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Cause of Failure Investigation of Masonry Retaining Walls at Logita Bridge Site, Sidama Southern Ethiopia

Bereket Bezabih[1], Henok Refissa, Thomas Bezabih

[1]Lecturer, Department of Civil Engineering, Hawassa Institute of Technology, Hawassa, Ethiopia
E-mail: bereket.bezabih@hu.edu.et Phone +251994783890

Abstract

A masonry retaining wall supporting the approach embankment of Logita Bridge suffered significant distresses, which led to a dispute between the client, the consultant, and the contractor. As a result, the construction work has been interrupted for a prolonged time causing frustration on all stakeholders. This paper presents the investigation work undertaken to establish the causes of the observed distresses. Based on preliminary investigations, a hypothesis was developed that the observed distresses were due to the choice of ambiguous structural system, use of materials with widely varying stiffness and the sole use of standard stability checks, which failed to verify the complex design. A numerical simulation was undertaken using FLAC 8.1 by assuming elastic behavior for the reinforced concrete and stone masonry parts of the wall and Mohr-Coulomb model for the various backfill materials and foundation. The values of parameters were either adopted directly from code provisions or were estimated using methods specified in relevant codes. The modeling was undertaken for the current construction state as well as the final designed state. The numerical analysis conclusively proves that the distress observed in Logita Bridge retaining wall is caused by development of tensile internal stresses at the face of the wall. If the final wall is constructed, the numerical simulation indicates that structure will collapse, not necessarily from stability of the wall but through deep-seated stability failure through the backfill and foundation.

Key Words: Retaining Walls, Failure Analysis, FLAC, Back Analysis

1 Introduction

1.1 Background

Retaining walls are common structures at bridge sites. They provide lateral support at the abutment, guide walls or approach embankments. Conventional retaining walls can be in the form of gravity or cantilever walls. One can employ a variety of forms such as semi gravity, counterfort, anchored depending of site-specific requirements. Other alternative solutions for lateral support include mechanically stabilized walls and embedded walls.

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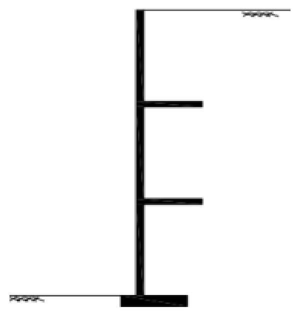
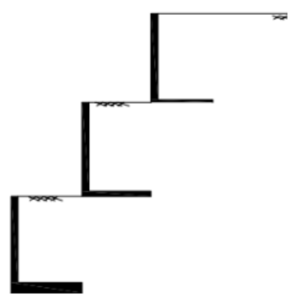
Site-specific conditions, such as high elevation support requirements, may demand adoption of a unique solution to deliver a safe and economic retaining wall solution. Some of these solutions include retaining walls with relief shelves and Cascading/Stacked walls, Figure 1. The design and construction of such systems requires special attention as the mechanism of resistance is complicated and not very well understood.



(a) Cantilever Walls with Relief Shelves



(b) Stacked Cantilever Walls

Figure 1 Relief Shelf and Stacked retaining walls

Despite the careful consideration during design and construction, retaining walls suffer from various types and levels of distress and complete collapse. Common types of distresses in retaining walls are excessive tilt and bowing, cracking and separation all the way to complete collapse. Marsh & Walsh (1996) identified excessive imposed loads, unsuitable backfill such as expansive and impervious clay, and use of excessive compaction equipment as potential causes of failure in their case studies. The possibility of poor construction and/or erroneous design is ever-present.

When significant levels of distresses are observed on a retaining wall, a forensic investigation shall be undertaken to establish the causes of the distress and identify potential rehabilitation schemes. A full-fledged forensic investigation shall take an independent review of the as-built structure and design parameters, characterize the degree of distress, develop a failure hypothesis, conduct diagnostic tests and deformation based back analysis (Babu, et al. 2016). This paper presents the investigation work undertaken to determine the causes of distresses observed on Logita Bridge approach embankment retaining wall.

1.2 Description of Project Site and Observed Distresses

The bridge site is located at 6º 30’ 11.86” N and 38º 49’ 25.43” E on Bansa Ware – Mikichu road project. Girder beams support bridge deck, which in turn is supported by piers and masonry abutments on the East and West sides. Unreinforced, natural stone with ordinary mortar, masonry ‘gravity’ walls on both sides of the roads support the approach embankment. The walls extend longitudinally up 20 m and have height up to 15 m. In order to address the relatively high elevation 15 m at its highest point, the designer has chosen a structural system, which appears to be cascading/stacked walls, see Figure 3.

Figure 2 Partial View of Logita Bridge, abutment and approach embankment retaining wall under construction

The brief notes on working drawing put forward by the designer indicate the retaining wall shall be founded on a well-compacted rock fill and on 20 cm, Type C concrete slab reinforced with diameter 12 bars spaced at 25 cm either way. The bottom foundation wall shall be constructed on a C20/40 slab. In the absence of competent foundation cyclopean concrete shall be adopted, (PES, 2018). The laboratory test results show C-25 concrete is used on the slabs.

Figure 3 presents a cross section at highest elevation of the approach embankment retaining wall at Logita Bridge site. The area of the cross section between the rock fill zones shown in figure 3 is back filled with a granular selected material as per the design and construction information.

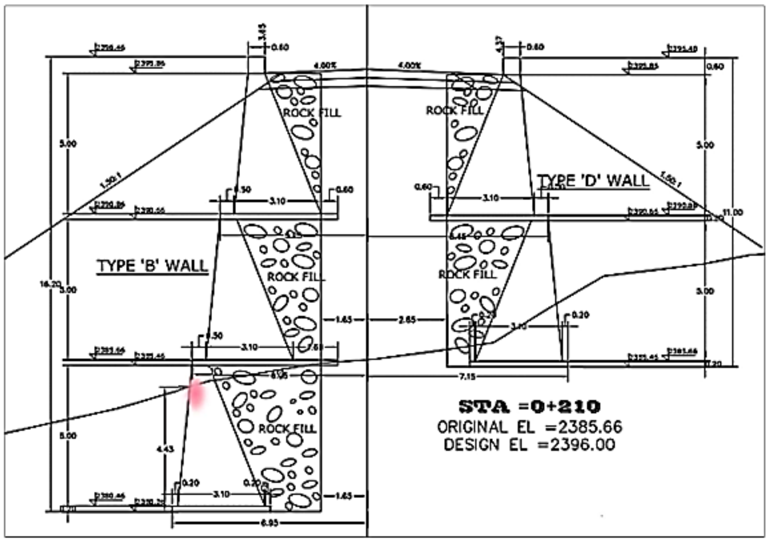


Figure 3 A Cross section of the Retaining Wall at Logita Bridge Site. (Prominent Engineering Solutions, 2018)

Horizontal cracks were observed on the first stage (bottom) wall on the north side of the wall, as highlighted by pink shades on Figure 3. These cracks are observed either at the interface of the reinforced concrete slab and masonry wall or at one to two masonry course below the reinforced concrete slab, Figure 4 (a), (b). The horizontal extent of these cracks goes beyond the vertical construction joints. It is worth noting the construction joint is incorporated only for masonry work. The reinforced concrete slab lays across the construction joints. There is little lateral bulging observed at the locations of the cracks.

Although the authors did not verify by measurements (i.e. a direct comparison between design spatial orientation and current orientation of the wall), physical inspections shows no sign of stability failure i.e. failure due to toppling, sliding or bearing capacity (excessive settlement or nearby budging due to foundation movement). Figure 4 (a) and (b) show pictures of the horizontal cracks observed at the bridge site. After these distresses were observed, construction of the retaining wall has been stopped after the second layer is back filled.

(a)

(b)

Figure 4 (a) Horizontal Cracks South Side (b) Horizontal cracks & Vertical Construction Joint South Side

2 General Approach & Methodology

2.1 General

The general approached followed to establish the cause of observed distresses includes activities such as field visit, review of design documents and prior investigative works, develop a hypothesis for the cause of failure, and test the failure hypothesis through back analysis.

Ideally, a fully-fledged forensic investigation requires independent review of the as built structure completed with field measurement, field and laboratory testing of design parameters and review of the construction documentation. In the present case, the nature of the distresses and absence of stability related distresses led to the decision that back analysis to evaluate the adequacy of design suffices.

Based on the field observation and review of design documents (what is available) a cause of failure hypothesis is developed and numerical method is used to verify its validity. Furthermore, prior investigation had put forward a potential solution to address the problem. The applicability of this proposal is reviewed.

2.2 Review of Design and Prior Investigation Reports

The client has conducted an investigative review by another consultant to establish the cause of the observed distresses. They have reported the following observations and their potential causes.

- Crack on the masonry wall
- No discernible bulging of the wall
- Poor masonry workmanship

The report speculates on potential causes of the observed cracks as;

- Poor bondage due to low quality mortar mix
- Foundation settlement
- Overloading
- Temperature and/or moisture fluctuations

Furthermore, the quality of the backfill and the construction of the weep holes are deemed to not satisfy the specifications provided by the designer. In order to alleviate these problems, the designing entity has put forward the following recommendations,

- The retaining wall masonry and the two backfills (rock fill and granular backfill) shall be removed to a depth of the first weep hole and weep hole shall be enveloped with high quality draining material
- The granular back fill shall be replaced with a rock fill
- The reinforced concrete slab shall extend to the corresponding slab on the other side of the road to form a continuous support with dowels on in the retaining wall

A review of available design/ construction documents of the wall has the following major shortcomings,

- No clear reference and citation to the use of appropriate code and standards
- No clear definition of the walls structural solution i.e. weather it is designed as *relief shelve* or *cascade retaining* wall
- No specification for material selection i.e. the quality of masonry stone, mortar, rock fill and granular back fill
- No specification of construction of important sections of the back fill such as rock fill i.e. gradation, maximum size, minimum size or compaction layer thickness etc.

The retaining wall system neither is a relief shelf nor stacked retaining wall system. There is ***no fixity*** at the face of the wall between the reinforced concrete slab and the masonry wall. Hence, the junction cannot provide fixed structural support. The width of the slab is not bound within the Rankin's shear zone. Such an arrangement cannot effectively reduce the lateral earth pressure as intended by the relief-shelf wall system, as it could potentially end up being simply supported by the backfill beyond the active state shear zone (Farouk, 2015).

In the absence of a design report, one can only presume the use of reinforced concrete slab is to serve as a foundation for the masonry wall and the system is designed as a *cascading wall* system. The retaining wall system is not a cascading/stacked system mainly because the reinforced concrete slab rests on the lower wall, not on the retained soils, preferably beyond the active shear zone. It is highly doubtful the design assumption of in Rankine's Theory (horizontal principal plane) is applicable. Hence, a simple limit equilibrium based stability analysis of the bottom wall will not adequately address the internal stress conditions developed due to complexity of the design.

Another relevant observation is the use of the same cross section of retaining walls for all three tiers of the wall system. Gravity retaining walls resist the lateral earth pressure due to their sheer weight. Hence, the width of the wall is expected to vary with the magnitude of lateral earth pressure.

2.3 Cause of Distress Hypothesis

The nature of the observed distresses and the ambiguity of the structural system along with the lack of evidence for global stability issues lend high credence to the assertion that the failure is related to *internal stresses*. The potential causes of the observed distress are the use of composite material (the difference in stiffness between the rock fill under the reinforced concrete slab), the lack of structural continuity between the materials to serve as relief shelf, and potential cantilever action.

The system is not reinforced to ensure structural continuity i.e. does not transfer moment and shear stress between the 'relief shelf' slab and the masonry stem/wall. The choice of the structural system and its intended mechanism of resistance need to be investigated using *deformation analysis* besides the *limit equilibrium stability analysis* conducted in the design of the retaining wall.

The analysis and design of retaining wall as per ERA or ES7 standards are required to satisfy stability requirements i.e. safety against overturning, sliding and foundation bearing failure. However, these requirements are expected to apply for conventional retaining wall of gravity and cantilever type walls, which assume the walls as rigid for stability considerations. If a designer selects a structural system to address special site problems such as very high walls, the onus is on the designer to verify the selected system satisfies safety and performance expectations. Hence, if relief shelves, cascading retaining walls, or composite materials are used in the design of the wall, the verification shall go beyond the minimum expectation of codes and standards.

2.4 Stress Analysis Using Numerical Method

A plain strain analysis using finite difference software FLAC 8.1 is used to simulate the appropriate material properties and loading conditions in the wall, the foundation, and the backfill in order to understand the state of internal stresses.

FLAC (Fast Lagrangian Analysis of Continua) is a 2D explicit finite difference numerical program for mechanical computation, (ITASCA, 2015). It is capable of modeling various loading conditions, constitutive models, and geometries. Under a prescribed load, the material may yield or flow. A quick indicator of the state of the computation is 'unbalanced force' at the end of each cycle of computation. If the unbalanced force converges to zero, the system has reached a static equilibrium. However, if the unbalanced force converges to none-zero value the system is flowing at constant velocity i.e. plastic flow has occurred.

The Logita bridge approach retaining wall is modeled using FLAC to analyze the stress state under the current level of construction (As-Is) and the designing lay out for the most critical profile of 15 m height. A 30 m by 36 m extent is used to model the retaining wall to ensure boundary conditions do not affect the analysis. Fixed boundary is used at the bottom and roller support is used on the sides of the model. Automatically generated course mesh is used to compute the internal stress and deformations. Figure 5 and 6 show the models generated for the Logita-AsIs and Logita-Design scenarios along with the finite element grids respectively.

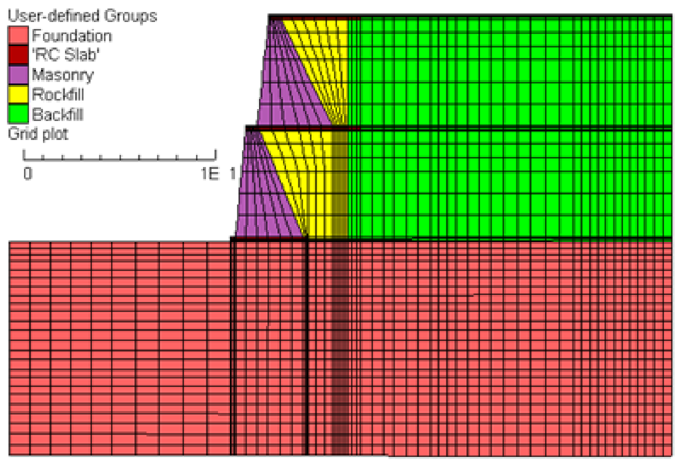


Figure 5 Finite Difference Model for Logita AS-IS

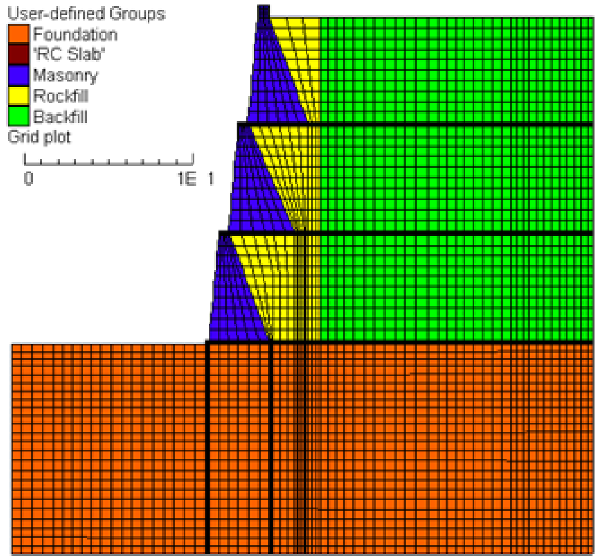


Figure 6 Finite Difference Model Logita –Design

2.5 Constitutive Models and Material Properties

FLAC is capable of modeling both linear and nonlinear mechanical behavior. The various zones of the designed and constructed retaining wall are modelled using either 'elastic' or 'Mohr-Coulomb' constitutive models. The 'elastic' model in FLAC requires Bulk Modulus and Shear Modulus or alternatively Young's Modulus and Poisson's ration. These values are estimated for the masonry wall as shown in this section. The values for a C-25 reinforced concrete are read from Euro Code 2/ES2 provisions. The 'mohr-coulomb' model requires internal angle of friction, angle of dilation, cohesion, and tension values in addition to the aforementioned 'elastic' parameters. These values are estimated from laboratory tests values and/or typical values are adopted from literature. The values used in the computation are presented in Table 1. If not specified default values are adopted.

Table 1 Material properties used in numerical computations

Description	Unit Weight γ (kN/m^3)	Angle of Friction ϕ' (deg)	Young's Modulus E, GPa	Poisson's Ratio
Masonry	20	N/A	5.4	0.22
Concrete	24	N/A	30	0.15
Rock Fill	19	42	3.7	0.22
Back Fill	18	30	0.03	0.3
Foundation	18	30	0.04	0.3

Masonry is composite material and the mechanical property tests can be done on the components, a small assembly, or a scaled masonry structure. Masonry behaves essentially as elastic material in compression up to 80 - 90 % of its strength, (Angellilo, et al. 2014). Masonry has very low tensile strength as compared to its compressive strength. Hence, it is used under the assumption of no tension capacity.

2.6 Compressive Strength of Masonry

As per (EN-1996-1-1, 2005) the characteristic compressive strength of masonry, f_k (N/mm^2)

$$f_k = K f_b^{\alpha} f_m^{\beta}$$

$where$

K, α, β	Constants $K = 0.45, \alpha = 0.7,$ $\beta = 0.3$ for natural stone in general purpose mortar,
f_b	Normalize mean compressive strength of masonry units in N/mm^2
f_m	Compressive Strength of Mortar N/mm^2

2.7 Modulus of Elasticity, E of Masonry

The short-term modulus of masonry can be determined as a secant modulus of stress-strain from tests result plots, In the absence of such test results, the short term modulus of elasticity can be determined using the equation,

$$E = K_E f_k$$

$where$

K_E	A constant with recommended value 1000
f_k	Characteristic compressive strength of Masonry

2.8 Shear Modulus, G

The shear modulus can be taken as 40% of the Young's Modulus E (EN-1996-1-1, 2005).

2.9 Compacted Rock fill

Rock fill materials with particles sizes above the standard gravel size are unsuitable for laboratory testing. As a result, only individual pieces can be tested, (Kutzner, 1997). Mechanical properties of the compacted fill must be inferred from the constituent properties. For instance, the shear strength of compacted rock fill can be given by τ (kN/m^2) (Novak, Moffat, & Nalluri, 2007)

$$\tau = \alpha(\sigma')^{\beta}$$

$where$

α, β Constants

$\alpha = 3$ $for\ Poor\ slate$ 6.8 $Good\ Quality\ sandstone$

$\beta = 0.67$ $Sand\ Stone$ - 0.81 $basalt$

σ' Compressive strength off rock material ((kN/m^2)

2.10 Modulus of Compression, E of Rock fill

The modulus of compressibility E of a rock fill is a function of rock type, strength, shape and gradation of rock size in the rock fill. It also depends on the roller type, compaction energy and the in situ confining stress (Fell, McGreoger, Stapledon, & Belt, 2005). A typical value for modulus of deformation for compacted rock fill is in the order of $20 - 50\, MN/m^2$ (Novak, et al. 2007).

Based on the review of properties of materials presented in this section and the assumption that good quality materials are adopted for masonry with ordinary cement mortars well as compacted rock fill, the values n Table 1 are estimated for the numerical modeling of Logita Bridge approach embankment retaining wall. Characteristic compressive strength of stone used for masonry 12 MPa (laboratory test) and ordinary mortar 12 MPa.

3 Results and Discussion

In FLAC, a quick indicator of the computation status is the ’unbalanced force’ value, the maximum nodal force vector. If the unbalanced force reaches a zero value, the computation stops as the systems has reached a static equilibrium. However, if the unbalanced force converges to none zero value, the system is flowing with constant velocity i.e. failure has occurred. In the analysis of the Logita bridge approach embankment, the system reached static equilibrium for the Logita-AsIs, See Figure 7.

The total vertical stress values are the preferred indicator to determine the cause of the distress observed on Logita retaining wall. In FLAC the sign convention is the compressive stress are negative and tensile stress are positive. The computation results for the two cases indicate the development of tensile stress at the face of the wall where horizontal cracks are observed. The range of magnitudes for the Logita-AsIs is case 0.5 - 0.7 MPa. See Figure 8. The total shear strain is a preferred indicator for movement within the model due to the prescribed loading. Figure 9 presents the total straining increment plot for Logita-AsIs

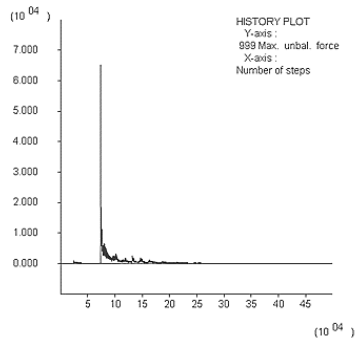


Figure 7 History Plot of Unbalanced for Logita As-Is

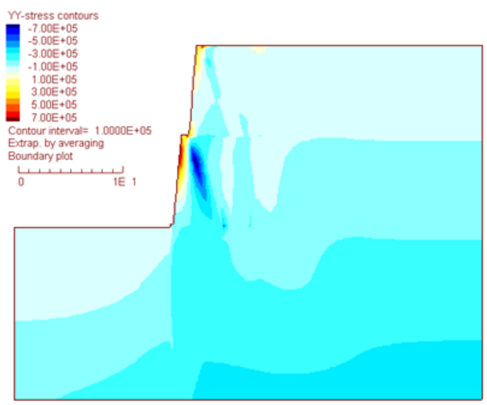


Figure 8 Total Vertical Stress Plot for Logita As-Is

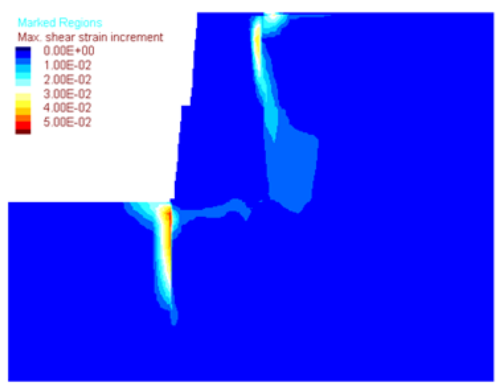


Figure 9 Total Strain Plot for Logita As-Is

For the final design scenario, Logita–Design case the model's unbalanced force ratio did not converge to a zero value after 10^6 steps. See Figure 10.

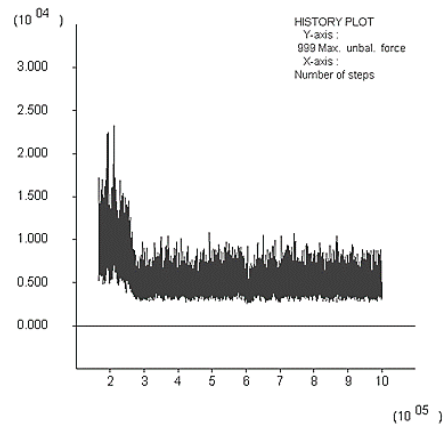


Figure 10 Unbalance force History plot for Logita-Design

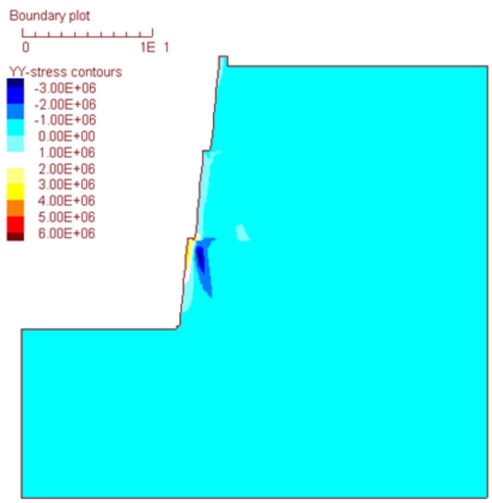


Figure 10 Total Vertical Stress plot for Logita-Design

The range of magnitudes vertical tensile stress for Logita-Design is 3-4 MPa, See Figure 10. Masonry wall is not expected to support tensile stress, and hence the cracks on the face of the wall and potential failure for the final design of constructed

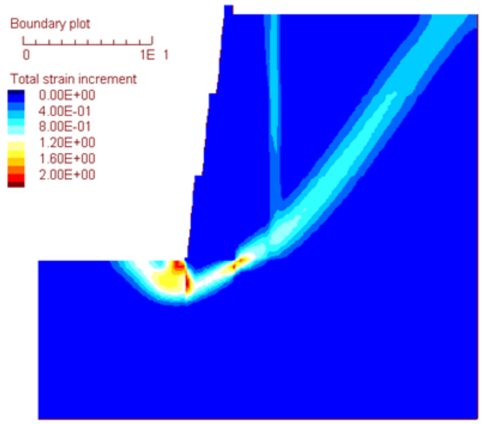


Figure 11 Total Strain Plot for Logita-Design

The major observation on the total strain plot shall be the potential for deep-seated failure through the granular back fill of the final design, if it were to be constructed. Of course, this is a function of the assumed foundation conditions.

It is important to note here that these computations have assumed certain factors, which can be a limitation on the accuracy of the true representation of the site condition. Chief among these assumption is the foundation condition is modeled as the same material as the granular backfill material. If the foundation is drastically different from granular back fill, the results may alter from those presented in this section.

There is a slight variation in the construction profile of the compacted rock fill and the design, as reported by the client. Although the design presumes a vertical face between the rock fill and the granular backfill, the contactors adopted a sloped face, without increasing the design thickness of the rock fill. In this analysis, the design profile is adopted because the analysis aims to address the hypothesis that the distress are due to design failures.

One of the suggested solutions to the problem of retaining wall failure is replacing the backfill with rock fill and extending RC slab across the section. The team attempted to evaluate the effect of this measure on the internal stress state of the retaining wall system. Both the left and right side retaining walls have been modeled with and without surcharge load. Figure 12, 13 & 14 present the adopted model, the total vertical stress, and total strain increment plots of the FLAC analysis.

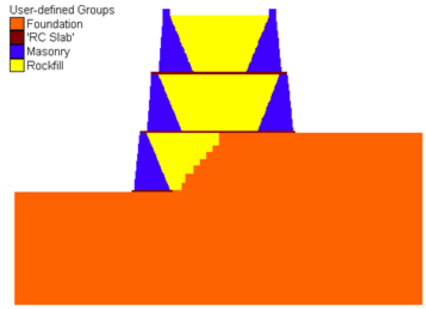


Figure 12 FLAC Model for Previously Recommended Solution

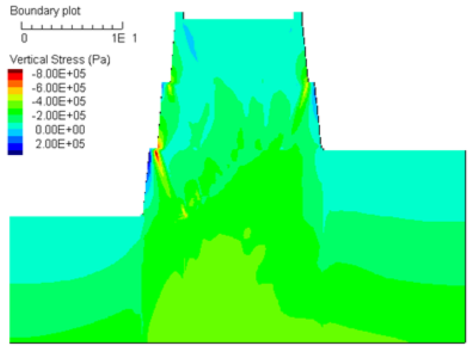


Figure 13 Total Vertical Stress Plot for Previously Recommended Solution

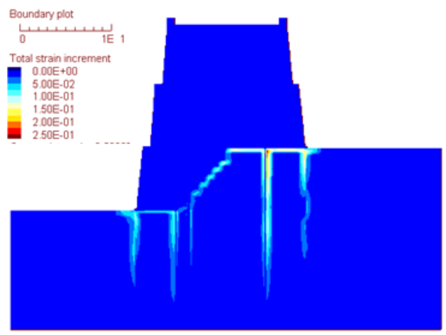


Figure 14 Total Strain Plot for Previously Recommended Solution

The replacement of the back fill material with a rock fill appears to avoid the potential deep-seated failure within the supported soil mass. Furthermore, the extension of the RC slab across the roadway section reduces but does not remove the development of tensile stresses at the face of the retaining wall.

If one adopts the multiple level solution for the high elevation problem, the design and analysis shall follow a satisfactory method which suite the type of retaining wall. Weight (2008) studied the various methods available for the design of multiple level retaining walls and indicated that there is significant variation in estimation of the sliding force and overturning moment estimates among these methods, ranging 5 to 15 times in magnitude. Furthermore, the most critical attribute in the geometrical design the offset distance between the top of the bottom wall and the toe of the top wall ‘C’, see figure 15.

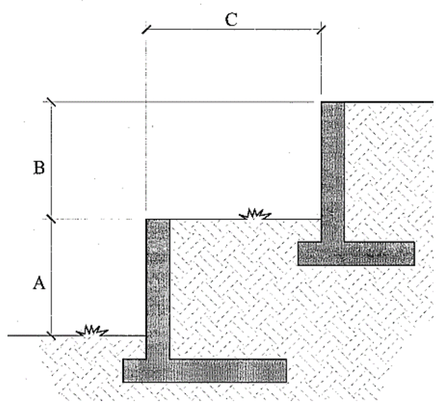


Figure 15 Geometric elements of Stacked Retaining Walls, (Weight, 2008)

At Logita Bridge, approach embankment although there is an offset of 0.5 m between the masonry walls. The foundation of the subsequent wall, however, does not have any offsets. The authors of this report simulated introduction of such an offset to evaluate the possibility of reducing the tensile stress on the face of the wall. Figure 16 shows the effect of cascading the walls by 1.0.

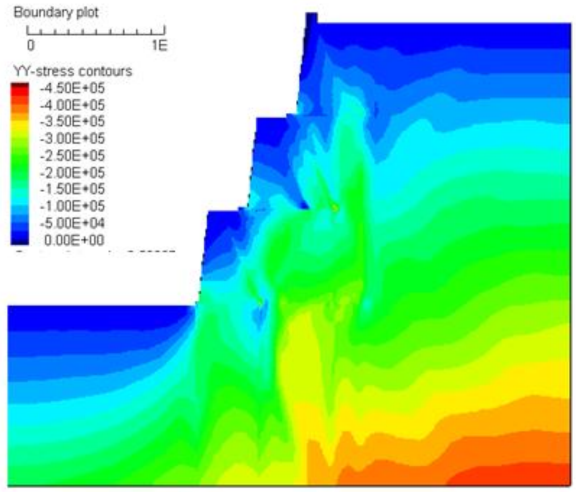


Figure 16 Total Vertical Stress Plot for 1 m offset Logita-Design

The above analysis shows offsetting the top wall from the head of the bottom wall in fact eliminates tensile stresses from the face of the wall. Weight (2008) identified a jump in factor of safety after the offset distance passes the point above the heel of the bottom wall. However, his modeled used cantilever retaining walls.

4 Conclusion and Recommendations

The numerical analysis conclusively proves that the observed distress are caused by development of tensile internal stresses at the face of the wall. If the final wall is constructed, the numerical simulation indicates that wall will collapse, not necessarily from stability of the wall but through deep-seated stability failure through the backfill and foundation.

The structural system adopted to deal with the relatively high elevation level is ambiguous and is not very well defined. As a result, the analysis conducted to verify the stability of the wall (presumably EQU, STA & GEO) fails to demonstrate the shortcomings of the geometry and material selection. The design of the retaining wall using limit equilibrium methods and stability consideration alone is inapplicable due to the complexity of the structural system adopted for the wall. Hence, the wall needs to be redesigned with a clearly defined structural system. As per the analysis presented above, the cause of the distresses can be attributed to a faulty design concept and implementation.

The codes and guidelines available do not require the analysis of internal stresses (STR) for retaining wall design. However, these codes do not prescribe a complicated structural system such as adopted at Logita bridge site either. Hence, it is the responsibility of the designer to make sure that it chooses a well-defined structural solution, and verify that it is safe and economical.

Some of the previously proposed solution such as replacing the backfill with a rock fill and extending the reinforced concrete slab across the road section do not fundamentally address the design shortfalls. These may address the potential failure due to plastic flow within the backfill, but fail to address the tension cracks on the face of the wall. It is worth noting that the reinforced concrete is not design for the proposed new loading condition and its response is unknown.

Any solution shall take into account the construction viability i.e. the need to dismantle significant portion of the existing wall. If the said modification requires significant demolition, the decision to implement shall be compared in terms of time and cost, with redesigning the retaining wall, as the later will likely have a higher degree of confidence in safety and performance.

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