DESIGN AND CONSTRUCTION OF A CELLULAR COFFERDAM FOR THE PACIFIC ACCESS CHANNEL

by

Antonio A. Abrego Maloff[1], Gonzalo De León[2], Maximiliano De Puy[3]

ABSTRACT

As part of the Panama Canal Long Range Master Plan, the Panama Canal Authority (ACP) initiated in 1997 the expansion of the capacity of the waterway. The Expansion Project included the construction of additional locks and navigation channels to allow the transit of Post-Panamax vessels. This project required a new navigation channel to connect the Gaillard Cut with the new Pacific Locks. This channel was designated as the Pacific Access Channel (PAC). The channel is approximately 7.8 km long and 218 m wide, with a water elevation at 26.82 m PLD (Precise Level Datum for the Panama Canal) and is separated from the Miraflores Lake (elevation 16.45 m PLD) by a new dam. ACP divided the Pacific Access Channel works into four separate construction packages.

One of those packages, named PAC-4 included the construction of one of the two embankment dams required to separate the new access channel from the Miraflores Lake, that have a difference elevation of almost ten meters. These dams were designated as Borinquen 1E and 2E; the last one was included in the Locks Contract while the Borinquen 1E dam was built during the PAC-4 contract. The design of the PAC-4 required that the contractor excavate to ground elevations below the Miraflores Lake level. Therefore, it was required the construction of a cellular cofferdam and an embankment cofferdam in order to provide the adequate conditions for the required excavation works. This cofferdam was designed by the ACP design team and the review process was performed by URS Corp., who was the consultant in charge of the design of the embankment 1E. The paper describes the design criteria assumed; the possible additional uses of this structures; and the construction process itself.

1. INTRODUCTION

In 1997, the Panama Canal Authority (ACP) initiated the expansion project of the waterway in order to increase its capacity. The Expansion Project included the construction of additional locks and navigation channels to allow the transit of Post-Panamax vessels. This project required a new navigation channel to connect the Gaillard Cut with the new Pacific Locks. This channel was designated as the Pacific Access Channel (PAC). The new channel is approximately 7.8 km long and 218 m wide, with a water elevation at 26.82 m PLD (Precise Level Datum for the Panama Canal) and is separated from the Miraflores Lake (elevation 16.45 m PLD) by a new dam.

ACP divided the Pacific Access Channel works into four separate construction packages. The first three contracts, denominated PAC-1, PAC-2 and PAC-3, were executed from 2007 until 2009, and the design contemplated to excavate from the natural ground elevation down to elevation 30.00 PLD. The last excavation contract, named PAC-4, was designed to remove all the remaining material down to elevation 9.14m PLD (bottom of navigation channel); this contract also included the construction of one of the two embankment dams required to separate the new access channel from the Miraflores Lake, that have a difference elevation of almost ten meters. These dams were designated as Borinquen 1E and 2E; the last one was included in the Locks Contract while the Borinquen 1E dam was built during the PAC-4 contract. The design of the PAC-4 required that the contractor excavate to ground elevations below the Miraflores Lake level; therefore, it was required the construction of a cellular cofferdam and an embankment cofferdam in order to provide the adequate conditions for the required excavation works. This cofferdam was designed by the ACP design team and the review process was performed by URS Corp., who was the consultant in charge of the design of the embankment 1E.

[1] Panama Canal Authority, Panama, AAbrego@pancanal.com
[2] Panama Canal Authority, Panama, GDeLeon@pancanal.com
[3] Geotechnical Consultant, Panama, Maxdepuy@gmail.com

Most cofferdams can be built either on rock or on sand and gravel; and the design varies depending on the soil condition. For the structure required in this project, given the geology of the area, the cofferdam was designed for the case where is resting on rock. It is important to note that the cofferdam was built as a separate structure from dam 1E, which will not depend on the cofferdam for its own stability.

The paper describes the design criteria assumed for the cofferdam, including its alignment, geological conditions under the structure and a summary of the design procedure used. In addition, the paper will address the construction process from the owner's point of view, including some changes proposed by the design team and also by the contractor due to problems at foundation levels.

2. PROJECT DESCRIPTION

The new Borinquen Dam 1E was part of the works for the Pacific Access Channel (phase 4), and begins at the Pedro Miguel Locks in the north and extends south across Miraflores Lake to Fabiana Hill, a rocky hill outcropping north of the Miraflores Locks (see Figure 1). This dam and the other three included in the Locks Contract, separates the Miraflores Lake (at elevation 16.75m PLD approximately) from the new navigation channel (at elevation 25.90m PLD approximately). Since the construction of this dam involved excavation of the foundation down to elevation 0.00m PLD, it was necessary that the contractor built a cofferdam in order to allow excavation in the dry.

The cofferdam was required where Dam 1E construction would extend into or near Miraflores Lake, between approximately Stations 1+000 and 2+700 (see Figure 1). The north end of the cofferdam was tied into the west approach wall of the Pedro Miguel locks. The south end of the cofferdam was tied into existing ground. The 1.8 km long cofferdam has a maximum height of about 17.00m.

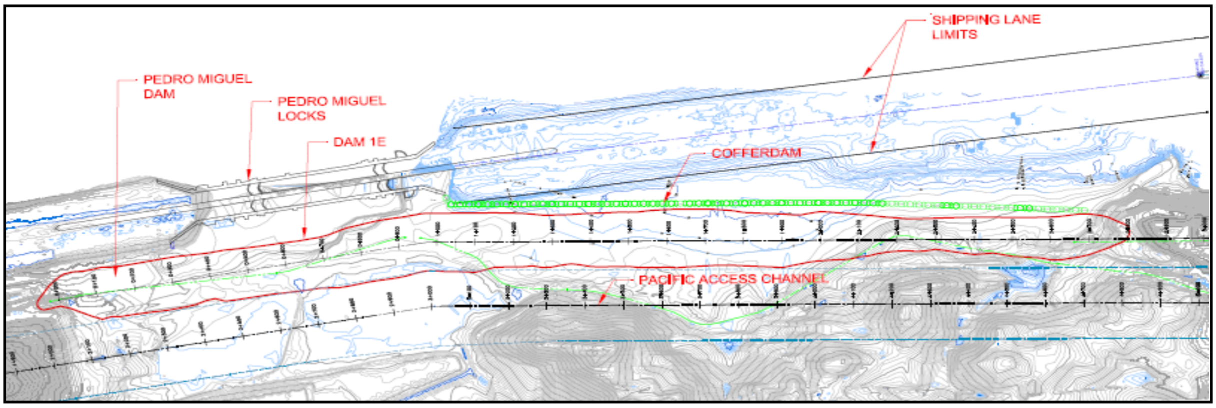

Figure 1: Dam 1E and cofferdam location

3. DESIGN CRITERIA

Most cofferdams can be built either on rock, where little or no overburden exists, or on sand and gravel; and the design varies depending on the soil condition. For the structure required in this project, given the geology of the site, the cofferdam was designed for the case where is resting on rock. It is important to note that the cofferdam was built as a separate structure from Dam 1E, which does not depend on the cofferdam for its own stability.

In general, the design of a cofferdam must satisfy the following criteria:

a. The structure must be able to withstand all the various loads applied to it;
b. The quantity of water entering the cofferdam must be controllable by pumping;
c. At every stage of construction the formation level must be stable and not subject to uncontrolled heave, boiling or piping;
d. Deflection of the cofferdam walls and bracing must not affect the permanent structure or any existing structure adjacent to the cofferdam;
e. Overall stability must be shown to exist against out of balance earth pressures due to sloping ground or potential slip failure planes;

3.1 Alignment

In order to accomplish the performance described above, the design of the cofferdam needs to consider several design criteria. One of them is the final alignment. This alignment was first outlined based on the design of Dam 1E (URS, 2008) and the final surface of the outcropping rock (weathered and sound).

The cofferdam was originally located approximately 30.00 m east of the outboard toe of Dam 1E and west of the shipping channel. The cofferdam was located in such a way not to interfere with the construction of Dam 1E and the shipping channel in Miraflores Lake. Due to the preliminary design of Dam 1E, the original cofferdam alignment required a PI between stations 3P+494.08 and 3P+778.71. This was done in order to maintain the cofferdam structure close to the toe of the dam and to reduce construction costs.

The final alignment of the cofferdam was defined based on various aspects as describe below:

a. The cofferdam must be located to avoid interference with construction of Dam 1E and the shipping channel in Miraflores Lake;
b. Minimum clearance between the cofferdam and the outboard toe of Dam 1E shall be 10 m;
c. The cofferdam must satisfy the operation requirements. In the middle section of the cofferdam, the structure will be used as a tie-up station (see fig. 2). The Cofferdam shall be straight in the area of the tie-up station. (approx. 500 m);
d. The northern closure will be the south end of Pedro Miguel Locks;
e. In the southern closure the cofferdam will extend until top of weathered rock reach elevation 18.00m PLD.

The final alignment in figure 2 shows the north and south ends and also where the structure changes it size and type. It changes size because the structure may be used as a tie-up station only in its middle portion. The rest of the structure will not be treated as such and therefore a more economical design has been implemented. On the other hand, as shown in Figure 2, the cellular cofferdam extends from the north end up to a point south of station 4P+249 where the geology allows a change in the type of cofferdam to be used. At this point, the rock is above elevation 18.00m PLD (see Section 4) and therefore a cellular cofferdam in uneconomical and an embankment cofferdam with sheet piles cutoff may be used as describe in Section 3.2.

In addition, since the cofferdam structure is required to keep the excavation area for the dam foundation completely dry, it was decide that in the north (Pedro Miguel Locks) the cofferdam would be extended and tied into an existing slope to reduce the risk of flooding. In order to ensure the water-tightness of the system, it was also decided to design a grout curtain or a similar structure extending from the cofferdam to the locks wall as shown in Figure 3. In the south end, the cofferdam will tie into the existing ground at Point #5 (see Figure 2).

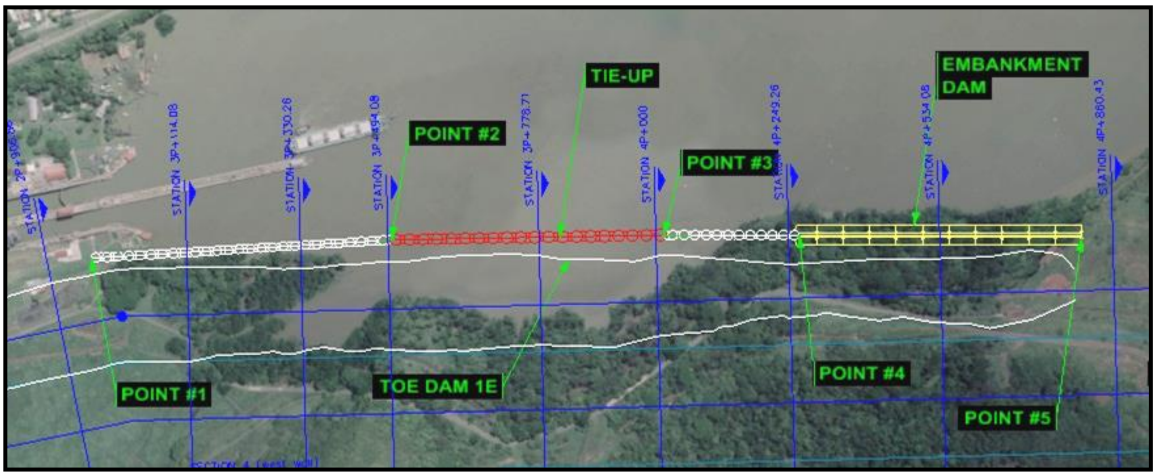

Figure 2: Cofferdam final alignment

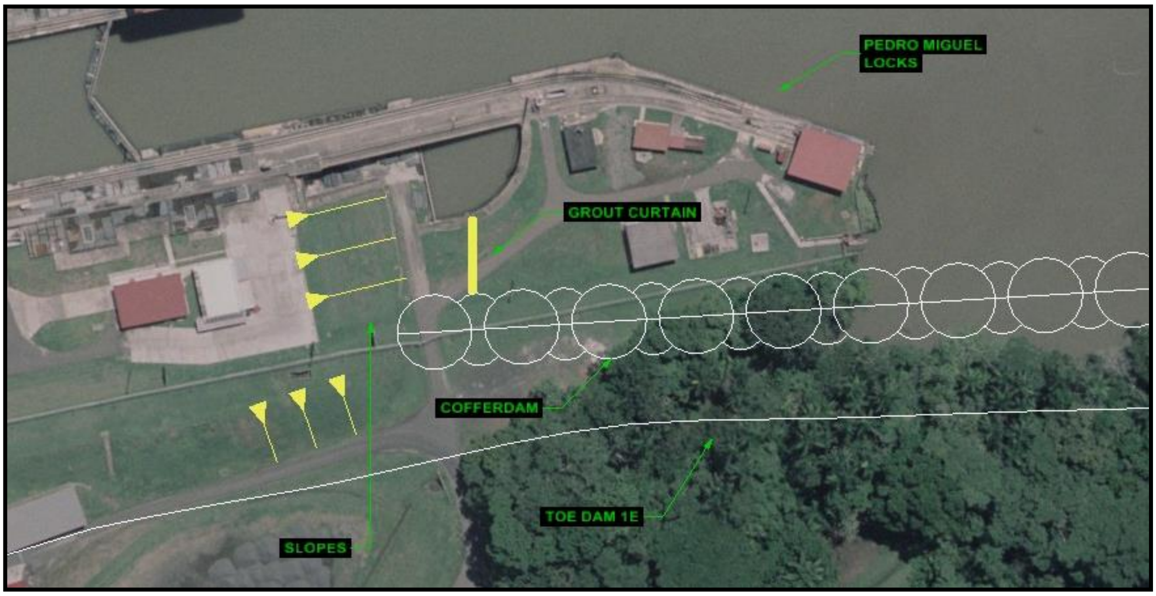

Figure 3: North tie-in of cofferdam (at Pedro Miguel Locks)

3.2 Cofferdam Alternatives

The original cofferdam concept required for the construction of Dam 1E consisted of a single-diameter cellular cofferdam to be built throughout the entire 1.8 km long alignment. However, analysis of the possible future uses of the structure and the geology of the area, lead the Design Team to consider two different alternatives along the alignment in order to allow tying up operations and also to reduce the cost of the structure:

a. Embankment cofferdam with sheet pile cutoff

This first alternative was considered for those areas where the rock elevation was above 18.00m PLD and the driving of the flat sheet piles (required for cellular cofferdams) was determined to be unnecessary (as well as impractical). Evaluation of the geology of the area (see Section 4) suggested that south of Station 4P+429 the rock elevation was above the required 18.00m PLD; therefore, and to avoid any seepage through the rock, it was decided that an embankment cofferdam with a sheet pile cutoff (see Figure 4) was enough to guarantee impervious condition during the excavation works required.

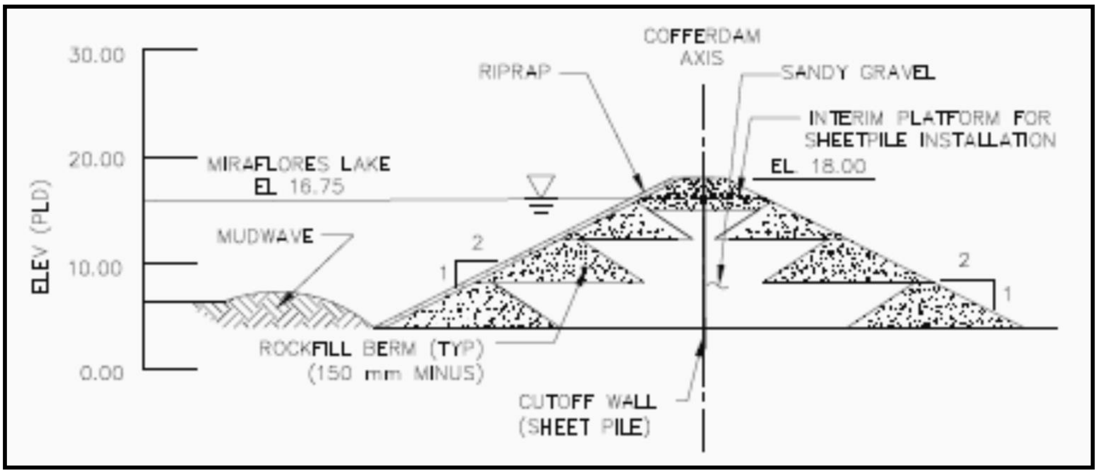

Figure 4: Typical Section of the Embankment cofferdam with sheet pile cutoff

This type of cofferdam was constructed in the south part of the cofferdam alignment, specifically from Station 4P+429 up to the south end where the structure must tie into the existing ground at Fabiana Hill.

b. Cellular sheet pile cofferdam

Those areas where the weathered rock elevation is below 18.00m PLD, the cellular sheet pile cofferdam alternative was used. This alternative was chosen over the Float-in-place concrete caisson cofferdam because is more economical and also because ACP has already design this type of structures in the past.

The Panama Canal operates another tie-up station in the Gaillard Cut which was design under the same concept. Having ACP the experience of designing a similar structure and considering that this existing cofferdam is presently being used as a tie-up station, the Design Team proposed that the remaining cofferdam was design as a cellular cofferdam instead of float-in-place concrete caisson. In addition the Team proposed that a portion of such a cofferdam might be used as a tie-up station.

The new cofferdam structure is located in the Miraflores Lake, a few hundred meters west of the existing navigation channel and north of the existing Miraflores Tie-up station. The existing tie-up station is composed of several buoys anchored to the lake floor and therefore there is no land access to this facility. The idea of having this structure design as a future tie-up station was extensively accepted because it would provide land facilities to the actual operation crew and an additional anchoring capacity at Miraflores, which may increase the vessel transit through these locks.

Therefore, it was decided that the middle portion of the structure, between station 3P+494 and 4P+000 (approximately 500m), were designed for the appropriate tie-up station loads and additional earth fill to be placed on top of the cells to reach the required freeboard.

3.3 General Design Criteria

In addition to the alignment the cellular sheet pile cofferdam design needed to consider several other general design criteria as shown in Table 1.

Feature/Issue	Criteria	Remarks
1. Cofferdam Crest Elevation	18.00 m.	The area between Dam 1E and the cofferdam will be backfilled to the cofferdam crest level.
2. Miraflores Lake Operating Levels	• Maximum: elevation 16.75 m • Minimum: elevation 16.45 m	
3. Operations	• For tie-up station, foundation level must allow for Panamax vessel draft. ○ Dredge line on Miraflores Lake side of cofferdam will be elevation 2.90 m. • Construction of the cofferdam shall minimize impact on shipping operation	A deck will be constructed on top of the cofferdam to allow for ship tie-up station.

4. Foundation	• Residual soils or rock:
	  ○ Sheet pile refusal on weathered rock, or
	  ○ At least 2 m below SPT N-value of 50.
	• Seepage cutoff must be adequate to prevent piping
	  ○ Foundation to be non-erodible, or hydraulic gradient to be less than 0.3 in erodible materials.
		• Materials must have sufficient strength for static and seismic stability, eliminate liquefaction potential, and minimize settlement.
		• All fill, alluvium, Pacific Muck and unsuitable soils will be removed from the cofferdam foundation in place under wet conditions.

5. Backfill	• Between Dam 1E and Cofferdam: Zone 3 rockfill.	Sheet pile cell fill is ACP specification.

Feature/Issue	Criteria	Remarks
	• Within sheet pile cells: Free-draining rockfill (minus 150 mm)	
6. Stability	• Must have adequate static and seismic stability. • Check overall cofferdam stability at tie-up station (draft to allow for Panamax vessel)	Ref: *Design of Sheet Pile Cellular Structures, EM 1110-2-2503*. U.S. Army Corps of Engineers, September 1989.
7. Dead Loads (Static)	• Loads from rockfill on west side of cofferdam. • Hydrostatic loads.	
8. Live Loads	• Ship berthing forces from a Panamax ship. • Operational deck loads. • Construction loads.	Loads to be determined during preliminary design of the cofferdam.
9. Seismic Loads	• Backfill loading. • Inertial loads of the cofferdam itself. • Hydrodynamic loads from Miraflores Lake.	
10. Drainage of Seepage	Provide for drainage of seepage from Dam 1E through cofferdam to Miraflores Lake.	Consider Miraflores Lake operating levels to set drainage invert elevation through cofferdam.
11. Corrosion Resistance	Must resist corrosion in brackish water conditions.	Consider wave splash and fluctuating Miraflores Lake levels.

TABLE 1: Design criteria for the cofferdam

4. FOUNDATION CONDITIONS

The proposed cofferdam footprint is underlain by varying depths of fill. Beneath the fill, the cofferdam is primarily underlain by the La Boca formation, and Pedro Miguel agglomerate. The designers of the Dam 1E concluded that the fill and residual soil materials are not suitable for dam foundation, so they will be stripped before construction of the embankment. However, for the case of the cofferdam this material is soft enough so the sheet piles will be driven through them until reaching hard rock (top of weathered rock), or the predefined elevation.

Once the cells are constructed and ready to be backfilled, this material will be removed. Only in the portion where the Tie-Up station will be constructed, this soft material will be dredged prior to the construction to avoid future interference of the dredge operations with the final structure.

4.1 Geological cross sections

Along the alignment of the cofferdam ACP drilled several boreholes to better characterize the geology right underneath the cofferdam structure. These new boreholes complemented many others already drilled in the area. As a result, ACP geologist developed a longitudinal cross section showing the location of the fill, overburden, top of weathered rock and top of sound rock. In addition, several cross sections were developed along the cofferdam alignment in order to provide the designer with more precise details on how the cofferdam would be founded on the soil. In this paper only two cross section are shown.

Using this longitudinal cross section, the Design Team was able to determine the location where the cofferdam would be switched from a cellular structure into an embankment cofferdam. It was decided that from station 4P+249.26 extending south across low lands to Fabiana Hill the rock elevation was high enough so a cellular cofferdam is no longer required. Therefore, in this section an embankment cofferdam would be used instead.

All the cross section developed by ACP were evaluated and analyzed by the Design Team. It was concluded that section 3P+330.26 (figure 5) was the most critical one within the non tie-up portion of the cofferdam, while section 3P+778.71 (figure 6) was the most critical within the portion assigned to the tie-up station.

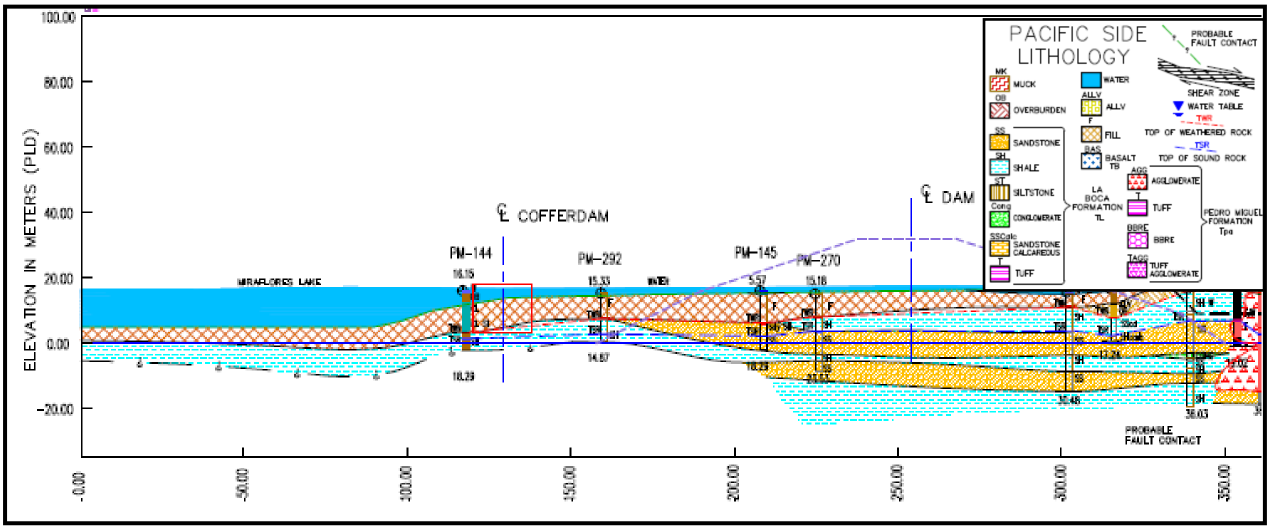

Figure 5: Cross section at station 3P+330.26 (non tie-up station)

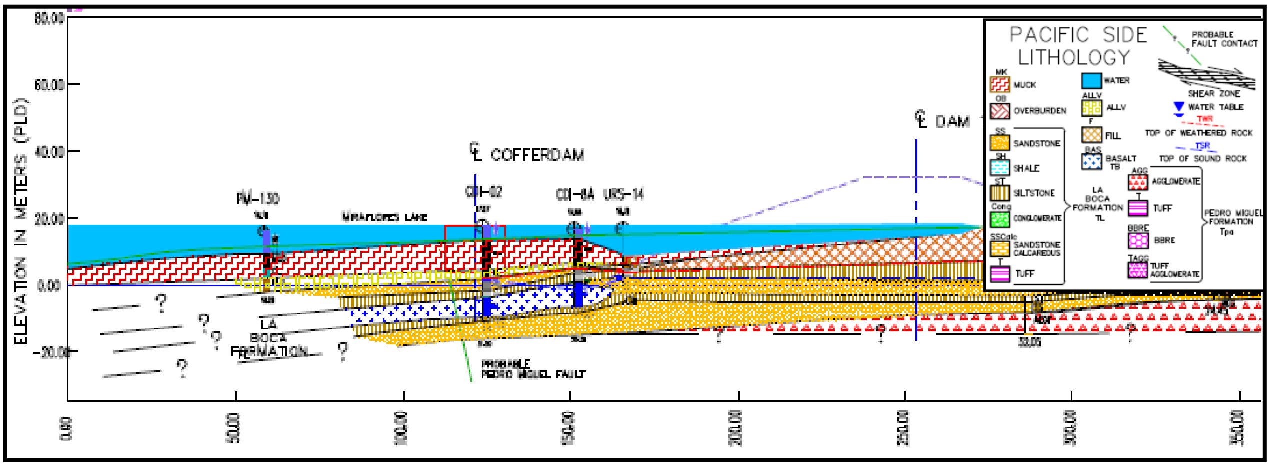

Figure 6: Cross section at station 3P+778.71 (tie-up station)

5. COFFERDAM DESIGN

The design of the cofferdam was done following the standard procedure developed by the Tennessee Valley Authority (TVA), also known as the Terzaghi's Method. Although there are other methods available, the TVA procedure is simple and one of the most commonly used around the world.

Basically, a cellular cofferdam is a gravity retaining structure formed from the series of interconnected straight web steel sheet pile cells filled with soil, usually sand, or sand and gravel. The interconnection provides water-tightness and self-stability against the lateral pressures of water and earth. The circular one (as the one being design), consists of individual large diameter circles connected together by arcs of smaller diameter. These arcs generally intercept the circles at a point making an angle of 30, 45 or even 90 degrees with the longitudinal axis of the cofferdam. The prime feature of the circular type cofferdam is that each cell is self-supporting and independent of the next.

The design of a cellular cofferdam proceeds much the same as that of an anchored wall. Before the design can be initiated, the necessary controlling dimensions must be set. In this case, the height of the structure is a known value so the next step was to choose an approximate diameter, D, and the equivalent width, B.

One additional parameter that needed to be defined was the location of the water table inside the cell, or the so called saturation line. This basically refers to the degree of saturation within the cell fill and its location is influenced by a number of factors including the condition of the pile interlocks, the permeability of the cell fill, whether a berm is used, and the number and position of weep holes on the inside row of piling. In the present design, a horizontal line, at an elevation so chosen as to represent the average expected condition of saturation, was assumed in order to simplify computations.

Once all these parameters were defined, the next step was to verify the stability of the cells. Since the cells will be founded on rock, several types of failures were needed to be checked: a) Sliding on the base; b) Overturning; c) Shear failure on centerline of cell; d) Horizontal shear; e) Excessive interlock tension; and f) Loss of internal stability

5.1 Design of the non Tie-Up and tie-up sections

The non tie-up section of the cofferdam applies to those areas that will not be used for berthing vessels. These areas have been identified as those close to the Pedro Miguel Locks (between point 1 and 2 in figure 2), and to the southern portion of the alignment inside the Miraflores Lake (between point 3 and 4 in figure 2). A review of the geological cross-section in these areas revealed that the most critical is the one identified as 3P+114.08; therefore, this section has been adopted as typical and the design will be performed based on its features.

On the other hand, the tie-up section of the cofferdam applies to that area that will be used for berthing vessels. This area has been identified in the middle section of the cofferdam alignment, between point 2 and 3 in figure 2. A review of the geological cross-section in this area revealed that the most critical is the one identified as 3P+778.71; therefore, this section has been adopted as typical and the design will be performed based on its features.

The design of the cofferdam has considered two different loading conditions that control the cell geometry and stability of the structure:

a. *Construction condition*: Condition that applies during the time the cofferdam is being constructed;
b. *Long term condition*: Condition that applies once the construction of the cofferdam has finished and all the permanent loads are applied.

For the evaluation in either condition, the material properties were maintained exactly the same. Table 2 shows a summary of these properties, while Table 3 summarizes all the assumptions made for the cofferdam design.

Material	**Condition**	**Friction Angle (ϕ)**	**Cohesion (kN/m^2)**	**K_a**	**K_o**	**γ_{sat} (kN/m^3)**	**γ' (kN/m^3)**
Fill/Muck	All	17^o	0.00	0.548	-----	16.60	6.79
Cell fill	All	34^o	0.00	0.254	-----	20.60	10.80
Backfill	All	42^o	0.00	0.183	0.331	22.00	12.19
La Boca Found. (seismic)	All	32^o	0.00	-----	-----	-----	-----
La Boca Found. (static)	Non Tie-up constr.	27^o	0.00	-----	-----	-----	-----
La Boca Found. (static)	Non Tie-up long term	29^o	0.00	-----	-----	-----	-----
La Boca Found. (static)	Tie-up constr.	26^o	0.00	-----	-----	-----	-----
La Boca Found. (static)	Tie-up long term	27^o	0.00	-----	-----	-----	-----

TABLE 2: Material properties

Parameter	Section Analyzed			
	Non Tie-Up		Tie-Up	
	Construction	Long Term	Construction	Long Term
Berm outboard	Not included	Not included	Not included	Not included
Passive resistance included	Not included	Not included	Not included	Not included
Pulling force from vessel	---	---	---	563.8kN
Loading from compaction surcharge	---	Not included	---	Not included
Aditional Surcharge	---	Not included	---	Not included
Height of cell	14.70m	14.70m	16.70m	16.70m
Additional height (for loading evaluation)	---	---	---	3.65m
Miraflores Lake Level (outboard side)	16.75m PLD	16.45m PLD	16.75m PLD	16.45m PLD
Saturarion level (inside cell - midpoint)	10.05m PLD	16.45m PLD	9.05m PLD	16.45m PLD
Unit Weight γ_{wet} assumed	γ_{sat}	γ_{sat}	γ_{sat}	γ_{sat}
Friction coefficient δ (outboard side)	0^o	---	0^o	---
Friction coefficient δ (inside cell)	22^o	22^o	22^o	22^o
Backfill condition	---	At Rest	---	At Rest
Water inside fill/muck (seismic design)	Restrained	---	Restrained	---
Water inside backfill (seismic design)	---	Free	---	Free

TABLE 3: Assumptions made for the design of the cofferdam

5.1.1 Construction stage condition

During this stage the cofferdam is analyzed assuming that the inboard side of the cofferdam has been fully excavated and the water on the Miraflores Lake exerts the maximum pressure on the structure. In addition, the overburden or fill material in the lake contributes to increase the active pressure. Figure 9 shows the typical cross-section assumed for this portion of the structure.

It is important to note from Table 3, that there is a difference in the height of the cell between the non tie-up and tie-up section. This difference results from the geological exploration, which indicated that at station 3P+778.71 (tie-up section), the top of the weathered rock was located at elevation 1.30m PLD and not at elevation 3.30m PLD.

As for the saturation line inside cell, it was assumed with a slope 1:1 from lake level to TWR; therefore the water elevation at the mid-point inside the cell are different for both cases. This slope is based on the filling material as shown in the USACE EM-1110-2-2503 manual (USACE, 1989);

In figure 7 it is shown the typical cross section used for design. Since the structure is located in a seismically active area, the stability of the cofferdam was verified for the static condition and also the seismic condition. Figures 8 and 9 show the load distribution on the cell for both conditions.

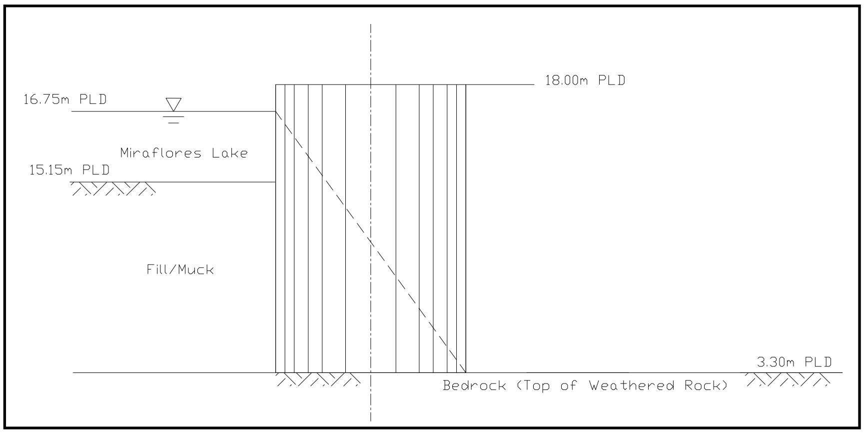

Figure 7: Typical cross-section for the non Tie-Up section during the construction stage

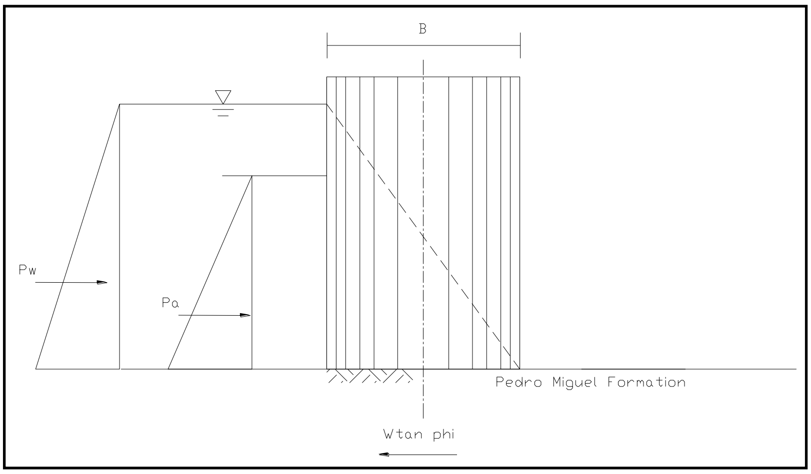

Figure 8: Horizontal pressure components acting on the cofferdam during the construction stage (static loading)

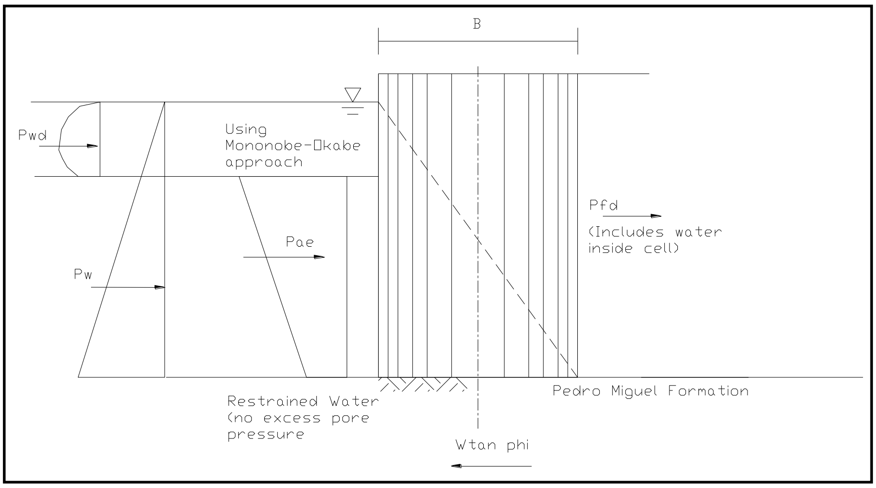

Figure 9: Horizontal pressure components acting on the cofferdam during the construction stage (seismic loading)

5.1.2. Long term condition

During this stage the cofferdam is analyzed with all permanent loads applied to the structure. The area between Dam 1E and the cofferdam has been backfilled to the cofferdam crest level. It is also assumed that on the inboard side of the cofferdam all the material (fill/muck) has been removed. This is assumed since this condition might happen in the future as part of normal operating procedures Figure 10 shows the typical cross-section assumed for this portion of the structure.

One significant difference at this stage between the non tie-up section and the tie-up section is the pulling force from vessels. For the non tie-up section, no pulling forces exerted from vessels is considered. For the tie-up section, the pulling force exerted on the future dock has been calculated assuming a Panamax ship with a DWT = 100,000 tons. The formulation of the load components has been derived from the literature (Tsinker, 1997). The resulting value is 563.8 kN (total load normal to the dock), which has been estimated assuming an horizontal angle (α) = 30^o, a vertical angle (β) = 20^o, a pulling force (QB) = 1,000 kN and a 20% increase due to non-uniformities.

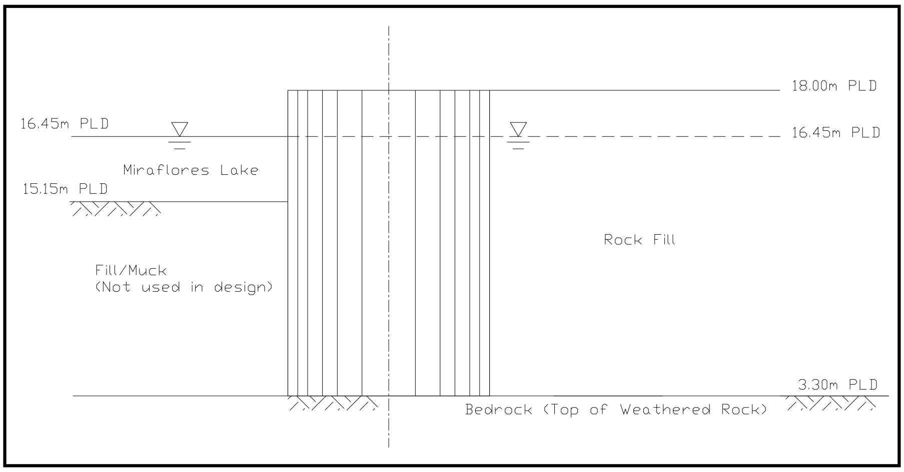

Figure 10: Typical cross-section for the non Tie-Up section for the long term condition

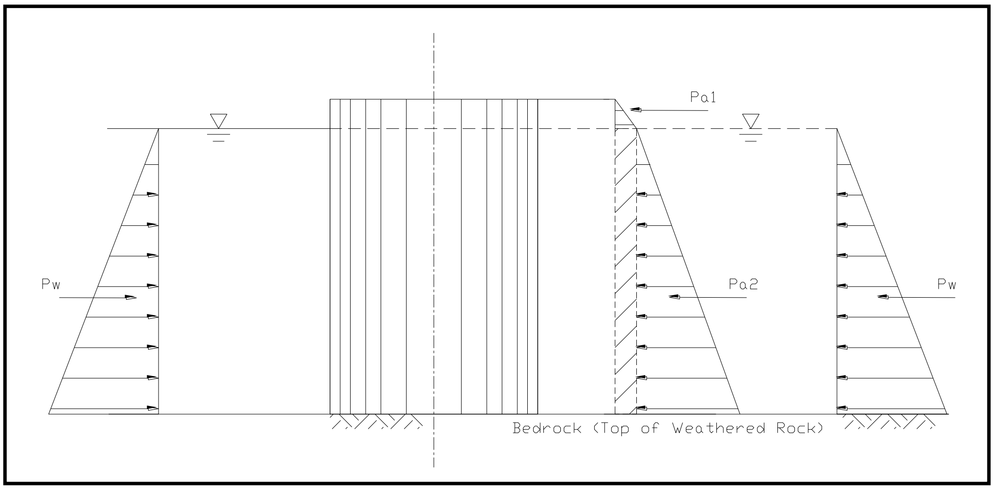

Figure 11: Horizontal pressure components acting on the cofferdam for the long term condition (static loading)

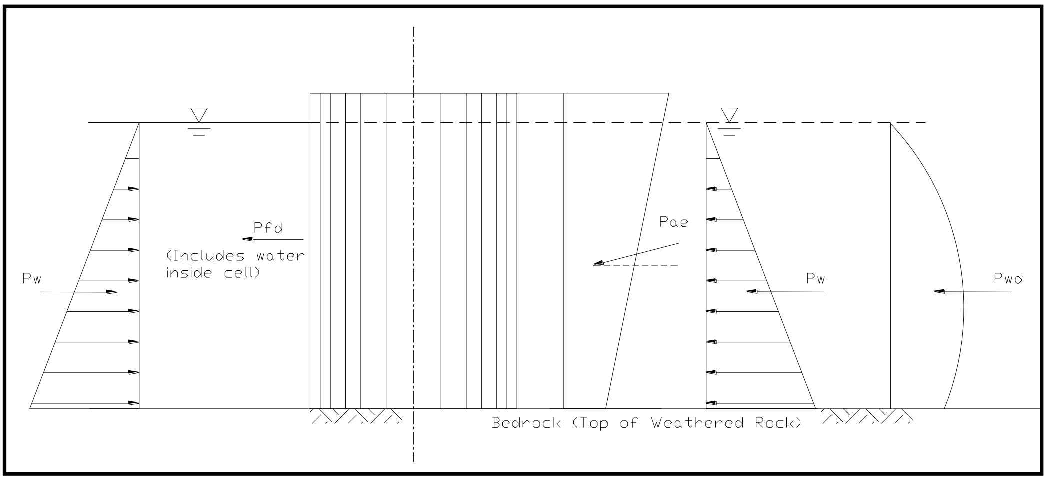

Figure 12: Horizontal pressure components acting on the cofferdam for the long term condition (seismic loading)

5.3 Results

Once the loading conditions have been established and the material properties chosen, the Design Team proceeded to evaluate and design the structure according to the criteria and procedures given above. It should be noted that following recommendations of USACE (1989), the stability of the structure has been evaluated for three different factors of safety, depending on the loading condition:

a) F.S. = 1.50 for the long term stage (permanent condition)
b) F.S. = 1.25 for construction stage (temporary condition)
c) F.S. = 1.10 @ 1.30 for seismic evaluation (see Tables 4 and 5)

All the stability analysis performed to the different cross-section and loading conditions were implemented in an Excel worksheet. In order to verify the accuracy of such worksheets, the stability analyses were also performed by hand and the results cross-checked with the computer results. It is important to note that such calculations were performed assuming an equivalent B=0.875D, where D=1.25H. The only fixed dimension was the height of the cell, which was a known value before the analysis was performed.

A summary table with all the final stability analysis results is shown in Tables 4 and 5. In addition to the resulting factor of safety, it is also presented the minimum strength required by the sheet piles and junction elements to comply with stresses imposed on the structure.

Failure Mode	Min. F.S. Required (1)	Construction Stage	Min. F.S. Required 1	Long Term
Sliding Stability	*1.25*	1.72	*1.50*	1.77
Overturning	*Middle 1/3*	Middle 1/3	*Middle 1/3*	Middle 1/3
Vertical Shear	*1.25*	2.17	*1.50*	3.61
Horizontal Shear	*1.25*	1.83	*1.50*	2.41
Bursting (piles of cell)	*< tu/r*	Req. Min. 3000 kN/m	*< tu/r*	Req. Min. 3000 kN/m
Bursting (Tees of cell)	*< tu/r*	Req. Min. 3000 kN/m	*< tu/r*	Req. Min. 5000 kN/m
Seismic Sliding Stability	*1.30*	1.42	*1.30*	1.09 (3)
Seismic Overturning Stability	*Middle 1/2*	Middle 1/2	*Middle 1/2*	Middle 1/2
Seismic Vertical Shear	*1.10*	1.20	*1.10*	0.77 (3)
Seismic Horizontal Shear	*1.10*	1.01 (2)	*1.10*	1.25

Notes:
[1] According to TVA, USS Steel Sheetpiling Design Manual & others
[2] Analysis does not include effect of berm located in the inboard face, which will increase the FS above 1.1
[3] Values below the minimum required factor of safety

TABLE 4: Summary of results for the non tie-up section

Failure Mode	Min. F.S. Required (1)	Construction Stage	Min. F.S. Required 1	Long Term
Static Sliding Stability	*1.25*	1.82	*1.50*	1.77
Overturning	*Middle 1/3*	Middle 1/3	*Middle 1/3*	Middle 1/3
Vertical Shear	*1.25*	2.37	*1.50*	2.89
Slipping between piling & fill	*1.25*	1.62	*1.50*	1.77
Horizontal Shear	*1.25*	2.07	*1.50*	2.16
Bursting (piles of cell)	*< tu/r*	Req. Min. 3000 kN/m	*< tu/r*	Req. Min. 3500 kN/m
Bursting (Tees of cell)	*< tu/r*	Req. Min. 3000 kN/m	*< tu/r*	Req. Min. 5500 kN/m
Seismic Sliding Stability	*1.30*	1.52	*1.30*	0.98 (2)
Seismic Overturning Stability	*Middle 1/2*	Middle 1/2	*Within Base*	Within Base
Seismic Vertical Shear	*1.10*	1.28	*1.10*	0.63 (2)
Seismic Horizontal Shear	*1.10*	1.12	*1.10*	0.85 (2)

Notes:
[1] According to TVA, USS Steel Sheetpiling Design Manual & others
[2] Values below the minimum required factor of safety

TABLE 5: Summary of results for the tie-up section

It is important to note that in Table 4 and 5 the footnotes 2 and 3 indicates that these values are below the minimum factor of safety required. This is due that the seismic analysis was done assuming that the response of the cell fill is governed by its drained strength. However, during an earthquake, its short-term strength is greater than the drained strength, suggesting that the real dynamic factor of safety is likely to be greater than the static one.

Once the equivalent width (B or w_e) was determined, the geometry of the cells was then defined. This was done with the help of tables or with computer programs. Several solutions are possible for the circular cells with a given equivalent width. In order to determine the final layout of the cells, the Design Team based the actual geometry of the cell on the ARCELOR's geometrical parameter tables found in the literature (Arcelor, 2005). The final equivalent width was chosen to be the closest to the one determined by empirical relationships; therefore, assuming a fixed B matching the ones presented in the table, the rest of the parameters were easily defined. The final values can be found in Table 6 and 7, and the corresponding definition in Figure 13.

Parameter	Value
Height (H)	14.70 m
Diameter (D)	17.93 (≈ 1.22H)
Equivalent Width (B)	16.34 (≈ 0.911D)
Radius of main cell (r_m)	8.97 m
Radius of connecting arcs (r_a)	5.98 m
System Length (x)	20.86 m
Junction Tipe	90°
Angle α	48.21°
N° of sheetpiles in cell (include 4 junction piles)	112 pcs
N° of sheetpiles in arc	19 pcs
Min. Interlock strenght requirement cell piles	3,000 kN/m
Min. Interlock strenght requirement T piles	5,000 kN/m

TABLE 6: Final layout for the non tie-up section

Parameter	Value
Height (H)	16.70 m
Diameter (D)	21.77 (≈ 1.30H)
Equivalent Width (B)	19.84 (≈ 0.911D)
Radius of main cell (r_m)	10.89 m
Radius of connecting arcs (r_a)	7.86 m
System Length (x)	26.29 m
Junction Tipe	90°
Angle α	47.65°
N° of sheetpiles in cell (include 4 junction piles)	136 pcs
N° of sheetpiles in arc	25 pcs
Min. Interlock strenght requirement cell piles	3,500 kN/m
Min. Interlock strenght requirement T piles	5,500 kN/m

TABLE 7: Final layout for the tie-up section

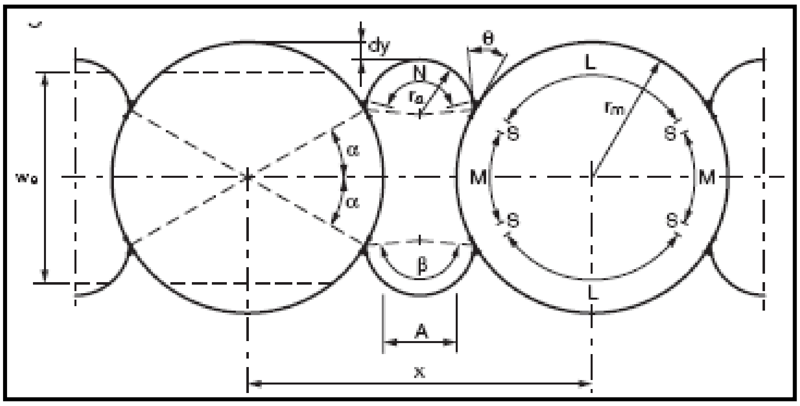

Figure 13: Geometrical values for circular cells

6. CONSTRUCTION OF THE COFFERDAM

The final layout of the cofferdam resulted in a structure consisting of 58 cells, 57 pairs of connecting arcs and 462 linear meters of cutoff wall (figure 14 and 15). The cellular cofferdam was designed for two different uses, therefore, the size of the cells varies according to the use. In the portion to be used as a tie-up station (Section “C” in figure 14) the final cell diameter is 21.64 m and the length is 499.51 m, which result in 20 cells. On the other hand, in the portion of the cofferdam not used as a tie-up station (Section “B” and “D”), the final cell diameter is 17.83 m and the total length is 667.52 m, which results in 33 cells. The remaining 4 cells, are located in the north end of the cofferdam, specifically built in an existing slope to reduce the risk of flooding (Section “A”). This 4 cells were also constructed with a cell diameter of 21.64 m.

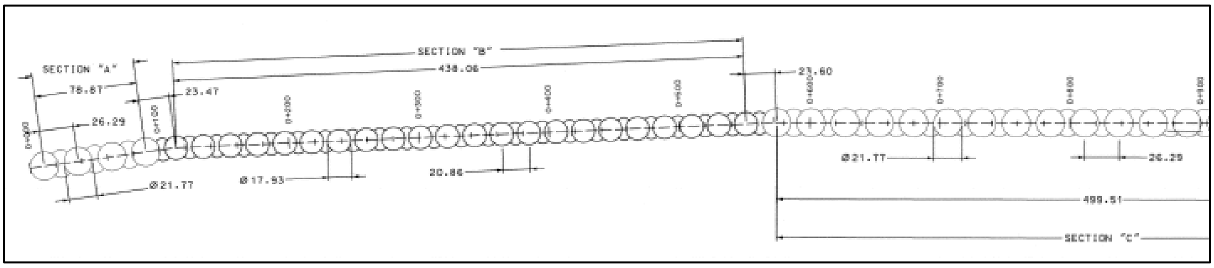

Figure 14: Final layout of the cofferdam (north end)

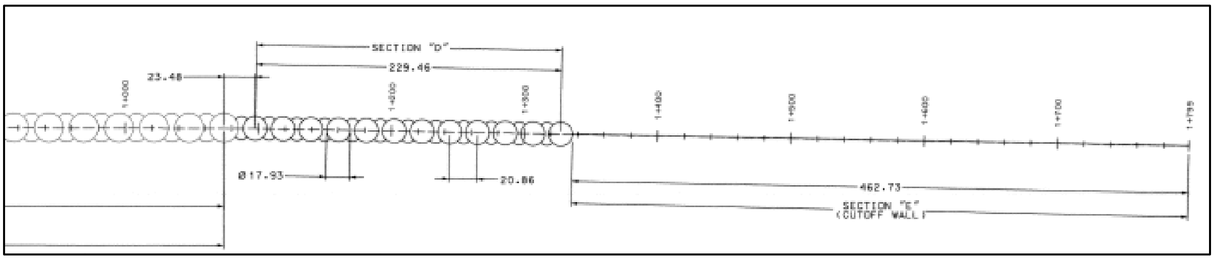

Figure 15: Final layout of the cofferdam (south end)

The south end of the cofferdam, was built using an embankment cofferdam with Z piles driven to refusal through the middle portion of the embankment (Section “E”). This portion of the structure has a length of 462.73 m and the average length of the piles is approximately 15.00 m.

6.1 Construction Methodology

The PAC-4 contract was awarded to the consortium ICA-FCC-MECO, and they subcontracted the consortium GOETTLE – ICONSA for the construction of the cellular cofferdam and the embankment cofferdam. Before this contract was awarded to the fore mentioned contractor, ACP had already purchased all the sheet piles required for the project, including the straight web piles for the cellular cofferdam and the Z piles for the embankment cutoff wall.

Works began in 2010 and the construction methodology proposed by the consortium was in accordance with international practice. First, all unsuitable material within the cell footprint was removed by the dredging work. Then, a bathymetric in-survey of the cofferdam installation area was perform to compare to the post-dredge bathymetric survey of the dredging subcontractor. This work was performed to anticipate the expected volume of cell fill.

6.1.1 Templates

In order to construct the cofferdam cell, it was necessary first to design and fabricate the templates which are used to drive the piles around. The templates consisted of two steel rings spaced vertically 3.0 m, which can be anchored into the correct position by means of ten spuds (tubular piles).

For this project, four different type of templates were required: a) for the 21.77 m cells, b) for the 17.93 m cells, c) for the connecting arcs between the same size cells, and d) for a special condition connecting arcs between two cells of different diameters. The templates were fabricated in New Orleans, USA, and then transported on barges to the project site in Panama; in figure 16 it is shown the templates.

Figure 16: Templates being transported

6.1.2 Marine cellular cofferdam

There were four cofferdam construction fronts: three fronts working from barges for cells in the dredged area, and one front working on land for cells outside the dredged area (cells 1-4 and 54-58). Each of the three marine teams were equipped with two floating templates. These templates were designed in such a way that the top ring of the template floated some 0.46 m above the water surface. The use by each marine crew of two templates with associated spud piles enabled the crews to work on a new cell while the previous cell were backfilled.

Once the template was floated to the cell footprint the spud piles were installed through spud wells to fix template on location; then, four sheet key (H-piles welded to a sheet pile) were driven into weathered rock to completely fix the template (see figure 17).

For the installation of the sheets between key sheets, the contractor first attached some of them to the template using spot welding directly to the template, then, in a sequential order, the sheet piles were vibrated into top of weathered rock ("near" refusal or to design tip elevation, whichever occurred first). This procedure was done in multiple passes, each pass was approximately 1.50 m in vertical length, and in a circular pattern. At the end, to reach the refusal criteria or to reach the design tip elevation, an impact hammer was used, centering the hammer over the sheet interlocks at the hammer's high impact energy and in conformance with the approved pile refusal criteria (see figure 18).

After reaching the refusal criteria or design tip elevation, the sheet piles were cut as required to install the cell fill using a conveyor. Initially, the cut off sheet were handled by the piling crane and later, after some cells were completely filled, a track excavator or other suitable equipment handled the cut off sheets.

Figure 17: Template in position

Figure 18: Driving piles in a cell by vibration

For the placement of the backfill, three temporary earthen access ramps were constructed prior to start of the works: one at cell 5, one at cell 14, and one at cell 46. At the end of each ramp there was a temporary access trestle that served as bridge between the fill and the cofferdam. These trestles were designed such that trucks and equipment could access the cell without damaging the sheet piles at that juncture.

The filling procedure of a given cell was only possible when such cell was closed and driven to refusal. Only the first cell of each working front was filled directly from the trestle using a 30.00 m long conveyor. The rest of the cells were filled using the same conveyor placed on top of the adjacent finished cells, with the discharge end over the center of the cell. The conveyor was fed by either a front-end loader or an excavator. Figure 19 shows this operation. Important to note that material from the conveyor was deposited in the cell at the center at all times, so the backfill was distributed evenly to avoid differential stresses or excessive development of interlock tensions.

Figure 19: Filling operation of the cells

At each cell, a drainage system was install according to the drawing plans and specifications. The drainage system consists of two 0.30 m diameter PVC pipes with filter fabric wrap placed in the inboard and outboard face of the cofferdam with the pipe discharge invert located at 17.00 m, PLD.

6.1.3 Connecting arcs

These structures are required to tie in independent cells all together, and are joined to the main cells by special T-piles. The placement of sheet piles in these arcs required the use of special templates also fabricated in New Orleans and transported in barges to the project site.

Prior to beginning the fill of the main cell, it is required to install two starter sheets at each of four SWC Weldon locations. The SWC is the point at which the arcs connect to the circular cell. The template was not used to set these starter sheets, which are not driven to refusal prior to initiating adjacent cell fill placement. The connecting arc template was only placed after completion of cell fill placement for both adjacent cells and was used to set the remaining sheets within the connecting arcs.

Only after the two adjacent cells were completely backfilled and the connecting arcs were driven to refusal, the connecting arcs were backfilled either directly from a front-end loader or excavator, or using chute, in both cases discharging at the center of the segment, so that the material was distribute evenly within the segment.

6.1.4 Landside cofferdams

For the construction of the landside cofferdam, the land crew utilized the same template as the marine crews. They began the construction in the north side of the alignment, in other words, the 4 land side 21.64 m diameter cells and then the same land crew built the remaining five 17.83 meter diameter land-side cofferdams located in the south end.

First, a trench was excavated the approximate width of the cofferdams to a depth sufficient to verify that no unsuitable material was encountered within the cells. This depth was approximately 4 m. The cofferdam template was then built in place, as shown in figure 20.

Figure 20: Landside cofferdam in the south end

Figure 21: Landside cofferdam in the north end after cutting off sheet piles

Unlike the cell built in water, no backfill was required for these cells and only driving the sheet piles to the refusal criteria or the design tip elevation was required. Once the crew finished using the template, it was removed in pieces and then re-used on the next and adjacent land-side cell, and so on (see figure 21).

6.1.5 Embankment cutoff wall

The work consisted in the construction of 464.45 meters (6.975 m2) of a sheet pile “Z-wall” using 15.00m long PZC-26 sheets, and the construction of embankment along sheet pile Z-wall.

The sheet piling was installed in a sequential manner from north (junction with the southernmost cellular cofferdam cell) to south. A template was installed at the starting location, and the surveyors were given the correct alignment and centerline of the wall.

The template consisted of a 24” steel I-beam whaler that lied down on the ground along the alignment of the face of the wall. This whaler was supported off the ground by 14” beams, 1.80m long, which lay on the ground, perpendicular to the whaler and all welded together as a unit. This frame, in turn, was fixed to the ground by driving steel H-piles, approximately 6.00 m long, vibrated into the ground adjacent to intersections of the whaler and support beams.

Figure 22: Driving sheet piles for the embankment cutoff wall

The sheets piles were driven in pairs. After 10 pairs are set on the template, the crane and hammer vibrated the sheets in a staggered pattern, so that the tip of every sheet was not more than 1.20 to 1.50m below that of any adjacent pile. After all the piles were driven to near refusal with the vibratory hammer, they were later impacted with the impact hammer to the required design elevation or to refusal, whichever occurred first.

6.1.6 Berm

After the construction of the cofferdam was finalized during the first quarter of 2011, the contractor was instructed to build a berm in the inboard side of the cofferdam before dewatering. This berm was originally included in the drawings and specifications of the contract, but not was a requirement for the design of the structure. In figure 23 it can be seen the structure finalized and the excavation works as they progressed later that same year. Note the berm already in place in the inboard side of the cofferdam. This is condition known as “construction stage”, where all the loads are acting from the Miraflores lake side.

Figure 24 shows the same cofferdam but working in the so called “long term” condition, where all the load is acting from the Borinquen dam side. The Panama Canal Authority has been monitoring the behavior of the cofferdam since it was built. For this, several prism are located in both sides of the cells and they are being monitored monthly using robotic instruments that read the real position of the prisms, triggering an alert if a displacement threshold previously set, is exceeded.

Figure 23: View of cofferdam from the south side (construction stage)

Figure 24: View of cofferdam from the south side (long term condition)

References

Arcelor (2205) “Piling Handbook,” Eight Edition

Tsinker, G. (1997) “Handbook of port and harbor engineering: geotechnical and structural aspects,” Published by Chapman & Hall.

URS (2008) Task Order No. 4, Borinquen Dam Design, Task B.1.8: Preliminary Design, Dam 1E, Draft Report, September 2008.

USACE (1989) “Design of Sheet Pile Cellular Structures Cofferdams and Retaining Structures,” EM1110-2-2503, September 1989.