CONSTRUCTION OF BORINQUEN DAM 1E AT THE PANAMA CANAL

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ABSTRACT

The recently completed Panama Canal Expansion project required construction of a 6.7-km-long channel at the Pacific entrance to the Panama Canal. The channel banks are partially formed by four dams, which retain Gatun Lake, the main waterway of the Panama Canal, approximately 27 m above the Pacific Ocean. The dams were designed as rockfill embankments with central impervious cores of residual soil. Design and construction of the dams posed multiple challenges including: 1) variable foundation conditions with occasional unpredictable weak features, 2) use of residual soils as core materials, 3) a wet tropical climate with a 4-month dry season, and 4) geologic faults across the dam foundations. The largest of the dams, Borinquen Dam 1E, is 2.2 km long and 32 m high. Construction of Dam 1E included: 1) erection of a 1.3-km-long cellular sheetpile cofferdam, 2) excavation of a 460-m-long cement-bentonite slurry cutoff wall, 3) injection of a 2.4-km-long grout curtain, and 4) installation of a 100-m-long secant pile wall. This paper provides an overview of the construction of Dam 1E and its key features and describes important design changes required during construction.

1. INTRODUCTION

The recently completed Panama Canal Expansion Project required construction of a 6.7-km-long channel at the Pacific entrance to the Panama Canal, to provide navigation access from the new Post-Panamax locks to the Gaillard Cut section of the Canal. The new channel, known as the Pacific Access Channel (PAC), is partially formed by four dams, referred to as Borinquen Dams 1E, 2E, 1W, and 2W. The dams retain Gatun Lake, the main waterway of the Panama Canal, approximately 11 m above the level of Miraflores Lake and 27 m above the Pacific Ocean. A view of the 5-km-long, northern portion of the new channel and of Dam 1E is shown in Figure 1.

Figure 1 : View of Pacific Access Channel (PAC) and Dam 1E, looking North

The dams were designed as rockfill embankments with central impervious cores of residual soil flanked by filter and drain zones of processed sands and gravels sourced from crushed rock (Mejia et al. 2011). Design and construction of the dams posed multiple challenges, including: 1) variable foundation conditions with occasional unpredictable weak features, 2) use of residual soils derived from rock weathering as core materials, 3) a wet tropical climate with a 4-month-long dry season, and 4) geologic faults across the dam foundations.

Construction of Dam 1E included the following main project elements: 1) erection of a 1.7-km-long, 19-m-high, cellular sheetpile cofferdam, 2) installation of a 30-m-long, 18-m-deep, triple-row, jet-grout cutoff wall, 3) excavation of a 460-m-long, 18-m-deep, cement-bentonite slurry cutoff wall, 4) dewatering and excavation of the dam foundation, 5) treatment and geologic mapping of the foundation, 6) injection of a 2.4-km-long, double-row grout curtain, 7) placement of a 2.4-km-long, 5.3-million-cubic-meter, zoned rockfill embankment, 8) installation of performance monitoring instrumentation, and 9) construction of a 97-m-long, 26-m-deep, secant-pile wall to provide closure against the structure of the Pedro Miguel Locks.

his paper provides an overview of the construction of Dam 1E, including borrow of the embankment materials from the required channel excavations and other sources, and of selected aspects of the above project elements. A more detailed description of the dam construction is presented by Mejia et al. (2017) and URS (2015a).

2. DAM LAYOUT

Borinquen Dam 1E was designed as a central earth core and rockfill embankment. To adequately control potential seepage and provide internal drainage, zones of chimney filters and drains flank the core. In addition, filter and drain blankets extend along the entire foundation of the outboard (downstream) rockfill shell (Mejia et al. 2011).

The dam is 2,420 m long, up to 32 m high, and has a crest 30 m wide. The southern end of the dam abuts against a small hill (Fabiana Hill), and the north end is connected to the northern monolith of the Pedro Miguel Locks by a 97-m-long secant pile cutoff wall, known as the North Tie-In wall. Figure 2 shows the general arrangement of Dam 1E and the PAC. A typical cross section of the dam is shown in Figure 3. The as-built embankment volume is approximately 5,290,000 m^3.

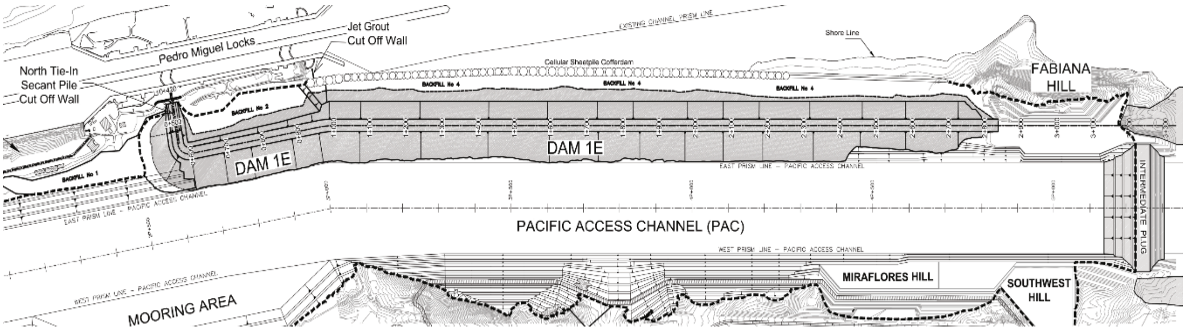

Figure 2 : Layout of Borinquen Dam 1E and Pacific Access Channel (PAC)

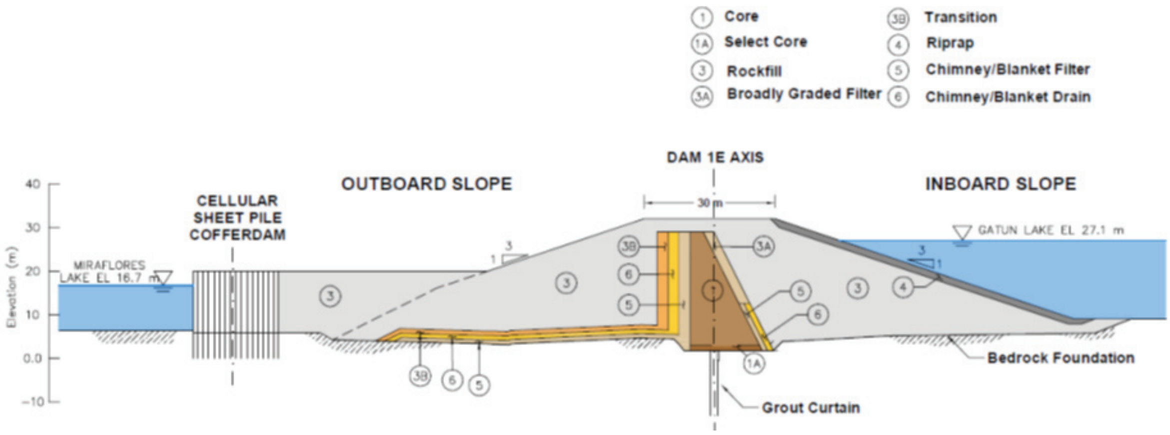

Figure 3 : Typical Cross Section of Borinquen Dam 1E

3. CHANNEL AND DAM FOUNDATION EXCAVATIONS

Following completion of the cellular cofferdam, the dam footprint was dewatered, and the foundation was excavated to remove muck from the former bed of Miraflores Lake (Fig. 1). Between 1 and 2 m of material were left above the foundation level for removal during final excavation. Figure 4 shows the dam foundation after bulk excavation and initial placement of rockfill on the inboard foundation. The volume of channel and foundation excavation totaled 26,989,000 m^3.

Figure 4 : Dam 1E foundation with cellular cofferdam in foreground, PAC excavation in background, and rockfill shell on the inboard dam foundation

4. DAM EMBANKMENT MATERIALS

The dam embankment materials were sourced from the channel excavation and reserve borrow areas of residual soil. Additional volumes of residual soils, rockfill, and sound basalt for aggregate production were obtained from external sources. An additional 3,910,000 m^3 was obtained from sources outside the channel limits, as modifications to the contract.

4.1 Rockfill

Rockfill was sourced from the channel excavations, Miraflores Hill (Fig. 2), and disposal stockpiles from previous PAC excavation contracts. The Basalt and Pedro Miguel Formations encountered in the channel yielded suitable rockfill (Zone 3 in Fig. 3). The basalt was a medium hard to very hard, strong to very strong, massive or columnar jointed rock. The welded tuff and agglomerate facies of the Pedro Miguel Formation were also strong and sufficiently durable for use as rockfill. The fine-grained tuffaceous facies of that formation were found to be weak and subject to slaking and were excluded from use in dam construction.

4.2 Core Material

The core material (Zone 1) was sourced from alluvial and residual soils derived from the weathering of local bedrock. Alluvial sources included deposits encountered in the dam foundation excavation and in a reserve borrow area. The alluvial soils classified as clays, sandy clays, silts, and sandy silts of high plasticity (CH/MH). Residual soils of Pedro Miguel agglomerate and of basalt were sourced from the channel excavations and from a nearby hill borrow site, respectively. The core material often classified as a sandy silt of high plasticity (MH), plotting just below the A line on the Casagrande plasticity chart. Liquid limits of the materials were in the range of 48 to 87, and plasticity indices were in the range of 14 to 49. The fines content (minus 75-micron particles) ranged from 33 to 92% and averaged about 71%.

The core soils were mined with excavators, hauled to stockpiles in 40-ton articulated dump trucks, and spread with bulldozers in 300-mm lifts. The alluvial borrow sources provided relatively homogeneous materials that required little mixing. The residual soil borrow sources were more variable and required mixing and removal of oversize rock. The alluvial soils were typically at the upper limit of the water content specification and workability. The in-situ water content of the residual soils was often drier than the minimum specified compaction water content, requiring addition of water in the stockpile.

4.3 Filter and Drain Material

Filter and drain materials were produced by onsite crushing and screening of basalt excavated from the channel and were also imported from offsite sources. Up to seven crushing plants were mobilized for the on-site production of fine filter (Zone 5), drain (Zone 6), and transition (Zone 3A/B) materials. To meet the embankment demand, Zone 5 and Zone 6 materials were also obtained from offsite quarries.

Considerable difficulties were encountered in configuring and calibrating the onsite plants, and in selecting suitable feed materials to achieve specification compliance. To facilitate production, the Contractor requested a few changes in the specified gradations, which were approved by the Engineer. The main changes consisted of increasing the maximum fines content from 3 to 5% in Zone 5, from 2 to 3% in Zone 6, and from 3 to 5% in Zone 3A.

5. FOUNDATION GEOLOGY

During excavation of the trench for the core foundation, several sections of the outboard cut slope slumped. These failures required regrading the slope locally to inclinations between 2H:1V and 3H:1V to remove the failed masses and stabilize the slope. No failures occurred on the inboard slope. The slumps developed on adversely oriented features such as bedding shears or joints, which caused wedge failure of the trench walls, as shown in Figure 5.

Dam 1E is underlain predominantly by Miocene-age sedimentary and volcanic rocks, including sandstone, siltstone, clay shale, basalt, agglomerate, and tuff (consolidated volcanic ash). The sedimentary rock is low-strength La Boca Formation consisting of interbedded, sandstone, siltstone, clay shale and conglomerate units and underlies the northern three-quarters of the dam foundation. Near the southern end, the La Boca Formation is overlain by Pedro Miguel Formation consisting of interbedded volcaniclastic sandstone with minor siltstone units (URS 2015b).

Figure 5 : Example failure of outboard core trench slope

The foundation geology was mapped to confirm compliance with the specification and to determine treatment requirements. After treatment and final cleaning, the foundation surface was inspected and approved for placement of embankment fill. Geologic mapping confirmed that the minimum rock strength and weathering objectives were achieved beneath the core and shoulders.

The design of Dam 1E included provisions to widen the chimney filter and drain zones within 50 m of fault traces deemed capable of the design fault displacement. Based on the geologic investigations carried out for design, two branches (East and West) of the Pedro Miguel Fault capable of such displacement were mapped to cross the dam foundation during design (Mejia et al. 2011). A prominent fault trace was encountered crossing the dam axis about 200 m north of where the West Branch of the Pedro Miguel Fault was mapped before construction (Schug et al. 2016). Two fault traces associated with the Pedro Miguel fault were encountered near where the East Branch (‘Stewart’ Strand) of the fault had been mapped at the foot of Fabiana Hill. Those faults were judged capable of the design fault displacement and the filter and drain widening provisions of the design were implemented in their vicinity.

Immediately after initial excavation, instability developed over a 100-m length of the PAC cut slope along the inboard toe of Dam 1E, about one-third of the dam length north of Fabiana Hill. The cut slope failed towards the floor of the channel leaving back-scarps extending approximately 5 m back from the top of the cut. Subsequent field investigations showed that the slope was cut in weak, fissured rock with adverse dips associated with faulting. These geologic conditions were deemed to pose a significant instability hazard to the dam. Thus, the foundation excavation design was revised over a 550-m length by removing the sheared rock and excavating a wide bench 2 m below the level of the PAC floor along the inboard dam toe. The excavation was backfilled with compacted rockfill to become part of the inboard dam shoulder.

6. FOUNDATION GROUTING

The grout curtain extended the full length of the dam to a nominal depth of 15 m. The curtain consists of two rows of holes, 1.5 m inboard and 1.5 m outboard, astride of the dam axis. The outboard row holes dip 70 degrees to the north whereas the inboard row holes dip to the south. A 3-m-deep concrete cutoff wall was installed along the inboard row. In addition, multiple rows of “stitch grout” holes were installed in shear zones across the dam axis (URS 2015c).

Super primary holes were drilled at 24-m spacings along the curtain, to a 25-m depth below the foundation. Primary holes, spaced at 6 m on centers, and secondary holes (at the mid points) were mandatory, resulting in a maximum 3-m spacing between holes along both rows. The design provided for “split-spaced” holes to be triggered if the grout take in any of the mandatory holes exceeded a predetermined quantity. A closure criterion of 25 kg of cement injected per meter of grout stage was established initially for grout take.

7. FOUNDATION TREATMENT AND CLEANING

Treatment of the core foundation was applied in two stages. To prepare fault and shear zones for stitch grouting, initial rock shaping and dental concrete were completed before grout curtain installation. The final treatment, performed after the grout curtain was completed, consisted of minor dental concrete, shotcrete, and slush grouting. Cleaning of the foundation surface to remove loose and encrusted materials was achieved with compressed air and water. The fine grained, non-welded clay shale units within the La Boca formation deteriorated rapidly after exposure. This required trimming and cleaning of the foundation immediately before placement of embankment materials.

Following initial cleaning and mapping of the core foundation, the concrete cutoff wall was installed along the inboard row of the grout curtain. The wall is 3 m deep and 0.6 m wide and was extended to the full length of the foundation. The wall trench was excavated with a trenching machine with a 0.65-m-wide blade. Compressed air, water, and suction were used to remove rock fragments, water, and debris at the base of the trench prior to backfilling with concrete (Fig. 6).

The core foundation contained numerous shear zones and faults ranging from less-than-50-mm-wide shear zones to sheared rock and gouge zones up to 1.5 m wide. The gouge materials typically consisted of low plasticity silt and sandy soil and included up to 5-cm-thick seams of high plasticity clays and silts. The shear zones and faults were treated by over-excavating them to a depth three times their width, up to a maximum depth of 1.5 m. The features were cleaned with compressed air or water and backfilled with concrete.

Figure 6 : Cleaning of concrete cutoff wall trench

Beds of highly fractured, weak rock were encountered in the La Boca Formation, generally in the finer grained units (clay shales). The beds were often sheared through their full thickness with polished surfaces. These weak and sheared beds were covered with a minimum thickness of 0.2 m of backfill concrete. Shotcrete was used on inclined fractured and weak rock surfaces, where regular concrete could not be vibrated in place.

8. EMBANKMENT CONSTRUCTION

8.1 Embankment Core

Because placement of the core could not be started until the nearby grout curtain was completed, the inboard shell was placed up to a height of 17 m in advance of the core, over a significant length of the dam. Once grouting was completed, core placement commenced at the northern end of the dam and progressed south (Fig. 7). The chimney filter zones were generally placed ahead of the adjacent core and rockfill zones. After placing a lift of filter material, a lift of core was placed and the adjacent rockfill zones were placed and compacted.

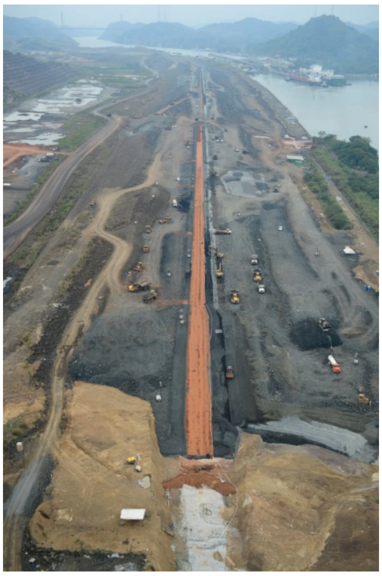

Figure 7 : Dam embankment construction, looking north from Fabiana Hill

A zone of select core material (Zone 1A) up to 1 m thick was placed at the base of the core to provide a good contact with the foundation. Alluvial soil without rock fragments was typically used for this zone. A 300-mm-thick lift was placed first to avoid damaging the foundation. A 200-mm loose thickness was used for subsequent lifts within Zone 1A. The materials were compacted with 8 passes of a weighted front-end loader, as shown in Figure 8. In confined areas inaccessible to the loader, gasoline-powered hand tampers were used with a reduced lift thickness of 150 mm.

Figure 8 : Compaction of core Zone 1A with front-end loader

The main body of the core (Zone 1) was placed in 225-mm-thick loose lifts. The earthfill was dumped and leveled to the required lift thickness by dozers. Each lift was disked and compacted with a self-propelled, quadruple-drum, pad-foot compactor or with a rubber-tired, single-drum, pad-foot roller. The earthfill became effectively unworkable at a water content of approximately +8% and required removal of lifts exhibiting deep rutting (close to or deeper than the thickness of the lift). At water contents higher than +8%, the minimum specified undrained shear strength of 75 kPa could not be achieved reliably. A large portion of Zone 1 had to be placed during the wet season, making control of water content critical.

Compaction was indexed by measuring the undrained shear strength of each lift using a field shear vane in accordance with standard ASTM D2573. Figure 9 shows a shear vane strength test in progress. This proved to be a rapid and reliable quality control test method. Periodically, field density tests were also performed as part of the quality assurance program, using the sand replacement method (ASTM D1556) at the same locations tested with the shear vane. The in-place average water content of Zone 1 was +5.7% above the standard optimum. The average vane shear strength was 114 kPa. The relative compaction of the material averaged 93.2% of the standard maximum dry density (ASTM D698).

8.2 Filter and Drain Zones

The chimney filter, drain, and transition zones flanking the core were widened from their design width of 2.5 m to 4.5 m within 50 m of the main fault crossings in the foundation. Depending on access constraints, the materials were placed by one of the following methods: (a) spreading with an excavator, (b) end-dumping from haul trucks, and (c) delivery by telescoping belt and tremmie hose. Once spread, the materials were leveled with excavators, bulldozers, or handheld screeds, and compacted by 4 passes of a 12-ton, smooth-drum, vibratory, self-propelled roller. Ample water was added to the materials during placement and spreading.

Protective mats were placed to allow haul trucks to cross over the chimney filters and drains. These methods proved effective at minimizing contamination and breakdown of the materials. Traffic over the blankets was avoided to the extent possible.

Figure 9 : Vane shear testing of compacted core material

8.3 Rockfill

Shoulder rockfill was placed by end-dumping from large trucks and by spreading out and leveling the material in 0.9-m lifts with a bulldozer. Water was added with water cannons or water trucks during spreading (Fig. 10). The rockfill was compacted by 6 passes of a 12-ton, smooth-drum, vibratory, self-propelled roller. Eleven in-situ density and gradation tests were completed in the rockfill. An average dry density of 2,310 kg/m^3 was measured using standard ASTM D5030.

Figure 10 : End-dumping, spreading, and watering of shoulder rockfill

8.4 Riprap

Riprap (Zone 4) was placed as protection against erosion on the inboard face of the embankment. Riprap was sourced by selectively stockpiling columnar basalt excavated from the PAC and other nearby sources. The material was placed using a 30-ton excavator with a general-purpose bucket. The rockfill was trimmed to the design slope to allow accurate placement of the riprap over the trimmed face.

9. NORTH TIE-IN CUTOFF WALL

The North Tie-In Cutoff Wall completes the Dam 1E water barrier between the dam embankment and the Pedro Miguel Locks. The wall is a plastic concrete secant-pile structure, 97 m long and up to 26 m deep. It penetrates into sound rock of the La Boca Formation, and overlaps vertically with the grout curtain. A cutter-tooth auger bit and a hydraulic oscillator successfully drove steel casing to support the holes in the cobble and boulder backfill of the Pedro Miguel Locks and the auger excavated the materials out (Fig. 11). The shaft bottom was carefully cleaned with a bucket auger.

The design required a minimum wall thickness of 0.6 m. A pile diameter of 1.4 m with a spacing of 0.86 m was selected, resulting in an overlap thickness of 1.1 m. This layout provided ample margin to meet the specified minimum wall thickness. The selected plastic concrete mix yielded a 28-day compressive strength between 1080 and 1100 kPa, and a permeability between 3.6×10^{-7} and 5.1×10^{-7} cm/sec, which met the design specifications.

Figure 11 : Excavation of secant piles

Upon completion of the wall, the concrete guides were removed, the top of the wall was trimmed to the design elevation, and the embankment was completed over the wall alignment. The joints between piles were delineated at the surface by a thin smear of clay/bentonite around the perimeter of the secondary piles (Fig. 12). The contact between piles and the integrity of the piles was verified at depth with drillholes. Core recovery and camera inspection of the holes showed a good contact between piles, and between the foundation rock and the base of the piles.

Figure 12 : Trimming the top of the cutoff secant-pile wall

Closure of the secant pile wall against the concrete monolith of the locks was made by two closure shafts drilled against the monolith. The gap between the closure shafts and the monolith wall was closed with three 850-mm-OD connection shafts. The connection shafts were cored to intersect each other and to overlap the monolith wall and the closure shafts. Camera inspection showed that the north closure shaft deviated from the monolith wall, and a fourth connection shaft was drilled to ensure a complete seal. The closure and connection shafts were backfilled with lean concrete.

10. CONCLUSIONS

Borinquen Dam 1E is a critical component of the Panama Canal and is vital to the Canal's operation. This paper has presented an overview of the construction of the dam, including the borrow of embankment materials from required excavations and other sources, and key aspects of construction of the embankment, its foundation, and seepage cutoffs. The most salient design changes required during construction were also described.

Construction of the dam was a complex undertaking involving multiple and diverse project elements. Several challenges were encountered during the work, such as: weak foundation conditions including cross-cutting geologic faults, core materials of variable residual and alluvial soils, and a wet tropical climate with a short dry season. These challenges were met and the dam was constructed successfully through: a) close collaboration between the contractor, the owner and construction manager, and the design engineer, b) diligent inspection and quality assurance testing of the works, c) flexibility in the design concept, d) diligent investigation and solution of unanticipated issues, and e) timely decision-making.

The reservoir was filled without incident and the dam has been performing satisfactorily since it was completed in September 2015. The response of the dam to reservoir filling and its performance under the permanent embankment and reservoir loads is consistent with the design assumptions and has been well within expectations.

ACKNOWLEDGEMENTS

The Authors thank the Autoridad del Canal de Panamá (ACP) for permission to publish this article. They would also like to acknowledge the large team of professionals from ACP and URS (the first author's former employer) who participated in the construction of Borinquen Dam 1E and who worked diligently to ensure its successful completion.

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