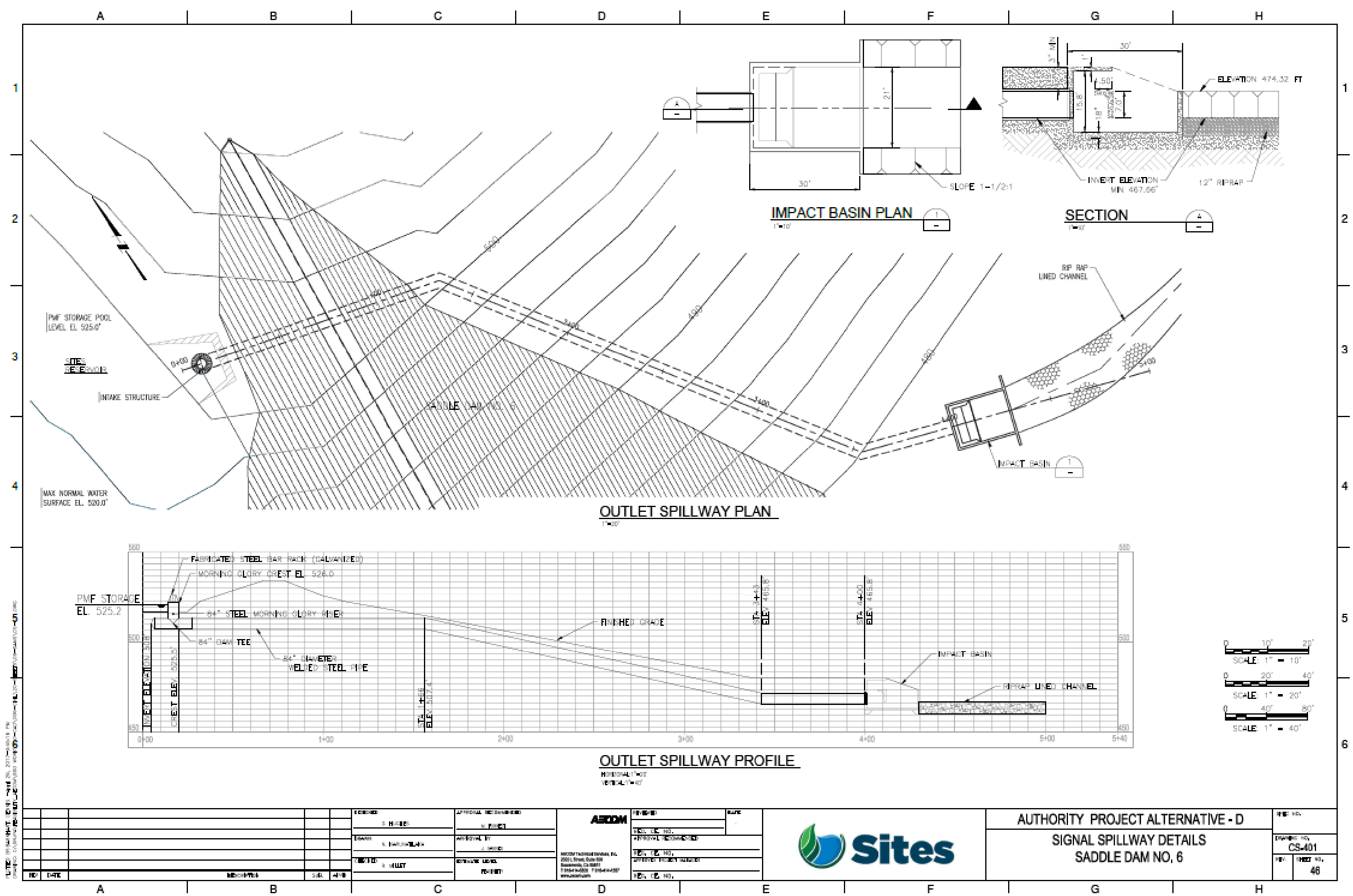


Figure B.3-15. Signal Spillway – Saddle Dam 6 – 1.8 MAF Reservoir – Plan and Elevation

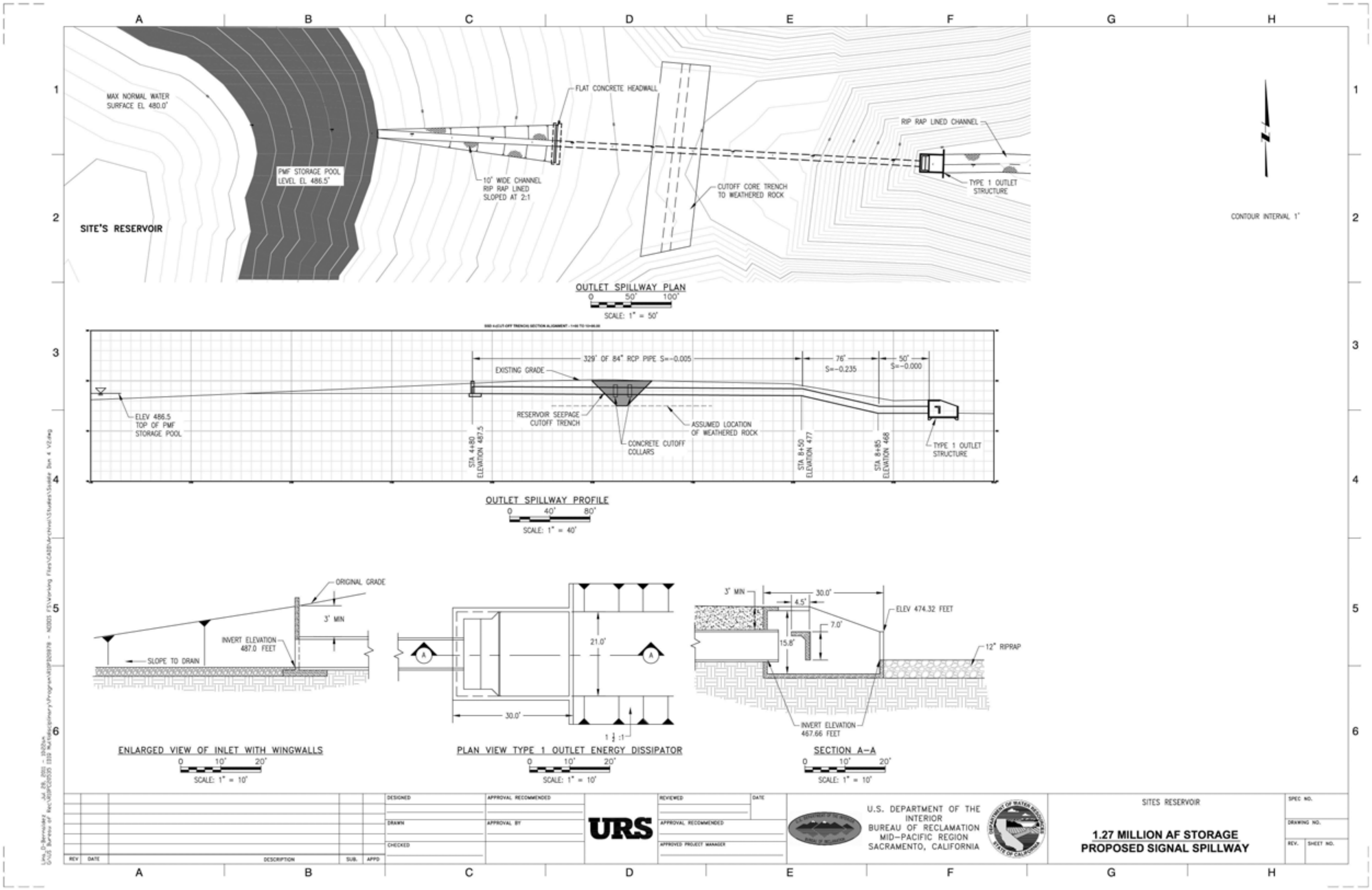


Figure B.3-16. Signal Spillway – Saddle Dam 6 – 1.3 MAF Reservoir – Plan, Elevation, Sections

Even though no outflows of significance are expected, the energy dissipating structure would be sized for a flow of approximately 600 cfs, which is the maximum expected outflow if the reservoir water level should approach the crest of the dam.

Sites Reservoir Clearing

General

Clearing in the reservoir for either reservoir size currently under consideration would require the removal of selected larger vegetation and demolition of a small number of structures used for residential dwellings and ranching operations.

Vegetation Removal

Approximately 92 percent of the reservoir inundation area footprint is composed of annual grasslands. As a result, clearing and grubbing would not be needed in this area. The remaining 8 percent consists of blue oak woodland, agricultural crops, and other vegetation, which would be cleared. To provide for unobstructed flow through the reservoir inlet/outlet works, selected larger vegetation would need to be removed from the inundation area prior to first filling. Reservoir clearing would consist predominantly of the removal of blue oak woodland, for a total cleared area of roughly 1,000 acres.

Structure Demolition

The reservoir inundation area includes the small community of Sites, which has about 19 residential dwellings, as well as approximately 15 scattered ranch compounds. These structures would have to be demolished and removed from the inundation area prior to first filling. Existing fencing would need to be removed.

Relocations

It would be necessary to relocate two existing cemeteries.

East Ridge Stability

General

Much of the reservoir's eastern rim is impounded by relatively high, steep ridges trending north-south. The stability of this natural ridge was evaluated under predicted reservoir loading. The focus of the preliminary feasibility-level ridge analysis was to determine the most critical ridge section, and to perform a static stability analysis to confirm that the proposed size of Sites Reservoir is technically feasible.

The critical section was found by examining USGS quadrangles of the proposed reservoir complex. The steepest and thinnest ridge sections were found to be immediately south of the Golden Gate Dam site, the most critical of which was modeled. This section has slopes approximating 30 degrees, with a crest at an elevation of approximately 555 feet, 35 feet above the reservoir's maximum WSE.

East Ridge Geology

Generally, the ridge on the eastern side of the reservoir is composed of upper Cretaceous marine sedimentary rocks of the Cortina and Boxer Formations. The rock characterized by these formations is interbedded sandstone and mudstone that strike generally north and dip approximately 50 degrees to the east. Exploratory drilling found generally good quality rock at depth, with light to moderate weathering and fracturing.

East Ridge Rock Strength Estimates

Conservative strength values used in the ridge stability analysis for sandstone and mudstone were developed from unconfined compressive strength testing on samples recovered from the initial phases of the preliminary geologic exploration program, in conjunction with published data. The unconfined compressive strength results were classified by rock type, condition (dry or saturated), and level of weathering. In the case of the mudstone, some friable materials were recovered in the exploratory drilling, so the assumption used in the model was that the critical failure surfaces would tend to occur along the mudstone bedding planes. In addition, some of the mudstone samples slaked (mostly parallel to the bedding plane), and could not be tested. Therefore, zero cohesion and a low-friction angle were used to model the mudstone bedding planes. Table B.3-6 provides a summary of the strengths used in the stability model.

Table B.3-6. Rock Strengths Used in East Ridge Stability Analysis

Rock Type	Cohesion	Friction Angle (degrees)
Sandstone	850 psi (122,000 psf)	35
Mudstone	132 psi (19,000 psf)	15
Mudstone (Bedding Planes)	0 psi (0 psf)	10

Key:
psf = pounds per square foot
psi = pounds per square inch

Assumed phreatic surfaces used in stability analyses include the following:

- **Full Reservoir –** Water surface at an elevation of 520 feet; phreatic surface linear from upstream water surface to downstream toe of slope.
- **Partial Pool –** Water surface at an elevation of 395 feet; phreatic surface (pore water pressure at atmospheric conditions) linear from the upstream water surface to the downstream toe of slope.
- **Rapid Drawdown of Reservoir –** Empty reservoir with phreatic surface coincident with the upstream toe-of-slope to the upstream high-water mark (elevation 520 feet), then linear to downstream toe-of-slope.

The downstream slope was evaluated on the presumption that downstream failures would initiate along the zero-cohesion, weak mudstone bedding planes. The upstream slope was modeled for two cases: failure surfaces encompassing the weak mudstone bedding planes; and failure surfaces that crossed the weak mudstone bedding planes. Figure B.3-17 illustrates a typical sliding surface evaluated as part of the stability analysis.

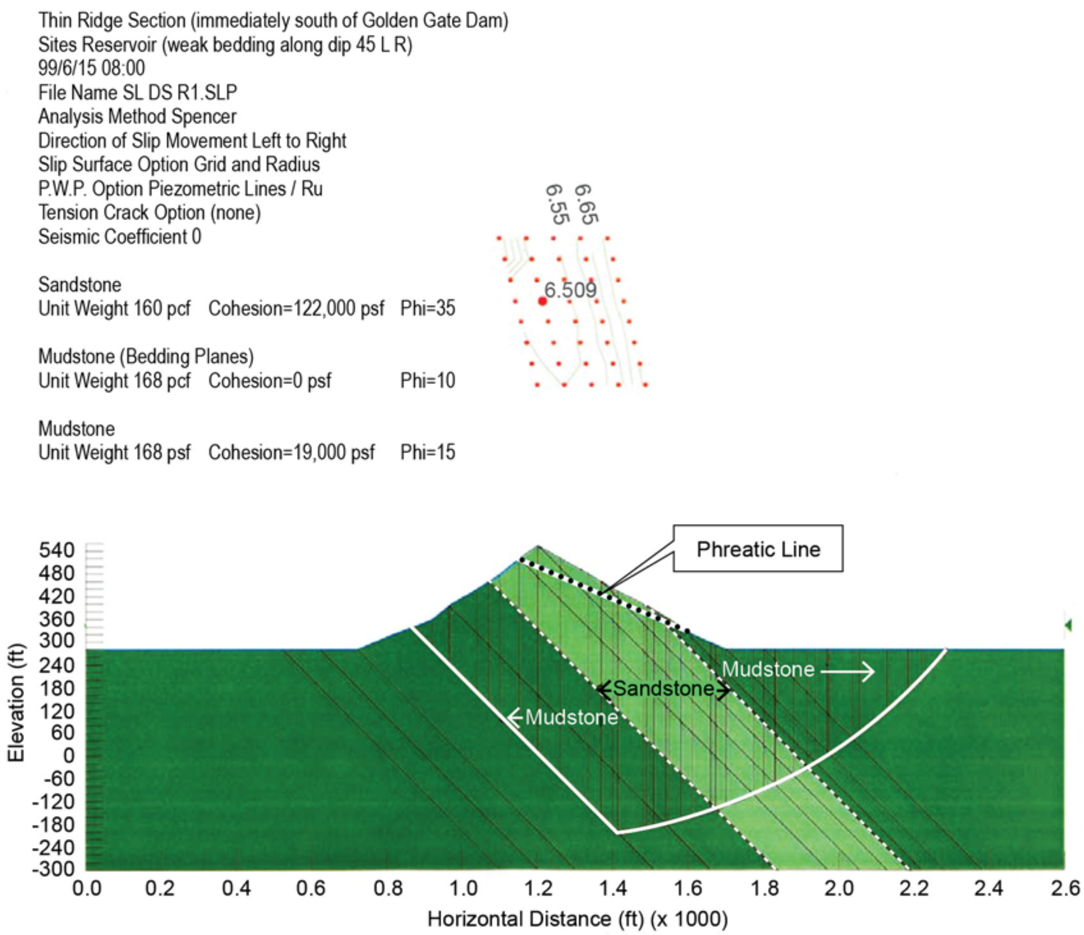


Figure B.3-17. Eastern Ridge Section (immediately south of Golden Gate Dam)

Stability Model Results

The minimum factor-of-safety of the critical sliding surface was found to be greater than six for the cases examined, indicating adequate stability of the eastern ridge under reservoir loading. Future design investigations should include an additional evaluation of the critical areas of the eastern ridge, with particular emphasis placed on incorporating refined geologic information (as it becomes available) into the stability models, in conjunction with a detailed evaluation of the effects of reservoir seepage through the ridge.

Reservoir Eastern Rim Grouting

The eastern reservoir rim narrows in locations from Golden Gate Dam north through the location of the northernmost saddle dam. Potential through-seepage could occur in these narrow ridge areas at maximum reservoir water level, depending on the depth to fresh rock. Additional geotechnical investigations would be needed during future phases of the NODOS/Sites Reservoir Project to further evaluate seepage conditions. A practical method to address rim seepage would be to: 1) extend the embankment foundation grouting for Golden Gate Dam and the saddle dams along the ridge through narrow areas of concern; and 2) add ridge grouting in areas of concern between dams. Based on a preliminary evaluation, it is assumed that approximately 6,000 lineal feet of grout curtain could be needed along the rim. The cost for primary and secondary grouting over this length has been included in cost estimates for Alternatives B, C, and D. For Alternative A, reservoir rim grouting is assumed to be approximately 3,000 lineal feet.

Sites Reservoir Inlet/Outlet

Options Evaluated, Selected Option

In the detailed technical report covering preliminary design of the Sites Reservoir appurtenant facilities, two options were evaluated for the inlet/outlet works: a low-level inlet/outlet with gate shaft, and a multi-level inlet/outlet. The two options provide different benefits, such as variability of the release water's temperature, and dissolved oxygen content of the outflow. A general description of the two options is summarized below:

- **Option 1 –** Low-level inlet/outlet structure with a gate shaft for an emergency fixed-wheel gate.
- **Option 2 –** Low-level inlet/outlet structure for emergency drawdown, plus a multi-level valved inlet tower and shaft with an emergency fixed-wheel gate.

Estimates developed for Options 1 and 2 indicate comparable costs for each.

Releases made from the low-level outlet included in Option 1 would come from the bottom of the reservoir pool, resulting in cold-water outflows. The multi-level tower included in Option 2 would provide flexibility in temperature control and dissolved oxygen content of the reservoir releases, which may provide benefits to downstream agricultural and environmental water users. Because the costs of Options 1 and 2 are similar, and Option 2 provides increased flexibility, Option 2 is the preferred alternative carried forward for further evaluation and costing for the two reservoir sizes and four associated NODOS/Sites Reservoir Project alternatives being considered. Option 2 details are described below. Details would be similar for both alternative reservoir sizes.

General

Figure B.3-18 presents a plan view of the inlet/outlet. Figure B.3-19 and Figure B.3-20 show profiles for the inlet/outlet tower and tunnel for the 1.3 MAF and 1.8 MAF reservoirs, respectively. The multi-level inlet/outlet tower was modeled after DWR's Castaic Dam Outlet Works, and has multiple inlet ports with the capability of drawing water at different levels into the reservoir. This tower would provide the flexibility to control the dissolved oxygen content and temperature of reservoir releases. The tower details would be similar for both alternative reservoir sizes currently being considered, but the tower elevations and number of inlet ports would be different. An on-slope intake should be evaluated against the vertical inlet/outlet tower as part of the final design. The on-slope intake could be a more cost-effective and reliable design, depending on the results of additional site investigations and site seismicity evaluations planned in future phases of the NODOS/Sites Reservoir Project.

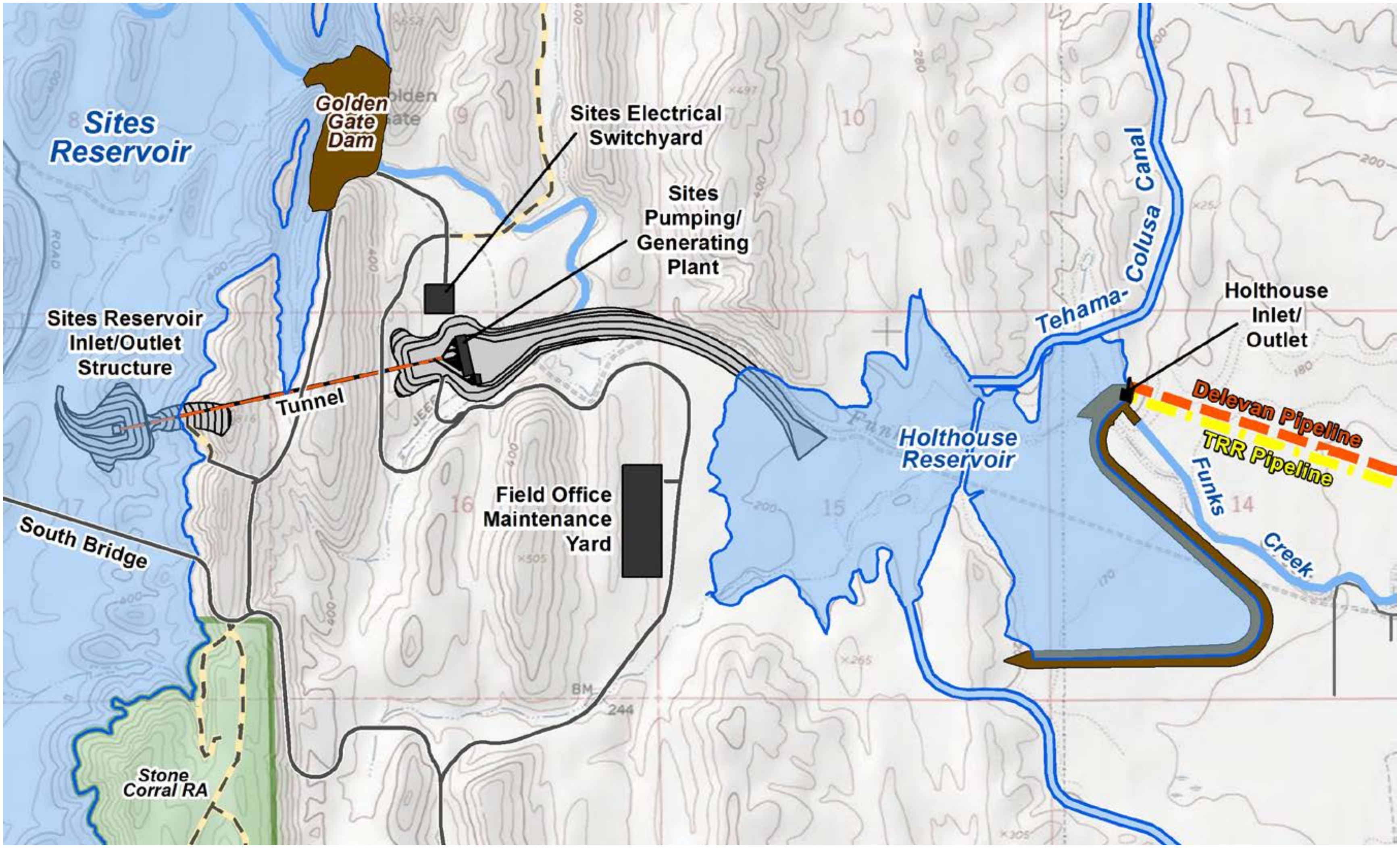


Figure B.3-18. Inlet/Outlet – General Plan

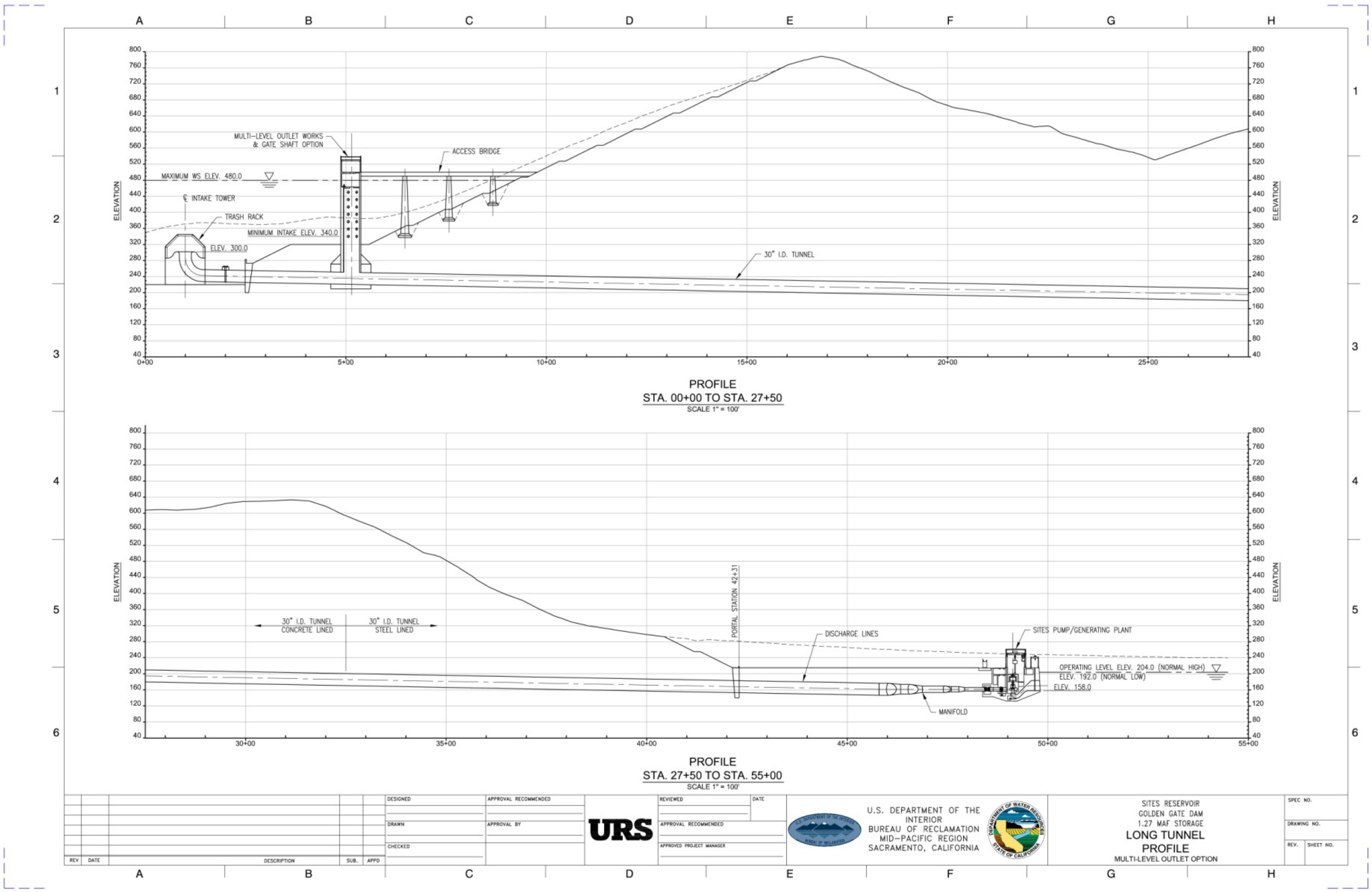


Figure B.3-19. Inlet/Outlet – Profile – 1.3 MAF Reservoir

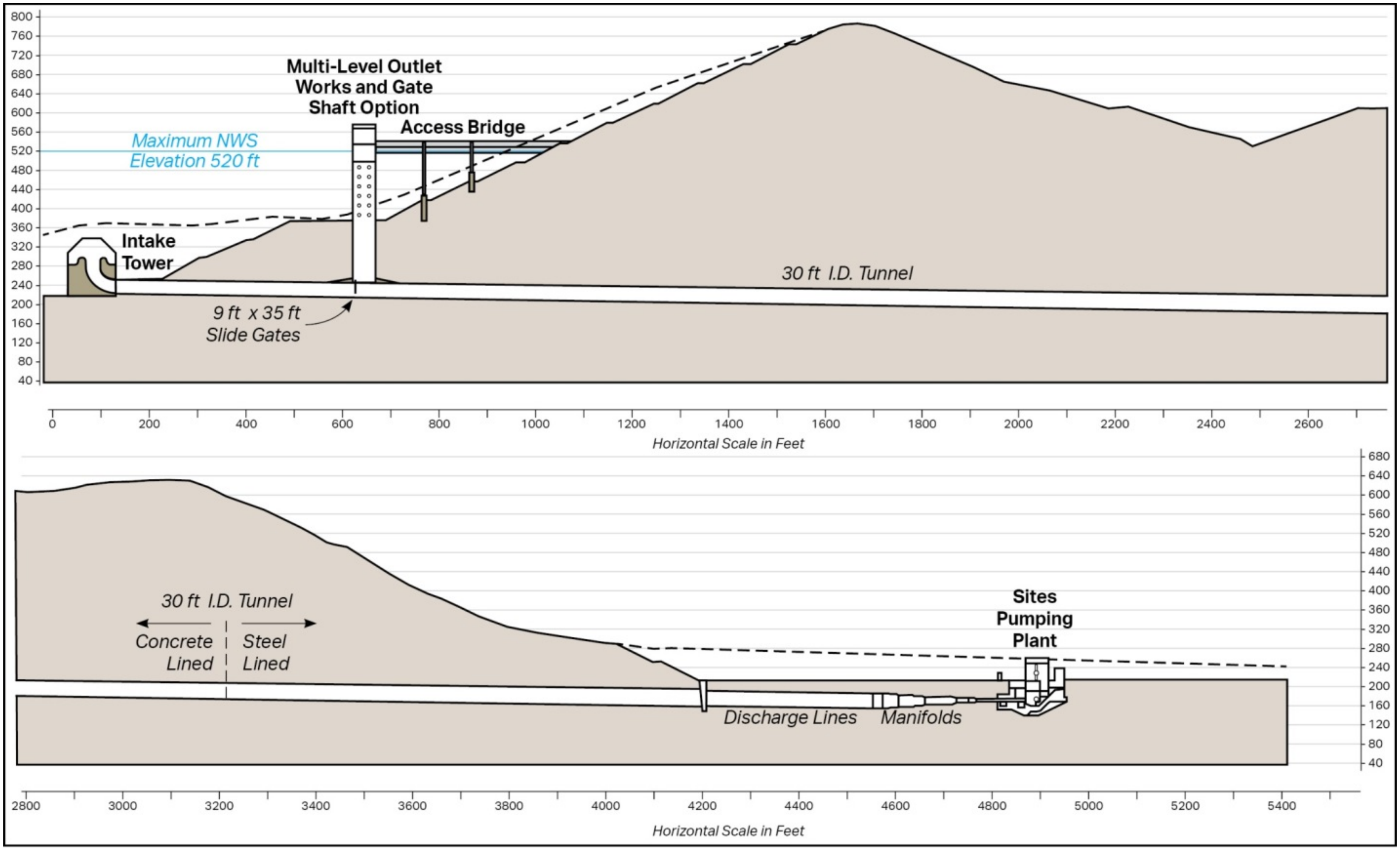


Figure B.3-20. Inlet/Outlet – Profile – 1.8 MAF Reservoir

The reservoir inlet/outlet would consist of a multi-level, valved inlet/outlet tower and gate shaft; a 30-foot-inside-diameter pressure tunnel approximately 4,000 feet long; a system of buried steel penstock piping connecting the downstream tunnel portal with the pumping/generating units in the SPGP; and an energy-dissipation valve structure to make emergency reservoir releases, or normal releases if the generating units are off-line. Under normal operating conditions, the reservoir outflows would pass through the SPGP units to generate electricity. In the event of an emergency requiring a rapid drawdown of the reservoir pool, the reservoir outlet works would be operated by bypassing the plant and directing the outflow to the energy-dissipation valve structure. The proposed size of the multi-level inlet/outlet would accommodate operational reservoir inflows and outflows up to the maximum required for each alternative. The low-level intake on the bottom of the reservoir at the upstream tunnel portal and the pressure tunnel are sized to accommodate the flow capacity required to make emergency reservoir drawdown releases; discussed in further detail in subsequent sections. The low-level intake with crest at elevation 300.0 feet could also be used to release stored water between the crest and the lowest ports in the vertical intake tower, if needed, during drought conditions.

Design Assumptions

Preliminary design of the reservoir inlet/outlet was performed in accordance with the state-of-practice for dam appurtenant structure design and conforming to dam safety criteria.

Inlet/Outlet Tower

Table B.3-7 provides a comparison of tower details for the two reservoir sizes under consideration.

Table B.3-7. Sites Reservoir Inlet/Outlet Tower Consideration

	1.8 MAF Reservoir	**1.3 MAF Reservoir**
Top Elevation	580.0 feet	540.0 feet
Bottom Elevation (Top of Bench)	320.0 feet	320.0 feet
Inside Diameter	32 feet	32 feet
Outside Diameter	39 feet	39 feet
Number of Ports	36 (4 each at 9 levels)	28 (4 each at 7 levels)
Functional Reservoir Release Elevations	520 feet to 340 feet	480 feet to 340 feet

Key:
MAF = million acre-feet

Figure B.3-21 presents section views of the proposed tower. The multi-level inlet/outlet tower contains trashracks with outlet ports controlled by butterfly valves. The valves are bolted onto thimbles embedded in the tower in tiers, with four valves spaced around each tier. The tower would contain movable fish screens in two tiers for operational purposes. Valves on any tier can be operated independently, or all valves can be operated together. The tiers are spaced approximately 20 feet apart down the tower, beginning approximately 30 feet below the maximum reservoir water level.

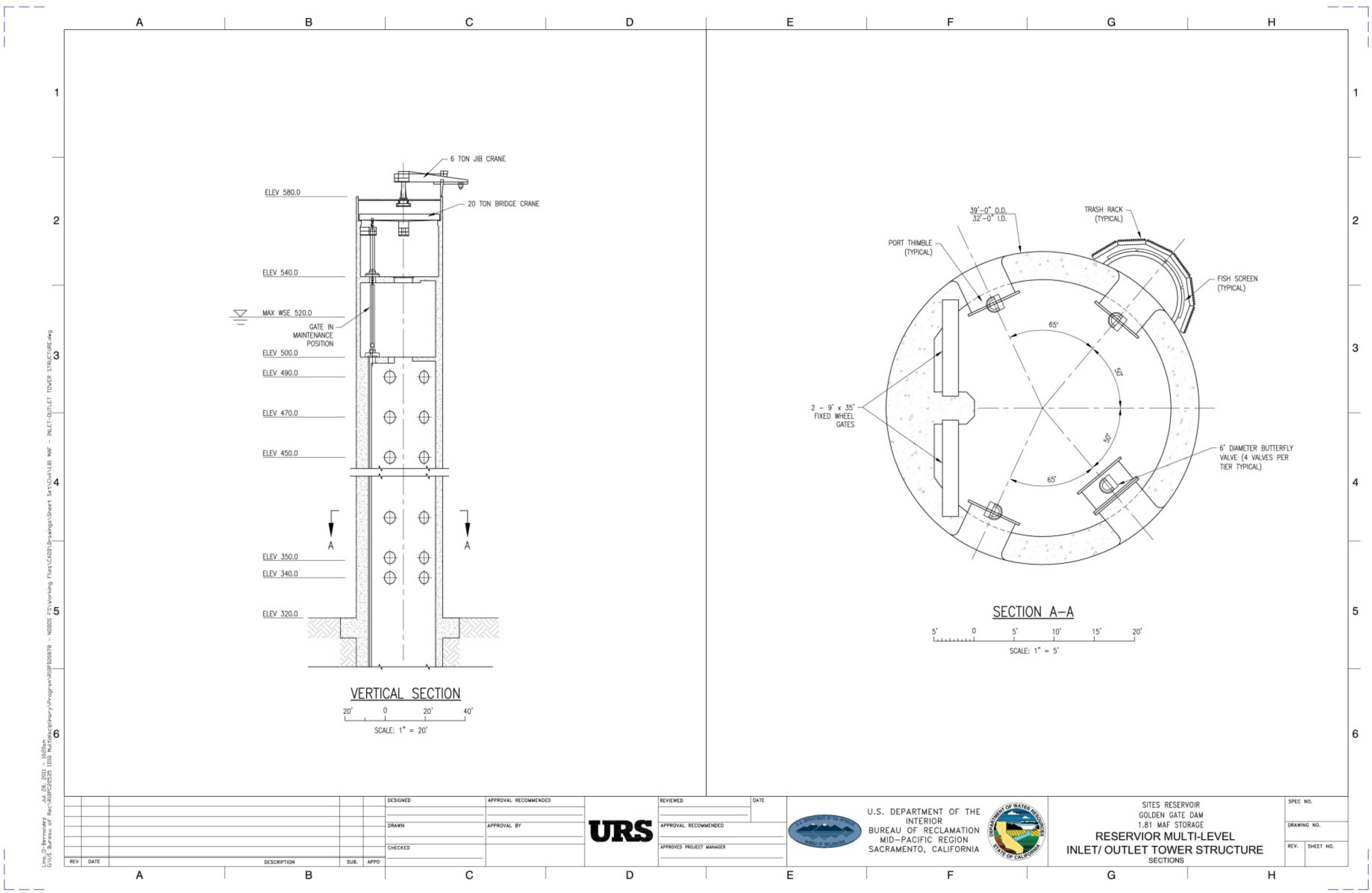


Figure B.3-21. Inlet/Outlet Tower Structure – Typical Sections

In addition to the valved outlets, the inlet/outlet tower/shaft would also contain wheel gates that can be extended down into the outlet pressure tunnel. It is possible to provide one large gate, or two smaller gates for improved operational reliability. This choice would be further evaluated during final design of the NODOS/Sites Reservoir Project. For evaluating alternatives, the use of two 9-foot by 35-foot fixed-wheel gates has been assumed. During normal operation, the gates would be down so that flows would enter and leave the reservoir through the ports in the vertical tower. With the gates down, the tunnel, vertical tower shaft, and penstock piping to SPGP can also be dewatered (at SPGP) for inspection and maintenance. If emergency releases were required, the gates would be raised to draw water through the low-level inlet on the bottom of the reservoir. Inspection of the tunnel upstream of the gate shaft would require placing bulkhead gates into slots in the low-level inlet on the bottom of the reservoir using a barge-mounted crane on the lake.

A bridge provides access to the multi-level tower from the nearby access road. The bridge length varies depending on reservoir size, and would have a superstructure consisting of simple spans of welded-plate girders acting compositely with a lightweight concrete deck. The girders are supported by the multi-level inlet/outlet tower; reinforced-concrete piers, which are excavated into the rock foundation; and a reinforced-concrete abutment. Roadway width would be 16 feet between barrier railings.

The foundations of the low-level, inlet/outlet gate shaft, and multi-level inlet/outlet tower structures are in the Boxer Formation.

Pressure Tunnel

Tunnel details, and details of the inlet control structure to the tunnel on the bottom of the reservoir, would be the same for the 1.8 MAF and 1.3 MAF reservoirs. The tunnel is modeled after DWR's Angeles Tunnel, and would be located through the reservoir eastern ridge at a location south of Golden Gate Dam. The finished inside diameter would be 30 feet over most of the tunnel length, except for a short, enlarged section at the intersection with the vertical inlet/outlet tower.

Two tunnel alignments (short and long) were investigated in the preliminary feasibility design of the Sites Reservoir appurtenant facilities. However, due to faulting issues along the short tunnel alignment, the long alignment was chosen for evaluating the NODOS/Sites Reservoir Project alternatives. The long tunnel is approximately 4,000 feet in length. The geology of the tunnel alignment consists of the Boxer Formation for the western 1,500 feet of the tunnel; and the Cortina Formation for the eastern 2,500 feet. Drilling and blasting is the assumed construction method for the tunnel.

The proposed inside diameter of 30 feet is selected to meet current DSOD emergency drawdown release requirements. This general guideline is that large reservoir outlet facilities should have a flow capacity capability that could lower the maximum reservoir storage depth by 10 percent within 10 days. In the case of Sites Reservoir, this correlates to approximately 22 feet of drawdown (300,000 AF), assuming a full reservoir at elevation 520.0. The maximum discharge release would be approximately 15,300 cfs, with a corresponding tunnel velocity of approximately 21.6 fps. Emergency releases would be made through a pipe bypass to four energy dissipating valves in a dissipating chamber adjacent to SPGP. In contrast with the emergency release, normal maximum pumping velocities through the tunnel for the SPGP would be approximately 8.4 fps for the 5,900 cfs design flow for Alternatives A, C, and D; and approximately 5.5 fps for the 3,900 cfs design flow for Alternative B. As part of final design, it may be possible to reduce the size the tunnel, based on risk-based evaluation methods. However, the reduction in size may not be important, and would not alter the evaluation of alternative projects. The lower normal maximum reservoir level for Alternative A could also slightly reduce the tunnel size for this alternative; however, this possible reduction also was not considered for this alternative.

The pressure tunnel would be concrete-lined to prevent rock fallout and erosion, and minimize seepage into the surrounding rock. Concrete lining also provides a smooth interior surface that reduces head loss. As long as the confining weight of rock cover over the tunnel exceeds the internal pressure (plus a safety factor), only concrete tunnel lining is provided. Where the confining weight of rock cover is less than the internal pressure, a steel liner is incorporated into the concrete lining. Figure B.3-19 and Figure B.3-20 present a profile view of the proposed tunnel for the two reservoir sizes under consideration, including where steel lining would be provided.

Gate Shaft

The gate shaft component of the inlet/outlet tower would be a shaft excavated in rock extending down from the base of the inlet/outlet tower at grade elevation, 320.0 feet to the invert of the outlet tunnel (Figure B.3-22). The gate shaft is modeled after DWR’s Angeles Tunnel gate shaft. The gate shaft would have the same inside diameter as the inlet/outlet tower or slightly larger, depending on the final selection of wheel gates for tunnel closure.

Penstock and Manifold

Flow in and out of Sites Reservoir from the SPGP would occur through buried steel penstocks and manifolds beginning at the downstream tunnel portal. The initial penstock diameter at the tunnel portal is 30 feet, and remains that size until the first bifurcation for the energy-dissipation valve structure. Subsequent bifurcations and size reductions create a manifold of piping that connects to the various piping and pumping/generating units in the SPGP. Figure B.3-22 presents a plan view of the penstock and manifolds for a 5,900 cfs SPGP. The manifold lines were not sized by an economic analysis, but rather for pumping flow velocities of 10 fps or less.

All buried penstocks and manifolds would be concrete-encased, and concrete anchor blocks would be used to resist the thrust forces on bends, reduction bifurcations, branches, etc. A maximum 45-degree angle between the main penstock and the bifurcations should be used for optimum efficiency. The penstocks would be designed for an internal pressure equivalent to the normal maximum static pumping head, and for internal surge pressures under normal operating and critical emergency shutdown conditions for pumping and generating configurations. A surge suppression system is not included in the current design, but could be further evaluated as part of final penstock design. Penstocks would also be evaluated for stability and stresses due to backfill loads over the pipes, with the pipes in a dewatered condition.

Steel—rather than concrete—penstock pipes were used because of size, handling weights, and internal pressures. A large amount of steel tunnel liner is already required; and economically, the short lengths of differing diameter manifold piping favor steel.

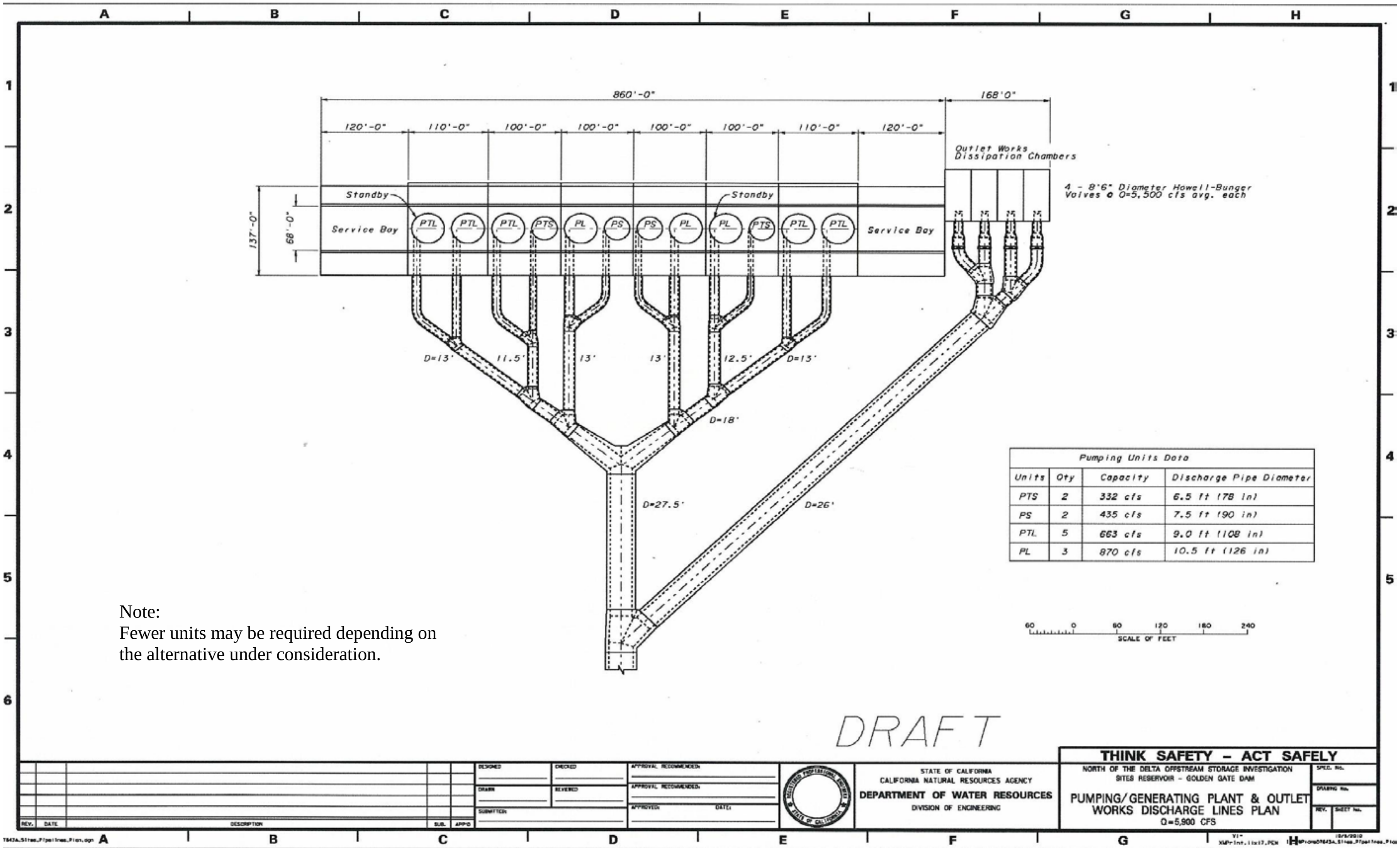


Figure B.3-22. Sites Pumping/Generating Plant – Discharge Lines Plan – Q=5,900 cfs

Emergency Release

The option considered to pass emergency releases from the reservoir would be a 26-foot-diameter bypass pipeline branching off of the 30-foot-diameter buried penstock near the downstream pressure tunnel portal, and leading to four energy dissipation valves. The 26-foot-diameter pipeline would bifurcate and reduce in size several times to join to four 8-foot, 6-inch fixed-cone dispersion (Howell Bunger) valves in reinforced-concrete energy dissipation chambers. Figure B.3-22 and Figure B.3-23 present plan and section views of the bypass piping and dissipation chambers.

It should be noted that the flow through the fixed-cone dispersion valves would be released downstream to Holthouse Reservoir, and would need to be routed to prevent downstream flooding. Further evaluation would be needed in detailed design regarding passing the emergency release in the natural channels downstream of Holthouse Reservoir. The new Holthouse Reservoir has an reinforced concrete spillway section sized to pass the emergency drawdown flow. In addition, it is recommended that the emergency drawdown requirements be revisited during future design investigations, because the DSOD generally examines evacuation requirements on a site-specific basis for large reservoirs.

Sites Pumping/Generating Plant

Introduction

The proposed SPGP would be approximately 3,300 feet southeast (downstream) of Golden Gate Dam (Figure B.3-18). The location and layout, including the plant/control building and conveyances, were determined on the basis of hydraulic and plant equipment requirements, foundation conditions, and the orientation of local faults. The final plant location should be determined by establishing a point of economic balance between the cost of the required excavation tunnel length and discharge lines, versus the cost of long-term pumping.

The SPGP for Alternatives A, B, C, and D was modeled after DWR’s Chrisman Pumping Plant, with modifications to reflect the pumping height difference, additional pumping units, and miscellaneous equipment needed for each alternative. The Chrisman Pumping Plant has flow capacities and head requirements similar to the SPGP design criteria. Equipment would include pumps and pump-turbines with dual-speed capability to accommodate head and flow variability. Preliminary criteria were developed to support the conceptual design of the equipment; however, these criteria are expected to be refined through future grid interconnection studies, the development of a hydropower strategy, and preliminary engineering. Using pump-turbines is feasible for the SPGP. The head difference between the pumping and generating functions is small, due to the large size of the inlet/outlet tunnel connecting the plant with Sites Reservoir.

For Alternative D, an optional equipment configuration was also evaluated. Pumping and generating functions would be provided using all pump-turbine units equipped with variable-frequency motor-generator drive technology to accommodate variable head and flow requirements. Although this equipment concept—and the more conventional concept for Alternatives A, B, and C—have similar capacities, the operational flexibility and cost vary considerably. For the Draft Feasibility Report, Alternative D is assumed to utilize the same pumping and pumping/generating equipment as Alternative C.

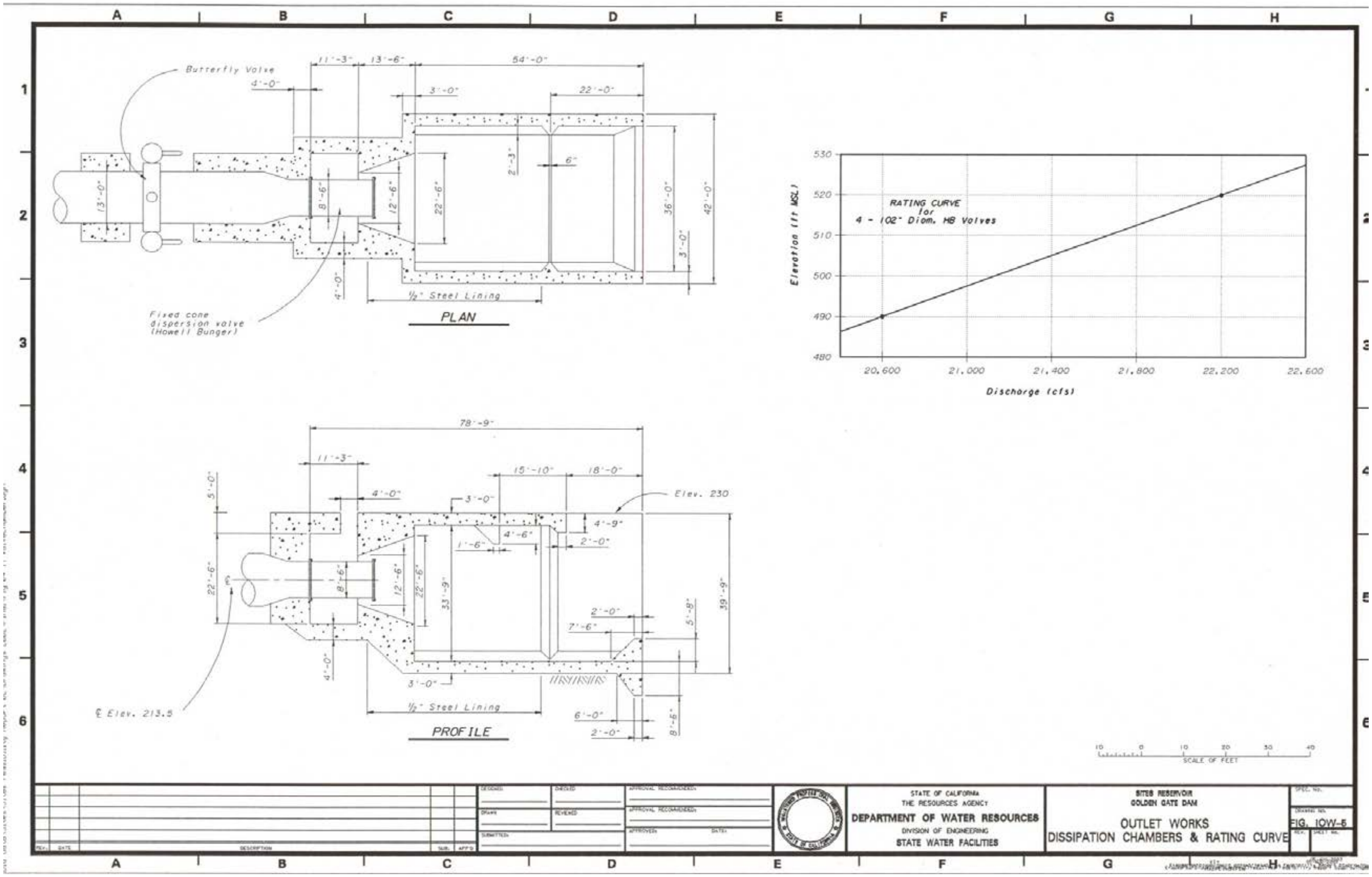


Figure B.3-23. Sites Pumping/Generating Plant – Dissipation Chambers and Rating Curve

Design Considerations

Preliminary design of the pumping/generating plant was performed in accordance with the state-of-practice for pumping plants, with conformance to current dam safety criteria where applicable. The SPGP was designed as a pumping/generation plant because all water stored in Sites Reservoir would have to be released back through the plant units. Making reservoir releases through the plant units provides an effective method of dissipating energy from the outflow. The configuration of the conceptual design relies on using the generating units for ordinary releases and scheduled releases for up to 6 hours per day on-peak, and a plant bypass with cone valves to make emergency releases.

Facility Layout

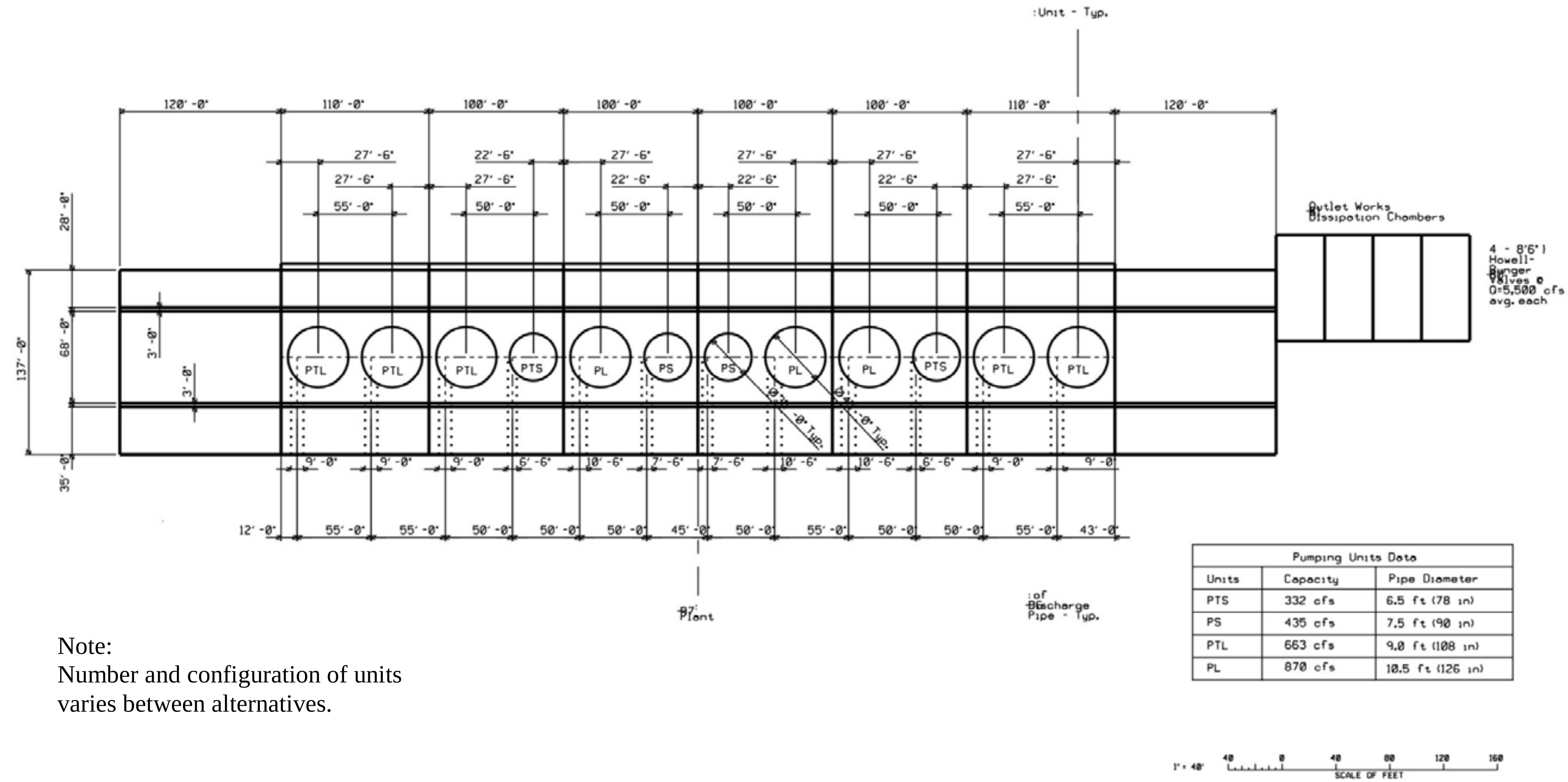
The approach for laying out the SPGP would be similar for the four alternatives. The SPGP would be on a relatively low, flat bench at an approximate elevation of 215 feet to minimize excavation volume. The materials excavated for the pumping/generating plant foundation would consist of colluvium, underlain by weathered and fresh sandstone of the Cortina Formation. Excavation for the approach channel is expected to consist of alluvium, underlain by weathered and fresh sandstone. Both excavations are likely to encounter groundwater, and require dewatering during excavation, because the maximum excavation depth would be at an elevation of 144 feet, and groundwater surface elevation would be 10 to 20 feet below the original ground level.

The proposed excavation would have 2H:1V slopes, terraced with 15-foot-wide benches at 40-foot vertical intervals. In the next design phase, studies should be performed to evaluate the stability of steeper excavation slopes.

The SPGP would lift water from Holthouse Reservoir into Sites Reservoir. The SPGP would be connected to Holthouse Reservoir by a long, excavated approach channel. Currently, Funks Reservoir operates in coordination with the T-C Canal between elevations of 203 and 205 feet. However, with the Holthouse Reservoir design, the SPGP would operate with tailwater elevations down to an elevation as low as 190.0 feet during pumping to take advantage of the full 6,500 AF active capacity of Holthouse Reservoir for subsequent generation. The 30-foot-diameter tunnel would be on the inlet side of the pumping/generating plant connecting it to Sites Reservoir.

Figure B.3-24, Figure B.3-25, and Figure B.3-26 present conceptual details for the SPGP for Alternatives A, B, C, and D. Details differ between the NODOS/Sites Reservoir Project alternatives, because the number of units and unit sizes may differ.

Figure B.3-27 through Figure B.3-31 present conceptual details for an alternative pumping/generating arrangement that was considered for the SPGP for Alternative D only. The alternative concept would employ all pump-turbine units with variable-frequency drives to provide similar flow capacities and operational flexibility as Alternative C. The arrangement was not pursued as part of evaluating the project alternatives because it would introduce higher equipment procurement costs and requires a larger structure to accommodate the variable frequency drive equipment. This arrangement still requires further evaluation to determine if its potential benefits offset the increased cost.



Pumping Units Data		
Units	Capacity	Pipe Diameter
PTS	332 cfs	6.5 ft (78 in)
PS	435 cfs	7.5 ft (90 in)
PTL	663 cfs	9.0 ft (108 in)
PL	870 cfs	10.5 ft (126 in)

Note:
Number and configuration of units varies between alternatives.

Figure B.3-24. Sites Pumping/Generating Plant – General Arrangement – Service Floor – Q=5,900 cfs

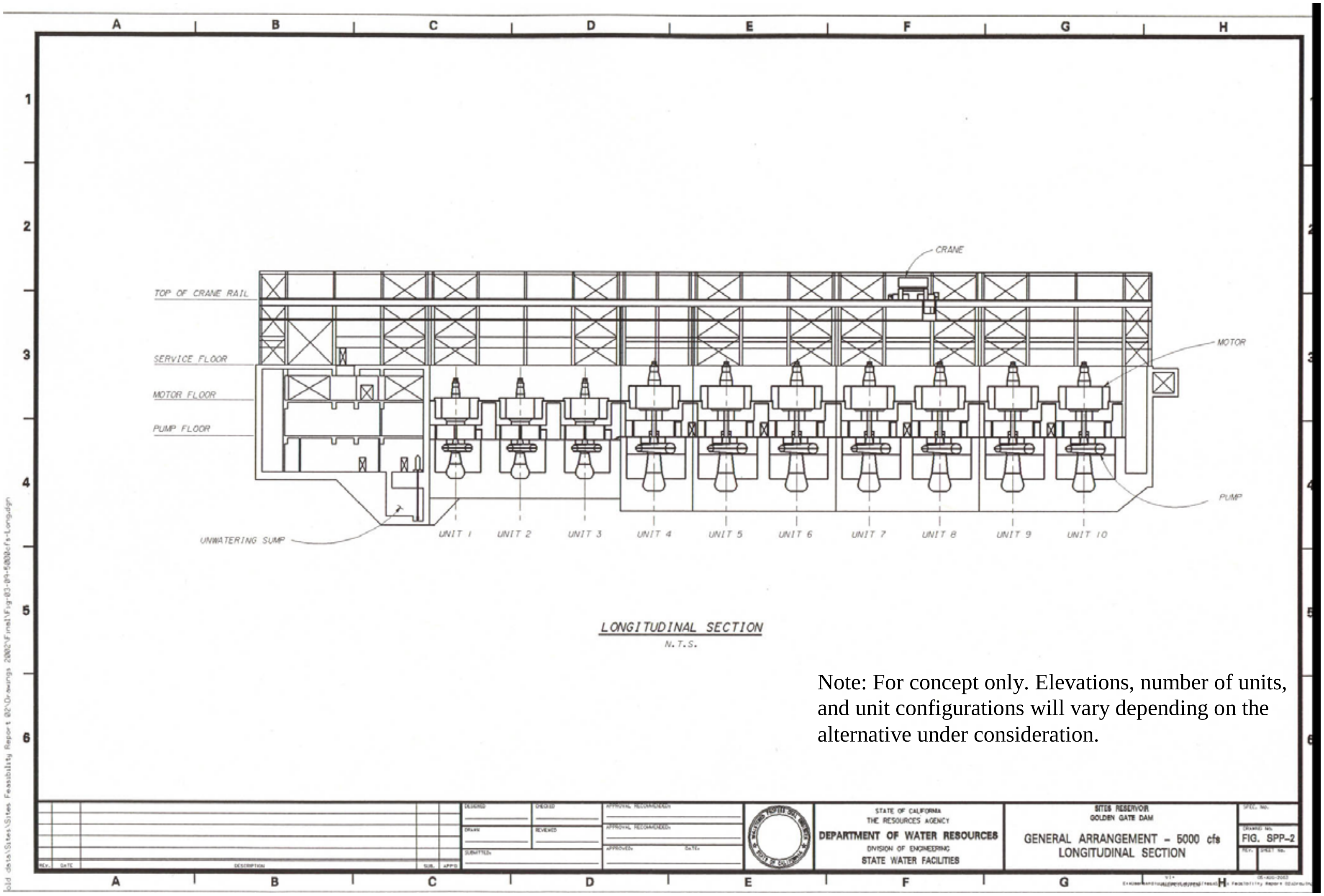


Figure B.3-25. Sites Pumping/Generating Plant – Q=5,900 cfs – Longitudinal Section

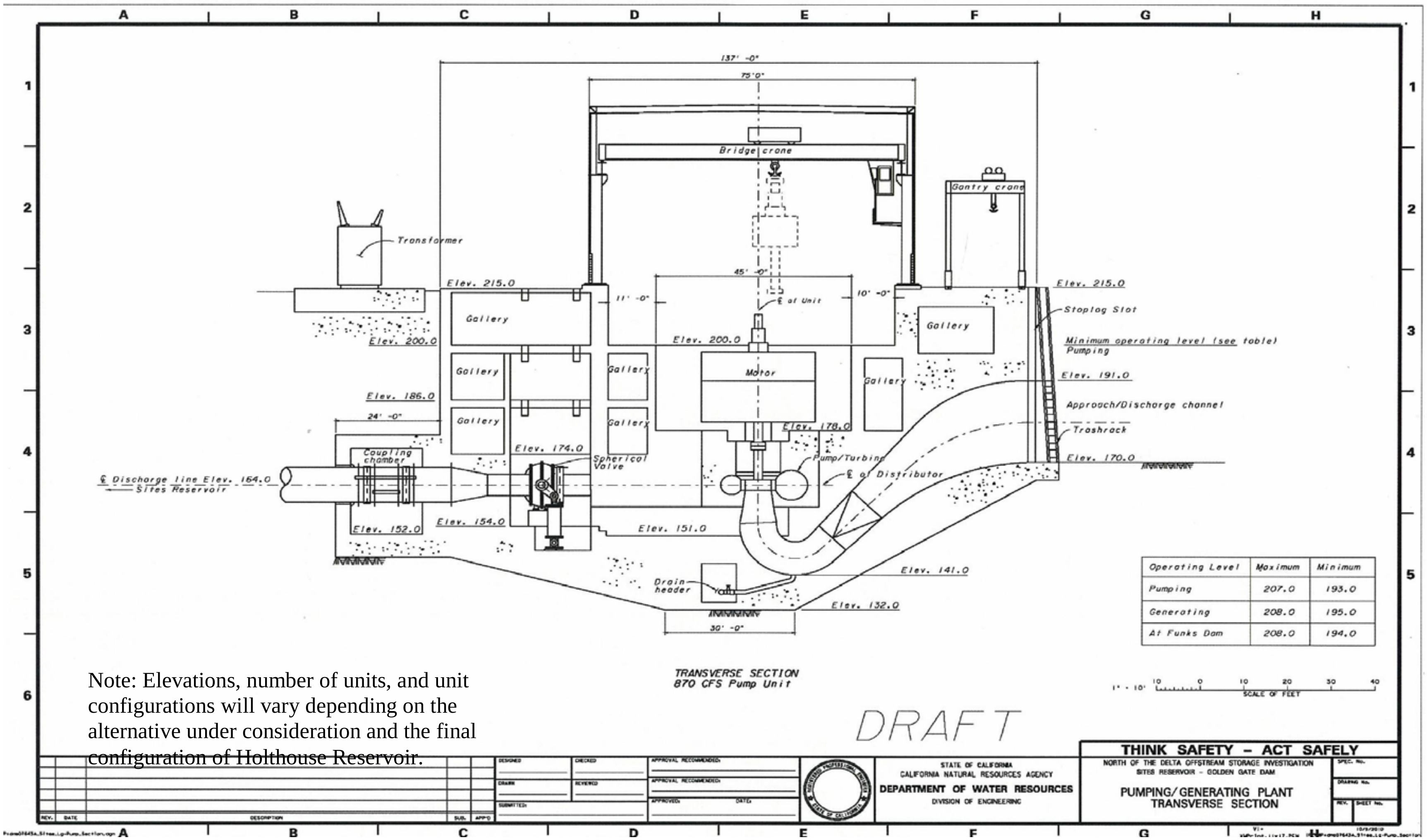


Figure B.3-26. Sites Pumping/Generating Plant – Transverse Section

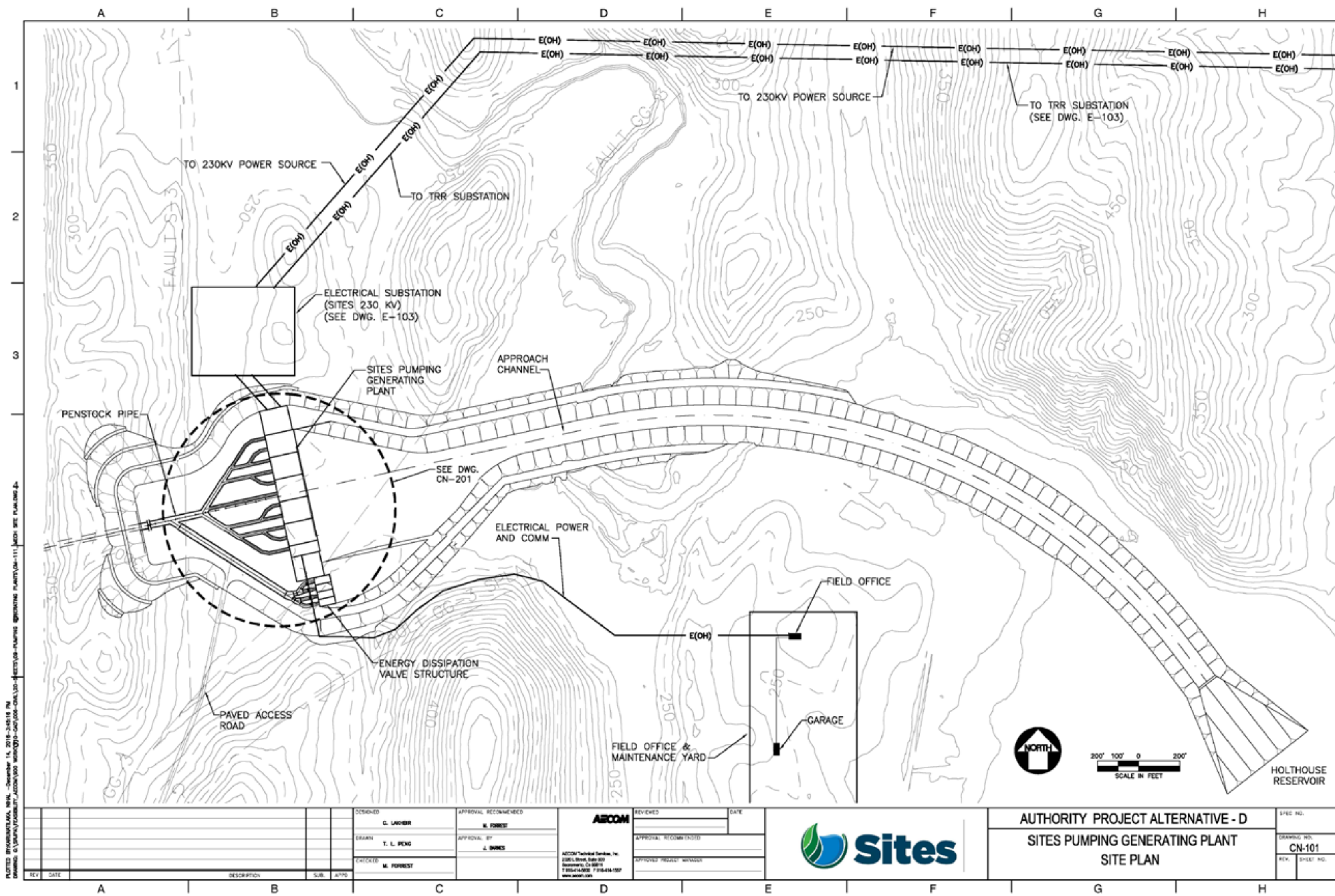


Figure B.3-27. Site Plan

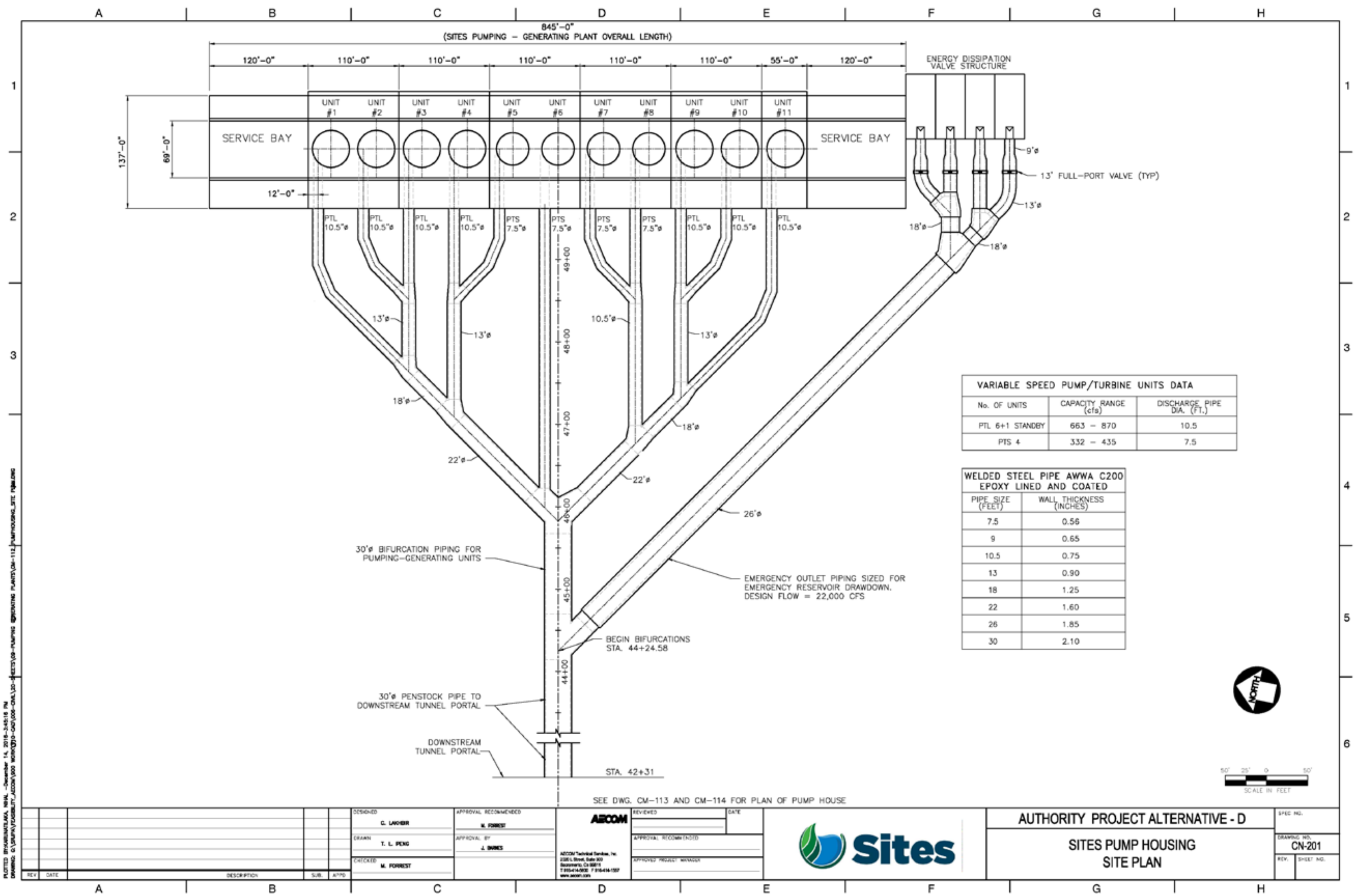


Figure B.3-28. Structure Plan

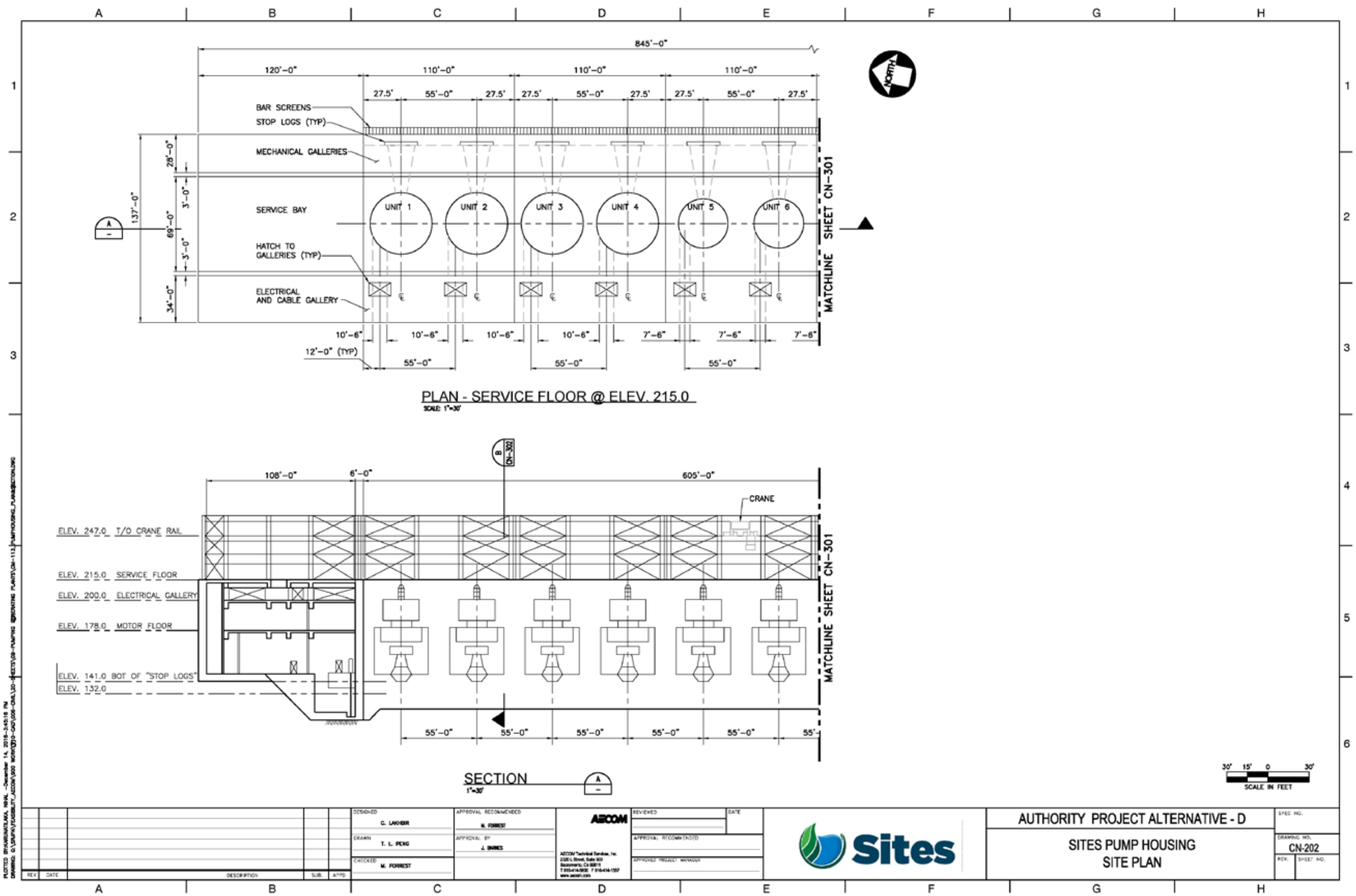


Figure B.3-29. Elevation Sheet 1

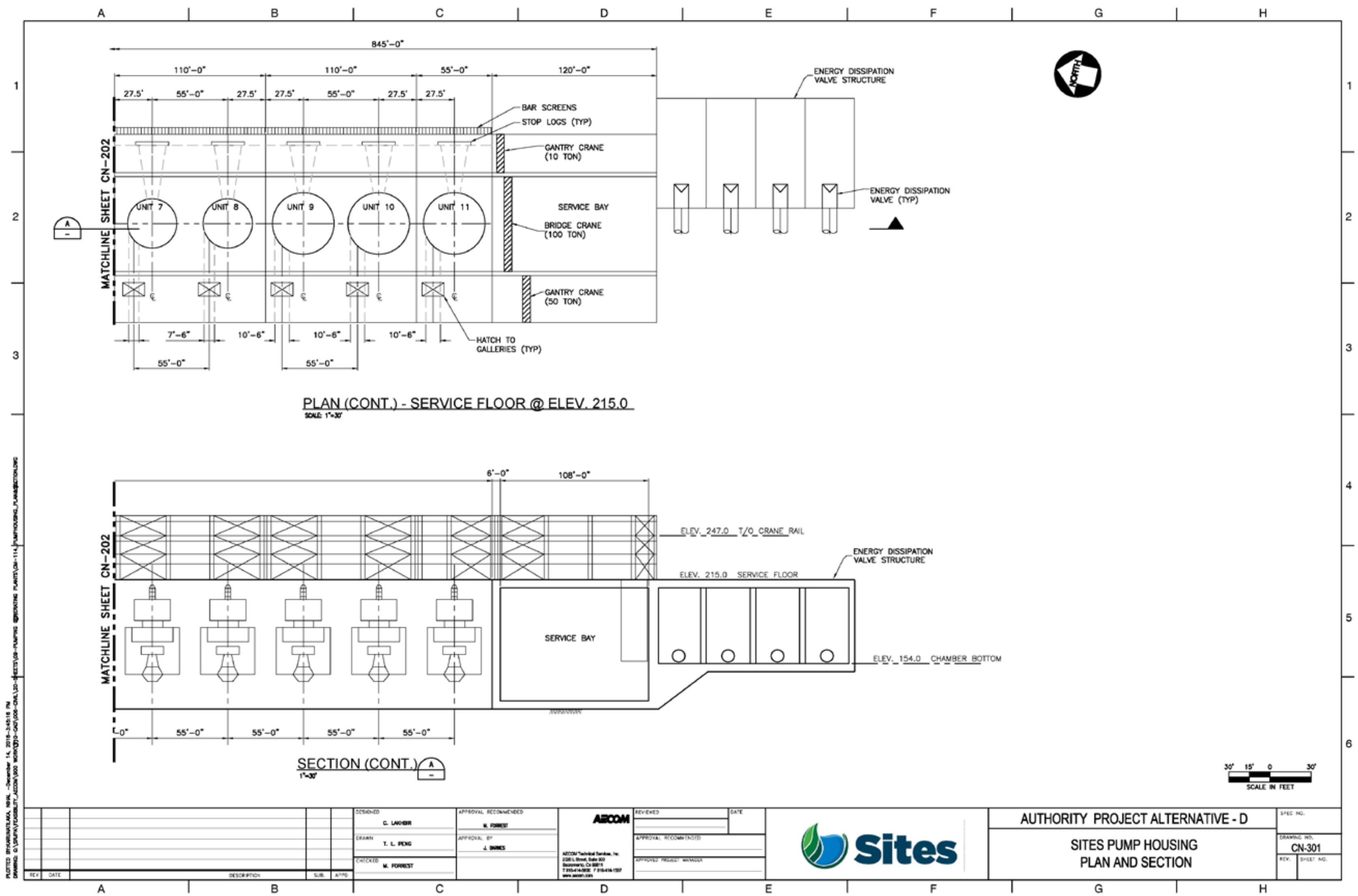


Figure B.3-30. Elevation Sheet 2

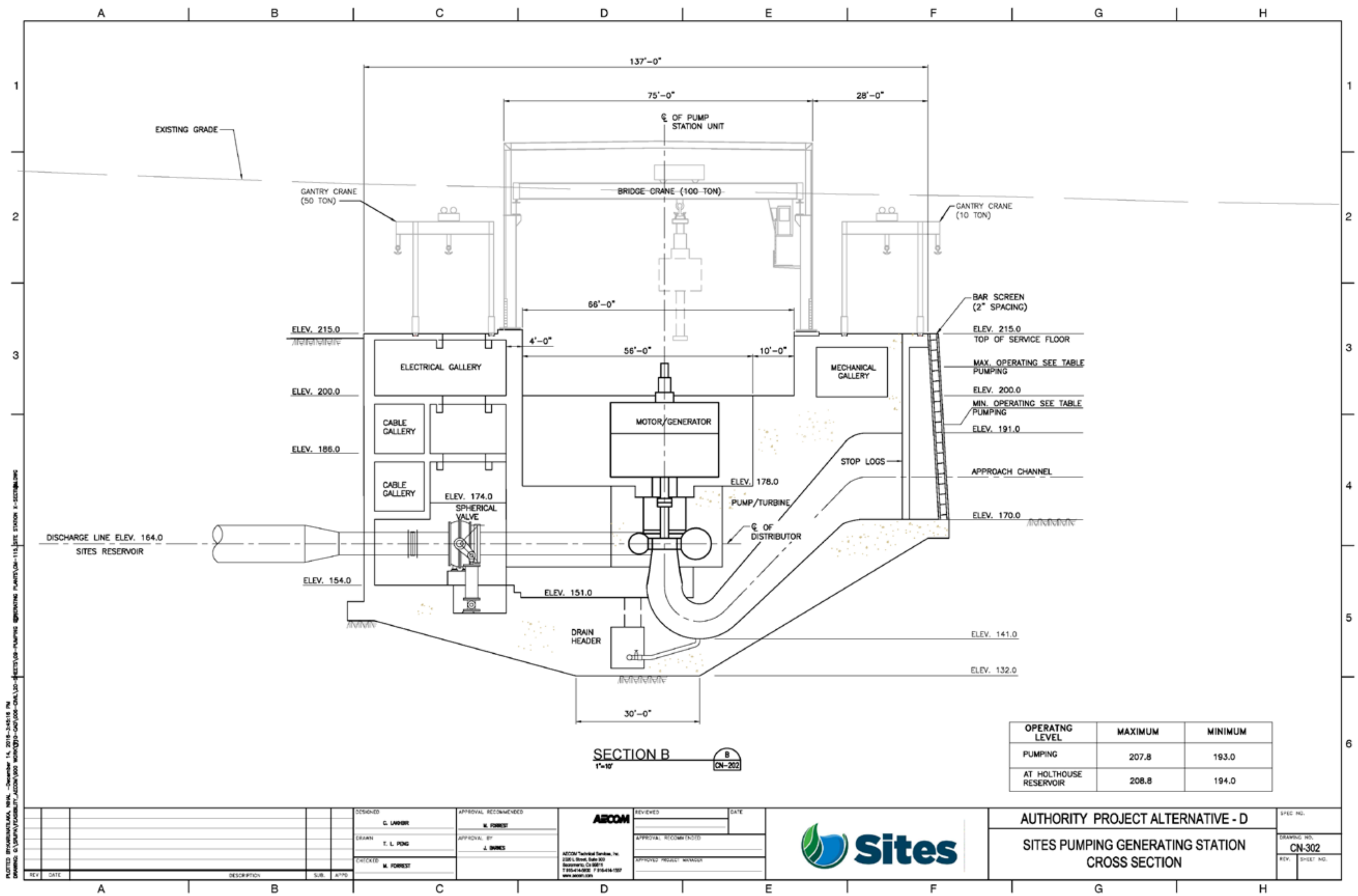


Figure B.3-31. Cross Section