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ABSTRACT

The Panama Canal Third Set of Locks Project is being constructed within a geologically diverse setting. Of particular interest is the upper chamber of the Pacific Locks Complex, which is 400 meters long and founded on basalt bedrock but crosses a 90 m wide fault zone. After completing the excavation to foundation grade, the fault zone was mapped and drilling investigations, in-situ geomechanical testing, and laboratory testing were performed. The fault zone contains highly fractured, faulted, and brecciated rock types that were grouped into two geomechanical classes, Class I and Class II.

Given the extent of the fault zone and variability of the geologic conditions within the fault zone, the foundation analysis was coordinated with the structural design of the lock walls to take into account deformation and sliding stability to meet the design and performance requirements. Two-dimensional (2D) and three-dimensional (3D) finite element analyses were performed using Phase2 and Abaqus 3D to estimate foundation settlements and evaluate the stresses within the lock walls. In addition, sensitivity analyses were performed using Abaqus 2D to evaluate bearing capacity and optimize concrete reinforcement.

A second less extensive fault zone was later encountered, and based on the experience gained in developing the mitigation measures for this fault zone, it was determined that similar mitigation measures were applicable.

INTRODUCTION

The Panama Canal Third Set of Locks Project will add a third lane to the existing Panama Canal locks to allow Post-Panamax size ships to traverse the Canal, greatly expanding shipping through the isthmus. The Third Set of Locks Project consists of a new lock complex at both the Atlantic and Pacific entrances to the Canal, which will allow vessels to move between Lake Gatun and sea level, an elevation difference of about 30 m. Both the Atlantic and the Pacific locks complexes contain three lock chambers, which are 55 m wide and 400 m long, separated by lock heads (LH) with rolling

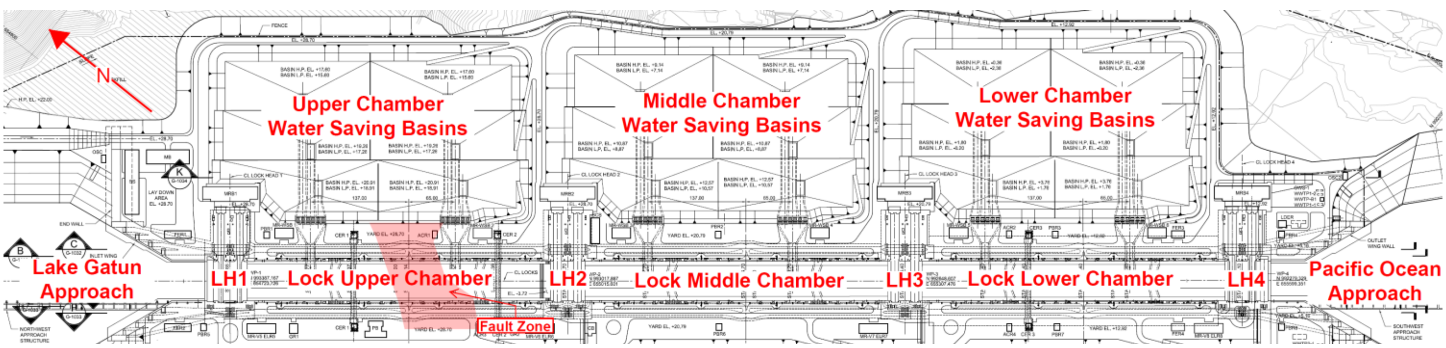

Fig. 1. PLC Layout and Approximate Location of LUC Fault Zone (highlighted)

gates. Adjacent to each lock chamber is a water saving basin (WSB) designed to save and reuse approximately 60% of the water used in a lockage cycle.

The project is constructed in a geologically diverse setting. Of particular interest is the Pacific Locks Complex (PLC) Lock Upper Chamber (LUC), which is primarily founded on relatively fresh and sound basalt bedrock but is bisected by a 90 m wide fault zone. Fig. 1 shows the PLC project layout, the location of the LUC, and the approximate location of the fault zone (shaded area).

GEOTECHNICAL EVALUATION OF FAULT ZONE

Geologic mapping was performed within the LUC excavations near the final foundation grade of the east and west lock walls and lock chamber floor. The excavation encountered a predominantly right-lateral strike slip fault zone with several *en echelon* segments accompanied with bedding plane shearing and many Riedel shears on meso- and mega-scale. The fault trend and sense of movement strikes NE–SW with the most distinct shears trending between N30° and N85° (azimuth) and dipping at approximately 65° to 85° to NW and N, respectively. The fault zone appears to have experienced multi-phase tectonism as many slip surfaces exhibit overlapping slickenside striations of various orientations.

As shown in Fig. 2, geologic mapping delineated a wide range of conditions, and several geologic units based on rock type, the degree of faulting/shearing, and the overall rock mass structure were identified.

The fault zone is comprised of Basalt and La Boca Formation, ranging from blocky to moderately to intensely sheared/faulted rock and fault breccia. The Basalt exhibits primarily brittle deformation with several fault planes and closely spaced intersecting shear planes. The rock units of the La Boca Formation, consisting of sedimentary rock types, exhibit brittle to ductile deformation with several fault planes and shear surfaces with overlapping slickensides.

The intact rock, character of the rock mass, and condition of discontinuities were documented and each subunit was subsequently categorized in general accordance with the Geologic Strength Index (GSI) system (Marinos and Hoek, 2005) using RocLab software (RocScience, Inc.).

Basalt

Four subunits of the Basalt were identified and include rock masses described as undisturbed basalt, partially disturbed/very blocky basalt, disturbed/sheared basalt, and intensely sheared/cataclastic basalt.

Undisturbed Basalt. This rock mass bounds the fault zone on both sides and consists of hard, fresh, and well-interlocked

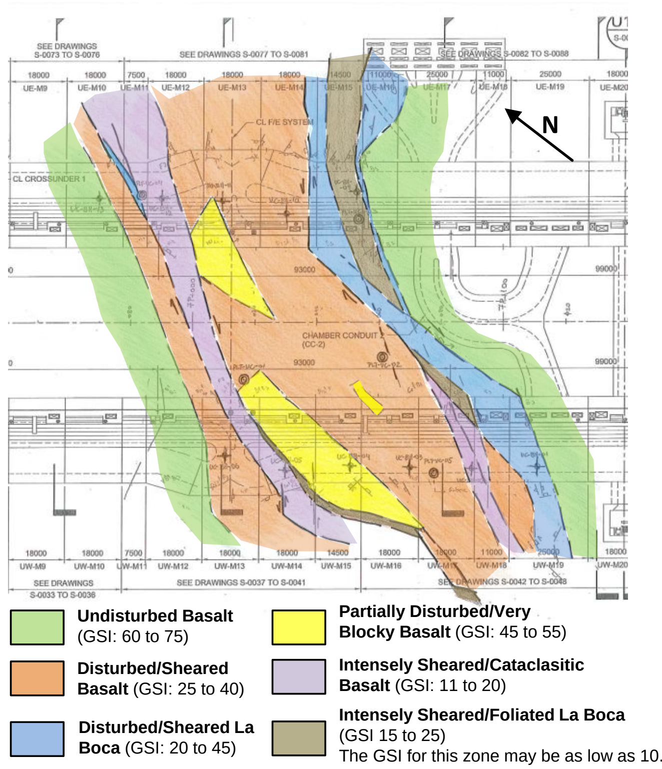

Fig. 2. Geologic Map of LUC Fault Zone

columnar basalt. The rock mass structure is characterized as blocky with good to very good joint surface conditions and has a GSI between 60 and 75.

Partially Disturbed/Very Blocky Basalt. This subunit occurs as isolated intact rock within a more highly disturbed and sheared rock mass. The intact rock is medium hard to hard, slightly weathered to fresh with multiple joint sets varying from smooth to rough, slightly to moderately weathered, and with occasional slickensides. The GSI for this subunit ranges from about 45 to 55.

Disturbed/Sheared Basalt. The rock mass is moderately to slightly weathered, medium hard to hard, and extensively sheared and faulted with multiple joint sets. Discontinuities are undulating to planar, typically smooth and slickensided, highly weathered, frequently filled with compacted clayey sand, subangular gravel, and calcite. Fault breccias, on the order of 10-cm to 50-cm-wide, are present throughout. The GSI for this subunit ranges from about 25 to 40.

Cataclastic Basalt. This subunit is a mixture of intensely sheared basalt and fault breccias, as shown in Fig. 3. Where basalt rock is present, it is typically soft to medium hard, moderately to highly weathered and friable. Discontinuities and shear planes are very closely spaced with planar, highly weathered, slickensided, with locally crushed rock material or filled with gravel and clayey sand. Fault breccia is up to 5 m wide and parallels predominant fault planes and can be locally poorly indurated and softened. The GSI for this rock mass ranges from about 11 to 20.

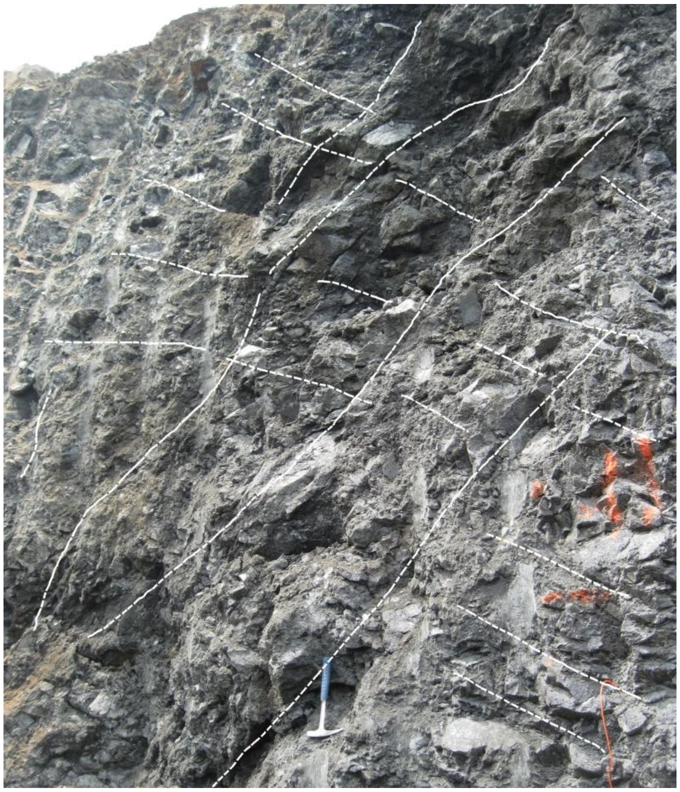

Fig. 3. Cataclastic Basalt. Persistent and very close joint spacing intersected by steeply dipping shear planes.

La Boca Formation

Within the fault zone, the La Boca Formation consists of: (1) thin to moderately bedded, bluish gray calcareous sandstone; (2) thinly laminated gray siltstone; (3) interbedded siltstone and sandstone; and (4) thinly bedded, dark brown to black carbonaceous/lignitic shale and siltstone. The La Boca Formation rock types are typically medium hard, although the carbonaceous/lignitic shale/siltstone is described as soft. Based on the character of the rock mass, the four rock types are mapped as two subunits.

Disturbed/Sheared La Boca. The rock mass includes medium hard and strong sandstone and interbedded sandstone/siltstone. The rock mass exhibits multiple intersecting sets of discontinuities, moderately to closely spaced, generally planar, tight to moderately open. Discontinuities are rough to slickensided and slightly to moderately weathered. The GSI for this subunit ranges from about 20 to 45.

Intensely Sheared/Foliated La Boca. This subunit consists of gray siltstone and carbonaceous shale/siltstone and has a width of approximately 5 to 10 m, as shown in Fig. 4. The rock mass is soft to moderately soft, with very closely to closely spaced discontinuities. The discontinuities are predominantly planar to undulating, moderately weathered, smooth and slickensided. Multiple shear planes exhibit occasional clayey sand filling and/or calcite or pyrite. The GSI for the rock mass ranges from about 15 to 25.

Fig. 4. Intensely Sheared/Foliated La Boca

The fault zone at the south end of the east lock wall consists of a 10 meter wide zone of black, argillaceous and carbonaceous shale. The rock is very fissile, laminated, and intensely sheared. The intact material is generally weak, but a 2 to 5 m wide zone is very weak and can be broken with moderate hand pressure, and the rock deteriorates when immersed in water. This rock is referred to as the soft black shale and may have a GSI as low as 10.

Engineering Classification of Foundation Conditions

Based on an analysis and taking into account the variability and complexity of the five subunits of the fault zone, it was concluded that; in terms of engineering geology, the entire fault zone can be classified into two geomechanical shear/fault conditions, each approximately 5 m to 15 m wide with adjacent shear/fault-disturbed zones.

Class I foundation conditions represent the poorest subunits and include Intensely Sheared/Cataclastic Basalt and Intensely Sheared/Foliated La Boca. Class II foundation conditions encompass the better subunits including Partially Disturbed/Very Blocky Basalt, Disturbed/Sheared Basalt and Disturbed/Sheared La Boca units. The two foundation classifications are represented in Fig. 5.

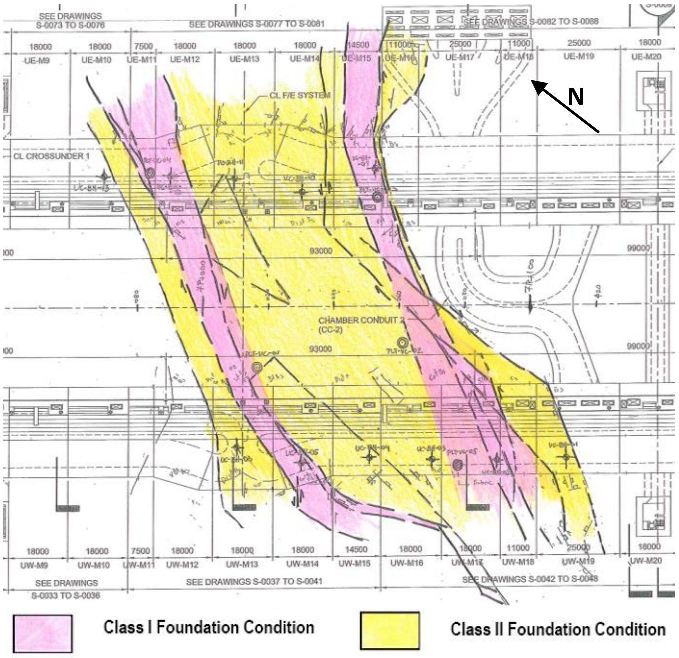

Fig. 5. Engineering Classification Map of LUC Fault Zone

GEOTECHNICAL EXPLORATIONS AND TESTING

To supplement field mapping, geotechnical investigations were performed to further characterize the fault zone conditions and materials. The investigations included 11 boreholes, uniaxial compressive strength (UCS) tests, six plate load tests, geophysical surveys, and field density tests. The testing was used to develop material parameters for lock wall foundation and structural analyses.

Drilling and Testing Investigation

Boreholes were drilled along the east and west lock wall locations and spaced approximately 20 m apart. Rock cores were logged and core samples were tested to measure the UCS and elastic modulus. Of the basalt encountered within the fault zone, 90% of core was classified as poor (rock quality designation, RQD, of 25 to 50%) to very poor (RQD less than 25%), and 65% of the La Boca formation was classified poor to very poor.

The average UCS for Basalt within the fault zone is 45 MPa, and the elastic modulus is 11,600 MPa. Tests on La Boca Formation indicated UCS of 37 MPa for sandstone and 15 MPa for siltstone. The intact modulus of the La Boca Formation was estimated using a modulus ratio (MR = Ei/USCi, Hoek and Diederichs, 2005) from other project test data available for the La Boca Formation. Using the MR, the estimated elastic modulus of sandstone is 8,500 MPa and siltstone is 3,300 MPa.

Plate Load Tests

Plate load tests were performed on Class I foundation materials using a rigid, 760 mm diameter plate and achieved a maximum bearing pressure of 1.0 MPa, which approximates the foundation bearing pressures under typical foundation loading. Four tests were performed in Class I Basalt and one test was performed on Class I La Boca Formation yielding an average modulus of about 390 MPa. The sixth test was performed on the Class I black shale and yielded a modulus of 230 MPa.

These test results were compared to the rock mass deformation modulus that was estimated using the GSI system and RocLab software (Rocscience Inc.) to provide a representative value.

Geophysical Survey

To supplement the borehole investigations by providing a continuous assessment of conditions with depth, a seismic refraction survey was performed. The seismic refraction measurements were coupled with velocity measurements from Multichannel Analysis of Surface Waves (MASW) testing. Investigations consisted of four survey lines, two survey lines along the alignment of each lock wall. Each survey line was 120 m long and included 48 geophones spaced 2.5 m apart. Two-dimensional profiles were developed for both P-wave and S-wave velocities along each survey line and extended to a depth of approximately 25 m. The profiles were consistent with surface observations and distinguished between zones of lower and higher seismic velocities, which correspond to Class I and Class II foundation materials, respectively.

In addition, the profiles indicated a zone of lowest velocities within the upper 4 to 7 m, which was attributed to the effects of blasting and construction traffic. These materials were removed during final excavation to foundation grade as part of the foundation treatment.

Density

The density of fault materials was measured using the water replacement method in a test pit described in ASTM D 5030. The results from in situ density testing for Class I and Class II foundation materials are summarized in Table 1.

Geotechnical Material Parameters

The results of the field investigations, testing, and subsequent geomechanic evaluations were used to develop geotechnical parameters for use in foundation and structural analyses of the lock walls. The geotechnical parameters for Class I and Class II are summarized in Table 1. Material properties for Undisturbed Basalt from previous project testing are provided for reference and were also taken into account in the analyses.

Table 1. Geotechnical Parameters

Parameter	Unit	Class I	Class II	Undisturbed Basalt
Unit weight, γ_n	kN/m^3	22.9	24.3	26.5
Poisson's Ratio, ν	--	0.30	0.30	0.30
P-wave velocity, V_p	m/s	1,000 – 3,000	1,000 – 4,000	--
S-wave velocity, V_s	m/s	500 – 1,500	700 – 2,000	--
Apparent Cohesion, c' ($\sigma_n > 0.5$ MPa)	MPa	0.07	0.34	0.65
Friction Angle, φ' ($\sigma_n > 0.5$ MPa)	Deg.	36	49	60
Rock Mass Modulus, E_{rm}	MPa	350	1,020	4,200
Dynamic Modulus, E_d	MPa	2,450	5,100	8,000

DESIGN CONSIDERATIONS

In addition to the structural stability of the walls, one of the design considerations for the lock walls was the allowable bearing capacity of the weaker foundation material, in particular as it relates to differential deformations between adjacent lock monoliths. Two culverts, 8.3 m by 6.5 m and 6.5 m by 6.5 m, used for the lock filling and emptying system, are located within the body of the lock monoliths; therefore, large differential deformations between adjacent monoliths have the potential to result in unacceptable hydraulic losses in the system and adversely impact the performance of the filling and emptying system. In addition, waterstops located at lock wall contraction joints can accommodate relatively small differential movements, so differential deformations have to be limited to avoid damage to the waterstops, thereby preserving the watertightness of the lock chamber. To minimize deformations to acceptable limits and meet allowable bearing capacity criteria, initial evaluations focused on the following foundation treatment options.

1. A reinforced concrete foundation mat bridging the fault zone. A similar design was developed for a solution channel at the Kentucky Locks (TVA, 1951) where a 34 m long and 12 m thick mat was constructed to bear on sound rock on either side of the 21 m wide solution channel and carry the structural load across the channel. Given the relative scale of the LUC Fault Zone, a nearly 150 m long and 25 m deep beam would be required. Preliminary analyses were performed on this option; however it was ultimately discarded because of cost and constructability concerns.

2. A concrete or RCC arch spanning the fault zone. Again, a similar concept was considered at the Kentucky Locks, however it was ultimately rejected. For the LUC Fault Zone, extensive excavation of the fault material would be required to determine the quality of the sound basalt that would accept the arch thrust, so this solution was determined to not be feasible.

3. Excavate and replace the Class I foundation material. Excavation and replacement with lean concrete is a common solution to treat weak foundations. Preliminary analyses indicated that this solution did not provide an appreciable reduction in total or differential deformations, so it was discarded.

4. Over-excavate the foundation deeper and wider than required for the lock wall construction and place a lean concrete slab located directly below the structural monolith. Preliminary analyses indicated this solution increased the allowable bearing capacity and helped control differential deformations, so this solution was further developed.

ANALYSES

Before beginning detailed numerical analyses, it was judged that deformations would control the final design of the foundation treatment. However, bearing pressures obtained from preliminary 2D finite element analyses of the lock walls under the critical earthquake time history using Abaqus indicated that bearing stability was a more critical design criteria than foundation deformations. In addition, the stability analyses indicated that a reinforced concrete floor slab, similar in concept to that employed elsewhere on the project, would be required to achieve an adequate sliding factor of safety for the lock walls and to protect the fault zone materials from erosion and deterioration over time. Thus, detailed analyses of bearing capacity, foundation deformations, and lock wall stability were performed in parallel using an iterative approach.

Bearing Capacity and Sliding Stability

The allowable bearing capacity was calculated for the weaker Class I foundation material. For monoliths partially founded on the Class I material, the allowable bearing capacity is assumed to be controlled by the weaker material because the majority of the monolith is founded on the Class I material in each case of a mixed foundation.

For very weak and disturbed rock or material it was considered more appropriate to calculate the allowable bearing capacity using the Mohr-Coulomb criterion described by the Terzaghi equation (USACE, 1994) than to use the Generalized Hoek-Brown strength equation. The allowable bearing capacity for the Class I foundation was estimated assuming general shear failure as defined by Equation 1.

$$q_a = [cC_cN_c + 0.5\gamma' B' C_\gamma N_\gamma + \gamma' D N_q] / F \qquad (1)$$

Where:

- q_a = allowable bearing capacity
- F = factor of safety
- c = apparent cohesion of rock mass
- $B' = B - 2e$, effective width of foundation
- B = total foundation width
- e = eccentricity parallel to foundation width
- γ' = effective unit weight
- D = embedment depth of foundation below ground surface
- C_c, C_γ = foundation correction factors per USACE, 1994 (

Table **2**)

N_c, N_γ, N_q = bearing capacity factors defined by the following equations:

$$N_c = 2N_{\varphi'}^{1/2}(N_{\varphi'} + 1) \qquad (2)$$

$$N_\gamma = N_{\varphi'}^{1/2}(N_{\varphi'}^{2} - 1) \qquad (3)$$

$$N_q = N_{\varphi'}^{2} \qquad (4)$$

where:

$$N_{\varphi'} = \tan^2(45 + \varphi'/2) \qquad (5)$$

and φ' = internal friction angle of rock mass

Table 2. Terzaghi Correction Factors (USACE, 1994)

Table 6-1
Correction factors (after Sowers 1979)

Foundation Shape	C_c N_c Correction	$C\gamma$ N_γ Correction
Circular	1.2	0.70
Square	1.25	0.85
Rectangular		
L/B = 2	1.12	0.90
L/B = 5	1.05	0.95
L/B = 10	1.00	1.00

The allowable bearing capacity was evaluated for three scenarios which impacted embedment depth: (1) no foundation treatment, (2) 1 m thick chamber floor slab, and (3) a 1 m thick chamber floor slab and the 2 m thick lean concrete below the lock wall. For the first scenario, the embedment depth was 3.5 m measured from the lock chamber floor, El. -2.64, to the bottom of the lock wall shear key, El. -6.14. Of this, the upper 1 m was assumed to deteriorate over time because of erosion of the chamber floor, so an embedment depth of D = 2.5 m was considered. In the second scenario, the full embedment depth of D = 3.5 m was used because the chamber floor slab would protect the chamber floor from erosion. For the final scenario, as shown in Fig. 6, the lock wall shear key toe and foundation treatment extends to El. -7.64, which is 5 m below the lock chamber floor, thus the embedment depth for the lock wall monolith is taken as D = 5.0 m.

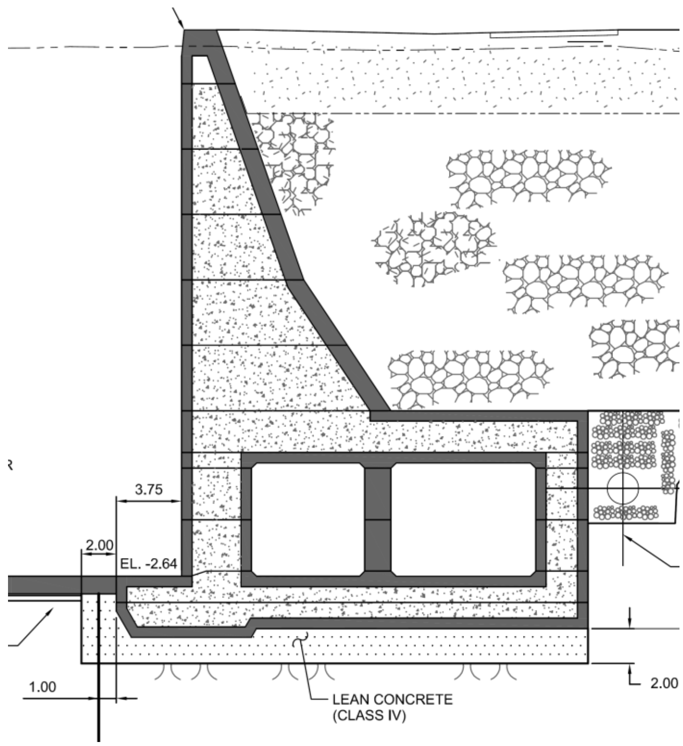

Fig. 6. Typical Lock Wall Section Modified for Fault Zone

For plane strain conditions ($C_c = C_\gamma = 1.00$), and no eccentric loads ($B' = B = 29.05$ m), and the strength parameters shown in Table 1, the allowable bearing capacity was calculated to be q_a = 3.0 MPa. Given the complex loading of the lock walls, eccentric loads needed to be accounted for. The effect of load eccentricity on the allowable bearing capacity was represented by Equation 6.

$$q_a = 0.071B' + 0.92 \qquad (6)$$

In order to evaluate the bearing pressures at the foundation, 2D finite element analyses were performed in Abaqus for the three scenarios described above. The results indicated that the bearing pressure exceeded the allowable bearing capacity and the sliding factor of safety (FS) was exceeded if no treatment was included. For the second scenario, the addition of a chamber floor slab significantly increased the sliding FS such that it met the requirement, but the allowable bearing capacity was still less than the bearing pressures. Finally, for the third scenario with the 2 m thick slab below the wall, the bearing pressures reduced and the bearing capacity increased, meeting the required criteria; the sliding FS remained acceptable due to the presence of the chamber floor slab.

2D Deformation Analysis

To assess the magnitude of potential foundation deformation within the fault zone, and to evaluate foundation treatment alternatives, 2D and 3D finite element analyses were performed. The 2D model, shown in Fig. 7, consists of a section taken through the culvert of the east lock along the length of the LUC. The model was developed using Phase2 (Rocscience Inc., 2010) to estimate foundation deformations within the fault zone; to assess the effectiveness of different foundation treatment depths; and to evaluate effects of construction sequence on foundation deformations.

The vertical boundaries of the model were restrained from movement in the direction normal to the boundary plane (rollers), and the bottom boundary of the model was restrained from movement horizontally and vertically. To minimize boundary effects, the bottom boundary is 65 m below the foundation grade, which is about two times the height of the wall plus the height of the backfill. Materials defined in the model include Class I (pink), Class II (yellow), and sound basalt (gray) foundation materials; concrete for foundation treatment and lock walls; and basalt rockfill (green) above the concrete lock wall culvert to the finished grade. The materials are modeled as linearly elastic, and the concrete-rock, rock-rock, and concrete-rock fill boundaries are modeled as material boundaries. The contraction joints between lock wall monoliths are modeled as frictionless joint interfaces to allow for independent movement of adjacent monoliths along the joint. The analysis considered the following construction stages:

1. Stage 1: In situ materials are in place.

2. Stage 2: Excavation occurs instantaneously. Materials are excavated to El. -5.64 m PLD along the entire lock wall and in all areas of foundation treatment. This stage is the reference stage to which deformations from subsequent stages are compared. Total deformations are set to zero at this stage to isolate deformations resulting from the construction of the foundation treatment, lock walls, and rock fills.

3. Intermediate Stages: Fault zone foundation treatment consisting of lean concrete is placed instantaneously. Lock wall monoliths are constructed sequentially after lean concrete placement.

4. Final Stage: Placement of rockfill to finished grade, El. +28.70 m PLD.

Effect of Foundation Treatment Depth. The total foundation deformations at the End of Construction (EOC) are shown on Fig. 8 for three cases: (1) no foundation treatment; (2) 2 m thick lean concrete slab below the lock wall; and (3) 5 m thick lean concrete slab below the lock wall.

The 2D analyses show that the maximum deformation is about 37 mm for the three cases, indicating that the slab has little effect on total deformations. However, a lean concrete slab serves to distribute load through the foundation and thus results in reducing differential deformations between adjacent monoliths, particularly near the interface between the fault zone and the undisturbed basalt. For the case without the slab, differential deformations are typically less than about 6 mm except at the contraction joint between monoliths M10 & M11 and between M15 & M16 where the differential deformations are about 11 mm and 12 mm, respectively. These contraction joints are closely aligned with the geologic contact between Class I and Class II foundation materials (M10 & M11) and Class I material and basalt (M15 & M16). Furthermore, the contrast in material properties, principally the modulus of deformation, appears to yield greater differential deformations in these locations.

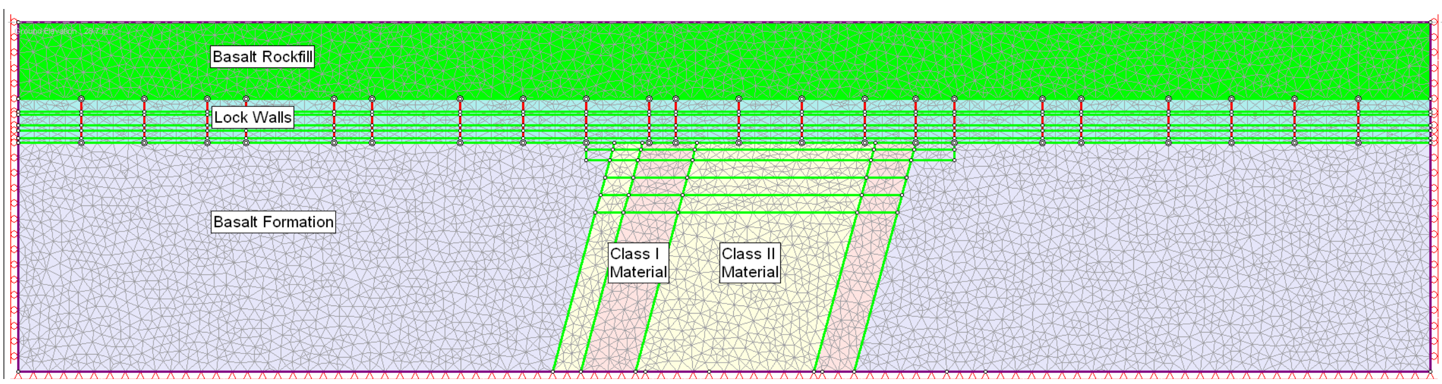

Fig. 7. 2D Model Layout of East Lock Wall Section (view looking east)

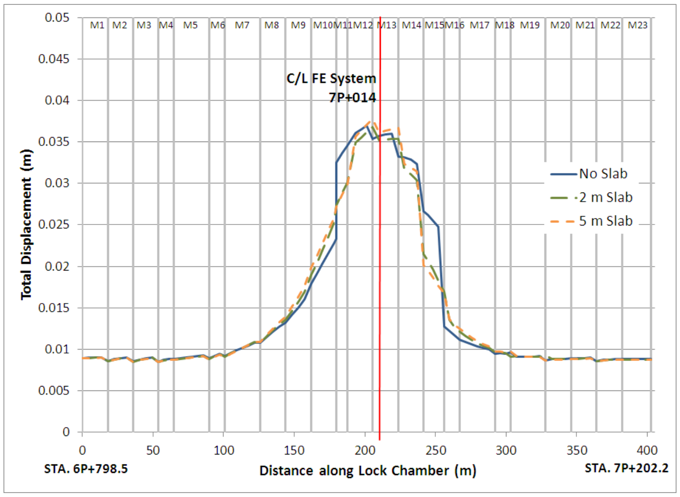

Fig. 8. Total Foundation Deformations at EOC (2D model)

For the models considering the 2 m and 5 m thick slabs, the differential deformations between adjacent monoliths are typically less than about 4 mm. However, at the contraction joint between M14 & M15, the differential deformation is about 9 mm with the 2 m thick slab, and about 11 mm with the 5 m thick slab. This contraction joint is closely aligned with the geologic contact between the Class I and Class II foundation materials. At the joint between M10 & M11 and the joint between M15 & M16, the slab reduces the differential deformations by about 50% compared to the case without the slab, resulting in more evenly distributed deformations.

The deformation analyses showed that although the slab has a minor effect on total deformations it does help control differential deformations. The model with the 2 m thick slab yielded similar results as the 5 m thick slab. Therefore, the 2 m thick slab, which was required to meet the bearing capacity requirements is preferred to control deformations and was then used for the subsequent deformation analyses.

Effect of Construction Sequence. Three construction sequences were modeled to evaluate total and differential deformations at the base of the east lock wall monoliths related to the concrete and backfill placement arrangement. The three sequences evaluated were:

1. Sequence 1: Construction of the monoliths and placement of the rockfill beginning at the fault zone and progressing outward on to sound basalt until reaching Lock Head 1 (LH1) and Lock Head 2 (LH2). These structures are located at each end of the LUC but were not modeled as they are outside the zone of influence.
2. Sequence 2: Construction of the monoliths and placement of the backfill beginning simultaneously at LH1 and LH2 and progressing inward on sound basalt until reaching the fault zone.
3. Sequence 3: Construction of the monoliths and placement of the backfill beginning at LH2 and progressing toward LH1.

The analyses showed that total and differential deformations are approximately the same for the three construction sequences, indicating that the construction sequence has negligible effect on foundation deformations.

3D Finite Element Analysis

In addition to the 2D stability and deformation analyses, a 3D Abaqus (Simulia, 2011) model of the LUC was developed to evaluate the structural and foundation behavior of the wall under various loading combinations. The model includes 17 Lock Wall (LW) monoliths at each side (east & west), the foundation, the fault zone, the 2 m thick lean concrete slab beneath the monoliths, the 1 m thick chamber floor slab, and the rockfill behind the walls. Materials defined in the model, shown in Fig. 9, include Class I (red), Class II (blue) and sound basalt foundations (grey), mass concrete for lock walls (tan), lean concrete slab for foundation treatment, and backfill (green) placed atop the monoliths to the finished grade.

The model and foundation block with the fault zone, shown in Fig. 10, has dimensions of 300 m long (parallel to lock centerline) by 200 m wide and extends 80 m in depth. All foundation materials were modeled as linear elastic using properties presented in Table 1.

The outside vertical boundaries of the 3D model are restrained from movement in the direction normal to the boundary plane (rollers) and the bottom boundaries were restrained from movement in the vertical direction. The lock wall contraction joints between monoliths are modeled and extend through the lean concrete slab foundation treatment, matching the lock wall contraction joints.

The contraction joints and other contacts are modeled using interfaces. The contraction joints between the monoliths were included to estimate differential settlements and to capture arching effects and stress paths between the monoliths. Additional interfaces were incorporated between the walls and the lean concrete slab, and conservatively between walls and backfill material.

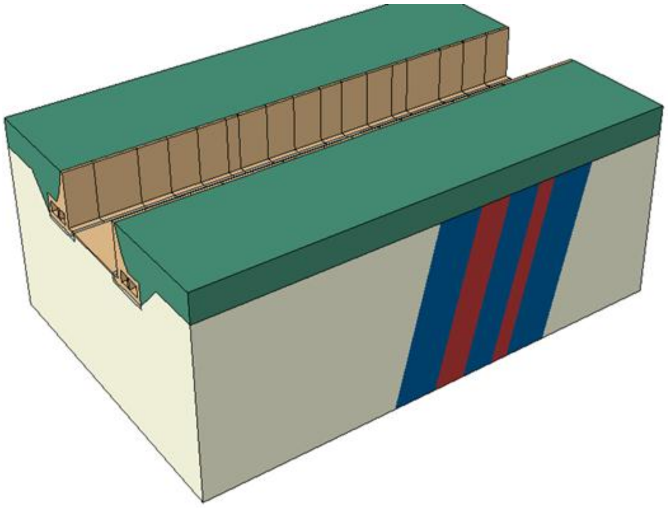

Fig. 9. 3D Model Layout (southeast view)

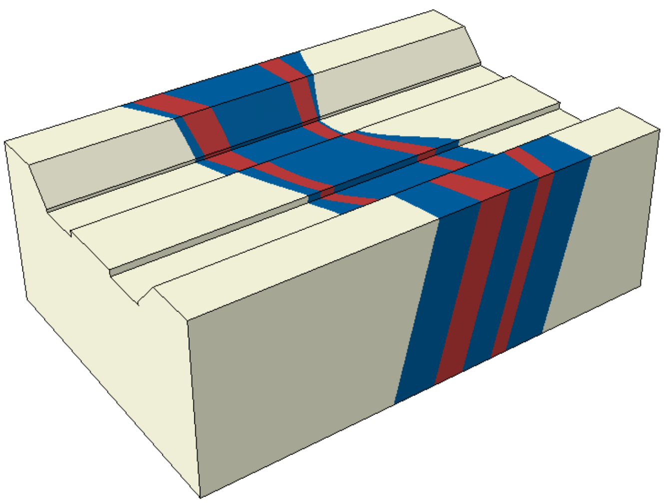

Fig. 10. Foundation including fault zone (3D model)

Based on the 3D analyses, the lock wall structures within the fault zone satisfied foundation bearing capacity and sliding stability criteria for the static loading condition and the Level I and Level II seismic loading conditions. The bearing stability requirements were achieved by including the 2 m thick lean concrete slab beneath the lock walls and widening the excavation at the toe of the lock wall by 2 m. Sliding stability requirements were met by providing a reinforced concrete floor slab on the chamber floor, which serves as a strut between the east and west lock walls to transfer load. The slab also provides long-term protection of the fault zone materials from erosion and deterioration over time. Moreover, the lock walls met the required stability criteria.

The 3D model also helped evaluate the stresses within the monoliths and along the interfaces and demonstrates how the lock walls behaved with a relatively modest 2 m thick concrete slab for foundation treatment. Fig. 11 shows the stress paths within the deformed (exaggerated for affect) lock walls geometry. The stress paths show an arching effect within the lock wall as the foundations beneath the monoliths deformed. It was found that the arching effect helped limit deformations for the cases considered, and only resulted in relatively low compressive stresses across the concrete interface, which were on the order of 1 MPa.

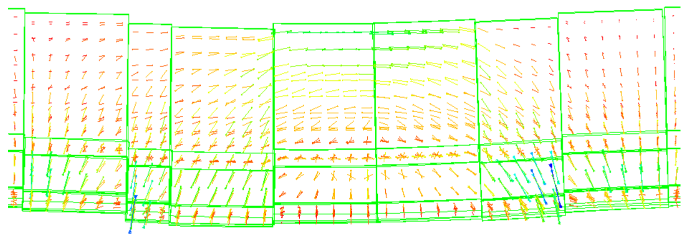

Fig. 11. Wall Arching Effect (deformations exaggerated for affect)

<u>Effect of Loading</u>. In addition to evaluating the lock wall stability, the 3D model enabled a more detailed and accurate representation of the 3D foundation behavior and lock wall configuration. This model primarily evaluated the effects of static loading conditions expected during normal operations on foundation deformations. The normal design process for the LUC founded on undisturbed basalt considered eight static and four seismic loading combinations. The deformations due to Load Combination No.1 (LC01) represent the EOC (unusual) conditions in which there is no hydrostatic pressure acting on the structure (i.e., no uplift). LC02 was evaluated because it represents the usual operating conditions as defined by the Employer's Requirements in which full hydrostatic pressure is exerted on the backside of the walls. LC02 was also considered because the uplift at the base of the structures is maximized due to the high water level in the chamber and in the backfill. The remaining static loading combinations were expected to have deformations between those of LC01 and LC02 and were not fully evaluated. The deformations from these two load combinations are plotted on Fig. 12 for the east lock wall.

The results of the 3D analyses indicate that the deformations are on the same order of magnitude as those for the 2D analyses. The maximum foundation deformations along the east wall range from about 25 mm for LC01 to about 31 mm for LC02, a difference of 6 mm. This difference represents the cyclic foundation deformation that is expected to occur during normal filling and emptying cycles of the locks. It is also noted that greater deformations for LC02 are likely due to the increased weight of the water within the lock chamber and behind the wall. The differential deformations range between 1 and 4 mm.

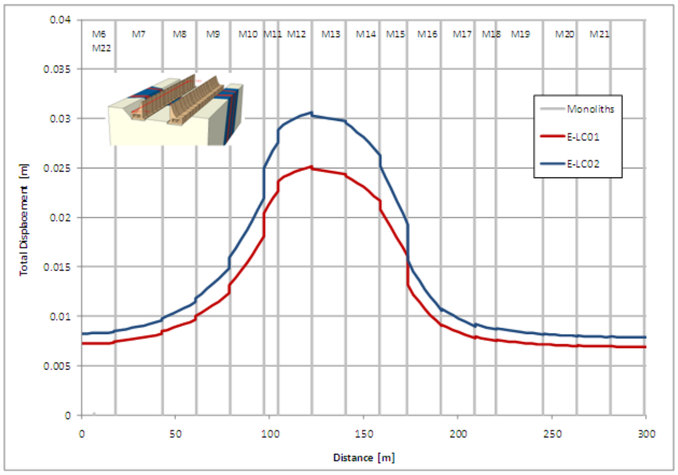

Fig. 12. LUC East Wall Vertical Deformations (3D model)

Given that the 3D model is more representative of the complex loading conditions, the 3D model is considered to provide a better representation of lock wall and foundation behavior than the 2D models. Through careful evaluation, appropriate foundation treatment were developed to achieve adequate bearing capacity, lock wall stability, and control foundation deformations over an expansive large fault zone.

SECOND FAULT ZONE

A second fault zone was later encountered within the LUC, and was located upstream of the main fault zone. The second fault zone was about 20 m wide, oriented subparallel to the 90 m wide fault zone, and was comprised of similar foundation material types.

Using the knowledge and experiences gained from the analyses performed on the larger fault zone, including an understanding of the influence of various design parameters on foundation and structural performance, similar foundation treatments were considered. Geologic mapping and rock mass characterization were performed for the second fault zone, but because the rock units were found to be of similar character to those encountered previously, no additional field investigations and testing were required. Additional analyses were performed to document the evaluation for this fault zone resulting in a similar foundation treatment.

CONCLUSIONS

Given the extent of the fault zone and variability of the geologic conditions within the fault zone, the foundation design was coordinated with the structural design of the lock walls to accommodate foundation deformations and stability to meet the design and performance requirements. Sensitivity analyses were performed using Abaqus 2D to evaluate bearing capacity and sliding stability. As a result of the analyses performed, the foundation of the typical lock wall monolith was modified to incorporate a 2 m thick lean concrete slab that extended 2 m from the face of the shear key toward the chamber. This treatment was needed to meet the bearing capacity and helped control the differential deformations between lock wall monoliths. The differential deformations are small enough for the waterstops to tolerate and maintain watertightness, and minor offsets between monoliths will not impact the hydraulics of the filling and emptying system.

A 1 m thick structural concrete chamber floor slab was also required to meet the sliding stability requirements during seismic loading conditions and to protect the chamber floor

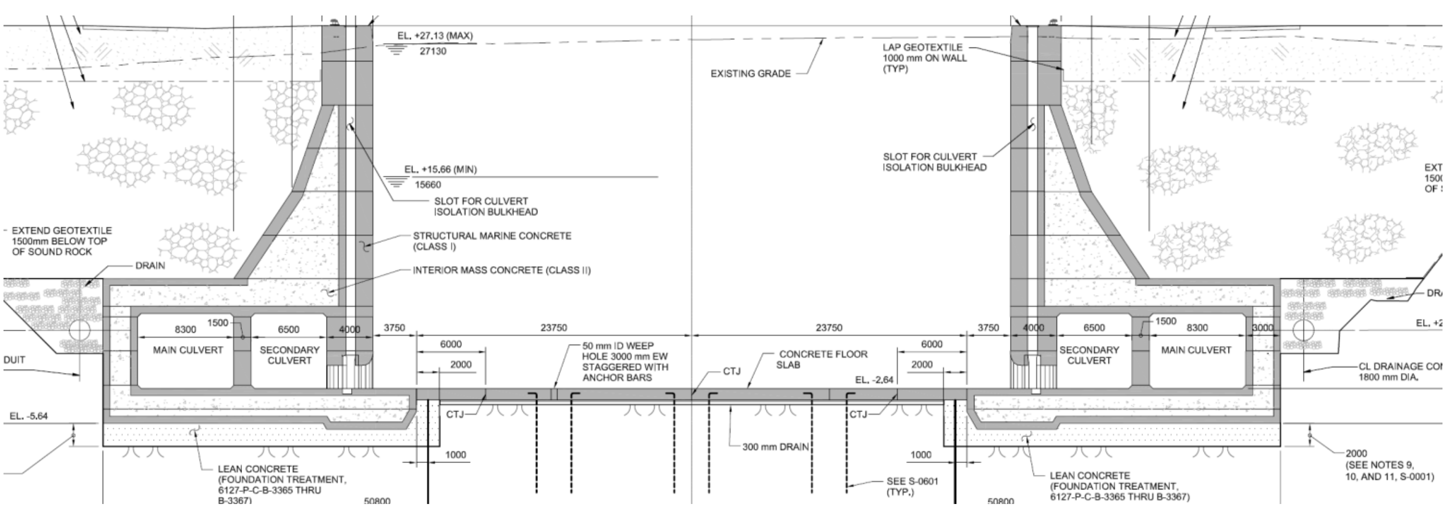

Fig. 13. Fault Zone Foundation Treatment including Lean Concrete and Chamber Floor Slab

from erosion. The slab is anchored into the rock and acts as a strut to transfer load between the walls. The slab includes a subdrain system and weep holes to control uplift pressures during rapid changes in the lock chamber water levels. The final lock wall foundation treatment and chamber floor slab design is shown in Fig. 13.

After the foundation treatment was finalized, further structural analyses indicated higher stresses within the lock wall monoliths at the fault zone, so the reinforcing design of the affected monoliths was modified to better accommodate the anticipated stresses.

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