loading Capacity

The computed moment and force demands (Table C-14) after application of load factors should be less than or equal to the factored uni-axial nominal capacities:

$$M_u \leq \phi \cdot M_n$$
$$V_u \leq \phi \cdot V_n$$
$$P_u \leq \phi \cdot P_n$$

From Table C-15 since:

$$P_u = Both\,(52{,}422.00)\ and\ (73{,}391) < 0.1 \cdot A_g \cdot f_c^{'} =$$
$$0.1 \cdot (14.63 \times 11.28 - 7.62 \times 10.97) \times 20{,}684.00 =$$
$$168{,}440.00$$

where A_g is the gross section area and f'_c is the concrete compression strength, the bending moment strength reduction factor of $\phi = 0.90$ applies. Otherwise a $\phi = 0.70$ should be employed in accordance with American Concrete Institute (ACI) 318-95, sec. 9.3.2.2.

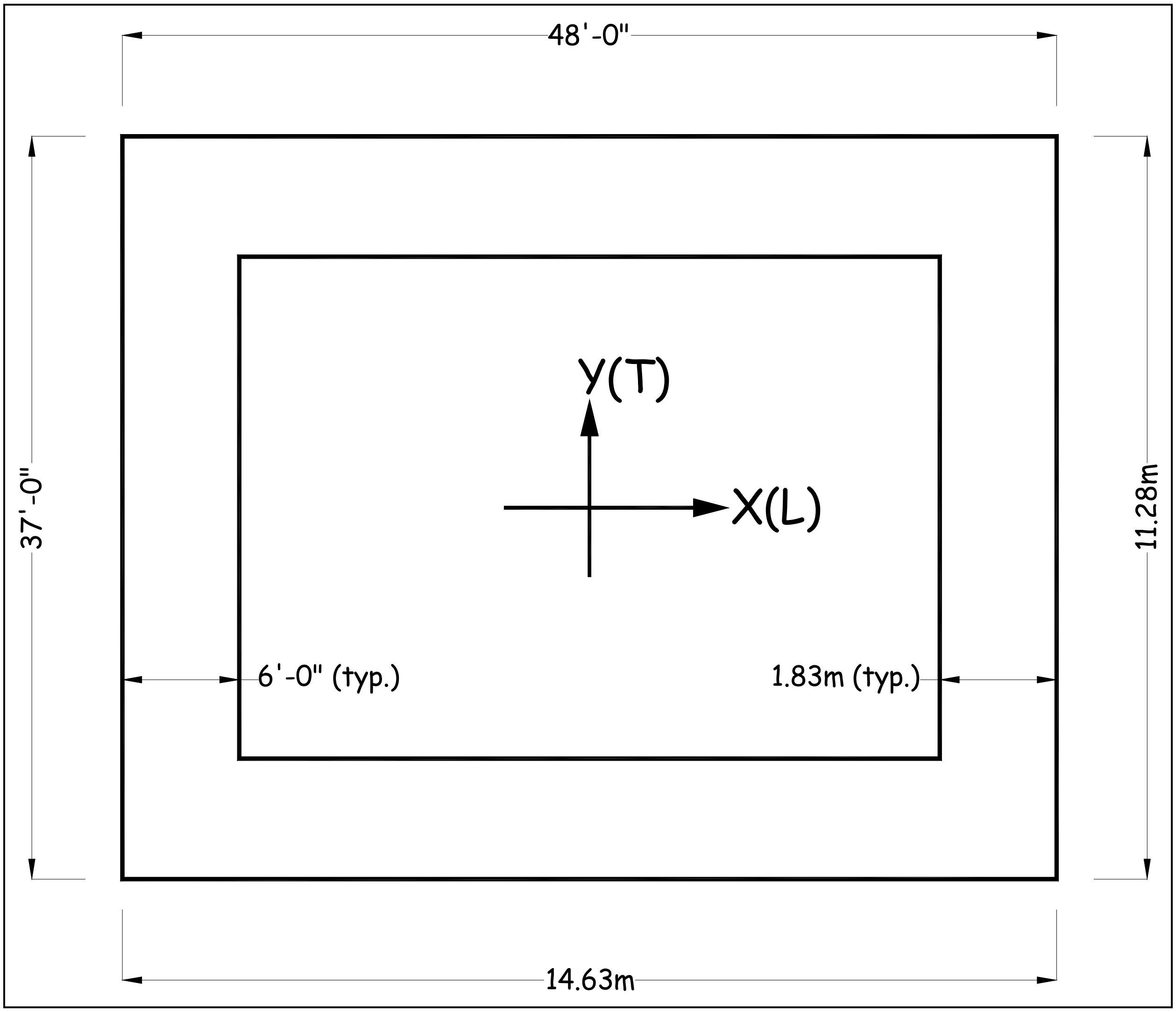


Figure C-19. Overall dimensions of critical section (section-1)

Table C-14
Summary of Computed Maximum Loads and Applicable Load Factors

Calculated Loads from Analyses		Longitudinal Direction		Transverse Direction		Load Factor Equations 4-4 & 4-5
			MDE			
Bending Moment	M	1,783,747 kN-m	(1,317,000 k-ft)	1,925,959 kN-m	(1,422,000 k-ft)	1.1
Shear Force	V	57,855 kN	(13,020 kips)	57,233 kN	(12,880 kips)	1.1
Axial Load	P	52,422 kN	(11,784 kips)	52,422 kN	(11,784 kips)	1
			OBE			
Bending Moment	M	863,025 kN-m	(637,200 k-ft)	931,693 kN-m	(687,900 k-ft)	1.5
Shear Force	V	27,995 kN	(6,300 kips)	27,701 kN	(6,234 kips)	1.5
Axial Load	P	52,422 kN	(11,784 kips)	52,422 kN	(11,784 kips)	1.4

Table C-15
Summary of Factored Maximum Loads and Preliminary Strength Reduction Factors

Factored Loads from Table C-14		Longitudinal Direction		Transverse Direction		Strength Reduction Factors (ACI 318-95 9.3.2)
MDE						
Bending Moment	M_u	1,962,122 kN-m	(1,448,700 k-ft)	2,118,555 kN-m	(1,564,200 k-ft)	0.9
Shear Force	V_u	63,641 kN	(14,322 kips)	62,956 kN	(14,168 kips)	0.85
Axial Load	P_u	52,422 kN	(11,784 kips)	52,422 kN	(11,784 kips)	0.7
OBE						
Bending Moment	M_u	1,294,538 kN-m	(955,800 k-ft)	1,397,540 kN-m	(1,031,850 k-ft)	0.9
Shear Force	V_u	41,993 kN	(9,450 kips)	41,552 kN	(9,351 kips)	0.85
Axial Load	P_u	73,391 kN	(16,498 kips)	73,391 kN	(16,498 kips)	0.7

If the tower is expected to fail in bending, the shear force strength reduction factor shall be taken as 0.85. While for the failure in shear, the shear force strength reduction factor shall be taken as 0.60 (See ACI 318-95 sec. 9.3.4.1).

C-14. Preliminary Selection of Reinforcement

a. Minimum reinforcement ratio per ACI 318-95 (sec. 11.10.9) is

Horizontal: $\rho_{h,\min} = 0.0025$

Vertical: $\rho_{v,\min} = 0.0025$

b. Maximum vertical reinforcement should not be taken more than 75 percent of the balanced vertical reinforcement ratio ρ_b:

$$\rho_{v,\max} = 0.75 \cdot \rho_b = 0.75 \times (0.0214\ ) = 0.0160$$

Where ρ_b is given by ACI 318-95 sec. 8.4.3 in psi units:

$$\rho_b = \frac{0.85 \cdot \beta_1 \cdot f'_c}{f_y} \cdot \left(\frac{87{,}000}{87{,}000 + f_y} \right) =$$

$$\frac{0.85 \times (0.85) \times (3{,}000)}{60{,}000} \cdot \left(\frac{87{,}000}{87{,}000 + 60{,}000} \right) = 0.0214$$

in which $\beta_1 = 0.85$ for concrete with $f'_c = 3{,}000$ psi

and f_y is the yield stress of the longitudinal reinforcement

c. According to the calculated reinforcement ratio limits, and from statistical information (Dove 1998) on existing towers it is reasonable to assume a reinforcement ratio of

$$\rho_a = 0.0035$$

d. The total area of the assumed reinforcement (A_s) will be:

$$A_s = 0.0035 \cdot A_g =$$
$$0.0035 \times (14.63 \times 11.28 - 7.62 \times 10.97) =$$
$$0.0035 \times (81.383)$$

$$A_s = 2848.40 \text{ cm}^2 \quad (441.50 \text{ in}^2)$$

e. At this point, the units for design calculations should be selected. Since the standard reinforcing bar sizes are in non-SI units, the design will be carried out in non-SI units and then converted into metric. Note that there is no guarantee that the standard non-SI-unit re-bars will be easily available in SI standards.

f. The minimum reinforcing spacing s_1 according to ACI 318-95 (sec. 11.10.9) is the minimum of $l_w/3$, $3h$ and 18 in., where l_w is horizontal length and h is thickness of the tower.

(1) Assuming reinforcing bar #11 with $A_{\#11} = 1.56$ in^2 = (10.06 cm^2), the approximate required number of reinforcing bars is obtained from

$$n = A_s / A_{\#11} = 441.5/1.56 = 283$$

Approximately 283 reinforcing bars should be fitted into the cross section shown in Figure C-19. The reinforcing bars should be arranged in not less than two rows. The approximate spacing is calculated as

$$s = \frac{2 \cdot (48 - 1 + 48 - 1 + 37 - 1 + 37 - 1)}{283} = 1.17 \text{ ft}$$

(2) The spacing is assumed as 12 in. (approximately 30 cm).

g. Provide two layers of reinforcement close to the wall surfaces. Provide horizontal and vertical reinforcement for confining the core, usually the same amount in each direction.

h. The preliminary arrangement of the reinforcement is given in Figure C-20.

C-15. Design for Seismic Shears (Diagonal Tension)

a. General. The tower capacity in shear is least along the diagonal direction of earthquake attack. The shear reinforcing steel requirements can be determined using Equations 4-14 through 4-17. The shear strength shall equal, or exceed, the shear demands as represented by Equations 4-4 to 4-8 and given by

$$U_{long} = D_{long} + L_{long} \pm E_{long} \pm 0.30\, E_{trans}$$
$$U_{trans} = D_{trans} + L_{trans} \pm E_{trans} \pm 0.30\, E_{long}$$

The basic design equation according to ACI 318-95 is

$$V_u / \phi \le V_n = V_c + V_s$$

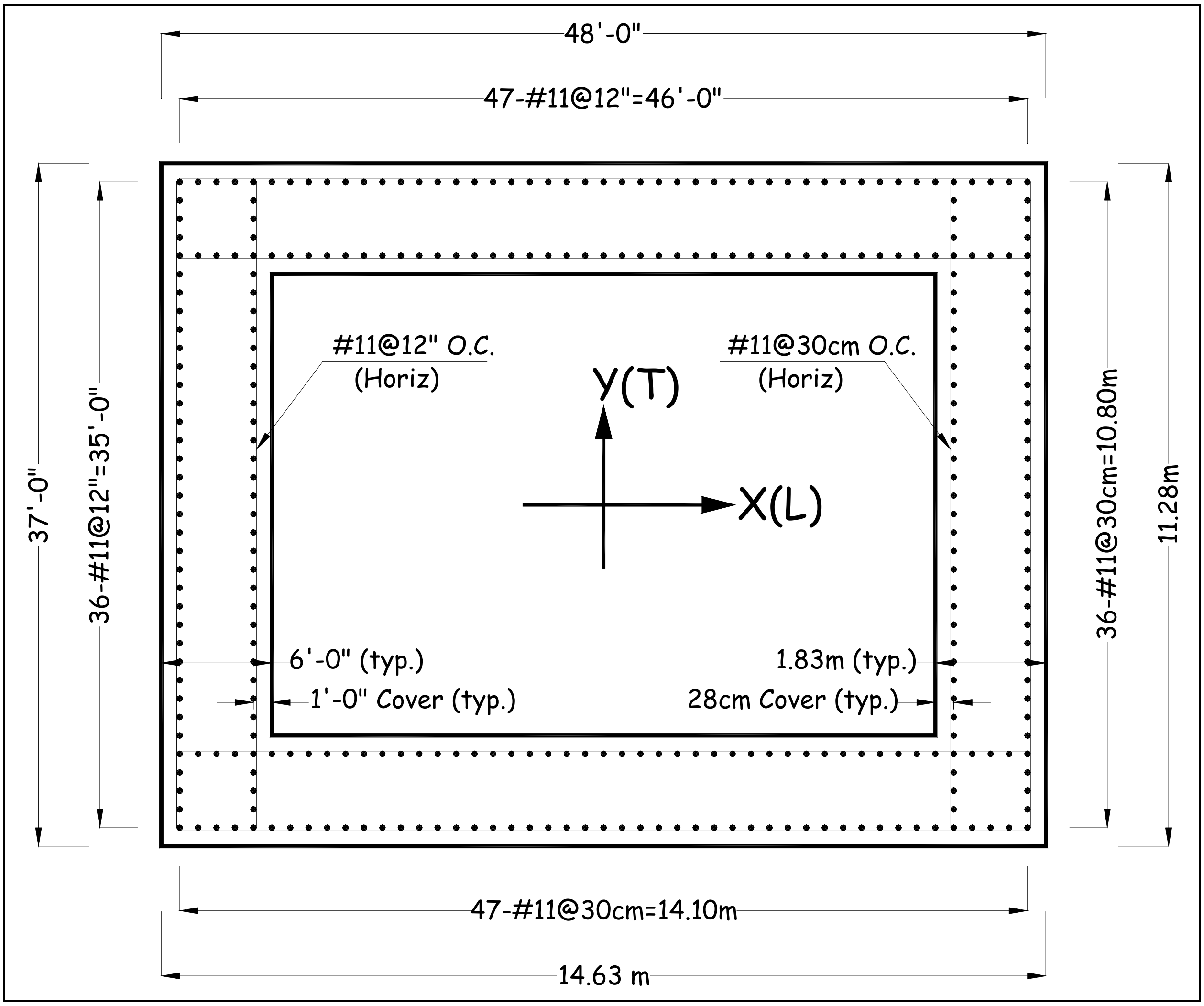


Figure C-20. Preliminary reinforcement arrangement

where

V_n = nominal shear resistance

V_c = nominal shear resistance provided by concrete

V_s = nominal shear resistance provided by shear reinforcement

b. Concrete nominal shear strength V_c. Using Equation 4-15, the nominal shear strength of concrete is calculated as follows:

(1) In MPa units:

$$V_c = 2 \left(K + \frac{P_u}{13.8 \cdot A_g} \right) 0.083 \sqrt{f'_{CA}} \cdot A_E$$

$$V_c = 2\left(0.5 + \frac{(52{,}422)/1{,}000}{13.8(81.383)}\right)0.083 \times$$
$$\sqrt{1.5(20.68)}(0.8)(81.383) =$$
$$= 32.910 \text{ MN} = 32{,}910 \text{ kN}$$

(2) In psi units:

$$V_c = 2\left(K + \frac{P_u}{2{,}000 \cdot A_g}\right)\sqrt{f'_{CA}} \cdot A_E$$
$$V_c = 2\left(0.5 + \frac{11{,}784(1{,}000)}{2{,}000(876)144}\right) \times$$
$$\sqrt{1.5(3{,}000)}(0.8)(876)144 =$$
$$= 7{,}402{,}014 \text{ lb} = 7{,}402 \text{ kips}$$

c. Shear steel requirements. Nominal shear strength provided by the reinforcement is calculated as:

$$V_s = \frac{A_v \cdot f_y (0.8 \cdot d)}{s}$$

(1) In the longitudinal direction:

$$V_{s,long} = \frac{A_v \cdot f_y (0.8 \cdot d)}{s}$$
$$V_{s,long} = \frac{4(10.06)(413.69 \times 1{,}000)(0.8)(14.63)}{0.3(10{,}000)}$$
$$V_{s,long} = 64{,}975 \text{ kN } (14{,}606 \text{ kips})$$

(2) In the transverse direction:

$$V_{s,trans} = \frac{A_v \cdot f_y (0.8 \cdot d)}{s}$$
$$V_{s,trans} = \frac{4(10.06)(413.69 \times 1{,}000)(0.8)(11.28)}{0.3(10{,}000)}$$
$$V_{s,trans} = 50{,}085 \text{ kN } (11{,}259 \text{ kips})$$

d. Ultimate shear capacity.

(1) Using Equation 4-14, the ultimate shear capacity is calculated as follows:

(a) In the longitudinal direction:

$$V_{u,long} = \phi \cdot (V_c + V_{s,long})$$
$$= 0.85\,(32{,}910 + 64{,}975)$$
$$= 83{,}202 \text{ kN } (18{,}707 \text{ kips})$$

(b) In the transverse direction:

$$V_{u,trans} = \phi \cdot (V_c + V_{s,trans})$$
$$= 0.85\,(32{,}910 + 50{,}085)$$
$$= 70{,}546 \text{ kN } (15{,}862 \text{ kips})$$

(2) The multidirectional shear demands are compared to the shear capacities following a principle similar to that for Equations 4-7 and 4-8, where $\alpha = 0.3$ is used for rectangular towers:

(a) For the longitudinal direction:

$$\frac{V_{E,long}}{V_{u,long}} + 0.30 \cdot \frac{V_{E,trans}}{V_{u,trans}} \leq 1.0$$

$$\frac{63{,}641}{83{,}202} + 0.30\,\frac{62{,}956}{70{,}546} = 1.033 > 1.0 \quad \Rightarrow \quad \text{NOT Okay}$$

(b) For the transverse direction:

$$\frac{V_{E,trans}}{V_{u,trans}} + 0.30 \cdot \frac{V_{E,long}}{V_{u,long}} \leq 1.0$$

$$\frac{62{,}956}{70{,}546} + 0.30\,\frac{63{,}641}{83{,}202} = 1.122 > 1.0 \;\Rightarrow \text{NOT Okay}$$

(3) Since shear capacity is not adequate, the amount of preliminary reinforcement should be increased. This can be achieved by reducing the re-bar spacing. Set new spacing to s = 25.0 cm (10.0 in).

(a) In the longitudinal direction:

$$V_{s,long} = \frac{4\,(10.06)\,(413.69 \times 1{,}000)\big(0.8\big)(14.63)}{0.25\big(10{,}000\big)}$$
$$= 77{,}970 \text{ kN } (17{,}527 \text{ kips})$$

$$V_{u,long} = \phi \cdot (V_c + V_{s,long})$$
$$= 0.85\,(32{,}910 + 77{,}970)$$
$$= 94{,}248 \text{ kN } (21{,}190 \text{ kips})$$

$$\frac{63{,}641}{94{,}248} + 0.30\,\frac{62{,}956}{79{,}060} = 0.914 < 1.0 \quad \Rightarrow \quad \text{Okay}$$

(b) In the transverse direction:

$$V_{s,trans} = \frac{4\,(10.06)\,(413.69 \times 1{,}000)(0.8)(11.28)}{0.25(10{,}000)}$$
$$= 60{,}102 \text{ kN } (13{,}511 \text{ kips})$$

$$V_{u,trans} = \phi \cdot (V_c + V_{s,trans})$$
$$= 0.85\,(32{,}910 + 60{,}102)$$
$$= 79{,}060 \text{ kN } (17{,}776 \text{ kips})$$
$$\frac{62{,}956}{79{,}060} + 0.30\frac{63{,}641}{94{,}248} = 0.999 < 1.0 \quad \Rightarrow \quad \text{Okay}$$

Shear failure should be prevented whenever possible due to its brittle mode. Thus, it is important that the bending strength is reached before shear failure. In this manner, if the tower is designed properly for bending, the desirable ductility could be achieved. If the bending failure cannot be achieved before shear failure, then shear strength reduction factor ϕ should be taken as 0.6 to provide some additional shear strength and shear design repeated. Figure C-21 shows the reinforcing bar arrangement after the design for shear resistance.

C-16. Design for Seismic Bending Moments

a. General. The bending capacity of the tower shall be computed similar to cantilever columns with distributed reinforcement. Approximate hand calculations should be carried out in order to obtain approximate required reinforcement and to verify the final design. The final design in this example is performed by the computer program CGSI.

b. Design for MDE. The values of nominal loading should be established first, in order to perform a design.

(1) Nominal axial load equals:

$$P_n = \frac{P_u}{\phi} = \frac{52{,}422}{0.7} = 74{,}889 \text{ kN} \quad (16{,}835 \text{ kips})$$

(2) The nominal design value of the bending moment in the longitudinal direction (Y-Y) is:

$$M_u = \frac{M_{calculated}}{R_M}$$
$$= \frac{1{,}962{,}122}{2.0} = 981{,}061 \text{ kN - m} \ \ (723{,}555 \text{ k - ft})$$

where R_M is the moment reduction factor. R_M should not exceed two.

(3) The nominal moment capacity M_n is obtained as follows:

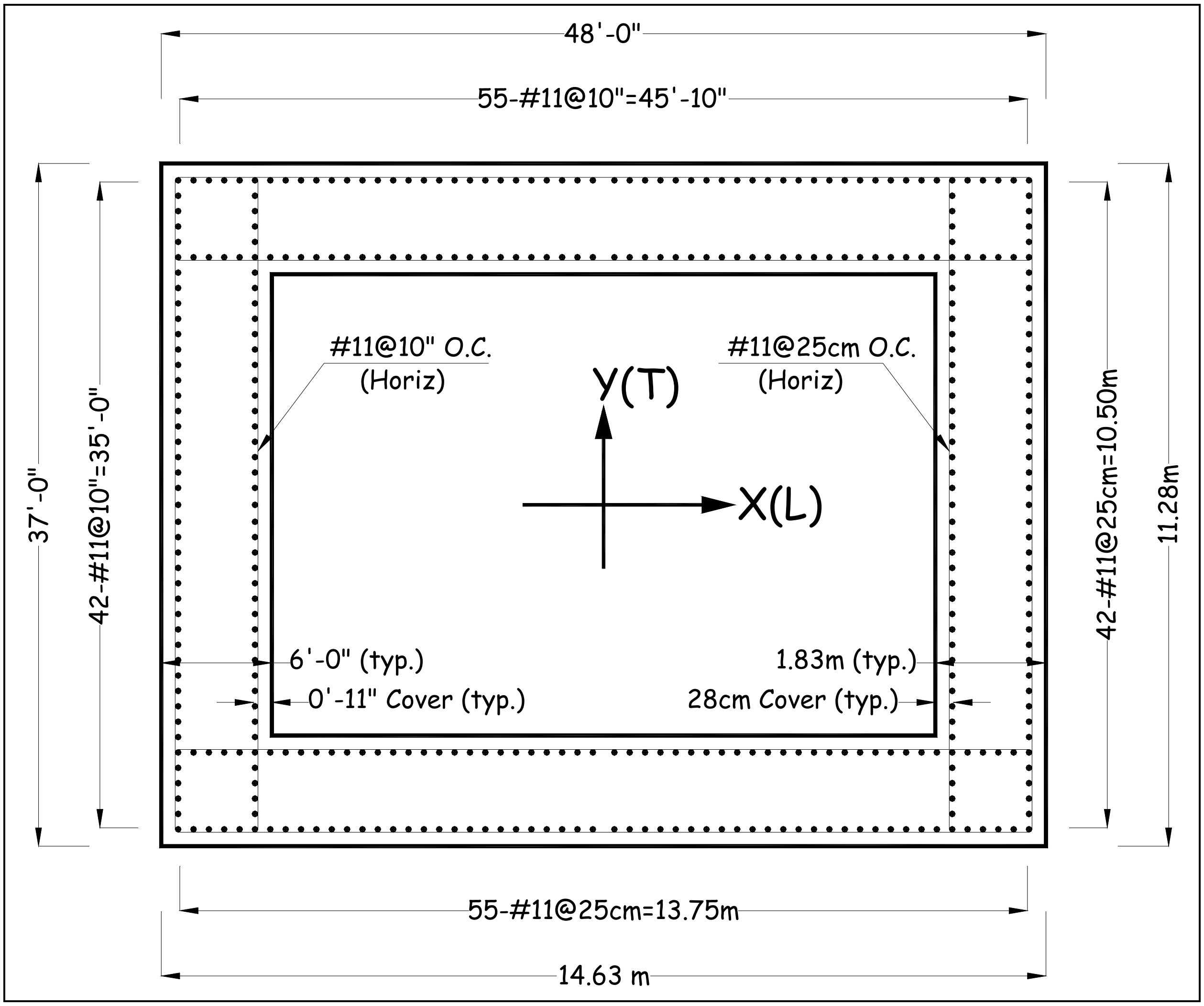


Figure C-21. Reinforcing bar arrangement after the design for shear resistance

$$\phi \cdot M_n \geq M_u$$

or

$$M_n = M_u / \phi = 981{,}061 / 0.9 =$$
$$= 1{,}090{,}068 \text{ kN - m } (803{,}950 \text{ k - ft})$$

(4) The eccentricity is calculated as indicated in Figure C-22:

$$e = \frac{M_n}{P_n} = \frac{1{,}090{,}068}{74{,}889} = 14.56 \text{ m} \quad (573.07 \text{ in})$$

(5) The compressive force in the concrete C_c can be obtained from force equilibrium in the vertical direction, as shown in Figure C-22:

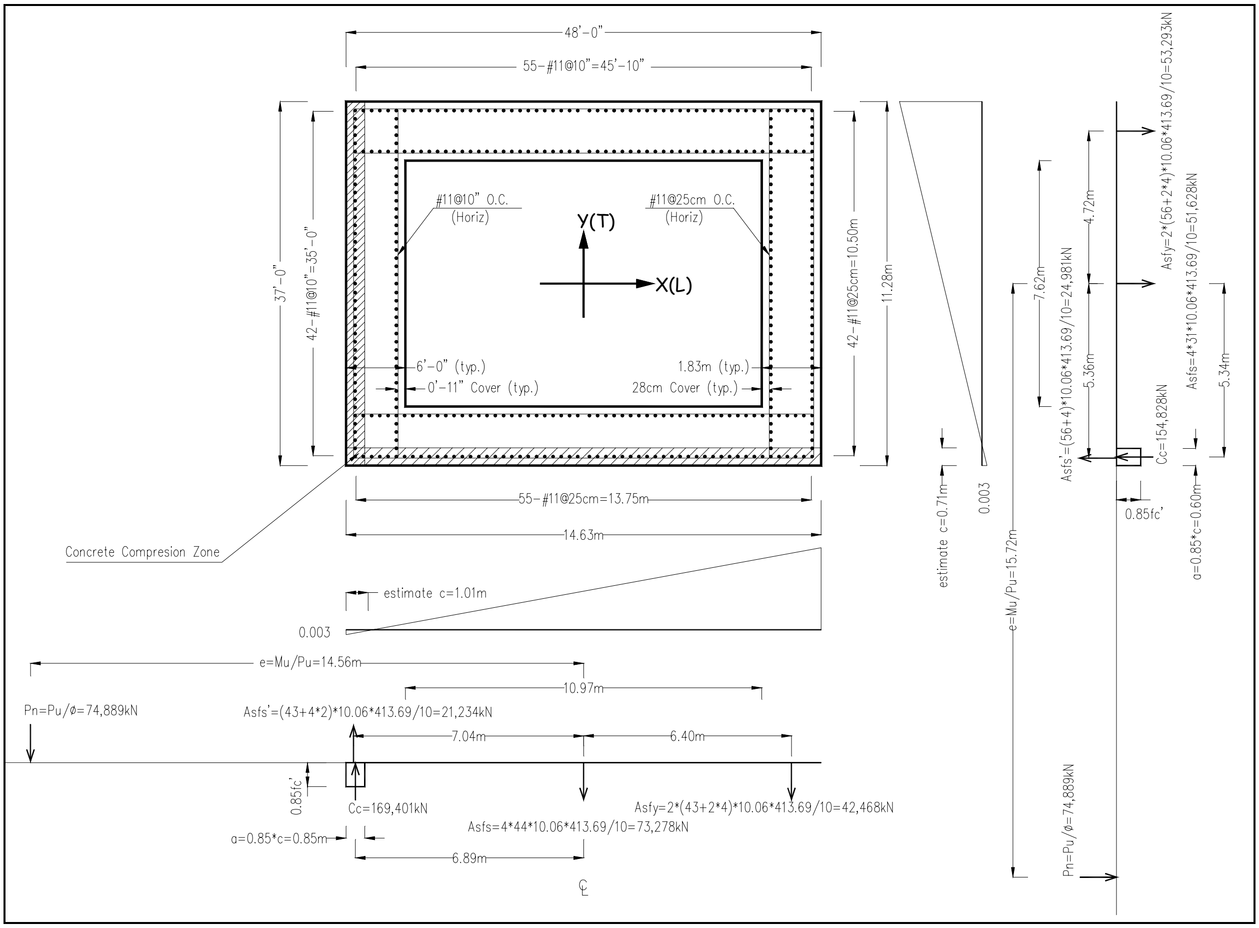


Figure C-22. Approximate design of the section-1 for bending moment resistance

$$\sum F_z = C_c + A_s f_s' + A_s f_s + A_s f_y + P_n = 0$$

$$\sum F_z = C_c + 21{,}234 - 73{,}278 - 42{,}468 - 74{,}889 = 0$$

$$C_c = 169{,}401 \text{ kN} \quad (38{,}081 \text{ kips})$$

(6) The depth of the concrete in compression can be obtained as

$$a = \frac{C_c}{0.85 \cdot f_c' \cdot 11.28}$$
$$= \frac{171{,}066}{0.85(20.68)(11.28)} = 0.85 \text{ m} \quad (33.64 \text{ in})$$

while the distance from extreme compression fiber to neutral axis is:

$$c = a / 0.85 = 0.85 / 0.85 = 1.01 \text{ m} \quad (39.57 \text{ in})$$

(7) The moment capacity of the section is obtained from the moment equilibrium with respect to the center line of the section:

$$\sum M_y = 0$$

$$\sum M_y = 42{,}468(6.40) + 73{,}278(0.0) +$$

$$21{,}234(7.04) + 169{,}401(6.89) = P_n \cdot e$$

$$P_n \cdot e = 271{,}830 + 0.0 + 149{,}398 + 1{,}166{,}836$$
$$= 1{,}588{,}064 \text{ kN-m} \quad (1{,}171{,}235 \text{ kip-ft})$$

$$1{,}588{,}064 \text{ kN - m} > 1{,}090{,}068 \text{ kN - m} \quad \Rightarrow \quad \text{Okay}$$

(8) Similarly, in the transverse direction (X-X):

$$M_n = \frac{M_u}{\phi \cdot R_M} = \frac{2{,}118{,}555}{0.9\ (2.0)}$$
$$= 1{,}176{,}975 \text{ kN-m} \quad (868{,}047 \text{ kip-ft})$$

$$e = \frac{M_n}{P_n} = \frac{1{,}176{,}975}{74{,}889} = 15.72 \text{ m} \quad (618.75 \text{ in})$$

As shown in Figure C-22:

$$\sum F_y = C_c + A_s f_s' + A_s f_s + A_s f_y + P_n = 0$$

$$\sum F_y = C_c + 24{,}981 - 51{,}628 - 53{,}293 - 74{,}889 = 0$$

$$C_c = 154{,}828 \text{ kN} \quad (34{,}805 \text{ kips})$$

$$a = \frac{C_c}{0.85 \cdot f_c' \cdot 14.63}$$
$$= \frac{154{,}828}{0.85(20.68)(14.63)} = 0.60 \text{ m} \quad (23.70 \text{ in})$$

$$c = a / 0.85 = 0.60 / 0.85 = 0.71 \text{ m} \quad (27.88 \text{ in})$$

$$\sum M_x = 0$$

$$\sum M_x = 53{,}293\ (4.72) + 51{,}628\ (0.0) + 24{,}981(5.36) + 154{,}828(5.34) = P_n \cdot e$$

$$P_n \cdot e = 251{,}779 + 0.0 + 133{,}884 + 826{,}452$$
$$= 1{,}212{,}113 \text{ kN-m} \quad (893{,}962 \text{ kip-ft})$$

$$1{,}212{,}113 \text{ kN - m} > 1{,}176{,}975 \text{ kN - m} \quad \Rightarrow \quad \text{Okay}$$

(9) Checking bi-directional criteria. Equation 4-6 should be used to account for earthquake bi-directional effects, as follows:

(a) In longitudinal direction:

$$\frac{M_{E,long}}{M_{u,long}} + 0.3 \cdot \frac{M_{E,trans}}{M_{u,trans}} \le 1.0$$

$$\frac{1{,}090{,}068}{1{,}588{,}068} + 0.3 \cdot \frac{1{,}176{,}975}{1{,}212{,}113} = 0.978 < 1.0 \Rightarrow \text{OK}$$

(b) In transverse direction:

$$\frac{M_{E,trans}}{M_{u,trans}} + 0.3 \cdot \frac{M_{E,long}}{M_{u,long}} \le 1.0$$

$$\frac{1{,}176{,}975}{1{,}212{,}133} + 0.3 \frac{1{,}090{,}068}{1{,}588{,}064} = 1.177 > 1.0 \Rightarrow \text{NOT OK}$$

Since the strength criteria are not satisfied, the reinforcing bar spacing is reduced from 25 cm (10 in) to 20 cm (8 in) and moment calculations repeated.

c. Redesign for MDE Load Case. Repetition of these calculations for the 20-cm spacing results in the following section forces and moment capacities for MDE load case.

(1) Longitudinal direction

$$C_c = 193{,}966 \text{ kN} \quad (43{,}603 \text{ kips})$$
$$a = 0.98 \text{ m} \quad (38.51 \text{ in})$$
$$c = 1.15 \text{ m} \quad (45.31 \text{ in})$$
$$P_n \cdot e = 1{,}871{,}237 \text{ kN-m} \quad (1{,}380{,}081 \text{ kip-ft})$$
$$1{,}871{,}237 \text{ kN - m} > 1{,}090{,}068 \text{ kN - m} \quad \Rightarrow \quad \text{Okay}$$

(2) Transverse direction

$$C_c = 174{,}397 \text{ kN} \quad (39{,}204 \text{ kips})$$
$$a = 0.68 \text{ m} \quad (26.69 \text{ in})$$
$$c = 0.80 \text{ m} \quad (31.40 \text{ in})$$
$$P_n \cdot e = 1{,}429{,}583 \text{ kN-m} \quad (1{,}054{,}351 \text{ kip-ft})$$
$$1{,}429{,}583 \text{ kN - m} > 1{,}176{,}975 \text{ kN - m} \quad \Rightarrow \quad \text{Okay}$$

(3) Checking bidirectional strength criteria

(a) In longitudinal direction:

$$\frac{M_{E,long}}{M_{u,long}} + 0.3 \cdot \frac{M_{E,trans}}{M_{u,trans}} \leq 1.0$$

$$\frac{1{,}090{,}068}{1{,}871{,}237} + 0.3 \cdot \frac{1{,}176{,}975}{1{,}429{,}583} = 0.829 < 1.0 \Rightarrow \text{OK}$$

(b) In transverse direction:

$$\frac{M_{E,trans}}{M_{u,trans}} + 0.3 \cdot \frac{M_{E,long}}{M_{u,long}} \leq 1.0$$

$$\frac{1{,}176{,}975}{1{,}429{,}583} + 0.3 \frac{1{,}090{,}068}{1{,}871{,}237} = 0.998 < 1.0 \Rightarrow \text{OK}$$

d. Check for OBE load case. The design now is checked for the OBE to ensure that the serviceability requirements are met. Note that the section capacity for the OBE condition should be computed since it differs from that for the MDE. The results are summarized below.

(1) Longitudinal direction:

$$C_c = 222{,}256\ \text{kN} \quad (49{,}962\ \text{kips})$$
$$a = 1.12\ \text{m} \quad (44.13\ \text{in})$$
$$c = 1.32\ \text{m} \quad (51.92\ \text{in})$$
$$P_n \cdot e = 2{,}060{,}336\ \text{kN-m} \quad (1{,}519{,}546\ \text{kip-ft})$$
$$2{,}060{,}336\ \text{kN - m} > 1{,}438{,}375\ \text{kN - m} \quad \Rightarrow \quad \text{Okay}$$

(2) Transverse direction:

$$C_c = 204{,}352\ \text{kN} \quad (45{,}938\ \text{kips})$$
$$a = 0.79\ \text{m} \quad (31.28\ \text{in})$$
$$c = 0.93\ \text{m} \quad (36{,}80\ \text{in})$$
$$P_n \cdot e = 1{,}576{,}442\ \text{kN-m} \quad (1{,}162{,}663\ \text{kip-ft})$$
$$1{,}576{,}442\ \text{kN - m} > 1{,}552{,}822\ \text{kN - m} \quad \Rightarrow \quad \text{Okay}$$

(3) Check for bidirectional strength criteria.

(a) In longitudinal direction:

$$\frac{1{,}438{,}375}{2{,}060{,}336} + 0.3 \cdot \frac{1{,}552{,}822}{1{,}576{,}442} = 0.994 < 1.0 \Rightarrow \text{OK}$$

(b) In transverse direction:

$$\frac{1{,}552{,}822}{1{,}576{,}442}+0.3\frac{1{,}438{,}375}{2{,}060{,}336}=1.19>1.0\Rightarrow \text{NOT OK}$$

Since the strength criteria are not satisfied, the section should be redesigned with 15 cm (6 in) of spacing.

e. Redesign for OBE load case.

(1) Longitudinal direction:

$$\begin{aligned}
&C_c = 261{,}809\ \text{kN} \quad (58{,}854\ \text{kips})\\
&a = 1.32\ \text{m} \quad (51.98\ \text{in})\\
&c = 1.55\ \text{m} \quad (61.16\ \text{in})\\
&P_n \cdot e = 2{,}539{,}236\ \text{kN-m} \quad (1{,}872{,}746\ \text{kip-ft})\\
&2{,}539{,}236\ \text{kN - m} > 1{,}438{,}375\ \text{kN - m} \quad \Rightarrow \quad \text{Okay}
\end{aligned}$$

(2) Transverse direction:

$$\begin{aligned}
&C_c = 237{,}661\ \text{kN} \quad (53{,}425\ \text{kips})\\
&a = 0.92\ \text{m} \quad (36.38\ \text{in})\\
&c = 1.09\ \text{m} \quad (42.79\ \text{in})\\
&P_n \cdot e = 1{,}945{,}437\ \text{kN-m} \quad (1{,}434{,}805\ \text{kip-ft})\\
&1{,}945{,}437\ \text{kN - m} > 1{,}552{,}822\ \text{kN - m} \quad \Rightarrow \quad \text{Okay}
\end{aligned}$$

(3) Check for bidirectional strength criteria.

(a) In longitudinal direction:

$$\frac{1{,}438{,}375}{2{,}539{,}236}+0.3\cdot\frac{1{,}552{,}822}{1{,}945{,}437}=0.81<1.0\Rightarrow \text{OK}$$

(b) In transverse direction:

$$\frac{1{,}552{,}822}{1{,}945{,}437}+0.3\frac{1{,}438{,}375}{2{,}539{,}236}=0.97<1.0\Rightarrow \text{OK}$$

f. Final Design Check. Since the design was changed to satisfy moment requirement, one last check is required to determine whether the tower fails in shear or in bending. This can be achieved by comparing the ratios of nominal and demand forces:

If $\dfrac{M_n}{V_n} < \dfrac{M_u}{V_u}$ then the tower will fail in bending.

In order to evaluate this inequality, shear capacities of the structure should be recomputed with the new re-bar arrangement.

(1) Final check for MDE.

(a) In longitudinal direction:

$$V_{n,long} = V_c + V_{s,long} = 32{,}910 + 127{,}904 =$$
$$= 136{,}692 \text{ kN } (30{,}731 \text{ kips})$$

$$\frac{2{,}355{,}697}{136{,}692} = 17.23 < \frac{1{,}962{,}122}{63{,}641} = 30.83 \quad \Rightarrow \quad \text{Okay}$$

(b) In transverse direction:

$$V_{n,trans} = V_c + V_{s,trans} = 32{,}910 + 98{,}593 =$$
$$= 111{,}777 \text{ kN } (25{,}131 \text{ kips})$$

$$\frac{1{,}801{,}361}{111{,}777} = 16.12 < \frac{2{,}118{,}555}{62{,}956} = 33.65 \quad \Rightarrow \quad \text{Okay}$$

(2) Final check for OBE.

(a) In longitudinal direction:

$$\frac{2{,}539{,}236}{136{,}692} = 18.58 < \frac{1{,}294{,}538}{63{,}641} = 20.34 \quad \Rightarrow \quad \text{Okay}$$

(b) In the transverse direction:

$$\frac{1{,}945{,}437}{111{,}777} = 17.40 < \frac{1{,}397{,}540}{62{,}956} = 22.20 \quad \Rightarrow \quad \text{Okay}$$

C-17. Final Design Using a Computer Program

Due to the relative complexity of the section, performing the final design using a computer program is recommended. In this example, the Concrete General Strength Investigation (CGSI) is used (Figure C-23 and Table C-16).

a. The bending moment strength shall equal, or exceed, the bending moment demands by the following equation:

(1) In longitudinal direction:

$$\mathrm{U}_{\mathrm{long}} = D_{long} + L_{long} \pm \frac{\mathrm{M}_{\mathrm{long}}}{\mathrm{R}_{\mathrm{M}}} \pm 0.3 \cdot \frac{M_{trans}}{R_M} \qquad \text{Load case 1 (Longitudinal)}$$

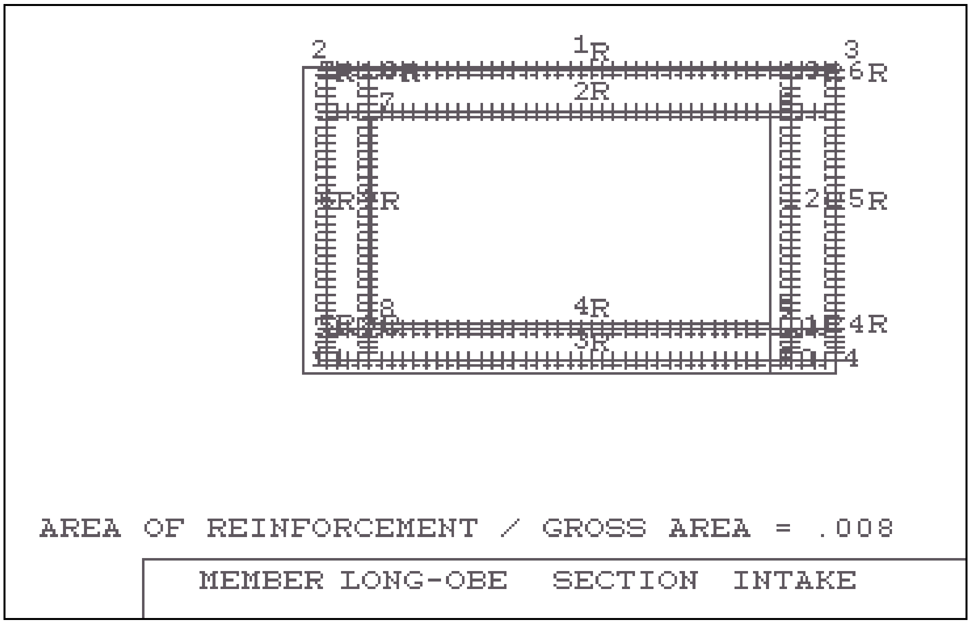


Figure C-23. Input geometry for CGSI program

Table C-16
Input Data for CGSI Analysis

In Longitudinal Direction	In Transverse Direction
1000 OBE-LONGITUDINAL	1000 OBE-TRANSVERSE
1010 MEMBER, LONG-OBE	1010 MEMBER, TRANS-OBE
1020 SECTION, INTAKE	1020 SECTION, INTAKE
1030 MATERIALS, 3000.0000, 60000.0000	1030 MATERIALS, 3000.0000, 60000.0000
1040 DIMENSION, FT	1040 DIMENSION, FT
1050 POINTS, 11	1050 POINTS, 11
1060 0.0000, 0.0000	1060 0.0000, 0.0000
1070 0.0000, 37.0000	1070 0.0000, 37.0000
1080 48.0000, 37.0000	1080 48.0000, 37.0000
1090 48.0000, 0.0000	1090 48.0000, 0.0000
1100 42.0000, 0.0000	1100 42.0000, 0.0000
1110 42.0000, 31.0000	1110 42.0000, 31.0000
1120 6.0000, 31.0000	1120 6.0000, 31.0000
1130 6.0000, 6.0000	1130 6.0000, 6.0000
1140 42.000, 6.0000	1140 42.000, 6.0000
1150 42.000, 0.0000	1150 42.000, 0.0000
1160 0.0000, 0.0000	1160 0.0000, 0.0000
1170 REINFORCE, 16	1170 REINFORCE, 16
1180 47, 3.12, 1.0, 36.0, 47.0, 36.0	1180 47, 3.12, 1.0, 36.0, 47.0, 36.0
1190 47, 3.12, 1.0, 31.0, 47.0, 31.0	1190 47, 3.12, 1.0, 31.0, 47.0, 31.0
1200 47, 3.12, 1.0, 1.0, 47.0, 1.0	1200 47, 3.12, 1.0, 1.0, 47.0, 1.0
1210 47, 3.12, 1.0, 5.0, 47.0, 5.0	1210 47, 3.12, 1.0, 5.0, 47.0, 5.0
1220 4, 3.12, 1.0, 2.0, 1.0, 4.0	1220 4, 3.12, 1.0, 2.0, 1.0, 4.0
1230 26, 3.12, 1.0, 6.0, 1.0, 30.0	1230 26, 3.12, 1.0, 6.0, 1.0, 30.0
1240 4, 3.12, 1.0, 32.0, 1.0, 35.0	1240 4, 3.12, 1.0, 32.0, 1.0, 35.0
1250 4, 3.12, 5.0, 2.0, 5.0, 4.0	1250 4, 3.12, 5.0, 2.0, 5.0, 4.0
1260 26, 3.12, 5.0, 6.0, 5.0, 30.0	1260 26, 3.12, 5.0, 6.0, 5.0, 30.0
1270 4, 3.12, 5.0, 32.0, 5.0, 35.0	1270 4, 3.12, 5.0, 32.0, 5.0, 35.0
1280 4, 3.12, 43.0, 2.0, 43.0, 4.0	1280 4, 3.12, 43.0, 2.0, 43.0, 4.0
1290 26, 3.12, 43.0, 6.0, 43.0, 30.0	1290 26, 3.12, 43.0, 6.0, 43.0, 30.0
1300 4, 3.12, 43.0, 32.0, 43.0, 35.0	1300 4, 3.12, 43.0, 32.0, 43.0, 35.0
1310 4, 3.12, 47.0, 2.0, 47.0, 4.0	1310 4, 3.12, 47.0, 2.0, 47.0, 4.0
1320 26, 3.12, 47.0, 6.0, 47.0, 30.0	1320 26, 3.12, 47.0, 6.0, 47.0, 30.0
1330 4, 3.12, 47.0, 32.0, 47.0, 35.0	1330 4, 3.12, 47.0, 32.0, 47.0, 35.0
1340 A77	1340 A77
1350 PLOT	1350 PLOT
1410 LOAD, LONG-OBE, 1	1410 LOAD, TRAN-OBE, 1
1420 1060835.000, 343573.0000, -16498.0000,	1420 318251.0000, 1145242.000, -16498.0000,
24.0000, 18.5000	24,0000, 18.5000
1430 EXIT	1430 EXIT

(2) In transverse direction:

$$\mathrm{U}_{\mathrm{trans}} = D_{trans} + L_{trans} \pm \frac{\mathrm{M}_{\mathrm{trans}}}{\mathrm{R}_{\mathrm{M}}} \pm 0.3 \cdot \frac{M_{long}}{R_M} \qquad \text{Load case 2 (Transverse)}$$

b. The examination of the results from CGSI (Figures C-24 and C-25) shows that the approximately designed section is adequate to carry the demand loads. Thus, it is confirmed that the section requires #11 reinforcing bar spaced at 6 in in horizontal and vertical directions, as shown in Figure C-26.

C-18. Sliding Shear Failure Check

A total number of 4*(95+73-4)=656 #11 bars (#11 @ 15cm (6 in.) center space) cross the potential horizontal sliding shear plane of the base of the tower. The nominal sliding shear resistance is determined using Equation 4-19:

$$\begin{aligned} V_{SL} &= P_u + 0.25 \cdot f_y \cdot A_{VF} \\ &= 52{,}422 + 0.25(413.69)(656)(10.06/10) \end{aligned}$$

$$\begin{aligned} V_{SL} &= 120{,}704\,\text{kN} \quad (27{,}134\,\text{kips}) \\ &> 74{,}871\,\text{kN} \quad (16{,}831\,\text{kips}) \quad \Rightarrow \quad \text{Okay} \end{aligned}$$

Since the nominal sliding shear resistance is significantly greater than the sliding shear demand, sliding shear is not a problem.

C-19. Anchorage of Moment Steel in Base Slab

a. Equation 4-11 can be used to determine the required anchorage length. The minimum length, however, should be at least 30 bar diameters.

$$l_a = \frac{k_s \cdot d_b}{\sqrt{f'_c}\left(1 + 2.5 \cdot \dfrac{c}{d_b}\right)} \qquad \text{(in psi units)}$$

where k_s is the re-bar constant calculated by

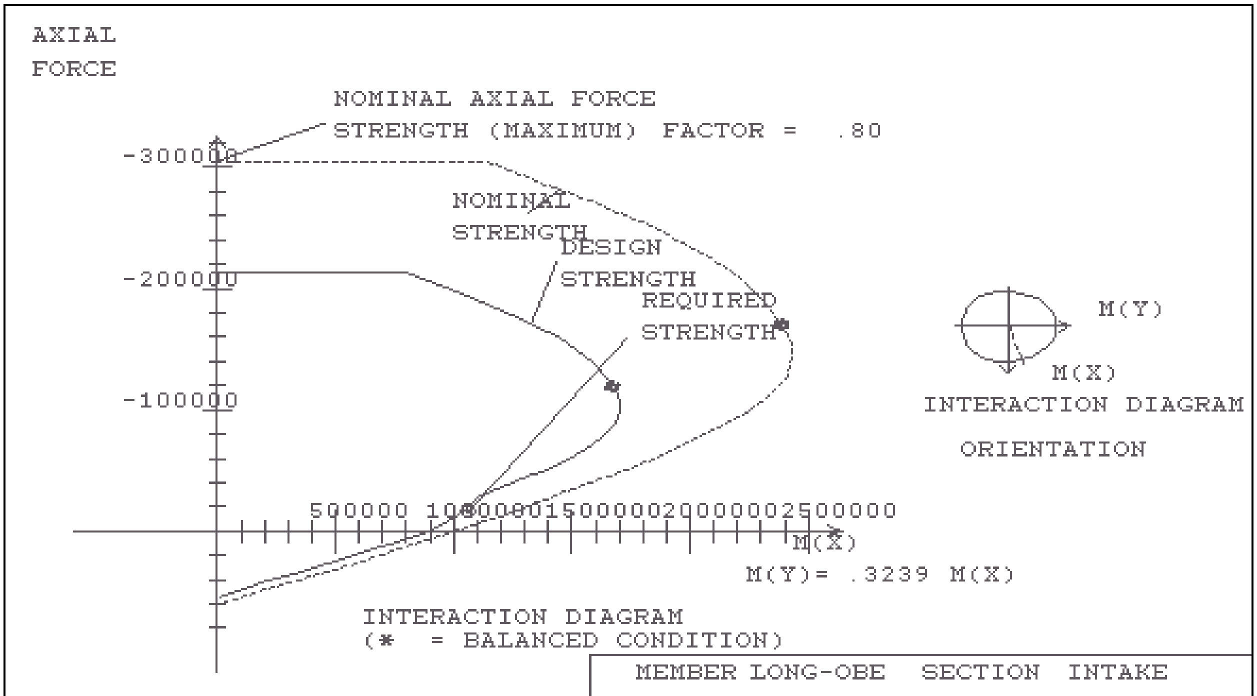
$$k_s = \frac{f_y - 11{,}000}{4.8} = \frac{60{,}000 - 11{,}000}{4.8} = 10{,}208$$

d_b = #11 bar diameter =3.58 cm (1.41 in)

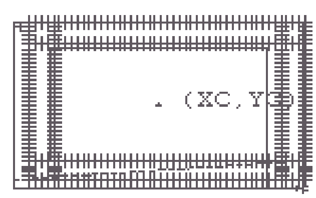
$$c = \min(13.45, 5.83) = 5.83\,\text{cm} \quad (2.3\,\text{in})$$

which is lesser of the clear cover over the bar or bars or half of the clear spacing between adjacent bars.

$$2.5\left(\frac{c}{d_b}\right) = 2.5\left(\frac{5.83}{3.58}\right) = 4.07$$

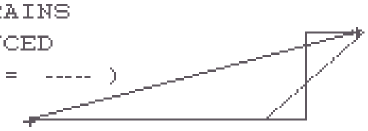



```
Fig. 2.  1.  Strain Analysis and Interaction
             Diagram for Load: LONG-OBE

        Required Strength (U)
        (Referenced to XC,YC)
          P(U) =          -16498.000 kips.
          M(UX)=         1060835.000 kip-ft.
          M(UY)=          343573.000 kip-ft.

          (XC,YC)= 288.000, 222.000

          Percentage of Balanced Reinforcement =    7.26%

          Strains at U/Phi (Phi =  .81)
          (Neutral Axis =  ----- )

          ACI 318-77 STANDARD-
             PHI FROM ACI CODE
             e MAX =   .00300
             FY NOT LIMITED
```



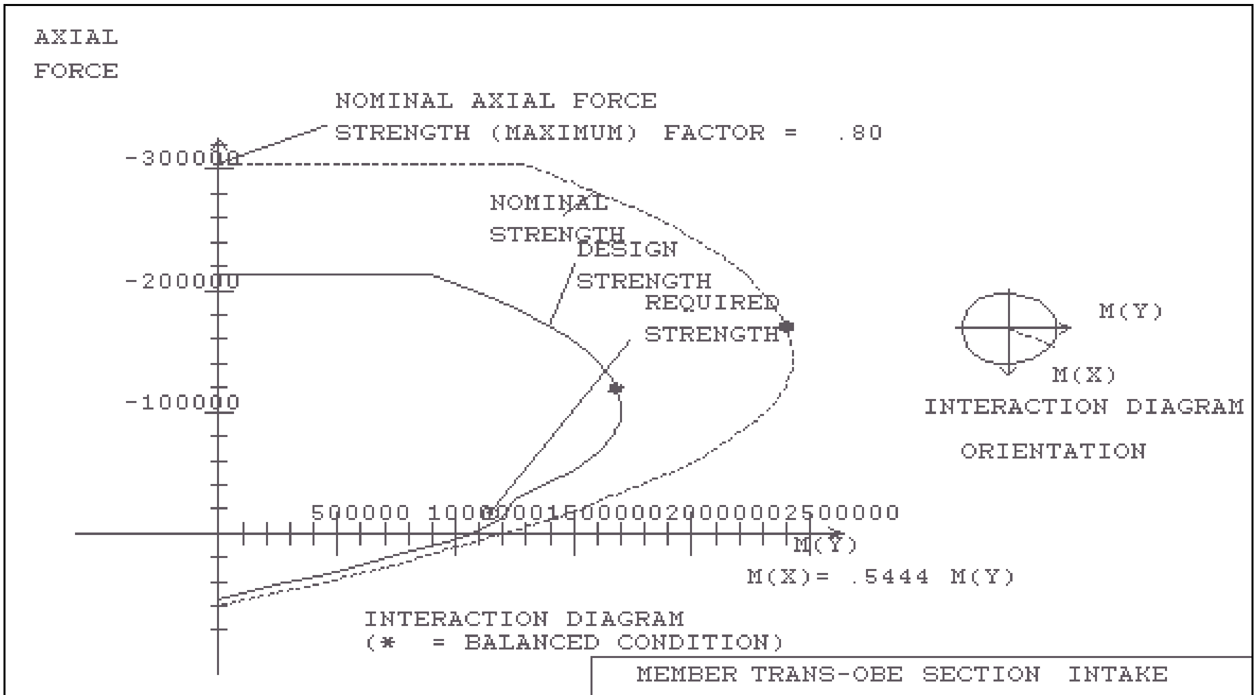
* MAX. CONC. STR.=.00268
\+ MAX. STEEL STR.=.01442

ACTUAL STRAINS
VS. BALANCED
(BALANCED = -----)

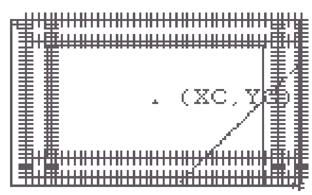


MEMBER LONG-OBE SECTION INTAKE

Figure C-24. CGSI analysis results in longitudinal direction

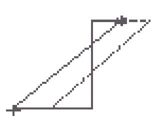



```
Fig. 2.  1.  Strain Analysis and Interaction
             Diagram for Load: TRAN-OBE

         Required Strength (U)
         (Referenced to XC,YC)
           P(U) =         -16498.000 kips.
           M(UX)=         623464.000 kip-ft.
           M(UY)=        1145242.000 kip-ft.

           (XC,YC)= 288.000, 222.000

           Percentage of Balanced Reinforcement =    6.10%

           Strains at U/Phi (Phi =  .81)
           (Neutral Axis = ----- )

           ACI 318-77 STANDARD-
              PHI FROM ACI CODE
              e MAX =   .00300
              FY NOT LIMITED
```



* MAX. CONC. STR.=.00154
\+ MAX. STEEL STR.=.00407

ACTUAL STRAINS
VS. BALANCED
(BALANCED = -----)



MEMBER TRANS-OBE SECTION INTAKE

Figure C-25. CGSI analysis results in transverse direction

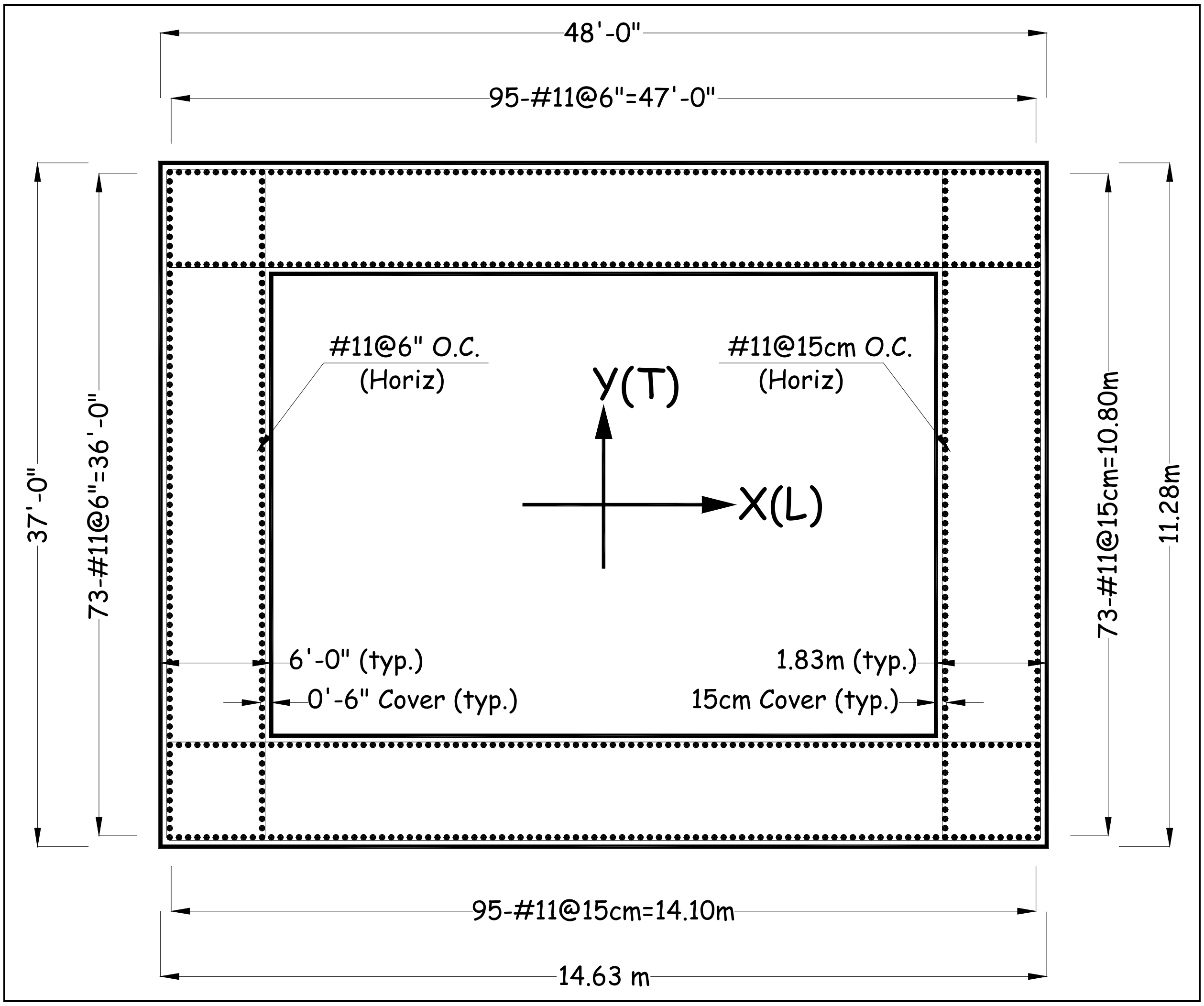


Figure C-26. Final reinforcing bar arrangement after designing for bending resistance

(ratio of $\left(c/d_b\right)$=1.63 should not exceed 2.5)

$$l_a = \frac{10{,}208\left(1.41\right)}{\sqrt{3{,}000}\left(1.0+4.07+0.0\right)} = 51.83\,\text{in}\;(131.65\,\text{cm})$$

l_a cannot be less than

$$l_a = 30 \cdot d_b = 30 \cdot 3.58 = 107.44 \text{ cm (42.3 in)}$$

The final length for straight anchorage is

$$l_a = 1.10 \text{ m (43.00 in)}$$

b. For 90-degree standard hooks, the effective anchorage length in inches is:

$$l_a = 1{,}200 \cdot \mathrm{d_b} \cdot \frac{f_y}{60{,}000\sqrt{f'_c}}$$

$$= 1{,}200(1.41)\frac{60{,}000}{60{,}000\sqrt{3{,}000}} = 30.89 \text{ in } (78.46 \text{ cm})$$

l_a cannot be less than

$$l_a = 15 \cdot d_b = 15 \cdot 3.58 = 53.72 \text{ cm } (21.15 \text{ in})$$

Final hooked anchorage length is

$$l_a = 0.80 \text{ m } (31.00 \text{ in})$$

C-20. Design of Reinforcing Steel Splices

The lap splice length required for new and existing towers should not be less than:

$$l_{s,\min} = 1{,}860\frac{d_b}{\sqrt{f'_c}} = 1{,}860\frac{1.41}{\sqrt{3{,}000}} = 48 \text{ in} \quad (1.25 \text{ m})$$

When concrete compressive strains exceed 0.002 in/in, transverse confinement steel should be provided at splice locations. The minimum area of transverse reinforcement necessary to prevent bond deterioration is given by Equation 4-13:

$$A_{tr} = \frac{s}{l_s} \cdot \frac{f_y}{f_{yt}} \cdot A_b = \frac{15}{125} \cdot \frac{413.69}{413.69} \cdot 10.06$$

$$= 1.21 \text{ cm}^2 \ (0.19 \text{ in}^2) < A_{s,\#11} = 10.06 \text{ cm}^2 \ (1.56 \text{ in}^2)$$

Since the required transverse reinforcing bar area is less than what is provided by #11 horizontal bars, the additional transverse reinforcement is not necessary.

C-21. Comprehensive Spalling Check

It can be assumed that the comprehensive spalling failures will not occur, when at ultimate load conditions, the ratio of the neutral axis depth to the total depth of the tower cross section is less than 15 percent (Equation 4-18):

a. In the longitudinal direction:

$$\frac{c}{d} = \frac{1.55}{14.63 - 0.28} = 0.140 < 0.15 \quad \Rightarrow \text{Okay}$$

where c = 1.55 is the location of neutral axis.

b. In the transverse direction:

$$\frac{c}{d} = \frac{1.09}{11.28 - 0.28} = 0.075 < 0.15 \quad \Rightarrow \text{Okay}$$

where c = 1.09 is the location of neutral axis.

C-22. Minimum Tensile Reinforcement Requirements

In the design of new intake towers, the nominal moment capacity should equal or exceed 120 percent of the uncracked moment capacity, or "cracking moment." This ensures that there is an adequate reinforcement to handle the energy, which is suddenly transferred from the concrete to the reinforcing steel. Assuming that the axial load is concentric, the cracking moment is determined by Equation 4-9:

$$M_{CR} = \left(\frac{I_g}{C}\right) \cdot \left(\frac{P_u}{A_G} + f_r\right)$$

In longitudinal direction:

$$I_g = \frac{11.28(14.63)^3 - 7.62(10.97)^3}{12}$$

$$I_g = 2104.16\ \text{m}^4 \quad (243{,}792\ \text{ft}^4)$$

$$C = \frac{14.63}{2} = 7.32\ \text{m} \quad (24\ \text{ft})$$

$$P_u = 52{,}422\ \text{kN} \quad (11{,}784\ \text{kips})$$

$$A_G = 14.63(11.28) - 10.97(7.62)$$

$$= 81.38\ \text{m}^2 \quad (876\ \text{ft}^2)$$

$$f_r = 0.62\sqrt{f_c'} = 0.62\sqrt{20.68} = 2.82\ \text{MPa}$$

$$f_r = 7.50\sqrt{f_c'} = 7.50\sqrt{3{,}000} = 410.79\ \text{psi}$$

$$M_{CR} = \left(\frac{2{,}104.16}{7.32}\right) \cdot \left(\frac{52{,}422}{81.38} + 2{,}820\right)$$

$$M_{CR} = 996{,}365\ \text{kN-m} \quad (734{,}843\ \text{kip-ft})$$

$$M_n = 1{,}945{,}437\ \text{kN-m} \quad (1{,}434{,}805\ \text{kip-ft})$$

$$\frac{M_n}{M_{CR}} = \frac{1{,}945{,}437}{996{,}365} = 1.95 > 1.20 \qquad \text{Okay}$$

C-23. Conclusions and Recommendations

a. Application of load and moment reduction factors and the use of different performance criteria for MDE and OBE make it difficult to determine the critical load case in advance. The design was therefore carried out for both MDE and OBE load cases. During the design it became necessary to increase the amount of reinforcing bars several times.

b. Since strength and serviceability performance criteria must be satisfied, the following design steps are recommended:

- Carry out design for both the MDE and OBE load cases.

- Check the design to ensure it meets the MDE performance criteria.

- Check the design to ensure it meets the OBE performance criteria.

- Use a computer program such as CGSI to supplement hand calculations and check the final design.

Appendix D
Refined Procedure for the Determination of Hydrodynamic Added Masses

D-1. General

a. This appendix presents a refined procedure for finding the hydrodynamic added masses for intake towers. The refined procedure, developed by Goyal and Chopra, applies to doubly symmetric cross sections in plan. The treatment here is abbreviated. For more complete details, see Goyal and Chopra (1989). The added-mass concept is used to account for the hydrodynamic effects associated with water inside the tower and surrounding the tower during an earthquake. The procedure can be used with all of the analysis procedures presented in Appendixes B or C and is applicable for both uniform and nonuniform towers. Analytical solutions for hydrodynamic added masses are available only for circular towers (tapered or uniform) and for elliptic towers (uniform). In the following approximate procedure for tapered rectangular towers, a square or rectangular section is transformed first into an equivalent elliptical section and then into an equivalent circular section. The solution for the equivalent circular section so obtained is then taken as an approximate solution for the square or rectangular section.

b. The following sections demonstrate the calculations for hydrodynamic added masses on a rectangular intake tower. The tower is a nonuniform tower with a non-circular cross section. Figure D-1 shows the geometry of the tower. The normal pool elevation both outside and inside the tower is 1016.82 m (3336.0 ft). The water depth outside the tower H_o is 41.45 m (136 ft), and the depth inside the tower H_i is 39.62 m (130 ft). The tower is separated into discrete elements. A discrete element is an element having uniform cross-sectional properties throughout its height, both inside and outside. The tower so discretized is shown in Figure D-2. Figure D-3 depicts the basic geometry and symbols for defining the cross section. The geometry and symbols in Figure D-3 are those used in later calculations to define the hydrodynamic added masses for a noncircular section. The dimensional and cross-sectional properties of each discrete element are determined and are listed in Table D-1. For nonuniform towers of arbitrary cross section having two axes of symmetry, the hydrodynamic added mass is determined by use of the following steps:

(1) Step 1. At each discrete element, determine the cross-sectional radius $\tilde{r}_o(z)$ of the equivalent circular tower for the outside hydrodynamic mass and the equivalent cross-sectional radius $\tilde{r}_i(z)$ for the inside hydrodynamic added mass. Assume that the entire mass of the structure and of the inside and outside added masses are lumped at the midpoint of each discrete element. Determine the distance z to the midpoint of each structural mass m_s, the distance z_o to each outside added mass m_a^o and the distance z_i to each inside added mass m_a^i. The values corresponding to this procedure are shown in Table D-1. The remaining columns in Table D-1 list the cross-sectional dimensions at the level of each lumped mass and the cross-sectional area of the concrete at each of those levels. These values will be needed in later calculations. From this point onward, the calculations are tabulated in the order made. To follow the calculations for the example tower, refer to the tables of calculations given at the end of this appendix. For earthquake motions parallel to the long axis, Table D-2 shows the calculations for outside added masses and Table D-3 shows the calculations for inside added masses. The rectangular cross section at each discrete element is transformed first into an equivalent ellipse through the following relationships: the cross-sectional dimensions are shown symbolically in Figure D-3 and the actual dimensions are those listed in Table D-1.

$\tilde{a}_o/\tilde{b}_o = a_o/b_o$ and $\tilde{a}_i/\tilde{b}_i = a_i/b_i$

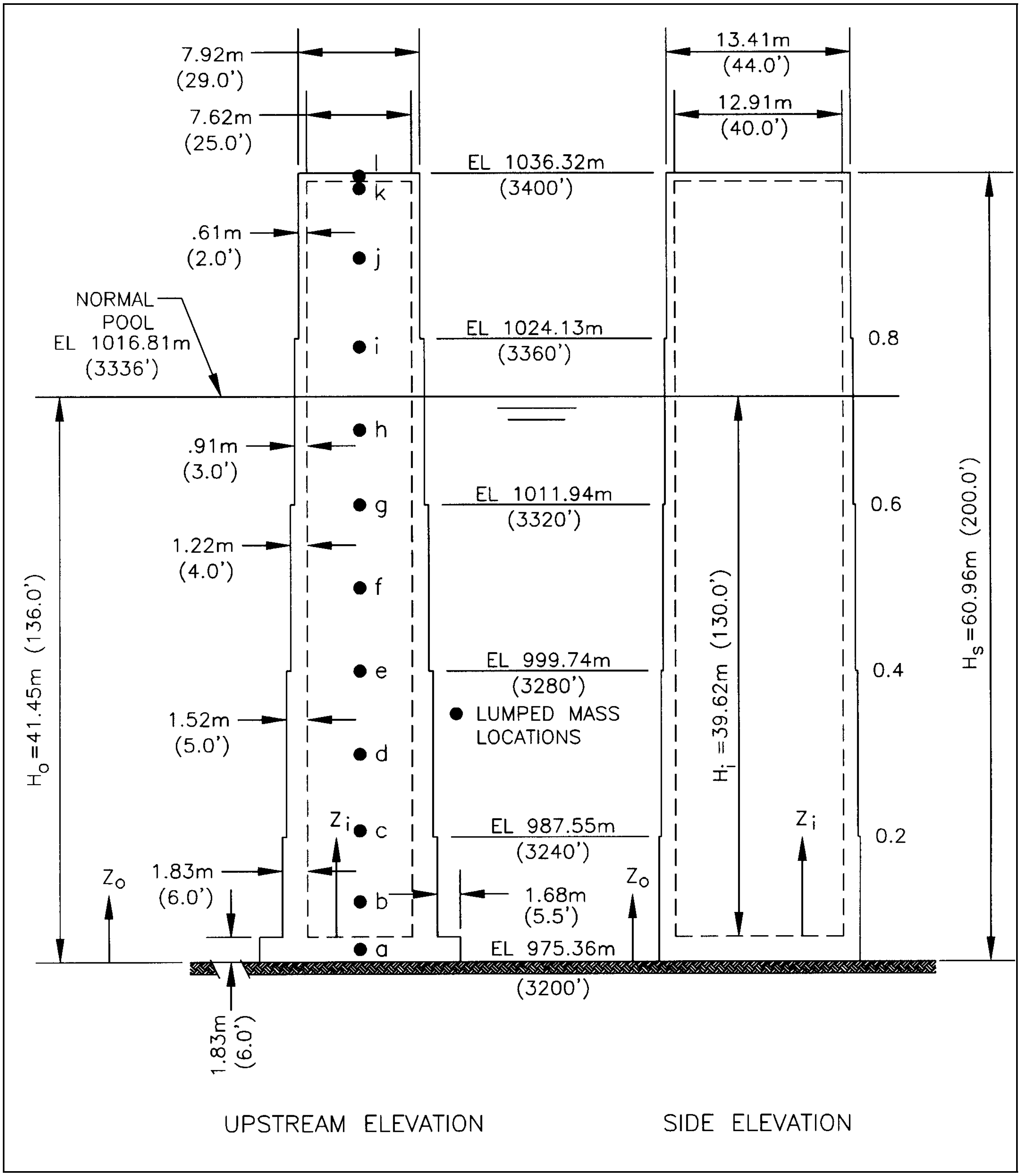


Figure D-1. Tower geometry

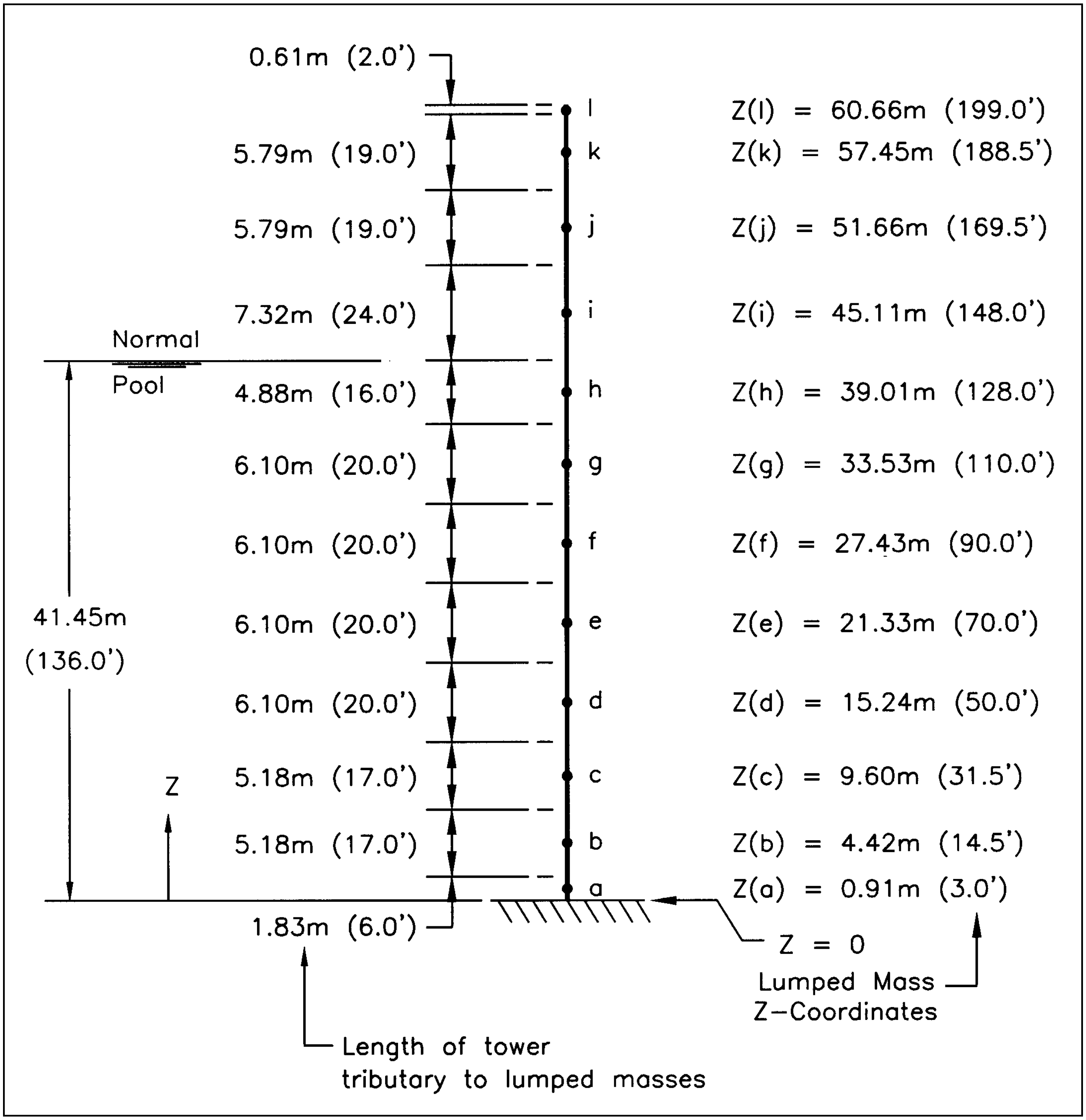


Figure D-2. Tower lumped-mass beam element idealization

where a_o, b_o, a_i, and b_i are depicted in Figure D-3 and $\tilde{a}_o$, $\tilde{b}_o$, $\tilde{a}_i$ and $\tilde{b}_i$ are the major and minor axes of the equivalent elliptical sections outside and inside the tower. The equivalent elliptical section is further transformed into an equivalent circular section. In towers for which _ $\leq a_o/b_o \leq 3$, the radius $\tilde{r}_o(z)$ is found through the use of Figure D-4 for the outside added mass; radius $\tilde{r}_i(z)$ is found similarly through the use of Figure D-5 for the inside added mass. Alternatively, the tabular form of Figure D-4 is given in Table D-4; it will, of course, yield the same results as Figure D-4. There is no comparable need for a tabular form of Figure D-5, since $\tilde{r}_i(z)$ can be calculated directly by the simple formula:

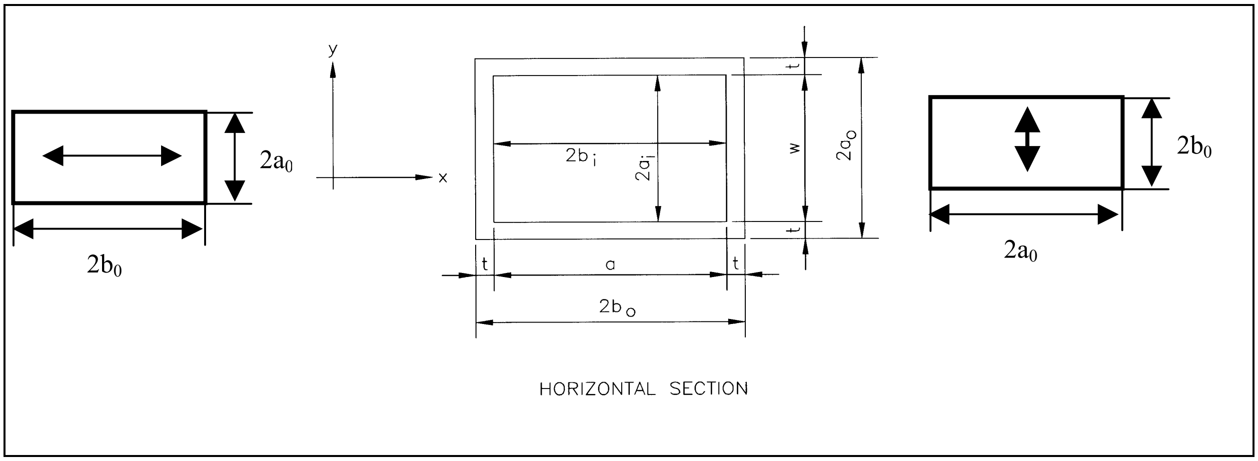


Figure D-3. Cross-sectional geometry

Table D-1
Physical Properties of the Example Tower

Mass	*z*	z_o	z_i	*w*	*d*	*t*	Outside Area a_o	Inside Area a_i
l	60.66			8.84	13.41			
	(199.0)	--	--	(29.0)	(44.0)	--	--	--
k	57.45			7.62	12.19	0.61		
	(188.5)	--	--	(25.0)	(40.0)	(2.0)	--	--
j	51.66	--	--	7.62	12.19	0.61	--	--
	(169.5)			(25.0)	(40.0)	(2.0)		
i	45.11	--	--	7.62	11.89	0.91	--	--
	(148.0)			(25.0)	(39.0)	(3.0)		
h	39.01	39.01	37.19	7.62	11.89	0.91	129.60	90.58
	(128.0)	(128.0)	(122.0)	(25.0)	(39.0)	(3.0)	(1395)	(975)
g	33.53	33.53	31.70	7.62	11.58	1.22	141.02	88.26
	(110.0)	(110.0)	(104.0)	(25.0)	(38.0)	(4.0)	(1518)	(950)
f	27.43	27.43	25.60	7.62	11.58	1.22	141.02	88.26
	(90.0)	(90.0)	(84.0)	(25.0)	(38.0)	(4.0)	(1518)	(950)
e	21.30	21.30	19.51	7.62	11.28	1.52	152.82	85.93
	(70.0)	(70.0)	(64.0)	(25.0)	(37.0)	(5.0)	(1645)	(925)
d	15.24	15.24	13.41	7.62	11.28	1.52	152.82	85.93
	(50.0)	(50.0)	(44.0)	(25.0)	(37.0)	(5.0)	(1645)	(925)
c	9.60	9.60	7.27	7.62	10.97	1.83	164.99	83.61
	(31.5)	(31.5)	(25.5)	(25.0)	(36.0)	(6.0)	(1776)	(900)
b	4.42	4.42	2.59	7.62	10.97	1.83	164.99	83.61
	(14.5)	(14.5)	(8.5)	(25.0)	(36.0)	(6.0)	(1776)	(900)
a	0.91	0.91	0.0	14.63	14.63	--	214.04	--
	(3.0)	(3.0)	(0.0)	(48.0)	(48.0)		(2304)	

Note: Dimensional values are in SI units of meters, with non-SI units of feet in parentheses. Area values are in SI units of m^2, with non-SI units of ft^2 in parentheses.
w = width
d = depth
t = thickness

Table D-2
Tabular Calculations for Outside Hydrodynamic Added Mass, Example Intake Tower, Earthquake Motion Parallel to the Longitudinal Axis

Mass	z	z_O	a_O	b_O	A_O
h	39.01 (128.0)	39.01 (128.0)	4.72 (15.5)	6.86 (22.50)	129.60 (1395)
g	33.53 (110.0)	33.53 (110.0)	5.03 (16.5)	7.01 (23.00)	141.02 (1518)
f	27.43 (90.0)	27.43 (90.0)	5.03 (16.5)	7.01 (23.00)	141.02 (1518)
e	21.30 (70.0)	21.30 (70.0)	5.33 (17.5)	7.16 (23.50)	152.82 (1645)
d	13.24 (50.0)	15.24 (50.0)	5.33 (17.5)	7.16 (23.50)	152.82 (1645)
c	9.60 (31.5)	9.60 (31.5)	5.64 (18.5)	7.31 (24.00)	164.99 (1776)
b	4.42 (14.5)	4.42 (14.5)	5.64 (18.5)	7.31 (24.00)	164.99 (1776)
a	0.91 (3.0)	0.91 (3.0)	2.31 (24.00)	7.31 (24.00)	214.04 (2304)

Mass	a_O/b_O	r_O/a_O	$\underline{r}_O/H_O$	$\underline{z}_O/H_O$	$\dfrac{m_a^\circ(z)}{m_\infty^\circ}$	$\dfrac{m_\infty^\circ}{\rho_w a_o}$	m_∞°	$m_a^\circ(z)$
h	0.689	1.18	0.135	0.941	0.49	0.88	114.11 (2379)	59.81 (1.166)
g	0.717	1.16	0.140	0.809	0.77	0.90	127.01 (2648)	97.61 (2.039)
f	0.717	1.16	0.140	0.662	0.88	0.90	127.01 (2648)	111.52 (2.330)
e	0.745	1.13	0.146	0.515	0.91	0.93	142.22 (2965)	120.11 (2.509)
d	0.745	1.13	0.146	0.368	0.94	0.93	142.22 (2965)	133.41 (2.787)
c	0.771	1.11	0.151	0.232	0.95	0.96	158.48 (3304)	150.27 (3.139)
b	0.771	1.11	0.151	0.107	0.96	0.96	158.48 (3304)	151.94 (3.172)
a	1.000	1.00	0.176	0.022	0.94	1.19	254.84 (5313)	239.05 (4.994)

Note: Mass values are in SI units of kN-sec^2/m^2 with non-SI units of kip-sec^2/ft^2 in parentheses. Dimensional values are in SI units of meters, with non-SI units of feet in parentheses. Area values are in SI units of m^2, with non-SI units of ft^2 in parentheses.

Table D-3
Tabular Calculations for Inside Hydrodynamic Added Mass, Example Intake Tower, Earthquake Motion Parallel to the Longitudinal Axis

Mass	Z	Z_i	a_i	b_i	A_i
h	39.01 (128.0)	37.19 (122.0	3.81 (12.5)	5.94 (19.5)	90.56 (975)
g	33.53 (110.0)	31.70 (104.0)	3.81 (12.5)	5.79 (19.00)	88.26 (950)
f	27.43 (90.0)	25.60 (84.0)	3.81 (12.5)	5.79 (19.00)	88.26 (950)
e	21.30 (70.0)	19.51 (64.0)	3.81 (12.5)	5.64 (18.50)	85.93 (925)
d	15.24 (50.0)	13.41 (44.0)	3.81 (12.5)	5.64 (18.50)	85.93 (925)
c	9.60 (31.5)	7.77 (25.5)	3.81 (12.5)	5.49 (18.00)	274.32 (900)
b	4.42 (14.5)	2.59 (8.5)	3.81 (12.5)	5.49 (18.00)	274.32 (900)
a	0.91 (3.0)	--	--	--	--

Mass	a_i / b_i	r_i	r_i / H_i	$\underline{z}_i / H_i$	$\dfrac{m^i_\infty}{\rho_w A_i}$	$m^i_\infty (z)$
h	0.641	6.71 (22.00)	0.169	0.938	0.55	49.74 (1.039)
g	0.658	6.53 (21.44)	0.165	0.800	0.91	80.18 (1.675)
f	0.658	6.53 (21.44)	0.165	0.646	0.98	86.36 (1.804)
e	0.676	6.36 (20.87)	0.161	0.492	1.00	85.83 (1.793)
d	0.676	6.36 (20.87)	0.161	0.338	1.00	85.83 (1.793)
c	0.694	6.19 (20.32)	0.156	0.196	1.00	83.50 (1.744)
b	0.694	6.19 (20.32)	0.156	0.065	1.00	83.50 (1.744)
a	--	--	--	--	--	--

Note: Dimensional values are in SI units of meters, with non-SI units of feet in parentheses. Area values are in SI units of m^2, with non-SI units of ft^2 in parentheses. Mass values are in SI units of kN-sec^2/m^2 with non-SI units of kip-sec^2/ft^2 in parentheses.

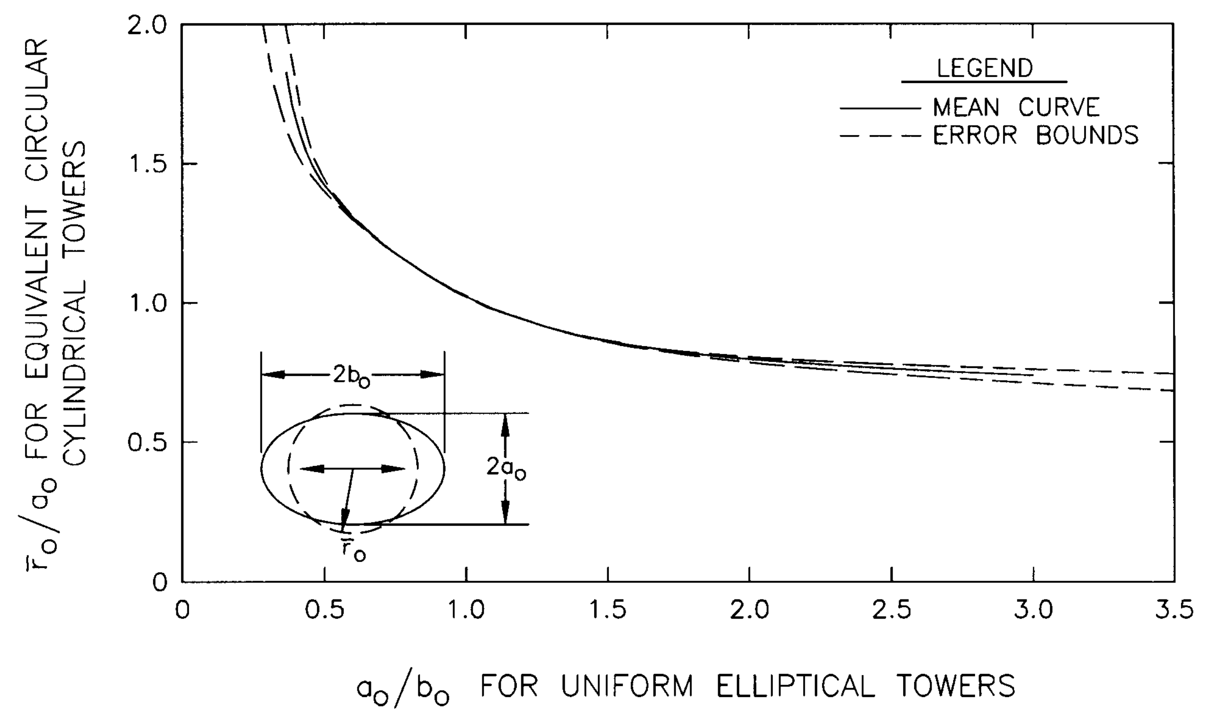


Figure D-4. Equivalent radius of circular towers for outside hydrodynamic added mass

$$\tilde{r}_i = \sqrt{\frac{A_i}{\pi} \bullet \frac{b_i}{a_i}}$$

Essentially all intake towers fall into the classification in which _ $\leq a_o/b_o \leq 3$. For those structures that do not fall into this classification, a more general solution is given in Goyal and Chopra 1989. The values of the ratios $\tilde{r}_o(z)/a_o$ and $\tilde{r}_i(z)/a_i$ will yield the radii of the equivalent step-tapered circular tower at each lumped mass (or discrete element). The values of r_o/H_o and r_i/H_i are calculated from these values for use in the next step, where, at each lumped mass,

$$r_o/H_o = \tilde{r}_o/H_o \quad \text{and} \quad r_i/H_i = \tilde{r}_i/H_i$$

The values of $\tilde{r}_o/H_o$ and $\tilde{r}_i/H_i$ at each lumped mass are entered into the tables of calculations, Tables D-2 and D-3, for outside and inside masses, respectively. The computed values of z_o/H_o and z_i/H_i are also entered.

(2) Step 2. Determine the normalized hydrodynamic added mass for the outside water $m_a^o(z)/m_\infty^o$ and for the inside water $m_a^i(z)/\rho_w A_i$ for each discrete element. The normalized hydrodynamic added masses are determined from Table D-5 for each value of r_o/H_o and from Table D-6 for each value of r_i/H_i. The values of $m_a^o(z)/m_\infty^o$ and $m_a^i(z)/\rho_w A_i$ thus found are entered in the appropriate columns of the tables of calculations, Tables D-2 and D-3. (As a matter of interest, the curves of Figures C-7 and C-8 given earlier in Appendix C are plots of the tabulated values given in Tables D-5 and D-6. Either the curves or the tables may be used to find the normalized hydrodynamic added masses.)

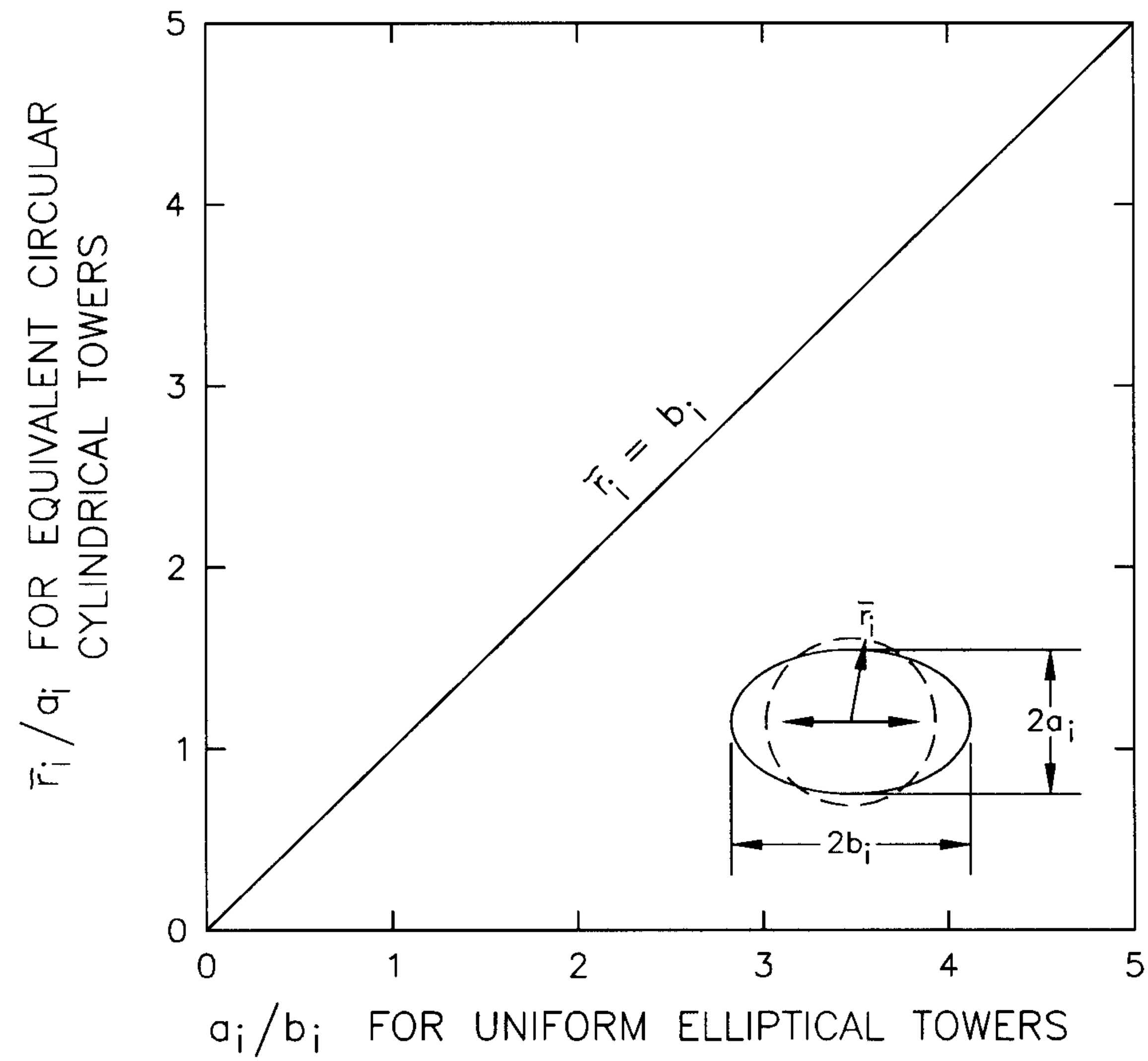


Figure D-5. Equivalent radius of circular towers for inside hydrodynamic added mass

Table D-4
Equivalent Radius of Circular Towers For Outside Hydrodynamic Added Mass (Adapted from Goyal and Chopra, 1989, Table 8-3)

a_o/b_o	$\tilde{r}_o/a_o$	a_o/b_o	$\tilde{r}_o/a_o$	a_o/b_o	$\tilde{r}_o/a_o$	a_o/b_o	$\tilde{r}_o/a_o$
0.33	1.80	1.00	1.00	1.70	0.85	2.40	0.79
0.40	1.63	1.10	0.97	1.80	0.84	2.50	0.79
0.50	1.39	1.20	0.94	1.90	0.83	2.60	0.78
0.60	1.27	1.30	0.92	2.00	0.82	2.70	0.78
0.70	1.17	1.40	0.90	2.10	0.81	2.80	0.77
0.80	1.09	1.50	0.88	2.20	0.81	2.90	0.77
0.90	1.04	1.60	0.86	2.30	0.80	3.00	0.76

Table D-5
Normalized Outside Hydrodynamic Added Mass $m_a^o(z)/m_\infty^o$ on Circular Cylindrical Towers

	r_o/H_o									
z/H_o	0.05	0.10	0.15	0.20	0.25	0.30	0.40	0.50	0.60	0.80
1.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.98	0.46	0.31	0.24	0.19	0.17	0.15	0.19	0.10	0.09	0.07
0.96	0.63	0.46	0.37	0.31	0.27	0.24	0.19	0.16	0.14	0.11
0.94	0.74	0.56	0.46	0.39	0.34	0.31	0.25	0.22	0.19	0.15
0.92	0.80	0.64	0.53	0.46	0.41	0.36	0.30	0.26	0.23	0.19
0.90	0.85	0.69	0.59	0.51	0.46	0.41	0.35	0.30	0.27	0.22
0.88	0.88	0.74	0.64	0.56	0.50	0.46	0.39	0.34	0.30	0.24
0.86	0.90	0.77	0.67	0.60	0.54	0.49	0.42	0.37	0.33	0.27
0.84	0.92	0.80	0.71	0.63	0.57	0.53	0.45	0.40	0.35	0.29
0.82	0.93	0.83	0.74	0.66	0.60	0.56	0.48	0.42	0.38	0.30
0.80	0.94	0.85	0.76	0.69	0.63	0.58	0.50	0.44	0.40	0.33
0.78	0.95	0.86	0.78	0.71	0.66	0.61	0.53	0.47	0.42	0.35
0.76	0.96	0.88	0.80	0.73	0.68	0.63	0.55	0.49	0.44	0.36
0.74	0.96	0.89	0.82	0.75	0.70	0.65	0.57	0.50	0.45	0.38
0.72	0.97	0.90	0.83	0.77	0.71	0.67	0.59	0.52	0.47	0.39
0.70	0.97	0.91	0.84	0.78	0.73	0.68	0.60	0.54	0.49	0.41
0.68	0.98	0.92	0.86	0.80	0.74	0.70	0.62	0.55	0.50	0.42
0.66	0.98	0.93	0.87	0.81	0.76	0.71	0.63	0.57	0.51	0.43
0.64	0.98	0.93	0.88	0.82	0.77	0.72	0.64	0.58	0.53	0.44
0.62	0.98	0.94	0.88	0.83	0.78	0.74	0.66	0.59	0.54	0.45
0.60	0.98	0.94	0.89	0.84	0.79	0.75	0.67	0.60	0.55	0.46
0.56	0.99	0.95	0.90	0.86	0.81	0.77	0.69	0.62	0.57	0.48
0.52	0.99	0.96	0.91	0.87	0.83	0.78	0.71	0.64	0.59	0.50
0.48	0.99	0.96	0.92	0.88	0.84	0.80	0.72	0.66	0.60	0.51
0.44	0.99	0.97	0.93	0.89	0.85	0.81	0.74	0.67	0.62	0.53
0.40	0.99	0.97	0.94	0.90	0.86	0.82	0.75	0.68	0.63	0.54
0.36	0.99	0.97	0.94	0.91	0.87	0.83	0.76	0.70	0.64	0.55
0.32	0.99	0.97	0.95	0.91	0.88	0.84	0.77	0.70	0.65	0.56
0.28	0.99	0.98	0.95	0.92	0.88	0.85	0.78	0.71	0.66	0.56
0.24	0.99	0.98	0.95	0.92	0.89	0.85	0.78	0.72	0.66	0.57
0.20	0.99	0.98	0.95	0.92	0.89	0.86	0.79	0.72	0.67	0.58
0.16	1.00	0.98	0.96	0.93	0.89	0.86	0.79	0.73	0.67	0.58
0.12	1.00	0.98	0.96	0.93	0.90	0.86	0.80	0.73	0.68	0.58
0.08	1.00	0.98	0.96	0.93	0.90	0.86	0.80	0.74	0.68	0.59
0.04	1.00	0.98	0.96	0.93	0.90	0.87	0.80	0.74	0.68	0.59
0.00	1.00	0.98	0.96	0.93	0.90	0.87	0.80	0.74	0.68	0.59

(3) Step 3. For the outside water, determine the hydrodynamic added mass per unit height m_∞^o for an infinitely long tower having the same cross section throughout its length as the base of the actual tower. For the inside water, determine the mass of the water per unit height contained inside each discrete element $\rho_w A_i$. The outside hydrodynamic added mass per unit height m_∞^o of the infinitely long tower can be found through the use of Table D-7. For a rectangular cross section, the ratio $m_\infty^o/\rho_w A_o$ is read directly from Table D-7 and entered in the table of calculations, Table D-6. This ratio $m_\infty^o/\rho_w A_o$ is multiplied by $\rho_w A_o$ at each discrete element to find the value of m_∞^o at that level; this is the value of m_∞^o entered into the table of calculations for outside hydrodynamic masses, Table D-2. The mass of the water inside each discrete element is simply $\rho_w A_i$. These values are calculated as needed.

(4) Step 4. Determine the final values of the hydrodynamic added masses per unit height $m_a^o(z)$ and $m_a^i(z)$. The calculation of the outside hydrodynamic added mass is quite direct. The normalized ratio $m_a^o(z)/m_\infty^o$ found earlier and listed in the table of calculations is multiplied by the value of m_∞^o just determined. The result is the final value of the outside hydrodynamic added mass per unit height at each discrete element, $m_a^o(z)$. The calculation of the inside hydrodynamic added mass is similar. The normalized ratio $m_a^i(z)/\rho_w A_i$ found earlier and listed in the table of calculations is multiplied by the value of $\rho_w A_i$. The result is the final value of the inside hydrodynamic added mass per unit height at each discrete element $m_a^i(z)$. The values of

Table D-6
Normalized Inside Hydrodynamic Added Mass $m_a^i(z)/\rho_w A_i$ on Circular Cylindrical Towers

	r_i/H_i									
z/H_i	**0.05**	**0.10**	**0.15**	**0.20**	**0.25**	**0.30**	**0.40**	**0.50**	**0.60**	**0.80**
1.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.98	0.59	0.39	0.30	0.24	0.20	0.18	0.14	0.12	0.10	0.08
0.96	0.81	0.59	0.47	0.39	0.34	0.30	0.24	0.20	0.18	0.14
0.94	0.91	0.72	0.59	0.50	0.44	0.39	0.32	0.27	0.24	0.19
0.92	0.96	0.81	0.68	0.59	0.52	0.47	0.39	0.33	0.29	0.24
0.90	0.98	0.87	0.75	0.66	0.58	0.53	0.45	0.39	0.34	0.28
0.88	0.99	0.91	0.81	0.72	0.65	0.59	0.50	0.44	0.39	0.32
0.86	0.99	0.94	0.84	0.77	0.70	0.64	0.55	0.48	0.43	0.35
0.84	1.00	0.96	0.88	0.81	0.74	0.68	0.59	0.52	0.47	0.38
0.82	1.00	0.97	0.91	0.84	0.78	0.72	0.63	0.56	0.50	0.41
0.80	1.00	0.98	0.93	0.87	0.81	0.75	0.66	0.59	0.53	0.44
0.78	1.00	0.99	0.94	0.89	0.83	0.78	0.69	0.62	0.56	0.47
0.76	1.00	0.99	0.96	0.91	0.86	0.81	0.72	0.65	0.59	0.49
0.74	1.00	0.99	0.97	0.92	0.88	0.83	0.74	0.67	0.61	0.51
0.72	1.00	0.99	0.97	0.94	0.89	0.85	0.77	0.70	0.64	0.54
0.70	1.00	1.00	0.98	0.95	0.91	0.87	0.79	0.72	0.66	0.56
0.68	1.00	1.00	0.98	0.96	0.92	0.88	0.81	0.74	0.68	0.58
0.66	1.00	1.00	0.99	0.96	0.93	0.90	0.82	0.76	0.70	0.60
0.64	1.00	1.00	0.99	0.97	0.94	0.91	0.84	0.77	0.72	0.61
0.62	1.00	1.00	0.99	0.98	0.95	0.92	0.85	0.79	0.73	0.63
0.60	1.00	1.00	0.99	0.98	0.96	0.93	0.87	0.81	0.75	0.64
0.56	1.00	1.00	1.00	0.99	0.97	0.94	0.89	0.83	0.78	0.67
0.52	1.00	1.00	1.00	0.99	0.98	0.96	0.91	0.85	0.80	0.70
0.48	1.00	1.00	1.00	0.99	0.98	0.97	0.92	0.87	0.82	0.72
0.44	1.00	1.00	1.00	0.99	0.99	0.97	0.94	0.89	0.84	0.74
0.40	1.00	1.00	1.00	1.00	0.99	0.98	0.95	0.90	0.86	0.76
0.36	1.00	1.00	1.00	1.00	0.99	0.98	0.95	0.92	0.87	0.77
0.32	1.00	1.00	1.00	1.00	0.99	0.99	0.96	0.93	0.88	0.79
0.28	1.00	1.00	1.00	1.00	1.00	0.99	0.97	0.93	0.89	0.80
0.24	1.00	1.00	1.00	1.00	1.00	0.99	0.97	0.94	0.90	0.81
0.20	1.00	1.00	1.00	1.00	1.00	0.99	0.98	0.95	0.91	0.82
0.16	1.00	1.00	1.00	1.00	1.00	0.99	0.98	0.95	0.91	0.82
0.12	1.00	1.00	1.00	1.00	1.00	0.99	0.98	0.95	0.92	0.83
0.08	1.00	1.00	1.00	1.00	1.00	1.00	0.98	0.96	0.92	0.83
0.04	1.00	1.00	1.00	1.00	1.00	1.00	0.98	0.96	0.92	0.83
0.00	1.00	1.00	1.00	1.00	1.00	1.00	0.98	0.96	0.92	0.83

$m_a^o(z)$ and $m_a^i(z)$ listed in Tables D-2 and D-3 are the values for the rectangular tower when the earthquake motion is parallel to the longitudinal axis. When the earthquake motion is transverse to the long axis, the dimensions a_o and b_o are reversed, as are a_i and b_i. To obtain the values of the added masses in the transverse direction, the entire computation is repeated for these new values of a_o/b_o and a_i/b_i. The procedure is identical to that for the longitudinal axis; only the numbers are different. In summary, the outside and inside hydrodynamic added masses per foot of height of the tower are shown in Table D-7 for earthquake motions in two directions. These are the values that should be added to the value of the structural mass at each discrete element to find the total magnitude of each lumped mass.

Table D-7
Summary of Added Hydrodynamic Masses (Mass per unit length of tower)

	Longitudinal Motion		Transverse Motion	
Point	**Outside Added Mass**	**Inside Added Mass**	**Outside Added Mass**	**Inside Added Mass**
h	59.81 (1.166)	49.74 (1.039)	96.27 (2.007)	63.46 (1.323)
g	97.61 (2.039)	80.15 (1.675)	164.76 (3.435)	85.67 (1.786)
f	111.52 (2.330)	86.36 (1.804)	191.19 (3.986)	88.31 (1.841)
e	120.11 (2.509)	85.83 (1.793)	210.38 (4.386)	86.00 (1.793)
d	133.41 (2.787)	85.83 (1.793)	217.33 (4.531)	86.00 (1.793)
c	150.27 (3.139)	83.50 (1.744)	230.19 (4.799)	83.65 (1.744)
b	151.84 (3.172)	83.50 (1.744)	230.19 (4.799)	83.65 (1.744)
a	239.05 (4.994)	--	238.77 (4.978)	--

Note: Mass values are in SI units of kN-sec^2/m^2 with non-SI units of kip-sec^2/ft^2 in parentheses.

Appendix E
Rotational Stability of Intake Towers

E-1. Rocking Potential

For a rigid tower, or a flexible tower idealized by a single-degree-of-freedom (SDOF) system, rocking will occur when the disturbing moment exceeds the restoring moment due to the weight of the tower. This criterion is based on satisfying the moment equilibrium of the rigid block model shown in Figure E-1:

$$MS_A\,(H/2) > Mg\,(B/2) \qquad \text{(E-1)}$$

where

M = the total mass of the tower including the added hydrodynamic mass of the surrounding and internal water

S_A = spectral acceleration

H = height of the block

g = acceleration due to gravity

B = base of the block

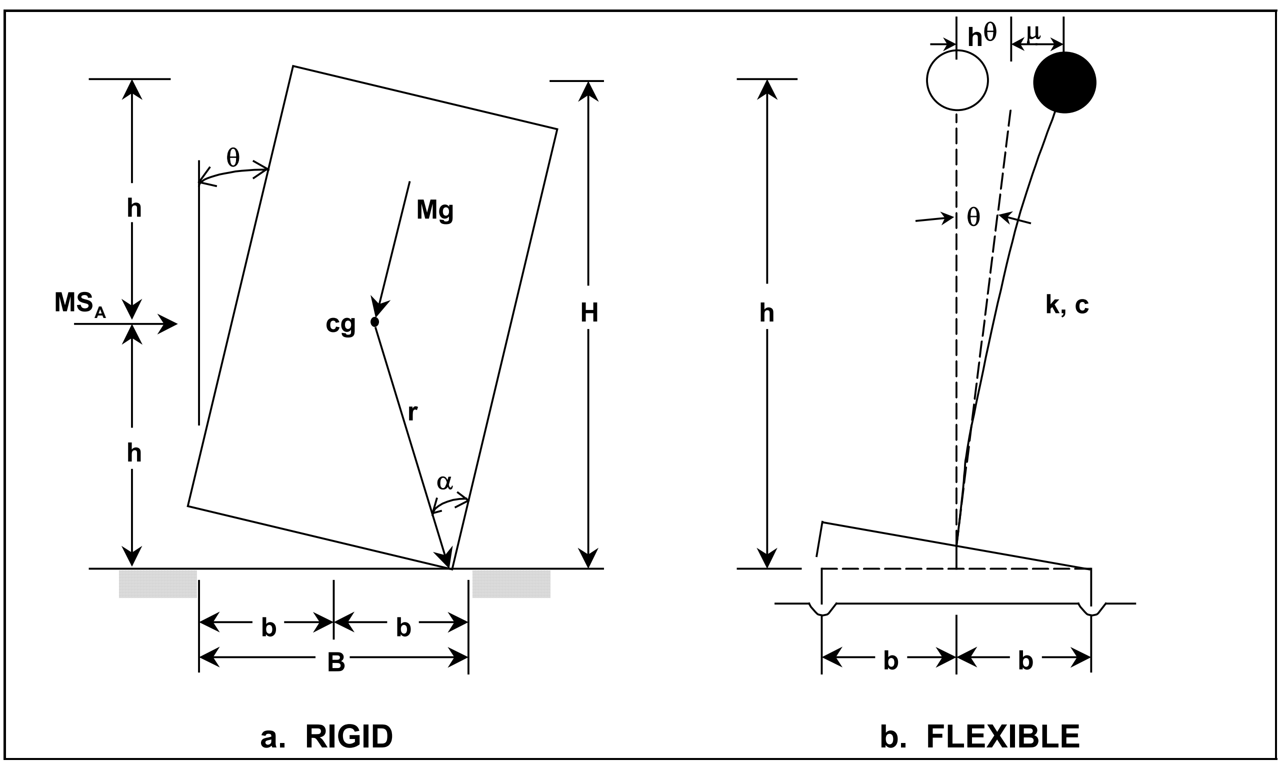


Figure E-1. Rigid block and SDOF models for dynamic rotational stability (where *c* = damping coefficient for the structure; θ = rigid body rotational degree of freedom; μ = flexural deflection of the top of the structure; and *k* = stiffness of the structure

Rocking occurs when the spectral acceleration S_A is greater than the product of the acceleration of gravity g and the ratio of the base width to the tower height:

$$S_A > g\ (b/h) \qquad \text{(E-2)}$$

where

$b = B/2$ (one-half the base of the block)

$h = H/2$ (one-half the height of the block)

E-2. Rocking Potential Example

The tower in Appendix C (Figure C-1) will be investigated for rotational stability using the response spectrum shown in Figure C-3. From Table C-11, the frequency of the first mode including hydrodynamic effects is 2.22 Hz or a period of 0.45 sec. From the response spectra (for 5 percent damping) shown in Figure C-3, the first mode spectral acceleration S_A is 0.62 g. The tower has a *b/h* ratio of 0.24; therefore, rocking can occur during the maximum design earthquake (MDE) represented by the response spectrum.

E-3. Housner's Rigid Block Model

a. Although rocking occurs, the tower will not necessarily be rotationally unstable during the operational basis earthquake (OBE) or MDE. Using spectral acceleration as the only indicator of earthquake demand is not a sufficient basis for evaluating overturning or tipping during earthquakes. Therefore two other dynamic motion parameters, i.e., spectral velocity and spectral displacement, must be included for evaluating overturning, and an energy-based characterization of the ground motion should be used. Housner (1963) developed a rigid block model for structures with a uniform mass distribution on a rigid foundation that relates overturning or tipping to the spectral velocity S_V. This criterion is based on satisfying conservation of kinetic and potential energies for the slender rigid blocks shown in Figure E-2. If a slender block is given an initial velocity that is just sufficient to rock the block to the point of impending static instability, then equating the initial kinetic energy of the block to the increased potential energy at the point of overturning results in the following equation:

$$\frac{WR\alpha 2}{2} = \frac{1}{2}\frac{W}{g}\frac{MR^2}{I}S_V^2 \qquad \text{(E-3)}$$

where

W = weight of the structure

R = distance from the center of gravity to the corner about which rotation occurs

I = mass moment of inertia about corner

The spectral velocity S_V approximates the maximum initial velocity of the block, and the critical angle of rotation α is a small angle (less than 20 degrees). For small angles $\alpha \approx b/h$ and $r \approx h$. For slender structures MR^2/I is almost equal to unity. For tall slender blocks Equation E-3 simplifies to

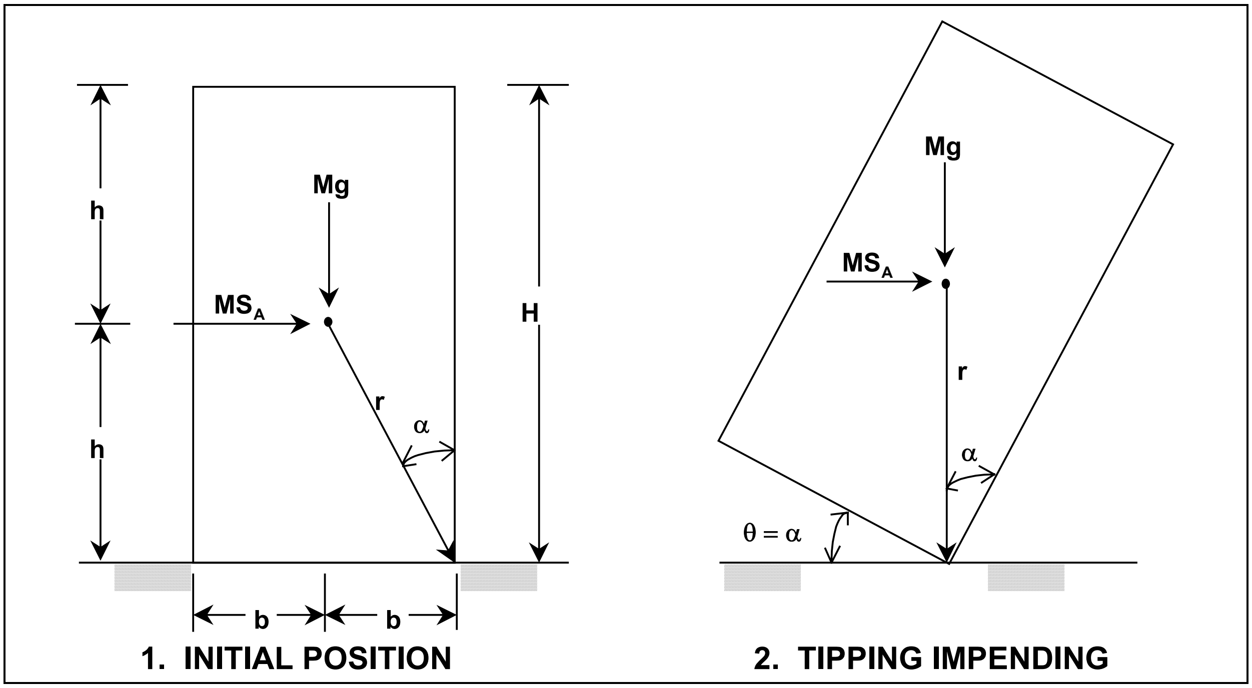


Figure E-2. Housner's model for slender rigid blocks

$$\alpha_{cr} = \frac{S_V}{\sqrt{gr}} \tag{E-4}$$

where

α_{cr} = critical value of the angle of rotation from the initial upright position of static equilibrium to the final tipping position of impending rotational instability. The instantaneous center of rotation is at the toe of the slender rigid block (radians).

S_V = the response spectral-pseudo velocity for the OBE or MDE ground motion (cm per sec)

g = acceleration of gravity (981.4 cm per sec squared)

r = the radial distance from the toe to the center of mass of the tower (cm)

b. According to Housner, this equation may be interpreted as stating that for a given spectral value S_V, a block that rocks through an angle α will have approximately a 50 percent probability of being overturned. However, a 50 percent probability of failure or survival is not always an acceptable criterion for stability because there is an unexpected scale effect that makes the larger of two geometrically similar blocks more stable than the smaller block.

c. The scaling effect can be estimated for a damped structure by using the relationships for the spectral-pseudo velocity S_V, spectral-pseudo acceleration S_a, and the spectral-pseudo displacement S_d. These relationships are appropriate because accelerograms recorded during earthquakes show the maximum relative displacements occur when the relative velocities are zero; the maximum values of the relative displacements are proportional to the maximum absolute accelerations. The shapes of the maximum peaks are approximately sinusoidal with a natural frequency of ω (rad/sec) = $T/2\pi$ where T is the period of the structure.

So

$$S_V = \omega\, S_d = S_a / \omega \qquad \text{(E-5)}$$

and

$$S_V^{\,2} = S_a\, S_d \qquad \text{(E-6)}$$

d. This relationship clearly shows that the earthquake demand acceleration and displacement are critical parameters for evaluating overturning, and that considering either S_a or S_d alone as the earthquake demand is not sufficient. Substituting Equation E-6 into Equation E-4 yields:

$$\alpha = \sqrt{\frac{S_a}{g}\frac{S_d}{r}} = \frac{b}{h} \qquad \text{(E-7)}$$

If S_a is sufficient to cause overturning of the rocking block, then combining Equation E-7 and the inequality E-2 yields:

$$\left[\frac{gr}{S_d}\right]\left(\frac{b}{h}\right)^2 \rangle\, g\frac{b}{h} \qquad \text{(E-8)}$$

For slender rigid blocks, r is approximately equal to h, so

$$S_d < b \qquad \text{(E-9)}$$

The scaling effect shows that if S_a is just sufficient to cause static overturning, the block will start to rotate; but, if the block is to overturn, its displacement as approximated by S_d must be equal to half the base width of the block. In general, S_d is never large enough to cause a typical tower to overturn. Conversely, these equations must be extended to evaluate the seismic stability of other structures subjected to significant net lateral loads.

E-4. Seismic Rotational Stability Example

a. For the tower in Appendix C, the geometry in the transverse direction is characterized by the following:

(1) Distance from the bottom of pedestal to c.g. h, of 23.25 m (75.729 ft) and a half base width b of 7.36 m (24 ft)

(2) An angle α = 0.3068 radian (17.58 deg)

(3) A radial distance r of 24.2 m (78.83 ft)

b. The earthquake demand for the site can be estimated using the response spectrum shown in Figure C-3.

(1) The spectral pseudo-acceleration is 0.62g for a period of vibration of 0.45 sec.

(2) The spectral pseudo-velocity and displacement are computed to be 0.436 m per sec (1.42 ft per sec) and 0.031 m (0.1 ft), respectively.

$S_V = S_a T / 2\,\pi = 0.62 \times 981.4 \times 0.45 / 2 \times 3.14 = 0.436$ m per sec (1.42 ft per sec)

and

$$S_d = S_V\, T / 2\,\pi = 0.436 \times 0.45 / 2 \times 3.14$$
$$= 0.031 \text{ m (0.1 ft)}$$
$$<< b = 7.36 \text{ m (24 ft)}$$

c. The critical angle of rotation is computed from Equation E-4 to be 0.015 rad (0.859 deg).

$$\alpha_{cr} = \frac{S_v}{\sqrt{gr}}$$

$$\alpha_{cr} = \frac{0.435}{\sqrt{981.4} \times 24.2} = 0.015 \text{ rad (0.859 deg)}$$

The actual angle α is less than the critical value from Equation E-4, and the spectral displacement is less than half of the base width. Therefore, rotational instability due to seismic rocking as represented by the response spectrum is highly unlikely.