PANAMA TSL INLET & OUTLET MONOLITHS, APPROACH STRUCTURES, AND WATER SAVING BASINS

by

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ABSTRACT

A summary of the Design of the Inlet and Outlet Monoliths, Approach Structures, and Water Saving Basins of the Panama Canal Third Set of Locks (TSL) is presented, including challenges encountered, areas of innovation for the features, and construction photos. The paper theme and topic are inland navigation, and waterway infrastructures associated with locks.

1. INLET AND OUTLET MONOLITHS

1.1 General

Inlet and Outlet Monoliths (Wing Walls) are very complicated 3-dimensional structures located at the beginning and end of the lock complexes that were built for the Panama Canal TSL Expansion, and they provide a smooth transition for water flow in and out of the structures.

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Figure 1: Pacific Outlet Monolith (Wing Wall) Construction, Tetra Tech(1)

1.2 Design Challenges

The design challenges for the Inlet and Outlet Monoliths of the Third Set of Locks included:

1) Very complex 3-D geometry monolith structures that required 3-D finite element analysis using ABAQUS nonlinear finite element software with acoustic water elements.
2) Thermal Analysis of massive concrete monolith sections.
3) High Seismic Zone Design requirements at the Pacific Site for heavy civil structures.
4) Discovery of an unknown fault at the Pacific Inlet (Lake Gatun) Monoliths.
5) Variable geotechnical conditions.
6) Differential Settlement of adjacent Monoliths.
7) 100-year Design Life Durability requirements in a marine environment.

1.3 Design Innovations

The design innovations for the Inlet and Outlet Monoliths of the Third Set of Locks included:

1) ***Design Optimization for Hydraulic Performance.*** The orientations of both the Inlet and Outlet Monoliths were optimized to provide a smoother transition for the water flow into and out of the culverts. This configuration resulted in better hydraulic performance and more efficient flow through the system with less hydraulic losses as the water enters and exits the system.
2) ***Design with State-of-the-Practice Seismic Analysis Techniques.*** Tetra Tech used state-of-the-practice seismic analysis techniques and tools to develop the reinforced concrete structure configurations to meet the stringent seismic criteria set by the Employer Requirements. Site-specific Time-History Seismic Analyses used seven worldwide historical earthquakes which were scaled and combined to determine the loads and stresses within the structures through the use of Abaqus software 3-D Finite Element Analyses. The 3-D nonlinear finite element analyses used soil-structure interaction and water-structure interaction with acoustic elements to model the water in the culverts and conduits during an earthquake.
3) ***Design for 100-Year Design Life Durability.*** The demanding 100-year design life durability requirement for the marine structures was met through: careful design and detailing of the

reinforced concrete structures, specification of good dense concrete mixes, and design of sufficient concrete cover in order to provided sufficient chloride ion penetration resistance to prevent the initiation of corrosion in the reinforcing steel.

4) ***Design with High Strength Reinforcing Steel.*** Higher strength reinforcing steel, Grade 75 ksi instead of the standard Grade 60 ksi, was used in many cases to reduce reinforcing congestion, improve concrete placement quality, and to reduce and overall project costs.
5) ***Design Monolith Geometry Optimization.*** Early gravity monolith concept designs were optimized to minimize concrete volume through the use of efficient flexural designed stem walls for the upper portions of the monoliths.
6) ***Design for Concrete Placement Thermal Stresses.*** Non-linear incremental thermal stress analysis was used in the monolith concrete design to limit the thermal stresses generated in the thick sections during construction to within acceptable ranges.

1.4 Description of Structures

Inlet and Outlet Wing Walls are located at the beginning and end of the lock complexes that were built for the Third Set of Locks Panama Canal Expansion. The Wing Walls function both as earth retaining structures and water intakes and outlets. Water enters and exits the lock chambers through the culverts in the Wing Walls. Four sets of Wing Walls were built for the TSL project, at the inlet and outlet of the Pacific and Atlantic Lock Complexes. The concrete monoliths vary in size and are each up to 14,000 cubic meters in concrete volume. The Wing Walls incorporated several challenging aspects into the design.

The Wing Walls incorporated several challenging aspects that had to be accounted for in the design. Most significantly, in order to reduce head losses and improve the performance of the filling and emptying system, as well as reduce the overall length of the wing walls, the culverts continue in a straight line and enter and exit through the angled face of the wing walls, rather than turning 90 degrees and intersecting the straight face as they would in a traditional design. This leads to a complex geometry that defies a traditional two-dimensional analysis. The culvert and wall geometry change continuously as well, meaning that each monolith is unique and had to be designed individually. ABAQUS software was used for the finite element analysis.

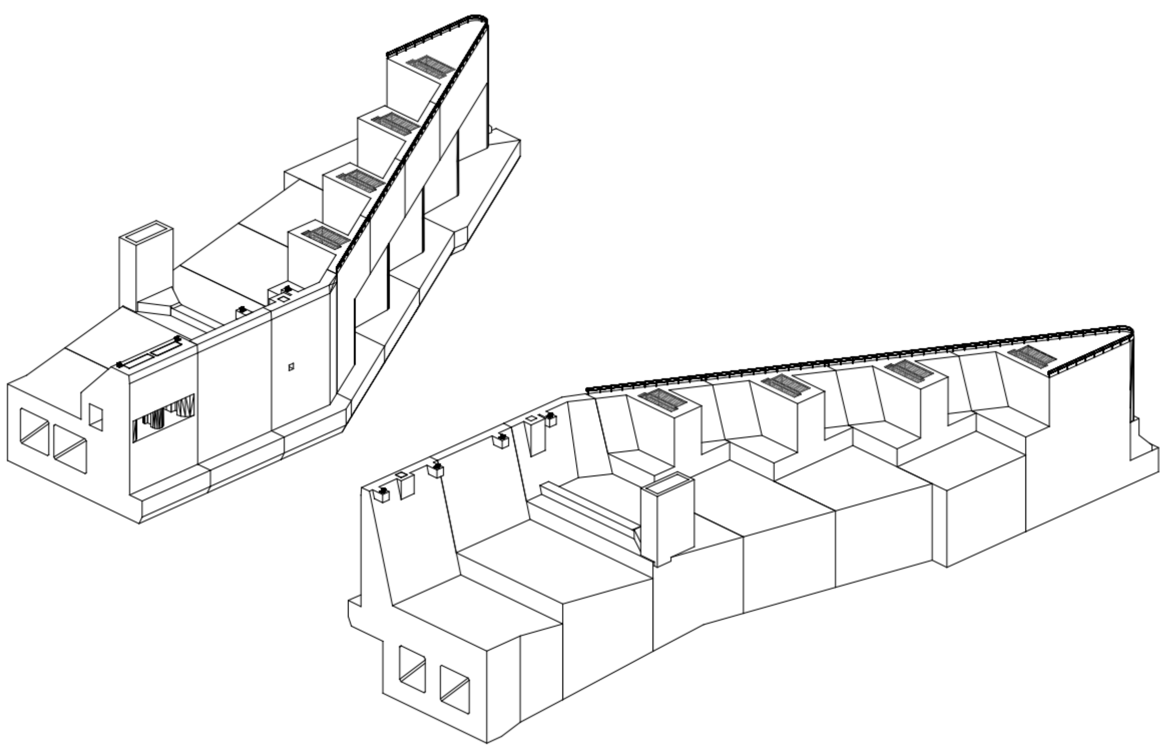

Figure 2: Isometric View of the Atlantic Inlet Wing Walls, Tetra Tech(1)

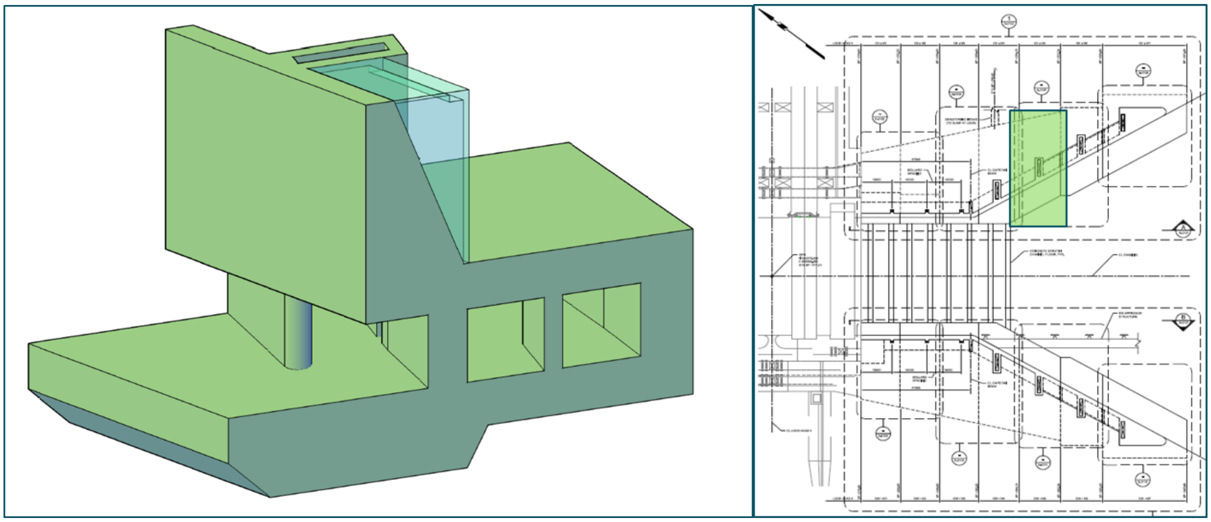

Figure 3: Isometric view and Plan View of a Wing Wall Monolith, Tetra Tech(1)

1.5 Two-Dimensional Analysis

Two-dimensional analyses were used for validation of the ABAQUS parameters and for the intermediate design of the wing walls. Planar slices of the wing wall were taken perpendicular to the channel centerline. The ABAQUS models were verified against the hand calculation by comparing the forces and moment calculated by the seismic coefficient method against the forces and moments generated from the time history analyses.

1.6 Three-Dimensional Analysis

For the final design, a three-dimensional SolidWorks model of each set of wing walls was developed and imported into ABAQUS. The 3D model was used for both static and dynamic analyses. Nonlinear elements were utilized for the foundation and the backfill. Nonlinear contact elements were used to model the interaction between the concrete monoliths and the foundation and the backfill. Acoustic elements were utilized for the water to model the hydrodynamic effects. The 3D model was necessary to model the complex structure of the wing walls.

Seven time histories were run on the model at two different return periods. Each of the time histories was deconvolved, scaled and rotated to match the site-specific conditions. The global model containing one set of the wing walls was meshed at a density sufficient to obtain accurate behavior of the model but not necessarily accurate stresses within the concrete monoliths.

For the reinforced concrete design, the global model was separated into a number of sub-models containing one or two monoliths, which were re-meshed with a finer mesh to generate more accurate stress results. A series of internal surfaces was inserted at key locations, such as at the mid-spans and supports over the culverts, for the purpose of extracting design forces. The displacements calculated from the global model runs were mapped onto the refined monoliths, and stresses and forces were extracted to be used for reinforcing design.

1.7 Extraction and Post-Processing of Data

Due to the extremely large data sets produced from the ABAQUS analysis, a workflow for the extraction of data, the design of concrete reinforcing, and the checking of the structure against the design criteria was developed. Extraction of the data was scripted using Python. The process for defining and designing the reinforcing sections was scripted in Visual Basic and Microsoft Excel to meet the design criteria. Scripting of the work made it possible to optimize the design quickly in order to meet the schedule.

Stability of the monoliths was assessed by checking the peak deflections during the time history runs and the residual deflections at the end of the runs. Deflections were measured at the top of the monolith relative to the base of the monolith and to a point in the rock foundation. Contact pressures at the interaction between the base of the monolith and the foundation below were also checked against the allowable pressures defined in the design criteria.

1.8 Settlement Analysis

During excavation of the Inlet Wing Walls at the Pacific Lock Complex, a fault zone with sheared and weakened rock was located under two of the monoliths. There was a concern that there would be differential settlement between the monoliths located in the fault zone and the ones outside of it. A standard analysis showed that there would be significant settlement. To improve on those initial results, a more sophisticated analysis using a staged method was used to better estimate the actual settlement. The staged analysis simulated the actual construction process of the monoliths. The model simulates the static case of the monolith in a series of steps, with more parts of the monolith added at each step with gravitational loads. The analysis adjusts the location of each subsequent piece of the monolith added to the model to eliminate any instantaneous settlement that occurs from the weight of the concrete. The analysis showed that the differential settlement was acceptable.

1.9 Purpose of the Concrete Thermal Analyses

The Inlet and Outlet Monolith (Wing Wall) structures include individual concrete lifts up to 3 meters in height, and 3 meters in width. Such massive concrete lifts can induce significant heat gain in the concrete, during the cement hydration process. The purpose of the finite element thermal analyses is to determine time-dependent temperature distributions in the concrete of the wing wall structures during construction, to calculate thermally induced strains, to evaluate the potential for concrete cracking in the concrete during construction, and to provide recommended concrete placement temperatures based on the analysis results.

1.10 Thermal Analysis Methodology

Three-dimensional non-linear thermal cracking and strain analyses were performed for the Atlantic and Pacific Inlet and Outlet wing wall structures of the Panama Canal Third Set of Locks Project. The thermal analysis was conducted to calculate the potential for thermal cracking of the large concrete wing wall monoliths, with the goal of limiting the potential for future corrosion of the concrete reinforcing bars. A “level 2” thermal strain analysis was conducted based on U.S. Army Corps of Engineers guidelines, and consisted of time-dependent mass and a surface gradient strain analyses. Site-specific climatological data, adiabatic heat-gain test results of the proposed concrete mix designs, concrete mix time-dependent strength gain, rock/soil foundation properties and proposed concrete lift geometry was included in the analyses.

Adiabatic time-dependent thermal forcing functions were derived for each proposed concrete mix design, and calibrated to provide the precise energy input required for the thermal models. The average daily ambient temperatures including the diurnal cycle were applied to the exposed surfaces of the concrete at each time step in the model, to accurately model the effects the ambient atmospheric conditions have on the concrete hydration process. Formwork acts as an insulator, so the time dependent effects of the formwork on the sides of the concrete lifts was included in the models. The theoretical heat gain of the large monolithic concrete Wing Wall structures was then modeled in 3D using the non-linear finite element analysis program ABAQUS. After the models for each monolith were run, time-dependent thermal data was then extracted and post-processed to calculate the potential for thermal cracking in the concrete during the concrete hydration process. During construction of the monoliths, thermocouples were placed within the concrete at specific locations to collect time-dependent heat gain data. This data was then used to further calibrate the adiabatic thermal forcing functions used in the ABAQUS models. Based on the results of these analyses, the concrete mix, lift geometry, and the concrete placement temperatures are modified to reduce the potential for thermal cracking, thus greatly improving the long-term durability of the Wing Wall structures. In summary, the thermal analysis for the Wing Walls and other thick concrete sections provided the following:

- 3D ABAQUS Thermal models capture time-dependent boundary layer effects (atmospheric variation, formwork)
- Nonlinear concrete material properties vary with curing time
- Concrete heat gain is controlled by derived adiabatic forcing functions
- Analyses are performed using assumed concrete initial placement temperatures
- Section thermal gradients are calculated based on post-processing of the model output data
- Post-processing of the thermal gradient data results in meaningful section strain and cracking results
- Concrete initial placement temperature and/or section geometry is adjusted as necessary to limit thermal strain demands and cracking potential
- Simplified Level 2 thermal modeling is cost effective

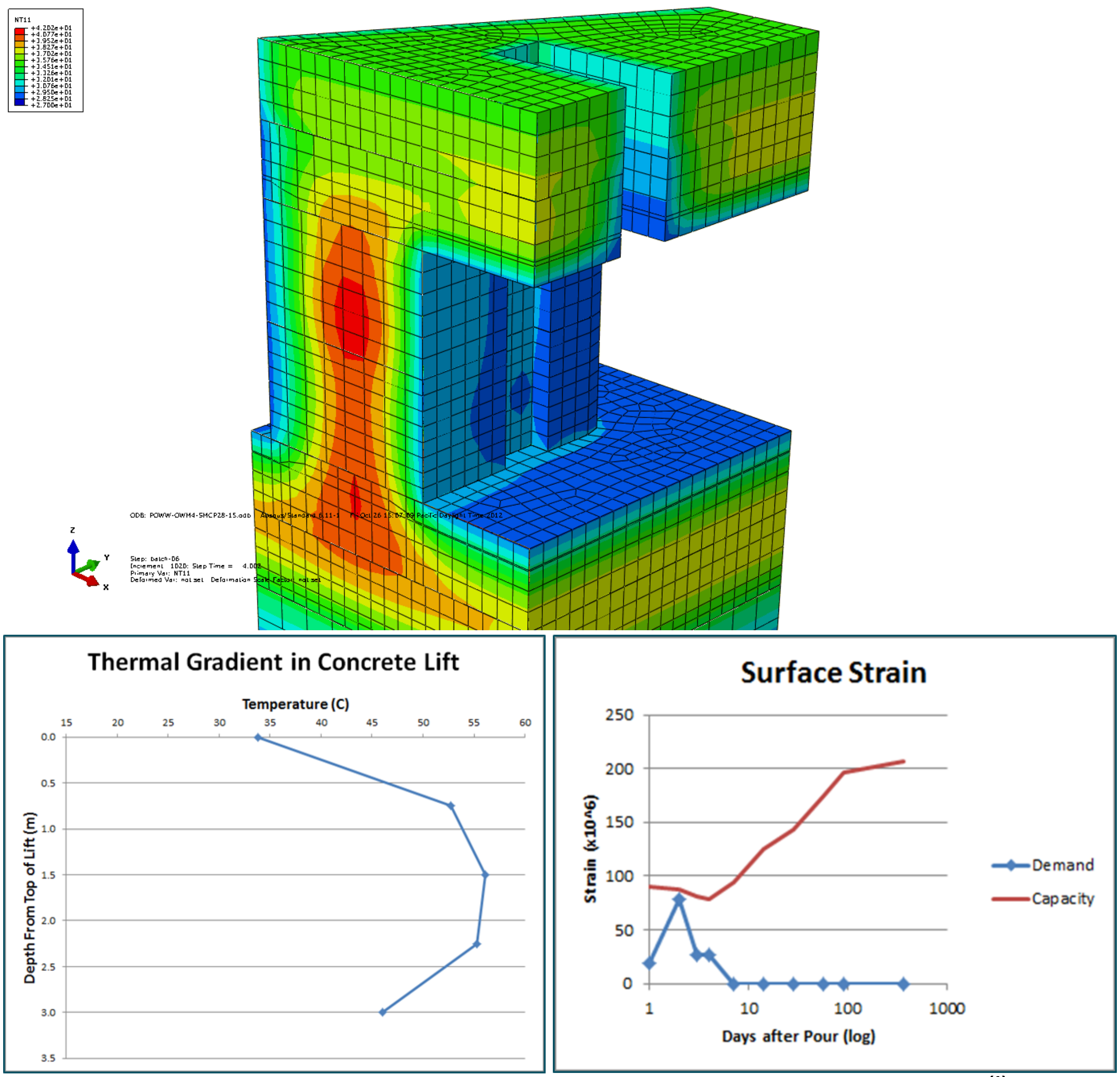

Figure 4: 3-D FEM Wing Wall Monolith Concrete Heat Distribution 1, Tetra Tech(1)

2. APPROACH STRUCTURES

2.1 General

The new Third Set of Locks for the Panama Canal includes Approach Structures at three of the four entrances. Approach Structures are required to help pilots approach, align, or pre-position vessels to reduce entrance time into the lock chamber. The Approach Structures also protect vessels and surrounding structures from accidental berth impacts. There are three Approach Structures; they are located at the Atlantic Ocean, Pacific Lake, and Pacific Ocean sites. Each Approach Structure is approximately 450 meters in length. The approach structures are designed as "hydraulically invisible" structures to ease vessel approach and exit. Fenders absorb energy from New Panamax ships approaching at a maximum of 2 knots. The following describes the design of the Approach Structures.

Figure 5: Pacific Outlet (Ocean) Approach Structure, Tetra Tech(1)

2.2 Design Challenges

1) Large ship impact loads for New Panamax ships approaching at a maximum of 2 knots.
2) Large seismic loads applied on tall hydraulic structures.
3) Large tidal variations at Pacific site compared with Atlantic site. The tide range for the Pacific site is 7.0 meters, compared with the tide range for the Atlantic site is 1.0 meter.
4) Varying geotechnical conditions, including fault zones.
5) Expansion and contraction over the 450-meter length of each approach structure had to be accommodated while carrying the large loads.
6) Approach structures were designed to be "hydraulically invisible" to ease vessel approach, and to easily allow water flow into and out of the lock monoliths and to allow mixing of the saltwater and freshwater.

2.3 Design Innovations

1) ***Design for Large Vessel Loads Nearly Continuously Applied.*** Large ship impact loads were transmitted to the structure by cell fenders with nearly continuous fender panels. This was

particularly challenging given the large 7-meter tide variations at the Pacific Ocean. The superstructure box girders were designed to provide good torsional resistance and strength to transfer the large loads to the piers.

2) ***Design for Varying Geotechnical Conditions.*** Varying geotechnical conditions were dealt with by designing varying lengths and quantities of drilled shafts as required by the actual geotechnical conditions. Rock bolts were used in some locations where the pier foundation rested directly on bedrock.
3) ***Design for Large Longitudinal Movements.*** Wide expansion joints were designed at 75-meter spacing over the 450-meter length of each approach structure while carrying the large transverse loads through elastomeric bearings.
4) ***Design for Hydraulic Transparency.*** Approach structures were designed to be transparent to easily allow water flow into and out of the lock monoliths and to allow mixing of the saltwater and freshwater.
5) ***Design for Ductility.*** An innovative feature of the design was the use of ductility for the seismic design. An efficient structural system was developed that utilized a ductile design allowing for more predictable seismic response and a lighter structure.

2.4 Design Development

The proposed configuration of the Approach Structures was physically tested at the Flanders Hydraulics Research Laboratory in Antwerp Belgium as part of the conceptual design phase. As the design was being further refined, ACP put forward a request that the design be changed to provide continuous or semi-continuous fenders in lieu of the 15-meter spacing from the tender and over-the-shoulder concepts. Given the high energy absorption required by the owner-specified vessel impact cases, cone fenders were the only realistic choice. Therefore, continuity was provided by reducing the fender spacing to 5 meters, sizing the fender panels so they nearly touched, and linking the panels with chains.

The Panama Canal is located in a seismically active area, and significant parts of the design for the Approach Structures at both lock complexes are controlled by earthquake loads. It was agreed by the design team and ACP to allow the design for seismic loads to take advantage of structural ductility. Therefore, taking advantage of ductility in the design allowed for significant savings.

Because the transverse seismic loads are similar in magnitude to the vessel impact loads (a little lower at the Atlantic, a little higher at the Pacific) this had little effect on the strength of the structure in the transverse direction. In the longitudinal direction, however, the most significant non-seismic loads were line forces on the bollards and friction on the fenders, both of which are substantially less than the seismic load. Therefore, the use of ductility in the design allowed a large reduction in the maximum lateral force in the longitudinal direction. With the load disparity, the most efficient pier section went from round to rectangular, long in the transverse direction so that the cracking strength of the pier at its base is higher than the demand from the vessel impact case, and narrow in the longitudinal direction to create flexibility and higher ductility.

The reduction in fender spacing meant that the deck girders would have to withstand vessel impact forces, meaning that they would have to be much stronger. Given the added transverse demand, the use of simple span girders no longer made sense and the design was changed to continuous spans with longitudinal movement joints every 75 meters. This change had the added advantage of introducing frame action in the longitudinal direction, further reducing the demand on the piers in that direction. In effect the approach structure had become a bridge subject to transverse loading from vessel impact.

The approach structure concept that emerged from this design process and proceeded to final design had a box-girder deck spanning 15 meters between rectangular piers with expansion joints in the deck every 75 meters, free to move longitudinally, but not laterally. The piers were 2 meters by 6 meters in cross-section, which gave them sufficient strength in the transverse direction to resist the vessel impact loads without cracking, but reasonable flexibility in the longitudinal direction. The piers in turn sat on H-shaped pile caps supported by drilled shafts. This configuration is shown in Figure 6 below.

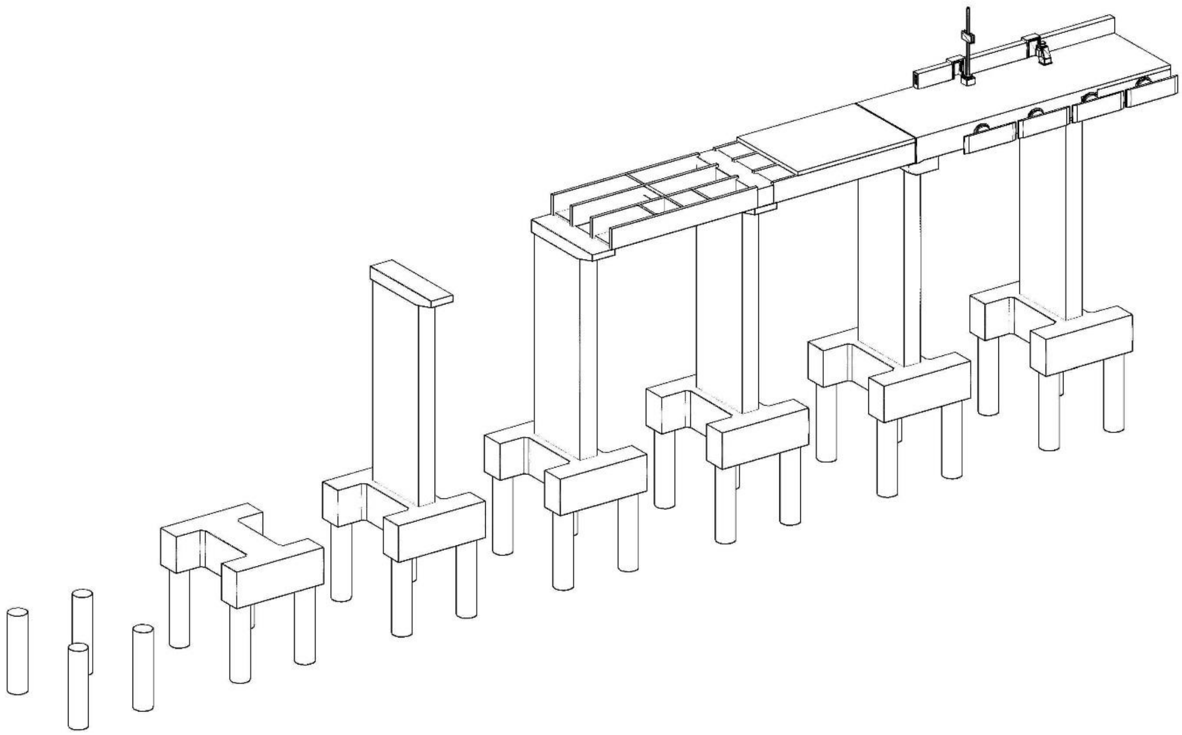

Figure 6: Approach Structure Configuration - Final Concept, Tetra Tech(1)

2.5 Design Criteria

The concrete lock structures for the Third Set of Locks—including the lock chambers and lock heads, the Wing Walls at each end, and the Water Saving Basins—were designed in accordance with US Army Corps of Engineers (USACE) design standards. These standards call for concrete hydraulic structures to remain essentially elastic under seismic loads, however, an applicable new design basis was needed. Due to the similarity of the Approach Structures to bridges, the American Association of State Highway and Transportation Officials (AASHTO) bridge code was selected to govern the design for seismic effects, while the USACE Engineering Manuals (EM's) were used for the remaining load cases.

2.6 Seismic Load Cases

The AASHTO code uses the concept of capacity design, where the designer selects the parts of the structures that will yield in a severe earthquake and the amount of ductility to be provided. The yielding regions are then detailed so that they will have the required yield strength and ductility, and the rest of the structure is designed to resist the maximum internal forces that will occur during the post-yield stage. While a strong earthquake can cause significant damage to a structure designed this way, the method provides a high level of protection against collapse, even for earthquakes larger than the design event. This is an appropriate standard for the Approach Structures, since they aren't necessary for operation of the locks, but they could block the canal if they were to collapse.

In the Approach Structures, the piers were chosen as the 'fuse' elements, and the foundations and the deck structures were designed to remain elastic under the maximum loads that can be delivered to them by the piers as the plastic hinges form and strain-hardening of the reinforcing sets in.

The designation of the piers as 'pier-columns' and 'columns' in their long and short directions respectively and the related ductility factors are a function of the depth-to-height ratio of the column and the reinforcing detailing provided. Columns are relatively slender elements with a high level of confining transverse reinforcing in their hinge zones that protects the concrete inside the outermost reinforcing hoop from crushing. A pier-column is a deeper element with less required transverse reinforcing and, consequently, less ductility.

2.7 Non-Seismic Load Cases

In addition to the Level I and Level II earthquakes, two non-seismic load cases were considered: Bollard Loads, and Vessel Impacts. For both of those cases, deck live load was included. The live load used in the load combinations was an envelope of a 10kPa uniform load, two patterns of 10kPa skip loading where some spans were loaded and others weren't, wheel loads from a HS-20 truck, and outrigger loads from a 60-ton crane.

Bridgestone, the fender supplier for the Approach Structures designed the fender system. The fenders selected were 900 mm cone fenders with UHMW-faced steel panels. The panels were all 4.7 meters long and linked together by chains for the full length of the approach structures, but the panel heights depended on the tidal ranges at the different approach structures. Bridgestone calculated the fender reactions for each of the design vessels for the Unusual vessel impact scenario and provided the maximum reactions to the designers. Calculations were done for two situations: one for an impact centered on a fender (meaning an odd number of fenders resisted the impact), and the other with the impact centered between two fenders (with an even number of fenders participating). Given the close spacing of the fenders and the size and bow radii of the design vessels, 5 fenders participated in the odd-number case and 6 fenders in the even-number case. The reactions provided by Bridgestone and diagrams of the two cases are shown in Figure 7 below.

Fender Loads			
5-Fender Impact		**6-Fender Impact**	
Fender	**Reaction (kN)**	**Fender**	**Reaction (kN)**
1)	893	1)	618
2)	850	2)	917
3)	791	3)	795
4)	850	4)	795
5)	893	5)	917
		6)	618
Total	**4,275**	**Total**	**4,659**

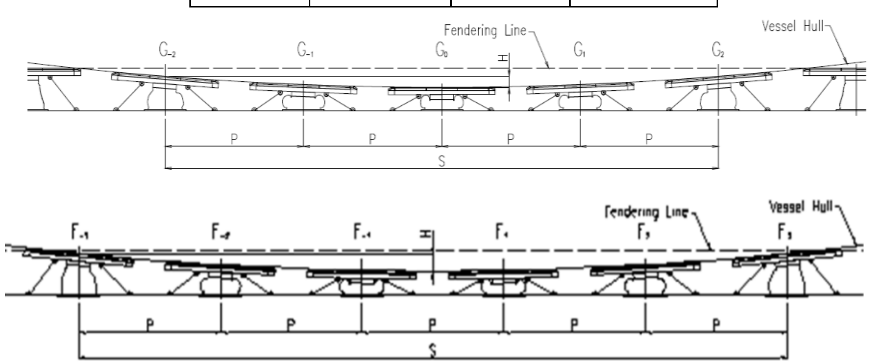

Figure 7: Fender Load Table and Diagrams

The fender reaction patterns for both the 5-fender and 6-fender cases were applied to the global analysis model starting at various locations along the deck to ensure that the maximum effects were captured. For instance, for the 5-fender case, Load pattern 1 would apply 893 kN to Fender 1, 850 kN to Fender 2 and so on to Fender 5. Load pattern 2 would apply the same pattern of loads to Fenders 2 through 6 and so forth along the full length of the model.

2.8 Geotechnical Conditions

Each of the Approach Structures had significantly different foundation conditions. At both the Atlantic and Pacific Outlets, the channel bottom was excavated into bedrock for the full length of the Approach Structures. At the Atlantic, the entire structure was founded on Gatún formation, which consists of soft sedimentary rocks. At the Pacific Outlet, the bedrock material for most of the length of the structure is La Boca formation, consisting of even softer sedimentary rocks than the Gatún formation. However the far end of the structure is founded in very hard Basalt rock. In between the Basalt and La Boca regions is an area referred to as 'Baked' La Boca, where the La Boca materials were hardened from the heat of the Basalt intrusions and are almost as strong as the Basalt itself.

At the Pacific Inlet, the bedrock surface lies 2 to 10 meters below the bottom of the channel. The intervening layer of overburden was potentially scourable and provided minimal resistance compared to the rock below even if it remained in place, so the overburden was neglected in design. Beneath the overburden, the bedrock was quite variable along the length of the structure, starting with basalt at the far end of the structure, transitioning to La Boca of varying hardness, then to a zone of highly sheared materials at an inactive fault and finally basalt again immediately adjacent to the wing wall. The geological profile developed for the Pacific Inlet Approach Structure is shown in the Figure below.

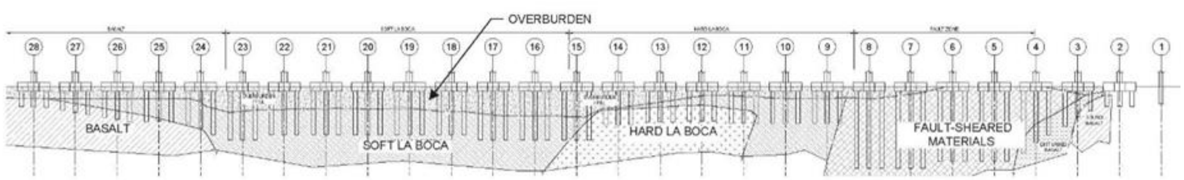

Figure 8: Geologic Profile at Pacific Inlet Approach Structure

2.9 Analysis and Design

The analysis and design of the Approach Structures involved a number of finite element models, with the seismic design following this basic sequence:

- Use a global model of the structure to determine the dynamic response and the shears and moments in the piers;
- Reduce the moments in the piers by the ductility factors and design the pier reinforcing to resist those moments;
- Calculate the maximum strength (plastic capacity) of the piers at the top and bottom;
- Apply the maximum pier base moment and associated shear to the foundations and design;
- Apply the maximum pier top moment at each pier location in a model of the deck and design;
- Revise the global model to reflect any changes to the deck or foundation, and repeat the steps above until no further modifications are necessary.

The global model for each of the approach structures was a three-dimensional frame model run in the program SAP 2000. The models each included three deck segments (the lengths between expansion joints): the section closest to the wing wall; a typical 75-meter long, 5-span segment in the middle; and the segment at the outboard end of the structure. Because all of the middle segments were essentially identical, these three-segment models were adequate to capture the behavior of the whole structure. Within these models, the piers and deck girders were represented by frame members with additional dummy members extending from the deck centerline to the fender locations, where the fender and bollard loads were applied.

The foundation stiffness was incorporated into the model using linear springs at the bases of the piers. The spring constants were taken from the output of the foundation analysis done using the program GROUP. Because the foundation stiffness affects the structural response, and therefore the load to the foundations, development of the spring constants was an iterative process, but one that converged quickly.

The linear multimode spectral analysis method—using the site-specific response spectra provided by ACP—was used for the seismic analysis. The structural masses were calculated by the program and lumped at the nodes. Due to the similarity of the piers to reservoir intake towers (fixed-base cantilever columns in water), the hydrodynamic added mass on the piers was calculated using the Goyal and Chopra method from Appendices C and D of EM-1110-2-2400, "Structural Design and Evaluation of Outlet Works." Because the sum of the structural and hydrodynamic mass of the piers is a significant portion of the total for the structure, the piers were broken up into 7 elements so that the mass was distributed along the length of the pier.

2.10 Pier Design

Because it is the 'fuse' element for the seismic design, and therefore controls the seismic load in all of the other elements, the pier was the starting point for the design process. The pier dimensions were selected so that the pier remained uncracked under the vessel impact loads. This requirement resulted in 6-meter by 2-meter piers for the Pacific approach structures, and 6-meter by 1.5-meter piers at the Atlantic Outlet Approach Structure, which is not quite as tall due to the smaller tidal range there.

The pier reinforcing was designed for an envelope of the seismic and non-seismic load combinations (with the seismic forces reduced by the ductility factors). Transverse reinforcing was provided per the prescriptive requirements in the AASHTO code that ensure the required ductility is provided. In addition to meeting the strength demands, the vertical reinforcing was sized to provide at least 1.2 times the expected cracking moment of the piers in order to maintain a ductile response.

Once the reinforcing was selected, the plastic moment capacity of the piers at the plastic hinges and the associated moments and shears at the top and bottom of the piers were calculated. In accordance with the AASHTO code, the plastic moment capacity was taken as 125% of the nominal moment strength of the pier.

2.11 Foundation Design

The Approach Structure foundations consist of 1.5-meter diameter drilled shafts supporting H-shaped pile caps. The drilled shaft diameter was selected by the Construction Contractor GUPC since it was the largest diameter that could be drilled in the foundation rock with equipment that was easily available in the local area. The drilled shafts were analyzed using the program GROUP and the axial capacities were calculated in accordance with the AASHTO code, considering only side friction. End bearing was neglected due to questions about compatibility between the side-friction and end-bearing responses, but also because the pullout demands on the shafts were nearly as high as the downward forces, meaning that the benefits of including end bearing were limited.

The loads applied in GROUP for the non-seismic cases were taken directly from the output of the global model, and for the seismic load cases, they were the plastic moments and shears calculated for the pier bases. Shaft reinforcing was designed based on the GROUP output, and the pile caps were designed using the results from a finite element model in SAP 2000. In addition to the design moments and forces for the shafts and pile caps, the pile cap stiffness matrix was extracted from GROUP. The matrix was then used to update the spring stiffnesses at the bases of the piers in the global SAP model.

2.12 Superstructure and Deck Design

A detailed study of the behavior of the deck under vessel impact loads using a three-dimensional shell model in SAP 2000 showed that a 'W' shaped girder with box cells at the front and back edges of the deck linked by the cast-in-place deck slab and a diaphragm at midspan across the open center bay was capable of distributing vessel impact loads through the whole assembly. By eliminating the need for a continuous connection at the bottom of the deck, precast construction became a much more attractive option, and was chosen for construction because of its accelerated construction benefits. A cross-section of the superstructure and deck construction is shown in Figure 8, and a view of the SAP shell model is shown in Figure 9. The shell model was also used to analyze the area of the girder wall at the fenders to ensure adequate capacity to resist the fender reactions.

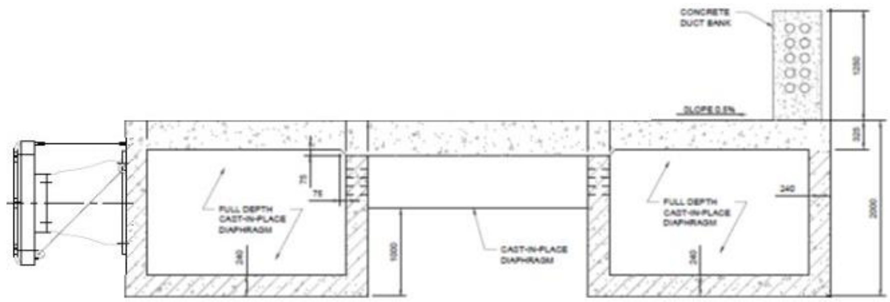

Figure 9: Approach Structure Superstructure Construction, Tetra Tech(1)

Figure 10: Finite Element Shell Model of Superstructure Deck, Tetra Tech(1)

For the non-seismic load cases, the moments and shears in the deck were taken from the global SAP model. However, because the seismic demand on the deck is due to the plastic capacity moments of the piers, the determination of the seismic forces in the deck required a separate model that included only the deck, modeled using frame elements, where the plastic capacity moments and shears at the tops of the piers were applied to the deck at the pier connections. The controlling moments and shears were then used to design the deck reinforcing.

In order to limit longitudinal stresses due to concrete shrinkage and temperature variations, expansion joints were provided in the deck at a maximum spacing of 75 meters (5 spans). The joints are free to open and close in the longitudinal direction, but are restrained in the transverse direction by a girder stop cast on the pier in the middle bay between the two closed cells. The required movement allowance was taken as twice the maximum deflection of a pier under seismic load without any ductility reduction. Based on this criterion, a movement gap of 100 mm was provided at the Atlantic and 200 mm at the Pacific approach structures. Given the large amount of movement that needs to be accommodated, Teflon-faced elastomeric slide bearings were chosen with two bearings on the corbel and one on the girder stop for each box girder. An isometric view of a typical expansion joint is shown in the Figure below.

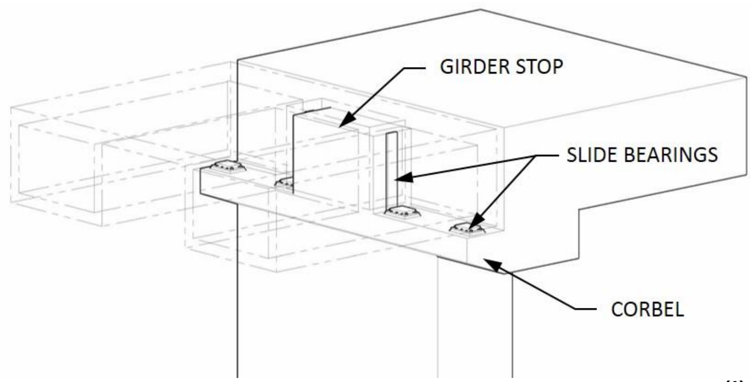

Figure 11: Expansion Joint Configuration, Tetra Tech[1]

The other significant superstructure detail is the girder splice through the pier to provide continuity at the pier-deck joint. Because the deck slab is cast in place, continuity for negative moments is provided by continuing the longitudinal reinforcing across the top of the deck pier. However continuity for positive moments caused by longitudinal movement under seismic loads was more difficult to achieve. Splicing bottom reinforcing within the pier was rejected due to the likelihood of interference between the two sets of girder bottom bars and the pier vertical reinforcing. Instead a depression was provided at each end of the precast girder that exposed the ends of the bottom bars. Splice bars running through the pier lap with the girder bottom bars on each side and a layer of cast-in-place concrete with transverse reinforcing and zee-bars tying it to the precast is used to provide continuity.

3. WATER SAVING BASINS

3.1 General

Water Saving Basins (WSB) have been designed for this Panama Canal TSL project for sustainability so that approximately 60% of the lock chamber water will be recycled. Even though the new Third Set of Locks are 60% wider and 40% longer than the existing locks, they will use less water due to the Water Saving Basins. The Water Saving Basins occupy an area equivalent to 54 football fields at each Pacific and Atlantic Complex. The components of the Water Saving Basins include: Trifurcations, Valve Structures, WSB Conduits, Basin Inlet Structures, Basin Floors, and Basin Dividing Walls. The Pacific and Atlantic Complexes have a total of 18 Basins, 36 Conduits & Intakes, 12 Valve Structures, and 12 Trifurcations. Dividing Walls (flood walls) provide barriers between basins, being 10-meter-tall reinforced concrete walls which have a total length of 4.8 kilometers for both complexes.

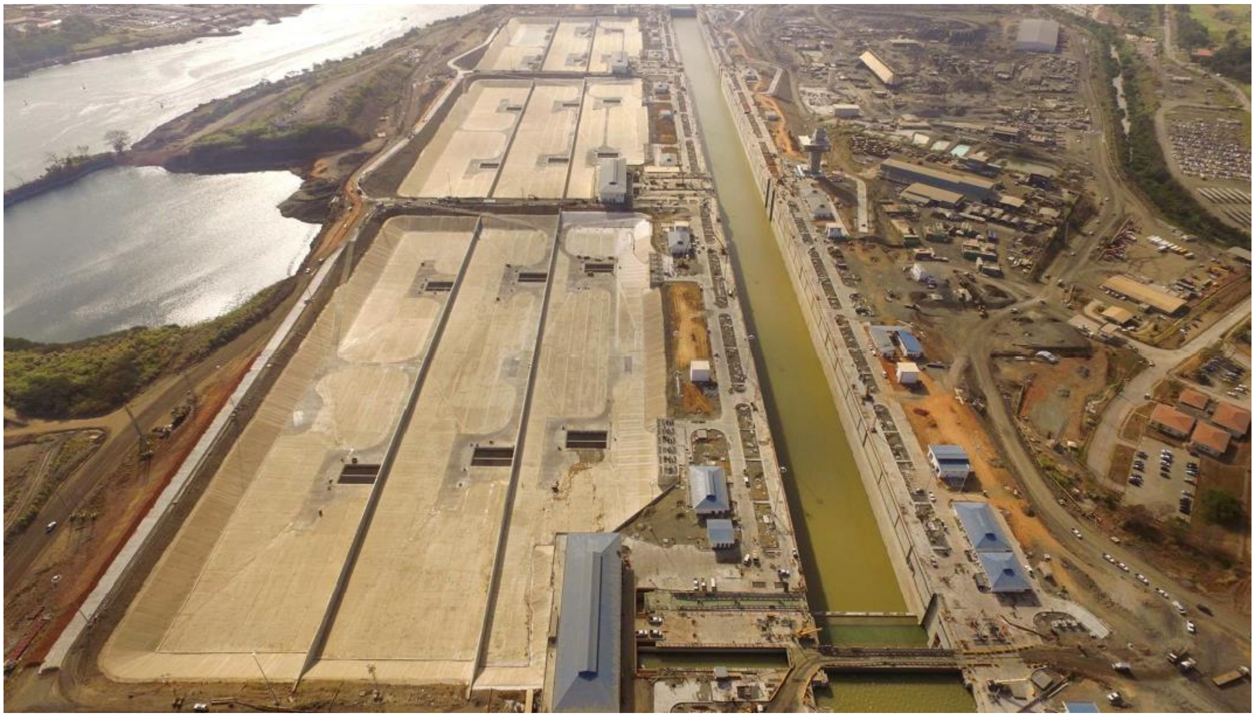

Figure 12: Pacific Complex Water Saving Basins Showing Dividing Walls and Inlets, ACP(2)

3.2 Design Challenges

1) Sustainable design to conserve Lake Gatun.
2) Large differential settlements had to be accommodated in the design of the Water Saving Basins and their components. The Pacific Basins were founded over a 30-meter-deep 1939 excavation which had to be filled in. Total Settlements of approximately 100 mm and Differential Settlements up to 49 mm were anticipated and were included in the design.
3) Varying geotechnical support conditions from bedrock to soil fill.

3.3 Design Innovations

1) ***Design for Sustainability.*** The Water Saving Basins save approximately 60 percent of the water from each lockage to maintain water availability and quality of Lake Gatun and waterways. Lake Gatun is the largest surface fresh water resource used by the people of Panama, so it is important to protect and conserve this vital natural resource. The water saving basins save approximately 2380 billion liters of water per year. The Panama TSL Water Saving Bains are one of the world's largest, if not the largest, water saving basin facilities ever designed or built in the world. Conduit valves allow a lock chamber to empty by gravity delivering water to fill each one of the three basins. Following lockage, gravity will move the water emptying each basin back to fill the lock chamber. The highly efficient hydraulic design uses 7 percent less water than the existing lock chambers, which are 40 percent smaller in volume. The sustainable design avoided the need for additional reservoir storage construction, preventing deforestation and displacement of residents and wildlife.
2) ***Design for Varying Geotechnical Conditions.*** In order to accommodate the large differential settlements while preventing leakage, the basin lining selected was a reinforced PVC Flexible Geomembrane Lining. Approximately 600,000 square meters of the Geomembrane Lining was used for the Panama TSL Water Saving Basins. The filling and emptying Conduits were designed with water-tight joint hinges that allowed settlement and articulation of the Conduit segments while keeping the Conduit joints water-tight and the reinforced concrete stresses within acceptable ranges. The reinforced concrete Dividing Walls separating the basins are approximately 10 meters tall, and maintain their stem alignment with embedded dowels while maintaining water-tight isolation joints throughout the differential settlement and filling and emptying operations of the

Basins. A total of 4.8 kilometers of Dividing Walls were designed for the varying foundation conditions.

3) ***Design for Conduit Geometry Optimization.*** During the early tender design phase, The Tetra Tech design team raised the Conduits and Valve Structures connected to the Water Saving Basins to a higher elevation and developed a transition structure (Trifurcation) to merge the water from three Conduits into one Conduit, which would pass water below the lock floor. This geometric change cut over 18 meters in height off each Valve Structure (on 12 Valve Structures), cut over 18 meters of Conduit depth (on 36 Conduit intakes), and cut 18 meters of excavation depth for these reductions, resulting in many millions of dollars in costs savings for design, excavation, and materials.

3.4 Water Saving Basin Design Criteria and Components

The design life required for all the Water Saving Basin structures is 100 years. The Basins and locks filling and emptying system are required to operate by gravity flow. The components of the Water Saving Basins consist of several filling and emptying structures as show in the Figure below and as further described in the following paragraphs.

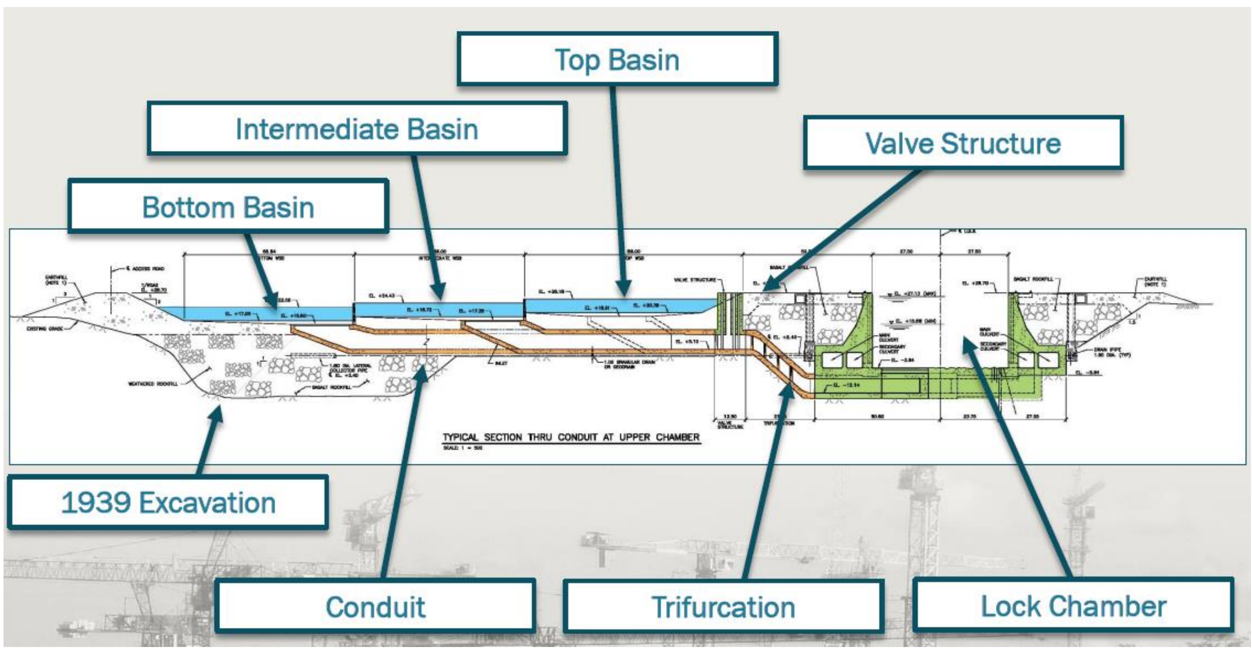

Figure 13: Section of Water Saving Basin Components

3.5 Trifurcations

The function of each Trifurcation structure is to connect the Conduit Valve Structure to the culverts at the back face of the lock walls, converging (necking down) six Conduits into one larger culvert. There are two Trifurcations per chamber. Tetra Tech designed 12 Trifurcations for the entirety of the project.

A hydraulic model determined the inside geometry of the Trifurcations. The thickness of the slabs and walls were determined through structural analysis. A SAFE model was created with static loads and an ABAQUS model was created to run a Time-History seismic analysis. The boundary conditions in the models were dependent upon the supporting material. The ABAQUS models demonstrated the non-linear yielding nature of the backfill and supporting material. The bottom slabs rest on the native bedrock which varies depending on their location. The Pacific site had Basalt and La Boca bedrock. The Atlantic site had Gatun (sedimentary) bedrock.

A finite element thermal analysis of the Trifurcation 2.5 meter thick bottom slab was performed to evaluate the potential for thermal strain and concrete cracking during the placement of concrete. The 2D finite element model was executed in ABAQUS and consisted of 11 lifts. Concrete mix design, maximum placement lift thickness, and placement temperaturas were tightly controlled to prevent termal cracking of the concrete during the concrete placement and curing period.

3.6 Valve Structures

The function of the Valve Structures is to control the flow of the water to and from the Basins. The Valve Structure connects the Trifurcation on the lock side to the Water Saving Basin Conduits on the basin side. There are two Valve Structures per chamber. Tetra Tech designed 12 Valve Structures for the entirety of the project. Each Valve Structure extends from the invert elevation of the conduits to the deck surface staging area at each chamber. It provides room for the conduit valve, two maintenance bulkhead slots and an access shaft for each of the six valves per Valve Structure.

The Valve Structure was designed using both a Seismic Time-History ABAQUS Shell FE model and a simplified structural analysis using the Seismic Coefficient Method. In ABAQUS, the supporting rock foundations for the Valve Structures were modeled using a Mohr-Coulomb non-linear material. The foundations further away from the Valve Structures were modeled with a linear elastic material because the non-linear behavior in the rock was expected to be limited to areas directly adjacent to the Valve Structure. The water was modeled as acoustic elements with the properties of water. The seismic analysis used seven historical earthquakes that were deconvolved and applied to the base of the structure. Site-specific scale factors were applied to the time histories prior to inputting them into the ABAQUS model. The damping parameters for the rock foundation, structure and back fill were defined by the Rayleigh damping equations.

A finite element thermal analysis of the Valve Structures was performed to evaluate the potential for thermal strain and concrete cracking during the placement of concrete. The finite element thermal model was executed in ABAQUS and consisted of five lifts for the conduit extension portion of the Valve Structure.

Figure 14: Valve structure under construction water saving basin side, April 2014, Tetra Tech[(1)]

3.7 Water Saving Basin Conduits

The function of the Conduits is to pass water to and from the basin inlet structures and the Valve Structure. Each Water Saving Basin per chamber is divided into three levels: top basin, intermediate basin and bottom basin. Two conduits fill up each basin level. Tetra Tech designed 36 Conduits for both the Pacific and Atlantic Complexes of the project. The conduit internal dimensions are typically 9.0 m wide by 6.0 m tall. The invert elevation of the conduit bottom slab remains the same per chamber. The inside geometry of the conduits was determined by a hydraulic analysis. The thickness of the slabs and outer walls was determined by a combination of a static structural analysis and a 2D Time-History analysis in ABAQUS. The outer wall and slab thicknesses were typically 1.75 meters. Figure 15 shows a section of three of the Conduits as modeled in REVIT, a facility information model. The shortest Conduit connects to the top basin inlet structure, the middle conduit connects to the intermediate basin inlet structure and the longest Conduit connects to the bottom basin inlet structure. The six Conduit openings at the interface with the Valve Structure necks down into three openings, one for each Conduit.

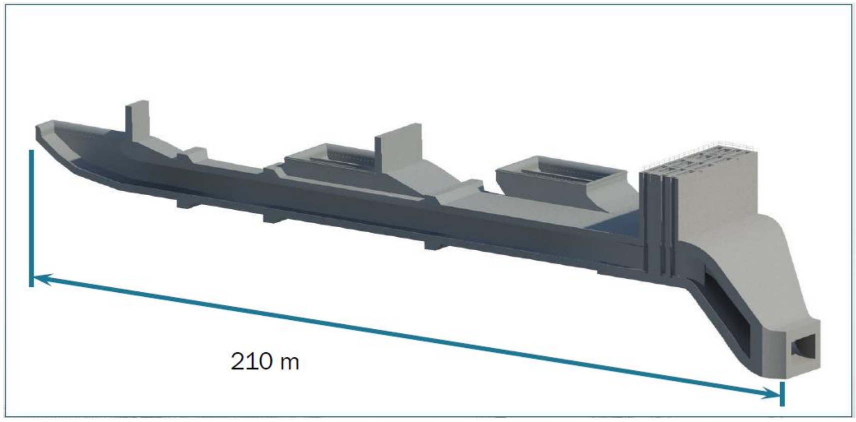

Figure 15: Conduit section, Valve Structure and Trifurcation REVIT model, Tetra Tech(1)

In 1939 a large portion under the upper and middle Pacific chambers had been excavated in anticipation of building a third set of locks. The work was abandoned after a few years of excavation and the partially excavated approach canal is now covered by the new Water Saving Basins. The Water Saving Basin Conduits at the upper and middle Pacific chambers span across the 1939 excavation and were constructed with flexible construction joints to withstand relative settlement of the supporting basalt rock fill. The supporting bedrock on either side of the rock fill is much stiffer, thus the joints were placed near changes in stiffness such as the edge of the rock support to allow a small rotation in the conduits. Corrosion resistant dowels combined with double waterstops provided the flexibility and water-tightness required for the Conduit construction joints.

The final seismic design of the conduits was performed using the site specific Time-Histories finite element analysis in ABAQUS. A static finite element analysis of the walls were compared with the ABAQUS results and the concrete sections were designed. The allowable bearing of the soil was checked for usual, unusual and extreme conditions. The maximum end pressures induced at the bottom corner of the conduit was compared with the allowable bearing pressure. Displacements and deflections were also checked in the finite element Time-History ABAQUS model. Maximum deflections occurred at the top corner of the conduits and was in the magnitude of 7.4mm and 11mm for the Pacific single cell conduit for level one and level two respective earthquake accelerations.

As mentioned previously the 1939 excavation at the Pacific site had to be filled with basalt rockfill. The excavation at the Atlantic site was carved out in nearly the precise shape of the conduits so that they could be built against the existing sedimentary Gatun rock.

3.8 Basin Inlet Structure

The inlet structure in each basin is located at the low point of the basin floor near the center of the basin. The inlet structure serves as a connection point between the Conduit and the basin. There are a total of six inlet structures per chamber, with varying geometry at the top, intermediate and bottom basins. Figure 15 above shows the inlet structures connected to the conduits in the REVIT model. Tetra Tech designed a total of 36 inlet structures for the entirety of the project. A trashrack covers each inlet structure entrance to prevent large debris and crocodiles from entering the conduits and lock chambers. The span between the inlet structure exterior walls is approximately 13 meters with an interior wall at midpoint.

The height of the inlet structure walls varies between 10.5 meters and 14.6 meters and the horizontal span opening is approximately 22.5 meters in length. The tops edges of the walls are rounded to help the water flow smoothly between the conduits and the basins. The minimum operation level of the water in the basin is 1.5 meters above the top of the inlet structure, however during maintenance the water can be fully drained.

3.9 Basin Floors and Basin Linings

The basin floors are typically a geodrain sub-base lined with a flexible PVC membrane lining. Limited areas of 250 mm thick concrete pavement for vehicle access slope down from outside the water saving basins down to the inlet structures. The sub-base directly beneath the basin liners is always drained by an underdrainage system so that it will not be subject to uplift groundwater forces. The floors of the basin are sloped toward the central inlet structures. A hydraulic model determined the sloped geometry of the water saving basins. Maximum and minimum elevation points were determined early on in the design process.

The exterior basin berms are sloped with engineered fill and/or cut into rock. The flexible membrane lining system rests on the floors and side slopes of each water saving basin to act as an essentially impermeable barrier, limiting leakage from the water saving basins. The lining system connects and is anchored with concrete ballasts to base of the slopes, the side slopes of the basins and throughout the entirety of the basin floor. The lining is anchored down with stainless steel bolts, clamp bars and epoxy at the concrete dividing walls, conduit inlets, manholes and cleanouts. Vehicular access ramps, traffic lanes and aprons around the conduit inlets are located on the lining for maintenance and inspection use. The lining was designed to withstand temperature, oxidation, ultraviolet light exposure, ozone exposure, hydrolysis, chemical attack, biological attack and various stress states due to cyclic loading, differential settlement, and vehicular traffic. The leakage from any basin per chamber is limited to approximately 10mm of water loss in any 24 hour period within any individual basin after accounting for precipitation and evaporation loss. During a level one earthquake, the lining must sustain no permanent damage, however if there is some damage it must be able to be repaired without interrupting the operations of the lock. In the event of a level two earthquake the lining may experience some damage that may require localized repair or limited replacement.

3.10 Basin Dividing Walls

The basin Dividing Walls are oriented parallel to the lock axis and separate the top, intermediate, and bottom basins from each other. They were designed as reinforced concrete cantilever walls, with a height up to 10.5 meters and a thickness up to 1.3 meters. The thickness of the stem tapers so that the top of the wall is about 0.6 meters thick and the base is up to 1.3 meters thick. The top of the wall had to be wide enough for a maintenance person to safely walk along the length. The top elevations of these walls were established by the hydraulic analysis results. The maximum operation level of the water in the basin is 0.8 meters below the top of the wall. The wall footings rest on basalt rockfill or bedrock which varies in the Atlantic and the Pacific Water Saving Basins. The walls were designed for normal operating water levels, as well as maintenance conditions and seismic events. Each dividing wall extends the entire length of each basin of approximately 450 meters long. Figure 16 shows one of the Dividing Walls under construction in March 2015.

Figure 16: Dividing Wall construction, March 2015, Tetra Tech(1)

Vertical expansion joints at the Pacific walls stems were placed approximately every 30 meters along the length of the Dividing Wall. While the footing remained continuous along the length, the expansion joints were placed for temperature, shrinkage and anticipated settlement over the 1939 excavation. The continuous footing offered some shear resistance, but to keep the top of the wall aligned, five smooth corrosion-resistant dowels were placed in the top of the stem at each expansion joint. The vertical expansion joints were covered with a flexible membrane liner to keep the water from passing through. The Atlantic Dividing Walls only required expansion joints at either side of the inlet structures because the supporting sedimentary bedrock was consistent along the entire length of the wall. However, contraction joints were placed in the wall stems approximately every 30 meters to allow for temperature and shrinkage. The sliding stability, overturning stability, floatation and bearing were checked for each wall for each load case.

4.0 REFERENCES

1. Tetra Tech (2016), Calculation and Design Basis Memorandum documents, Bellevue, WA, USA, www.tetratech.com
2. Panama Canal Authority, ACP (2016) http://www.acp.gob.pa/